

Influence of ground motion characteristics on drift demands in steel moment frames designed to Eurocode 8

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Abstract

This paper investigates the influence of the frequency content of ground motion, as well as the structural characteristics and level of inelasticity, on the global and inter-storey drift demands in frames designed to comply with Eurocode 8 provisions. To represent a wide range of structural characteristics, a suite of 39 steel moment frames is considered by varying the number of storeys, intensity of loading, performance requirements and compliance criteria. Extensive incremental dynamic analyses are carried out on the study frames using 72 far-field ground motion records, which are scaled to simulate four levels of behaviour factor. It is shown that the salient parameters that influence the global drift are the ratio of fundamental period to mean period and the behaviour factor. In addition to these, the relative storey stiffness ratio and the first-mode participation factor have a direct, though less influential, effect on the maximum inter-storey drifts. Based on the parametric assessments, regression models are proposed for predicting the global and storey drifts. Finally, the implications of the findings on the rules incorporated in European and United States design codes are highlighted and discussed.

Keywords: seismic behaviour; dynamic response; steel structures; moment frames; nonlinear analysis; Eurocode 8

1. Introduction

The prediction of drift demands plays an important role in seismic assessment and design due to their suitability for correlation with structural and non-structural damage, particularly in framed structures. These demands are typically examined in terms of global drift or storey drifts. The former usually serves as an indicator of overall ductility whilst the latter is usually associated with damage distribution. Many studies have therefore focused on evaluating drift demands in

multi-storey framed structures, with these investigations broadly classified into the following three approaches: (i) idealisation of the structure as an equivalent single degree of freedom system (SDF); (ii) adoption of generic multi-storey frames with controlled stiffness and strength distribution; and (iii) use of frames that are designed to the requirements of a specific code.

Although the idealisation of a structure as a SDF system disregards the influence of various factors such as higher mode effects as well as strength and stiffness distribution, it offers a simple approach to examine the overall inelastic response. This method is employed in several guides such as Annex B of EC8 (CEN, 2004) as well as FEMA-356 (FEMA, 2000) and FEMA-440 (FEMA, 2005) for the seismic assessment of existing structures. Previous studies adopting this approach have shown that the inelastic seismic response of a SDF system is a function of the elastic period (T_e), level of inelasticity (expressed in terms of the ductility demand ' μ ' or force reduction factor ' R '), hysteretic behaviour, post yield stiffness and frequency content of ground motion (Miranda, 2000; Chopra and Chintanapakdee, 2004; Kumar et al., 2011).

The second approach, whereby generic multi-storey frames are utilised, enables an assessment of higher mode and distribution effects. For example, Medina and Krawinkler (2005) used generic frames in which the stiffness and strength are controlled to produce a largely linear first mode profile and such that yielding occurs simultaneously at all storeys. The study concluded that the storey drifts are dependent on the fundamental period (T_1) and number of storeys (N) of the frame. The study was however confined to generic frames, and the adopted suite of ground motion records came from a limited moment magnitude range of 6.5 to 6.9.

The most realistic approach for predicting inelastic drift is to consider frames which are designed to a specific code or design philosophy. Following this procedure, Uang and Maarouf (1994) modelled four existing structures that included braced and moment steel frames as well as reinforced concrete buildings of heights between 2 and 13 storeys. The study illustrated that the global drift amplification factor (which is the ratio of inelastic to elastic global drift for a given ground motion) depends on the degree of inelasticity. This factor was found to be higher than unity for structures with T_1 lower than 0.3 s, and in the range of 0.7-0.9 for other structures. In contrast, the maximum drift amplification factor (which is the ratio of inelastic to elastic maximum storey drift for a given ground motion) can be much higher than 1.0 particularly for frames with a weak first storey. It was also concluded that the fundamental period of a structure

does not influence the drift amplification factor (except for periods lower than 0.3 s). However, the findings are based on only four frames and eight ground motion records seven of which having predominant periods between 0.35 to 0.60 s and one with 1.10 s.

Pettinga and Priestley (2005) examined the inelastic dynamic response of five reinforced concrete frames between 4 and 20 storeys high, and designed in accordance with the direct displacement based procedure. Artificial accelerograms matching the EC8 (CEN, 2004) design spectrum were employed. It was shown that frames above 12 storeys high exhibited significant drift amplification at the top storey. The application of an additional shear force at the top storey was proposed in order to improve the performance. This procedure is consistent with other studies (Medina, 2004; Paulay and Priestley, 1992) as well as earlier US and New Zealand seismic codes (NZS 4203:1992; IBC, 2000).

Karavasilis et al. (2008) performed nonlinear time history analysis on 72 steel frames and varied the number of storeys (ranging from 3 to 20), number of bays, beam-to-column stiffness ratio at mid-height of the frame (ρ), ratio of average plastic moment capacity of the bottom storey column and the average of plastic moments of resistance of the beams of all storeys of the frame (α_{avg}). The frames had T_1 values ranging from 0.53 s to 2.82 s. The study concluded that the global drift is not affected by T_1 and the corner period ' T_c ' of ground motion for $T_1 > 0.5$ s, and is only dependent on the relative intensity (expressed in terms of the behaviour factor). On the other hand, the storey drift was found to be dependent on the number of storeys as well as ρ and α_{avg} , and independent of T_1 . This observation however differs from the findings of Medina and Krawinkler (2005) regarding period independency. It should also be noted that the equations proposed by Karavasilis et al. (2008) for normalised peak storey-to-roof drift satisfy the physical constraint of reducing to unity for single-storey frames.

In terms of codified procedures, design guides typically adopt a simplified approach for the estimation of drift demands. EC8 (CEN, 2004) generally recommends the equal displacement rule. Accordingly, the drifts at roof and storey levels are determined from elastic lateral load analysis factored with the behaviour factor (q) assuming that the drift profile remains unchanged. On the other hand, US guides such as NEHRP (2003), ASCE/SEI (2005) and NEHRP (2009) adopt specific displacement modification factors (C_d) for different lateral resisting systems rather

than the direct use of the force reduction factor (R). Inspection of the relation between R and C_d values shows that the higher the R value the lower is the corresponding C_d .

As discussed above, a review of previous research studies and design provisions indicate significant inconsistencies over the parameters that influence the global and storey drifts of a multi-storey framed structure. Moreover, in most cases, the set of records employed in previous studies excluded large magnitude ground motions that are known to contain significant contributions at long periods of vibration. Consequently, there is a need for a detailed investigation to quantify the parameters that influence the global and maximum drift demands of multi-storey frames. To this end, the main aim of the study presented in this paper is to examine the parameters that influence the global and storey drift demands in moment resisting steel frames which are designed to comply with Eurocode 8 (EC8) provisions. For this purpose, a set of 39 frames is utilised in conjunction with a suite of 72 far-field ground motions which are scaled to simulate four levels of behaviour factor.

2. Structural Characteristics and Modelling Assumptions

To investigate the influence of key parameters on the nonlinear dynamic response, a large set of steel moment frames that satisfy the design provisions of EC3 (CEN, 2005) and EC8 (CEN, 2004) were considered. Figure 1 shows a plan and elevation of the structural system which consists of three lateral resisting moment frames, each of 3 bays of 6.0 m span, with a first storey height of 4.5 m and other storeys of 3.5 m each. The orthogonal direction of the system was assumed to have a separate lateral resisting system. The interior moment frame selected in this study was initially designed for gravity loading according to EC1 (CEN, 2002) and EC3 (CEN, 2005). Dead loads of 4.75 kN/m^2 and 5.75 kN/m^2 were considered for the roof and the remaining floors respectively, whereas live load proportions of 1 kN/m^2 and 2 kN/m^2 were considered for the roof and the remaining floors. Subsequently, seismic design was carried out according to EC8, using various combinations of PGA, soil conditions, and drift limits. European steel profiles were used for the columns (HE) and the beams (IPE), assuming a nominal yield strength of 275 N/mm^2 . The same sections were used for the internal and external columns for each storey. Beam profiles were also kept uniform for a given storey. Equivalent lateral seismic loading based on the

first mode of response was adopted, since the structure satisfies EC8 regularity conditions. Accordingly, the lateral load was distributed using following expression:

$$F_i = F_b \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \quad (1)$$

where, F_i is the horizontal force acting on storey i ; F_b is the seismic base shear obtained from the code spectrum; m_i and m_j are the storey masses; and s_i and s_j are the displacements of masses m_i and m_j , respectively, in the fundamental mode shape.

A total of 39 frames with 3, 5 and 7 storeys were designed using this procedure (Kumar, 2012). A summary of the final member cross-sections as well as the structural characteristics are given in Table 1. The frames were then modelled within the finite element program OpenSees (2008) to conduct nonlinear analyses of the structures. The beam and column elements of the frames were modelled using two force-based elements (with 7 Gauss points) per member. A cyclic bilinear stress-strain curve for steel with a post-yield stiffness of 0.5% was employed. Vertical gravity loads consisting of the dead loads and an allowance of 30% of live loads were applied on the beams. Initial stiffness proportional damping was considered with 2% viscous damping with respect to the first mode of vibration.

In order to enable a detailed assessment of inelastic dynamic behaviour, several characteristics were directly considered based on the frame geometry, or subsequently evaluated using Eigenvalue analysis, simple structural analysis or pushover response. For this purpose, the conventional form of pushover analysis was employed using the lateral load pattern obtained from the first mode shape of the frame. These characteristics include the following:

1. Overall height of the frame, H . As noted before, 3, 5 and 7 storey frames are considered, with total heights of 11.5, 18.5 and 25.5 m, respectively. The distribution of structural characteristics is shown for each of these frame heights in Figure 2.
2. Fundamental period (T_1), obtained using Eigenvalue analysis. The distribution of the fundamental period of the study frames ranges from 0.40 s to 1.75 s as can be appreciated from inspection of Figure 2. Naturally, the fundamental periods of the shorter and taller frames

tend to be toward the lower and upper end of this range, respectively. However, one can note from Figure 2 that there is a region of overlap among the three frame heights.

3. Plasticity resistance ratio (α) (also referred to as α_u/α_1 in EC8), determined as the ratio of base shear (V_y) when the plastic mechanism has developed in the structure to the base shear corresponding to first yield (V_1) (evaluated from pushover analysis of the frame). The distribution of the plasticity resistance ratio within the study frames ranges from 1.39 to 2.42, as indicated in Figure 2. It should be noted that this parameter (α) is preferred to the parameter ' α_{avg} ' proposed by Karavasilis et al. (2008) since ' α ' is obtained directly from the pushover curve rather than an approximation.
4. Beam-to-column stiffness ratio (ρ) of the frames, calculated for the storey closest to the mid-height of the structure, using the expression:

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

In the above equation, ' I ' and ' l ' are the second moment of inertia and length of the member, respectively, and subscripts ' b ' and ' c ' refer to the beam and column respectively. The distribution of beam-to-column stiffness ratios of the study frames is shown in Figure 2, and ranges from 0.07 to 0.21.

5. Relative storey stiffness ratio (β), determined using the inter-storey drift profile corresponding to the first mode shape of the frame obtained from Eigenvalue analysis. While other parameters mentioned above have been employed in previous studies to examine the inelastic response, this parameter is proposed in this investigation primarily to account for variation in the stiffness or strength of the upper storeys. This parameter may play a role considering that comparatively flexible/weak upper storeys may result in early yielding and relatively high drift demands at upper storeys due to higher mode effects. This parameter can be determined in several ways in order to capture the relative stiffness effect. Three different variations are proposed here: (i) β_1 , evaluated as the ratio of the inter-storey drift at the top storey of the frame to the maximum value in other storeys; (ii) β_2 , determined as the ratio of the maximum inter-storey drift for the upper 1/3rd of the frame to the maximum value in the lower 2/3rd; (ii)

β_3 , calculated as the ratio of the maximum inter-storey drift for the upper half of the frame to the maximum value for the lower half. For a frame with an odd number of storeys, the upper 1/3 and 1/2 of number of storeys is rounded off to the lower number. For example, for a 7 storey frame, β_2 is calculated as the ratio for the top 2 storeys to the bottom 5 storeys; similarly, β_3 is estimated as the ratio for the top 3 storeys to the bottom 4 storeys. The distribution of β_1 , β_2 and β_3 of the study frames is given in Figure 2. The evaluation of β_1 , β_2 and β_3 is illustrated for a 7-storey frame in Figure 3 which depicts the inter-storey drift profile obtained from Eigenvalue analysis. β_1 for this frame is 0.42, which is the ratio of inter-storey drift of the top storey (0.21%) and the maximum inter-storey drift for other storeys (0.50% at the 3rd storey). β_2 for the frame is 0.58, which is the ratio of the maximum inter-storey drifts for the upper 1/3rd (top 2 storeys) found to be 0.29% at the 6th storey and for other storeys (0.50% at 3rd storey). Similarly, β_3 for the frame is 0.78 (ratio of 0.39% at the 5th storey to 0.50% at the 3rd storey). It should finally be recalled that the geometry of all the study frames (i.e. number of bays, beam span, and storey heights) was kept constant, and related parameters were not considered as key variables within this investigation.

6. First mode participation factor (γ), computed following the eigenvalue analysis. This is used as another proxy measure for the contribution of higher mode effects in the drift response. The greater the value of this first mode participation factor, the less influential the contribution of higher mode effects will be. The distribution of this first-mode participation factor is also shown (by frame height) in Figure 2. It is clear that the two taller frames heights have similar first-mode participation, and that the lower height frames are more first-mode dominated (as expected).

3. Ground Motions and Frequency Content

In order to assess the influence of ground motion frequency content, the Mean Period (T_m) was chosen based on the studies of Kumar et al. (2011; 2013) and Karavasilis et al., (2011). The Mean Period, originally proposed by Rathje et al. (1998; 2004) as a frequency content measure, is determined as the weighted mean of the periods of the Fourier Amplitude Spectrum (FAS) over a pre-defined frequency range, where the weights are assigned based on the Fourier amplitudes and calculated using the following relationship:

$$T_m = \frac{\sum_i C_i^2 \times \frac{1}{f_i}}{\sum_i C_i^2} \text{ for } 0.25 \text{ Hz} \leq f_i \leq 20 \text{ Hz, with } \Delta f \leq 0.05 \text{ Hz} \quad (3)$$

In the above equation, C_i is the Fourier amplitude coefficient corresponding to a frequency (f_i) obtained from a discrete Fast Fourier Transform (FFT) for frequencies between 0.25 and 20 Hz; and Δf defines the spacing of the frequencies for which the FFT is performed.

To investigate the influence of the frequency characteristics of ground motion, 72 far-field records from 21 earthquake events that include a wide range of magnitude, distance and soil conditions (according to the NEHRP classification), were identified. The distribution of earthquake records used in this study with respect to moment magnitude, rupture distance and site class is depicted in Figure 4. Only one horizontal component from each recording station was selected. In order to limit the study to far-field ground motion, the records were chosen with rupture distances in the range of 0-80 km (closest distance from fault rupture) for moment magnitudes between 5.5 and 6, and within the range of 20-80 km for magnitudes greater than 6. The list of earthquakes used in this study, alongside the main parameters, is presented in Table 2. More detailed information on the selected records can be found elsewhere (Kumar, 2012).

4. Assessment of Seismic Drift Demands

4.1 Analysis Procedures

Incremental dynamic analyses (IDA) of the frames were carried out by scaling the records with respect to the fundamental period (T_1) to attain various levels of relative intensities (represented by the behaviour factor 'q' in EC8). The scaling factor (S_F) required for an individual record to attain a given behaviour factor is determined as follows:

$$S_F = q \times \frac{V_1}{S_a(T_1) \times m \times \gamma} \quad (4)$$

Where $S_a(T_1)$ is the spectral acceleration of a given record at the fundamental period of the frame; V_1 is the base shear corresponding to the formation of first yield in the frame, as obtained from static pushover analysis using a force profile based on the fundamental mode shape; m is

the seismic mass of the structure; and γ represents the mass participation ratio corresponding to the first mode.

The ground motions were scaled in order to achieve four behaviour factor (q) levels: 3, 4, 5 and 6. For each analysis, the maximum roof displacement (Δ_{max}) and the maximum inter-storey drift (θ_{max}) were recorded. The results from each analysis were then processed to determine the following:

- Global drift modification factor (δ_{mod}), determined as the ratio of the maximum roof displacement (Δ_{max}) recorded from IDA for a given behaviour factor to the product of behaviour factor (q) and the roof yield displacement ($\Delta_{1,roof}$) (i.e. roof displacement at first yield obtained from pushover analysis using a force profile based on the fundamental mode shape). This can be expressed as:

$$\delta_{mod} = \frac{\Delta_{max}}{q \times \Delta_{1,roof}} \quad (5)$$

- Maximum drift modification factor (θ_{mod}), determined as the ratio of the maximum inter-storey drift (θ_{max}) (recorded from IDA for a given behaviour factor) to the product of the behaviour factor (q) and the maximum inter-storey drift at the formation of first yield ($\theta_{1,max}$) (obtained from pushover analysis using a force profile based on the fundamental mode shape). It should be noted that for a given dynamic and pushover analysis, θ_{max} and $\theta_{1,max}$ may not occur at the same storey. This factor can be expressed as:

$$\theta_{mod} = \frac{\theta_{max}}{q \times \theta_{1,max}} \quad (6)$$

The results obtained from extensive incremental dynamic analyses were processed to examine the influence of various parameters on the global and maximum drift modification factors, as discussed below.

4.2 Global drift modification factor (δ_{mod})

Initial inspections of the IDA results indicated that the predictor variables that exerted the greatest influence upon the global drift modification factor were the period ratio (T_1/T_m) and the

behaviour factor (q). In order to assess the influence that other structural characteristics have upon the behaviour of the global drift modification factor, the dominant trends with respect to T_1/T_m and q were first modelled. Figure 5 shows the variation in δ_{mod} as a function of T_1/T_m for the considered values of q . The shaded regions represent the 95% confidence interval in the estimate of the mean logarithmic drift. The open symbols denote median values within bins, while the closed symbols are the mean values. The dashed lines indicate the plus/minus one standard deviation of $\ln\delta_{mod}$ for the model fitted and shown by the solid line. It is clear from this figure that the modification factor is relatively insensitive to the period ratio (or tuning ratio) when this variable takes values roughly between 1 and 2.7. For longer period ratios the modification factor tends to scale linearly with the period ratio, while at shorter period ratios the scaling is nonlinear. From Figure 5 it is also clear that this nonlinear scaling at short period ratios is a function of the level of nonlinearity, as expressed by the behaviour factor q . The reason for the change in scaling of the global drift modification factor with respect to the period ratio can be explained using physical arguments. For T_1/T_m lower than 1, δ_{mod} increases as T_1/T_m decreases. This trend is similar to that noted for the inelastic response of SDF systems found in previous studies (e.g. Kumar et al., 2011). In the short T_1/T_m range, the elongated fundamental period of a given structure is closer to the T_m of ground motion leading to relatively higher δ_{mod} . For the same reason, δ_{mod} increases with the increase in q in the short T_1/T_m range. For the intermediate T_1/T_m range, the elongated fundamental period of the structure is higher than T_m ; therefore the influence of T_1/T_m on δ_{mod} is relatively insignificant given that nonlinearity shifts the frames away from first-mode resonance. However, it is observed that the increase in q , which results in further elongation of the fundamental period, reduces δ_{mod} in this range. For large T_1/T_m ratios, the T_m of the ground motion becomes closer to the higher mode periods particularly the second mode. Hence, the δ_{mod} increases with the increase in T_1/T_m . It should be noted that for the frames considered in this study, the ratio of the fundamental period to the second-mode period is just over 2.7. It is observed that for a given T_1/T_m in this range, the higher the value of q , the lower the corresponding value of δ_{mod} . However, close examination shows that the slope of the fitted model is not a function of q over these period ratios suggesting that the higher-mode periods do not experience significant elongation.

It can also be appreciated from Figure 5 that there is significant dispersion associated with the model that has been fitted in this case. The plus and minus one standard deviation for the model

is shown using the dashed lines, while the shaded regions denote the 95% confidence interval in the estimate of the mean value of the global drift modification factor for a given bin of the period ratio. With the dominant trends in T_1/T_m and q accounted for, it is important to now see if this dispersion can be explained by other characteristics of the frames being considered. In order to understand the influence that other structural characteristics have upon the computed global drift modification factors, the residuals (observed minus modelled) of the model fit shown in Figure 5 are plotted against other structural characteristics in Figure 6. The residual panels on the left show the total residuals, while those on the right show the mean residuals in various bins for the same structural characteristics. In the lower six panels, the solid and dashed red lines denote linear trend lines and their 95% confidence interval, while the blue lines are locally-weighted quadratic polynomial trend lines for showing local fluctuations in the residuals. The continuous line in the upper right panel shows the heteroskedastic standard deviation of the model. The figure shows that by modelling the dependence of the global drift modification factor on the period ratio and the behaviour factor, the main trends in the data have already been accounted for. There appears to be very little influence associated with the height of the frame once the information regarding the fundamental period has been included in the period ratio. The consideration of the residuals, as grouped by the behaviour factor, reveals a degree of heteroskedasticity with respect to q . That is, the dispersion in the drift modification factors is lower for lower values of q . The dispersion increases nonlinearly and reaches a plateau at a behaviour factor of around 6. This behaviour is to be anticipated, and one should expect that this dispersion would decrease down towards zero as q tends to unity. It is important to account for this heteroskedasticity when developing the final prediction model to ensure that unbiased performance is obtained for all levels of nonlinearity.

The lower panels of Figure 6 emphasise two key features of the dataset being considered. The first is that there is a significant amount of dispersion associated with the global drift modification factors. If one considers the plot of the residuals against the fundamental period, it is clear that there are no statistically-significant trends, but there are also numerous cases of the residuals having absolute values in excess of 0.5. Using natural logarithmic units, such residual values are associated with factors of roughly 1.65 on the average response. The second point that is reinforced in Figure 6 is that a very large number of analyses have been conducted within this study. The implication of working with such a large dataset is that formal regression approaches

lead to the conclusion that there are statistically-significant trends in these residuals with respect to the plasticity resistance ratio, the relatively storey stiffness ratio and, to a lesser extent, the beam-to-column strength ratio (not shown in Figure 6). The removal of even mild trends results in a significant reduction in the log-likelihood value obtained in the regression analysis and in a consequent reduction in both the Akaike or Bayesian Information Criteria. For example, the lower two panels on the right-hand-side of Figure 6 both indicate that the global drift modification factor increases with increasing values of α_u/α_1 and β_3 , and that this dependence is strongly supported from a formal statistical perspective. These results very strongly support the inclusion of terms within a prediction equation that relate to these independent variables. However, over the ranges of parameters considered herein, the difference in median predictions that one would obtain after removing the trends with respect to α_u/α_1 and β_3 amount to 3-5%. Such differences should be viewed in the context that the standard deviation in the residuals is ranging from just under 0.2 to just over 0.3, as a function of q . These standard deviations imply that around 70% of the observations fall within 20% of the median at $q = 3$, and to within just under 40% of the median at $q = 6$. Furthermore, the inherent variability in earthquake-induced ground-motions must also be noted. While the above context must be borne in mind, the mild trends observed with respect to the plasticity resistance ratio and the relative storey stiffness ratio are consistent with the expectations of which structural characteristics lead to greater nonlinear demand. The higher values of α_u/α_1 and β_3 can be interpreted as being associated with higher overall levels of plasticity and greater contributions from higher-mode effects, respectively.

4.3 Maximum drift modification factor (θ_{mod})

The same process as described in the previous section was also followed in order to understand the relationship between the maximum drift modification factor and the various structural characteristics under consideration. The initial inspection of the data again indicated that the dominant trends could be explained through consideration of a model based primarily upon the T_1/T_m period ratio (or tuning ratio) and the behaviour factor q . Figure 7 shows a model that was fit to the maximum drift modification factors in order to allow an investigation of the model residuals to be undertaken. As for Figure 5, the dashed lines represent the plus/minus one standard deviation of the logarithmic drift modification factor, while the solid line represents the fitted model. The shaded region shows the 95% confidence interval in the estimate of the mean

modification factor in a given bin. This figure illustrates that the general scaling features that were observed for the global drift modification factor are again seen for the maximum drifts. However, in this case the range of period ratio over which the maximum drift modification factor is essentially constant for a given q value is greatly reduced. From Figure 7 it can be seen that this plateau extends roughly from a period ratio of unity through to a ratio of around 1.7 (as opposed to the ~ 2.7 seen for the global drifts). It can also be appreciated that for low period ratios the maximum drift modification depends nonlinearly upon the period ratio and is also a function of q , while for period ratios beyond roughly 1.7 this dependence on q is negligible.

Figure 8 shows the residuals obtained from the model shown in Figure 7 against other structural characteristics. The residual panels on the left show the total residuals, while those on the right show the mean residuals in various bins for the same structural characteristics. In the lower six panels, the solid and dashed red lines denote linear trend lines and their 95% confidence interval, while the blue lines are locally-weighted quadratic polynomial trend lines for showing local fluctuations in the residuals. Whereas the residuals for the global drift modification factors were very mild after consideration of the base scaling with respect to the period ratio and the behaviour factor, the residuals in Figure 8 show much stronger trends. Once again it must be emphasised that even these strong trends are relatively mild once one considers the degree of dispersion of the residuals shown in this figure. However, the trends shown with respect to both the first mode participation factor and the relative storey stiffness ratio would result in estimates of the maximum drift that are biased by up to 10% over the range of parameters considered here. Note that the beam-to-column strength ratio, while not shown in Figure 8, also exhibits mild trends which are similar in magnitude to those shown for the plasticity resistance ratio.

Although the relative storey stiffness ratio shown in Figure 8 is that defined as β_3 earlier, the residual trends with respect to the other definitions of the relative storey stiffness were also considered. From these considerations it was observed that all metrics were able to explain some of the residual trends but that they did not act equally over all period ratios. For example, the parameter β_1 exerted the strongest influence for relatively large values of the period ratio. Both parameters β_2 and β_3 performed similarly in terms of their ability to explain residual trends over a broad range of period ratios. Hereafter, β_3 is utilised as a predictor variable, but the selection of this metric is somewhat arbitrary. The reason for including a parameter relating to the relative

storey stiffness ratio was to characterise the influence of higher-mode effects. When higher modes play a role in the response, the interstorey drift values in the upper storeys are expected to increase and this behaviour is precisely that shown in Figure 8.

While Figure 8 indicates that there are residual trends with respect to the plasticity resistance ratio, these trends are mild and it will later be demonstrated that these arise as a result of the inherent correlations among the structural characteristics within the dataset. For example, once the trends that are shown with respect to the first mode participation factor and the relative storey stiffness ratio are accounted for, the mild trends associated with the plasticity resistance ratio no longer exist.

The influence of the first mode participation factor has not been discussed previously. Figure 8 suggests that the maximum drift modification factors are quite sensitive to this parameter, but it may be thought that this is related to the trends with respect to the relative storey stiffness ratio. That is, if the objective of the relative storey stiffness ratio is to indirectly represent the influence of higher-mode effects, the first mode participation factor would also seem to represent this. However, it is found that after accounting for the trend with respect to β_3 a trend with respect to the first mode participation factor remains. Given that the process of computing one of these parameters is very similar to that required for computing the other, there is no reason why both parameters cannot be used to enhance the predictions of the maximum drift response.

Figure 9 shows the improved behaviour of residuals that one obtains when including a linear dependence upon both the first mode participation factor and the relative storey stiffness ratio in the model for the maximum drift modification factor. It should be noted that the lower four panels only show the average residuals in bins. Also, the overall dispersion is better appreciated from the boxplots in the upper panels as well as the full residual plots in Figure 8. In Figure 9, it can be appreciated that the slight bias that was observed for the shorter frames in Figure 8 has now disappeared and that the trend with respect to the plasticity resistance ratio has also gone. There is, however, a mild trend remaining with respect to the beam-to-column strength ratio, but ignoring this trend results in a bias of no more than 2% over the range of parameters considered herein.

5. Prediction Models for Drift Demands

Based on the analysis of residuals described in the previous section, prediction models for the global and maximum drift modification factors are proposed and discussed in this section. Regression analysis for deriving the models that follow was conducted in R (R Development Core Team, 2012).

5.1 Global drift modification factor (δ_{mod})

Previously it was shown that the parameters that have the most significant influence on δ_{mod} are: T_1/T_m and q . While it was noted that the inclusion of the plasticity resistance ratio or the relative storey stiffness ratio within a predictive model would be justified on statistical grounds, the improvement is not deemed justified in light of the additional effort required to compute these parameters. It is common practice to assume that interstorey drift response is log-normally distributed, despite formal tests indicating that this is not always the case (e.g., Buratti et al., 2011). This assumption has a direct impact upon how one performs the regression analysis. In the present study it was found that both the global and maximum drift modification factors were not strictly log-normally distributed. This can be appreciated in Figures 6 and 8 in which one can see that the distribution of residuals is slightly positively skewed (these residuals are computed assuming that log-normality is appropriate). During the regression analysis, generalised nonlinear models were considered as were variable transformations. However, as the variance has quite a strong dependence upon q , the transformations that are required result in a more complicated and less intuitive predictive model. The reason for this is that the most appropriate transformations involved regressions on a transformed variable such as $y = 1/\delta_{mod}^{f(q)}$, where $f(q)$ is a quadratic function in q . Rather than complicate matters by making use of such transformations, a more pragmatic approach was adopted. The regressions were performed on the log-transformed global drift modification factors directly. This has two advantages: (1) it makes the approach adopted here consistent with other studies, and hence more familiar to end users; and (2) it is conservative as the positively skewed distribution will result in slightly higher values of the drift modification factors than would ideally be the case. This latter point can be appreciated from inspection of Figure 5 in which it can be seen that there is generally a difference between the mean (solid markers) and median (open markers) levels of the modification factors.

The functional form used for the model for the global drift modification factor is shown in Equation (7), in which the coefficient 2.782 is the mean ratio of the first to second mode period of the frames studied herein. The use of a less precise number, such as 2.8, would have little effect upon the performance of the model and could be invoked to simplify this expression slightly.

$$\ln \delta_{mod} = c_0 + c_1 q + (c_2 + c_3 q) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] + c_4 \ln \left[\max \left(\frac{T_1}{2.782 T_m}, 1 \right) \right] \quad (7)$$

The regression coefficients for the above equation are presented in Table 3. As previously discussed, the variance of $\ln \delta_{mod}$ is heteroskedastic with respect to q and so this was accounted for during the regression analysis through the use of a generalised nonlinear least squares algorithm. The resulting model for the standard deviation of the logarithmic drift modification factor is given by the functional form:

$$\sigma_{\ln \delta_{mod}}(q) = \sigma_0 \sin \left[\frac{\pi}{10}(q - 1) \right] \quad (8)$$

The value of σ_0 is the maximum value of the standard deviation that is obtained when $q=6$ and is given in Table 3. For values of q above 6 the standard deviation should be taken as σ_0 . The scaling of this model has previously been shown in the upper right panel of Figure 6 as this functional form was that initially developed in order to cater for the scaling of the period ratio and the behaviour factor.

5.2 Maximum drift modification factor (θ_{mod})

Similar considerations were given for the development of the model for the maximum drift. In this case, however, the degree of skewness of the residuals when assuming a log-normal distribution for the maximum drift modification factor is more significant (see Figure 7). For the same reasons stated previously, the regression analysis was still conducted using the log-transformed maximum drift modification factors. Alternative regressions were also conducted using the full q -dependent transformed variables in order to check that the general scaling of the model obtained using the simpler log-transform approach was consistent with the more elaborate analyses. It was found that while the formal transformations are justified and are strictly more

appropriate, the performance of the model developed using the log-transformed maximum drift modification factor is generally still of a very high quality and so we opt for simplicity here when presenting the model.

The functional form adopted for the prediction of the maximum drift modification factor is very similar to that used for the global drifts, but includes linear terms for accounting for the influence of the first-mode participation factor and the relative storey stiffness ratio.

$$\ln \theta_{mod} = c_0 + c_1 q + (c_2 + c_3 q) \ln \left[\min \left(\frac{T_1}{T_m}, 1 \right) \right] + c_4 \ln \left[\max \left(\frac{T_1}{1.7 T_m}, 1 \right) \right] + c_5 \gamma + c_6 \beta_3 \quad (9)$$

A heteroskedastic variance structure is again incorporated into the model development using the functional form shown in Equation (10).

$$\sigma_{\ln \theta_{mod}}(q) = \sigma_0 \sin \left[\frac{\pi}{10} (q - 1) \right] \quad (10)$$

The regression coefficients for the above equations are presented in Table 4. Again, when implementing the model for values of q above 6, the standard deviation should be taken as σ_0 . The general behaviour of this model can be appreciated from inspection of Figure 7. While the model fit shown in Figure 7 relates to the model without the linear corrections for the first mode participation factor or the relative storey stiffness ratio, these terms account for the minor trends shown in Figure 8. The improved performance of this complete model is evidenced by the very good behaviour of the residuals shown in Figure 9.

The models for both the global and maximum drift modification factors are able to capture the main scaling well considering their use of a relatively small number of coefficients. In both cases, the models can be regarded as having a plateau that depends upon q for values of the period ratio ranging between unity and either ~ 2.7 (for the global drift) or 1.7 (for the maximum drift). This scaling is reflected by the first two terms of Equations (7) and (9). The two logarithmic terms that involve the minimum and maximum functions in these equations simply take care of the q -dependent scaling for low period ratio scenarios, and of the q -independent scaling for high period ratio scenarios. The heteroskedastic standard deviations make intuitive sense (increasing variance with increasing nonlinearity) and also help to ensure that the models are not biased for lower

values of q (which would be the case had this heteroskedasticity not been accounted for). The assumption of lognormality for the drift modification factors enables probabilities of exceeding particular levels of drift response to be readily computed and used within a performance-based design framework.

6. Comparative Studies and Design Considerations

In this section, the prediction models proposed in this study are compared, where possible, with the findings of previous research studies as well as current European and US design guidance.

6.1 Comparison with Previous Research

As discussed in the introduction, previous studies are not in full agreement with regards to the parameters that influence the global and maximum drift demands. However, the findings of this investigation cannot be meaningfully compared with those from other research as most previous studies have not incorporated this range of structural characteristics and frequency content parameters. A limited comparison is nevertheless possible with the studies conducted on steel frames by Uang and Maarouf (1994) and Karavasilis et al. (2008) as discussed below.

Uang and Maarouf (1994) examined the influence of various parameters on δ_{mod} and θ_{mod} . For two steel frames: one moment resisting frame (MRF) with $T_1 = 2.1$ s, and one braced frame (BF) with $T_1 = 0.3$ s. For these structures, δ_{mod} and θ_{mod} were found to be dependent on the degree of inelasticity (measured in terms of q_μ , where q_μ is defined as the ratio of q and α_u/α_1). The use of q_μ in their study complicates the comparison a little. However, Uang and Maarouf (1994) used values of q_μ from 2 to 4; therefore, by making use of the average values of α_u/α_1 in our study (of 1.75) we can regard this range as being equivalent to a range of q from 3.5 to 7. For q_μ of 2 to 4 (hence, for q of 3.5 to 7), δ_{mod} was found to decrease from 0.8 to 0.72 for the MRF and to increase from 0.82 to 1.1 for the BF. On the other hand, for the same range of q_μ , θ_{mod} was found to increase from 1.1 to 1.3 for the MRF, and from 1.1 to 1.2 for the BF. To enable a meaningful comparison with this study herein, δ_{mod} is determined using Equation 7 for values of q corresponding to q_μ of 2 and 4 (q is obtained by using the average value of $\alpha_u/\alpha_1 = 1.75$ for our dataset), T_m of 0.55 s and 0.80 s, and T_1 ranging from 0.2 s to 2.2 s. On the other hand, θ_{mod} is evaluated from Equation 9 using the same values of q , T_m and T_1 , but also using values of $\gamma =$

0.82 and $\beta_3 = 0.77$ (corresponding to the average values of all frames used in this study). T_m of 0.55 s is used as it corresponds to the average value of T_m for the records used by Uang and Maarouf (1994). It should be noted that Uang and Maarouf (1994) used the characteristic period (T_c) as a frequency content measure. The average T_c of the records used was found to be 0.52 s, which was converted to T_m using Equation 11, as proposed in recent studies (Kumar et al., 2011).

$$T_m = 1.05T_c \quad (11)$$

The comparisons for δ_{mod} and θ_{mod} are presented in Figure 10. It is observed that δ_{mod} and θ_{mod} predicted by the models are comparable to those of Uang and Maarouf (1994) (except θ_{mod} for the MRF and q_μ of 4), considering the uncertainties involved and the limited number of data points. Moreover, it is shown that δ_{mod} and θ_{mod} increase with q_μ for the BF, which is a similar trend to that shown by the proposed model. However, Uang and Maarouf (1994) concluded that δ_{mod} and θ_{mod} are insensitive to T_1 unless $T_1/T_c < 0.3$. In contrast, the models proposed in this study show that δ_{mod} and θ_{mod} vary with T_1 and T_m for $T_1/T_m < 1$ (in the short T_1/T_m range) as well as with $T_1/T_m > 2.782$ (i.e., above the ratio of T_1/T_2) and $T_1/T_m > 1.7$, respectively, for δ_{mod} and θ_{mod} (in the long T_1/T_m range). This discrepancy may be due to the small number of frames and low number of records (8), employed by Uang and Maarouf (1994), which mostly consisted of relatively short period records.

In another study, Karavasilis et al. (2008) showed that δ_{mod} is a function of q , as indicated by Equation 12 (which has been re-arranged to be consistent with δ_{mod}), whereas θ_{max} is a function of θ_r , N , ρ and α_{avg} .

$$\delta_{mod} = (q + 0.39) / 1.39q \quad (12)$$

$$\theta_{max} = (1.0 - 0.193 \times (N - 1)^{0.54} \times \rho^{0.144} \times \alpha_{avg}^{-0.19}) / \theta_r \quad (13)$$

The comparison of δ_{mod} using Equation 12 and the model proposed in this paper is presented in Figure 11 for q of 3 and 6, and T_1/T_m range of 0.4 to 4. It is noted from the figure that the predictions of Karavasilis et al. (2008) are reasonably close to the prediction of the model proposed in this study in the intermediate T_1/T_m range. However, the model severely under-

predicts δ_{mod} in the short and long T_1/T_m ranges. This discrepancy may be attributed to the nature of the set of records used by Karavasilis et al. (2008), which consisted of 30 far-field ground motions. Out of the 30 records used in the study, the characteristic period (T_c) of 28 records ranged between 0.20 s to 0.60 s and the average T_c of the records was found to be 0.44. On the other hand, a direct and meaningful comparison of the model proposed herein for prediction of maximum drift demands with that proposed by Karavasilis et al. (2008) was not possible due to different set of parameters required for both models.

6.2 Comparison with Design Provisions

Seismic design provisions typically prescribe a behaviour factor (q), also referred to as the force reduction factor (R), that are dependent on the structural system and ductility class/level. The base shear applied on the structure is reduced using the selected behaviour factor to account for inelastic behaviour. The reduced base shear (design base shear, V_d) should ideally correspond to the base shear at the formation of first yield of the structure (V_1) or significant yield (V_p) according to EC8 and US provisions, respectively. It should be noted that significant yield may differ slightly from first yield, as it refers to the formation of a full plastic hinge in a member. In most cases, V_d tends to be lower than V_1 and V_p , due to several factors including the effects of material and design overstrength coupled with the large behaviour factors proposed in codes (Elghazouli, 2010).

For the estimation of drift demands, EC8 (CEN, 2004) uses the equal displacement rule in most cases. In other words, the global and maximum drift modification factors for determining the inelastic drift demands are considered as unity. The US provisions (NEHRP, 2003; ASCE/SEI, 2005; NEHRP, 2009) on the other hand, propose a dedicated factor (referred to as the seismic drift amplification factor, C_d) which depends on the structural system. For ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF), R factors of 3.5, 4.5 and 8.0 respectively are proposed, and corresponding values of 3, 4 and 5.5 are suggested for C_d . It is noted that the C_d factors proposed in US provisions are in general either equal to or lower than the corresponding force reduction factor.

For the purpose of comparing the drift provisions in EC8 and US codes with the proposed predictive relationships, it is assumed that V_d , V_1 and V_p are the same. Thus, the definition of

global and maximum drift modification factor for EC8 and US provisions can be evaluated using Equations 14 and 15 respectively, as follow:

$$\theta_{mod} = \delta_{mod} = 1 \quad (\text{for EC8}) \quad (14)$$

$$\theta_{mod} = \delta_{mod} = \frac{C_d}{R} \quad (\text{for US}) \quad (15)$$

Accordingly, based on US provisions, δ_{mod} and θ_{mod} for OMF, IMF and SMF are found to be 0.86, 0.89 and 0.69 respectively. In order to evaluate θ_{mod} , the mean value of β_3 for all frames (found to be 0.84) is used along with the mean value of γ (found to be 0.817).

The left-hand panel of Figure 12 depicts the prediction of the proposed model for δ_{mod} for q of 3.5, 4.5 and 8.0, and the corresponding factors proposed by EC8 and US provisions. It can be observed that the EC8 demand is notably higher than the predictions of the model for all cases except for the very short T_1/T_m range. On the other hand, the US provisions are less conservative when compared to EC8. However, compared with the model predictions, it is observed that the demand proposed in US provisions is higher in the intermediate ranges of period ratio. For the short T_1/T_m range, the US demands are significantly lower than the model predictions. The difference between the model predictions and US provisions, in this range, generally increases with the increase in q . For the long T_1/T_m range, the model predictions are higher for q of 8 and are roughly consistent for q of 3.5 and 4.5. It should also be noted here that the model developed in this study is being extrapolated beyond the range of data for which it was derived when making comparisons for q of 8.

Similarly, the proposed model for θ_{mod} (for q of 3.5, 4.5 and 8.0) and corresponding factors proposed in EC8 and US provisions are plotted in the right-hand panel of Figure 12. In comparison with EC8, the model predictions are typically lower, with few exceptions particularly the notable under-prediction except in the short T_1/T_m range for q of 8. On the other hand, US provisions are now in closer agreement with the model predictions for q of 3.5 and 4.5; the model predictions for these two q factors are lower than the US prescriptions for intermediate T_1/T_m values, and are higher for short and long T_1/T_m ranges.

7. Concluding Remarks

This paper investigated the influence of structural characteristics, level of inelasticity and ground motion frequency content on global and maximum drift demands of frames designed to EC8. It was observed that the global drift modification factor (δ_{mod}) is most notably dependent on the period ratio (T_1/T_m) and behaviour factor (q). The study showed that the influence of T_1/T_m on δ_{mod} can be divided into three ranges, namely: short, intermediate and long. It was noted that the influence of q on δ_{mod} is also dependent on the T_1/T_m range. Based on the parametric studies, a regression model was developed for predicting δ_{mod} as a function of T_1/T_m and q . The predictions of the model showed that in the short range ($T_1/T_m \leq 1$) δ_{mod} increases with the decrease in T_1/T_m . The maximum drift modification (θ_{mod}) was examined and shown to be strongly dependent on T_1/T_m and q , but that it also had a weaker dependence upon both the relative storey stiffness ratio (expressed as β_3) and the first mode participation factor (γ). The influence of T_1/T_m on θ_{mod} was based on three ranges, as in the case of δ_{mod} . The regression models that were developed for predicting the two different drift modification factors have been shown to agree well with the data over a very wide range of ground-motion characteristics and structural properties, despite being based upon a relatively simple functional form. The regression models recognise that there is greater dispersion in the modification factors for higher values of q .

Comparison with findings from other research studies was limited due to the lack of directly comparable studies incorporating the influence of frequency content, fundamental period and higher mode effects on the drift demands. The model predictions were also compared with European and US provisions for drift modification factors. In general, it was observed that EC8 provisions are highly conservative except for short period ratios with large behaviour factors. On the other hand, US provisions appear to under-predict the global and maximum drift modification factors for short period ratios, a trend that becomes more pronounced for large behaviour factors. In general, the US provisions are relatively close to the model predictions for intermediate period ranges. For long period-ratio ranges, δ_{mod} is under-predicted by US provisions for large behaviour factors, while θ_{mod} is consistently under-predicted for all behavior factors.

The findings of this investigation emphasise the oversimplified nature of drift demand criteria adopted in design codes, particularly in the European provisions. As shown in this paper, a significant enhancement in the reliability of these approaches can be achieved by adopting improved models that can capture the influence of key structural and loading characteristics.

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Tables

Table 1: Design details and corresponding structural characteristics of the frames used in the study (beam and column sizes shown starting from first storey upwards)

Frame ID	Design Details		Structural Properties							
	Beam	Columns	T_1 , s	H, m	α	ρ	β_1	β_2	β_3	γ
A01	IPE500, IPE500, IPE450	HEB500, HEB500, HEB450	0.53	11.50	1.47	0.2	0.73	0.73	0.73	0.88
A02	IPE550, IPE550, IPE500	HEM600, HEM600, HEM600	0.40	11.50	1.42	0.12	0.78	0.78	0.78	0.87
A03	IPE400, IPE400, IPE360	HEB400, HEB400, HEB400	0.76	11.50	1.9	0.18	0.77	0.74	0.74	0.88
A04	IPE550, IPE550, IPE500	HEM550, HEM550, HEM550	0.42	11.50	1.45	0.15	0.85	0.75	0.75	0.89
A05	IPE330, IPE330, IPE300	HEB340, HEB340, HEB340	1.00	11.50	2.42	0.14	0.74	0.74	0.74	0.88
A06	IPE360, IPE360, IPE300	HEB400, HEB400, HEB300	0.85	11.50	2.05	0.12	0.82	0.82	0.82	0.88
A07	IPE450, IPE450, IPE400	HEB450, HEB450, HEB400	0.62	11.50	1.57	0.18	0.77	0.73	0.73	0.88
A08	IPE360, IPE360, IPE360	HEB450, HEB450, HEB400	0.77	11.50	2.04	0.09	0.79	0.79	0.79	0.85
B01	IPE750x161, IPE750x161, IPE750x161, IPE750x161, IPE450	HEM900, HEM900, HEM900, HEM900, HEM600	0.43	18.50	1.43	0.14	0.70	0.74	0.74	0.83
B02	IPE500, IPE500, IPE450, IPE450, IPE300	HEM600, HEM600, HEM550, HEM550, HEM400	0.83	18.50	1.74	0.07	0.91	0.93	0.93	0.78
B03	IPE450, IPE450, IPE450, IPE450, IPE300	HEM600, HEM600, HEM550, HEM550, HEB400	1.00	18.50	1.69	0.14	0.62	0.73	0.73	0.82
B04	IPE750x137, IPE750x137, IPE600, IPE600, IPE400	HEM650, HEM650, HEM650, HEM650, HEB400	0.54	18.50	1.39	0.14	1.00	1.03	1.03	0.80
B05	IPE450, IPE450, IPE400, IPE400, IPE300	HEM550, HEM550, HEB550, HEB550, HEB550	0.97	18.50	1.94	0.07	0.82	0.93	0.93	0.77
B06	IPE450, IPE450, IPE400, IPE400, IPE300	HEB550, HEB550, HEB550, HEB550, HEB400	1.00	18.50	1.69	0.07	0.80	0.91	0.91	0.79
B07	IPE400, IPE400, IPE400, IPE400, IPE300	HEB500, HEB500, HEB500, HEB500, HEB400	1.16	18.50	1.99	0.1	0.64	0.79	0.79	0.82
B08	IPE500, IPE500, IPE500, IPE500, IPE300	HEM550, HEM550, HEB550, HEB550, HEB400	0.81	18.50	1.74	0.15	0.69	0.80	0.80	0.81
B09	IPE750X137, IPE750X137, IPE600, IPE600, IPE400	HEM700, HEM700, HEM600, HEM600, HEB400	0.54	18.50	1.4	0.17	1.00	1.00	1.00	0.80
B10	IPE360, IPE360, IPE330, IPE330 IPE300	HEB400, HEB400, HEB400, HEB400, HEB300	1.48	18.50	2.21	0.09	0.64	0.86	0.86	0.81

Table 1 (continued)

Frame ID	Design Details		Structural Properties							
	Beam	Columns	T_1 , s	H, m	α	ρ	β_1	β_2	β_3	γ
B11	IPE400, IPE400, IPE400, IPE400, IPE300	HEB500, HEB500, HEB400, HEB400, HEB300	1.18	18.50	2.06	0.18	0.64	0.79	0.79	0.81
B12	IPE500, IPE500, IPE450, IPE450, IPE300	HEB550, HEB550, HEB550, HEB550, HEB300	0.90	18.50	1.6	0.11	0.96	0.96	0.96	0.81
B13	IPE400, IPE400, IPE360, IPE360, IPE300	HEB500, HEB500, HEB450, HEB450, HEB300	1.22	18.50	2.01	0.09	0.73	0.88	0.88	0.78
B14	IPE400, IPE400, IPE400, IPE400, IPE300	HEB450, HEB450, HEB450, HEB450, HEB300	1.22	18.50	1.96	0.13	0.62	0.74	0.74	0.82
B15	IPE400, IPE400, IPE400, IPE400, IPE300	HEB500, HEB500, HEB500, HEB400, HEB400	1.15	18.50	1.98	0.12	0.60	0.80	0.80	0.82
C01	IPE550, IPE550, IPE550, IPE550, IPE550, IPE550, IPE450	HEM700, HEM700, HEM700, HEM600, HEM600, HEM600, HEM550	0.96	25.5	1.73	0.12	0.41	0.58	0.77	0.81
C02	IPE500, IPE500, IPE500, IPE500, IPE500, IPE500, IPE300	HEM600, HEM600, HEM600, HEM600, HEM600, HEM600, HEB450	1.13	25.5	1.82	0.09	0.56	0.61	0.76	0.81
C03	IPE500, IPE500, IPE500, IPE500, IPE500, IPE500, IPE450	HEM600, HEM600, HEM600, HEM550, HEM550, HEM550, HEB450	1.13	25.5	1.88	0.11	0.40	0.57	0.76	0.81
C04	IPE550, IPE550, IPE550, IPE500, IPE500, IPE500, IPE450	HEM600, HEM600, HEM600, HEM550, HEM550, HEM550, HEB450	1.04	25.5	1.56	0.11	0.49	0.69	0.89	0.80
C05	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	1.21	25.5	1.73	0.07	0.66	0.75	0.91	0.80
C06	IPE450, IPE450, IPE450, IPE450, IPE400, IPE400, IPE300	HEB550, HEB550, HEB550, HEB500, HEB500, HEB500, HEB300	1.44	25.5	1.72	0.14	0.65	0.73	0.85	0.79
C07	IPE550, IPE550, IPE550, IPE550, IPE550, IPE550, IPE300	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	1.02	25.5	1.67	0.21	0.75	0.75	0.79	0.80

Table 1 (continued)

Frame ID	Design Details		Structural Properties							
	Beam	Columns	T_1 , s	H, m	α	ρ	β_1	β_2	β_3	γ
C08	IPE400, IPE400, IPE400, IPE400, IPE400, IPE400, IPE300	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	1.75	25.5	1.92	0.18	0.43	0.55	0.75	0.82
C09	IPE750x137, IPE750x137, IPE750x137, IPE600, IPE600, IPE600, IPE400	HEM800, HEM800, HEM800, HEM700, HEM700, HEM700, HEB450	0.73	25.5	1.43	0.17	0.75	0.79	0.97	0.79
C10	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300	HEB450, HEB450, HEB450, HEB450, HEB450, HEB450, HEB300	1.57	25.5	1.64	0.13	0.57	0.69	0.89	0.81
C11	IPE550, IPE550, IPE550, IPE550, IPE500, IPE500, IPE300	HEM550, HEM550, HEM550, HEB550, HEB550, HEB550, HEB300	1.08	25.5	1.60	0.15	0.79	0.79	0.90	0.79
C12	IPE450, IPE450, IPE450, IPE400, IPE400, IPE400, IPE300	HEB450, HEB450, HEB450, HEB400, HEB400, HEB400, HEB300	1.59	25.5	1.60	0.18	0.56	0.68	0.90	0.80
C13	IPE500, IPE500, IPE500, IPE450, IPE450, IPE450, IPE300	HEM550, HEM550, HEM550, HEM550, HEM550, HEM550, HEB450	1.46	25.5	1.78	0.19	0.50	0.56	0.75	0.82
C14	IPE450, IPE450, IPE450, IPE450, IPE400, IPE360, IPE300	HEB550, HEB550, HEB550, HEB500, HEB500, HEB360, HEB300	1.46	25.5	1.70	0.14	0.73	0.91	0.91	0.78
C15	IPE550, IPE550, IPE550, IPE500, IPE500, IPE360, IPE360	HEM600, HEM600, HEM600, HEB550, HEB550, HEB550, HEB300	1.15	25.5	1.56	0.21	0.84	0.92	0.95	0.77
C16	IPE550, IPE550, IPE550, IPE500, IPE500, IPE400, IPE300	HEM550, HEM550, HEM550, HEB550, HEB550, HEB450, HEB400	1.11	25.5	1.57	0.15	0.89	0.94	0.94	0.78

Table 2: Catalogue of earthquakes used in the study and related information. In the column for mechanism of earthquake (RV = Reverse, SS = Strike slip, and RO = Reverse Oblique).

Earthquake Name	Moment Magnitude	Mechanism	Number of Records	T _m , sec (min - max)
Friuli, Italy-03 1976-09-11	5.5	RV	1	0.49
Point Mugu 1973-02-21	5.65	RV	1	0.70
Coyote Lake 1979-08-06	5.74	SS	3	0.39-0.47
Coalinga-05 1983-07-22	5.77	RV	3	0.33-0.42
Livermore-01 1980-01-24	5.8	SS	3	0.36-0.71
Taiwan SMART1(5) 1981-01-29	5.9	RV	2	0.41-0.46
Whittier Narrows-01 1987-10-01	5.99	RO	3	0.38-0.59
N. Palm Springs 1986-07-08	6.06	RO	3	0.44-0.55
Parkfield 1966-06-28	6.19	SS	1	0.40
Morgan Hill 1984-04-24	6.19	SS	4	0.53-0.98
Coalinga-01 1983-05-02	6.36	RV	9	0.49-0.87
Friuli, Italy-01 1976-05-06	6.5	RV	2	0.35-0.73
Imperial Valley-06 1979-10-15	6.53	SS	2	0.53-0.58
Superstition Hills-02 1987-11-24	6.54	SS	1	0.45
San Fernando 1971-02-09	6.61	RV	1	0.35
Northridge-01 1994-01-17	6.69	RV	9	0.31-0.81
Loma Prieta 1989-10-18	6.93	RO	8	0.53-0.79
Duzce, Turkey 1999-11-12	7.14	SS	1	0.80
Landers 1992-06-28	7.26	SS	4	0.64-0.90
Kocaeli, Turkey 1999-08-17	7.51	SS	3	0.59-0.85
Chi-Chi Taiwan 1999-09-20	7.62	RV	8	0.52-0.93

Table 3: Regression coefficients for the global drift modification factor (δ_{mod})

c_0	c_1	c_2	c_3	c_4	σ_0
-0.1177	-0.0387	0.3805	-0.1949	0.5027	0.3121

Table 4: Regression coefficients for the maximum drift modification factor (θ_{mod})

c_0	c_1	c_2	c_3	c_4	c_5	c_6	σ_0
0.5888	-0.0462	0.3308	-0.1517	0.3104	0.3564	-1.1000	0.2817

Figures

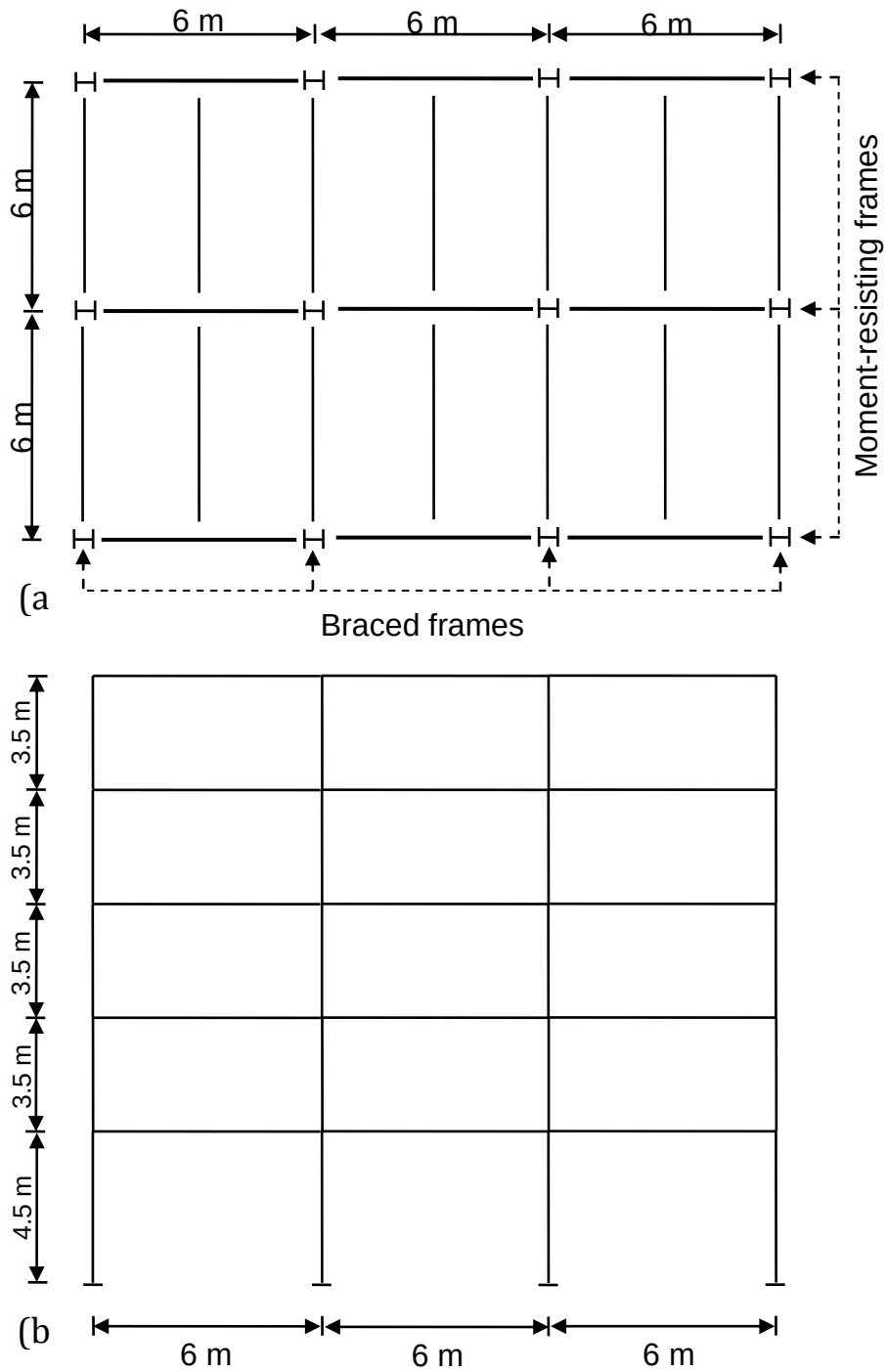


Figure 1: Plan and elevation of typical moment resisting steel frames adopted in the study

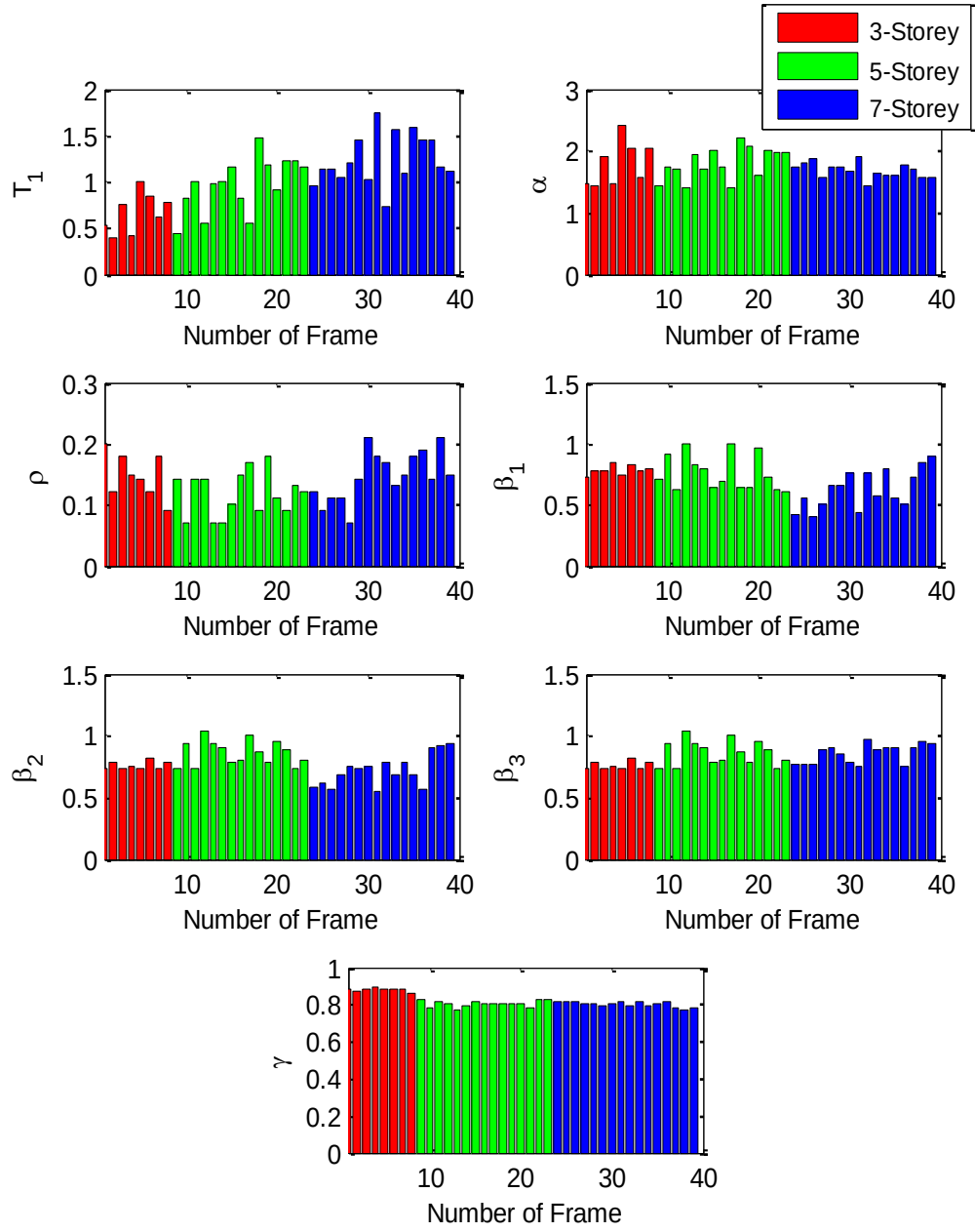


Figure 2: Distributions of structural properties of the frames used in the study, plotted according to the number of storeys of the frame.

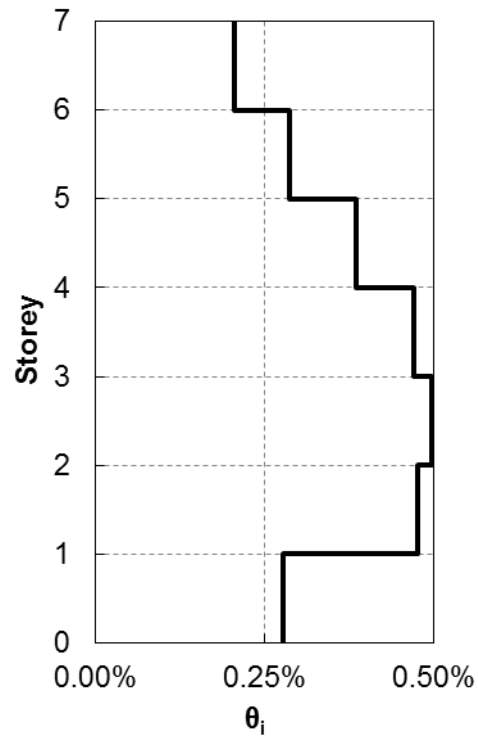


Figure 3: Inter-storey drift profile of a 7-storey frame obtained from eigenvalue analysis

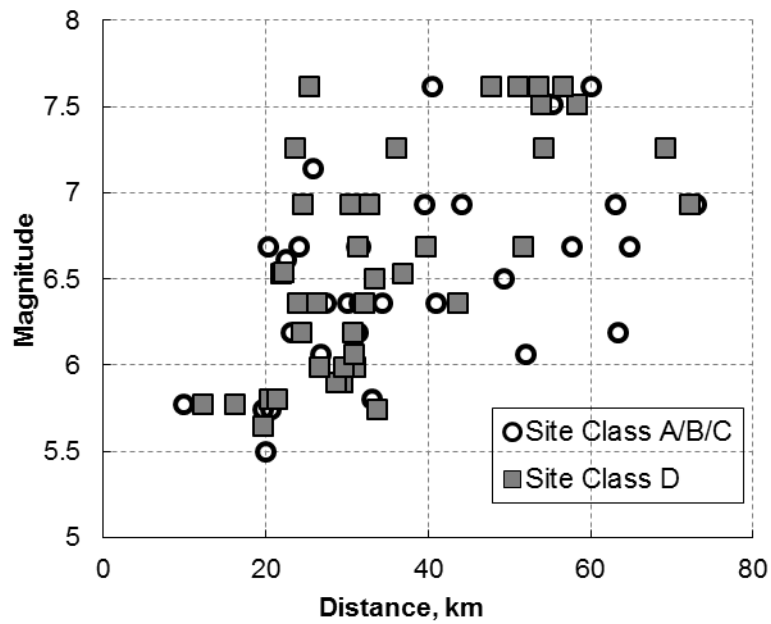


Figure 4: Distribution of moment magnitude, rupture distance and site conditions for the earthquake records used in the study

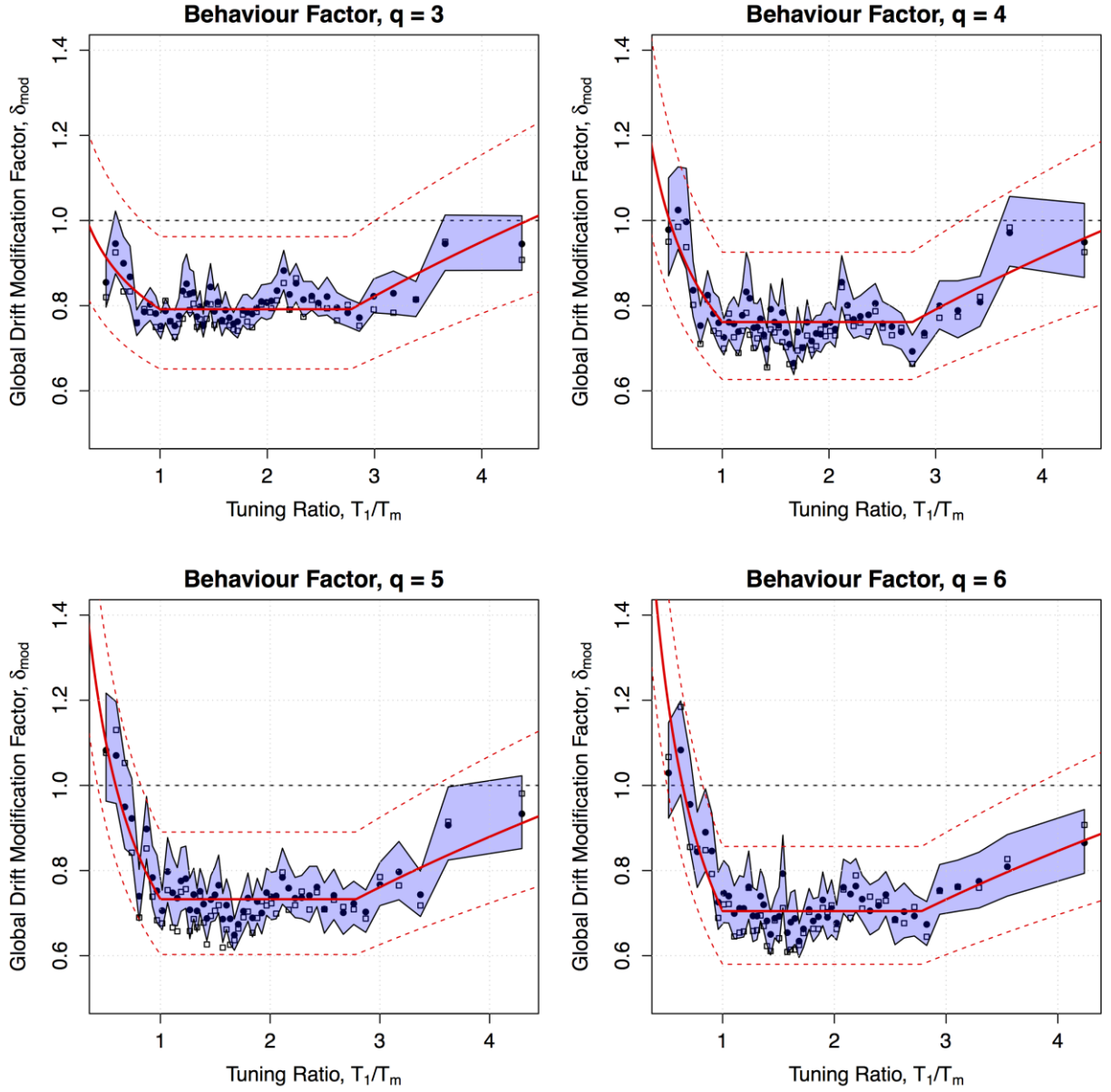


Figure 5: Mean global drift modification (δ_{mod}) factor for various period ratios and behaviour factor bins.

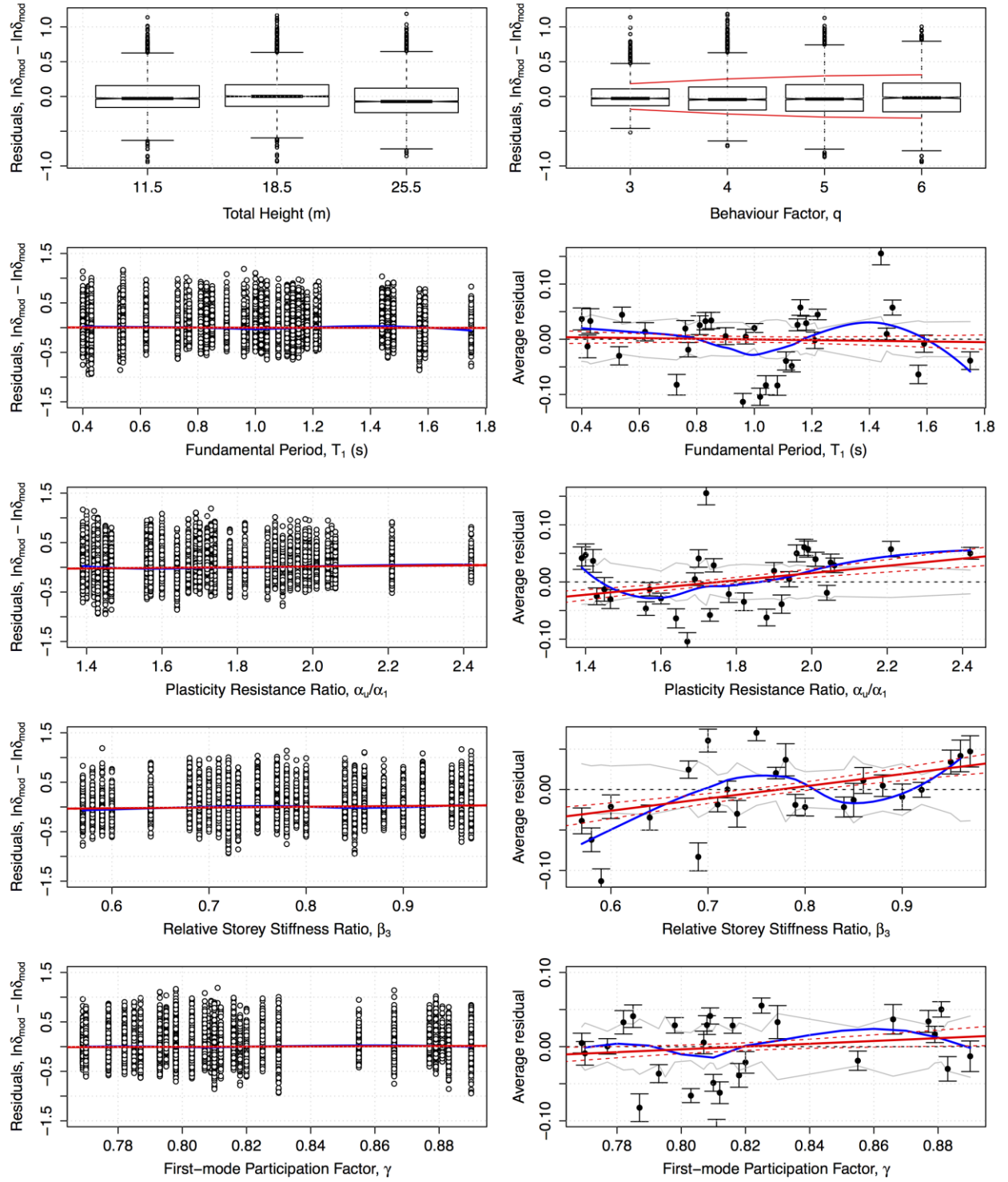


Figure 6: Residuals of the model for δ_{mod} shown in Figure 5 plotted against various structural characteristics.

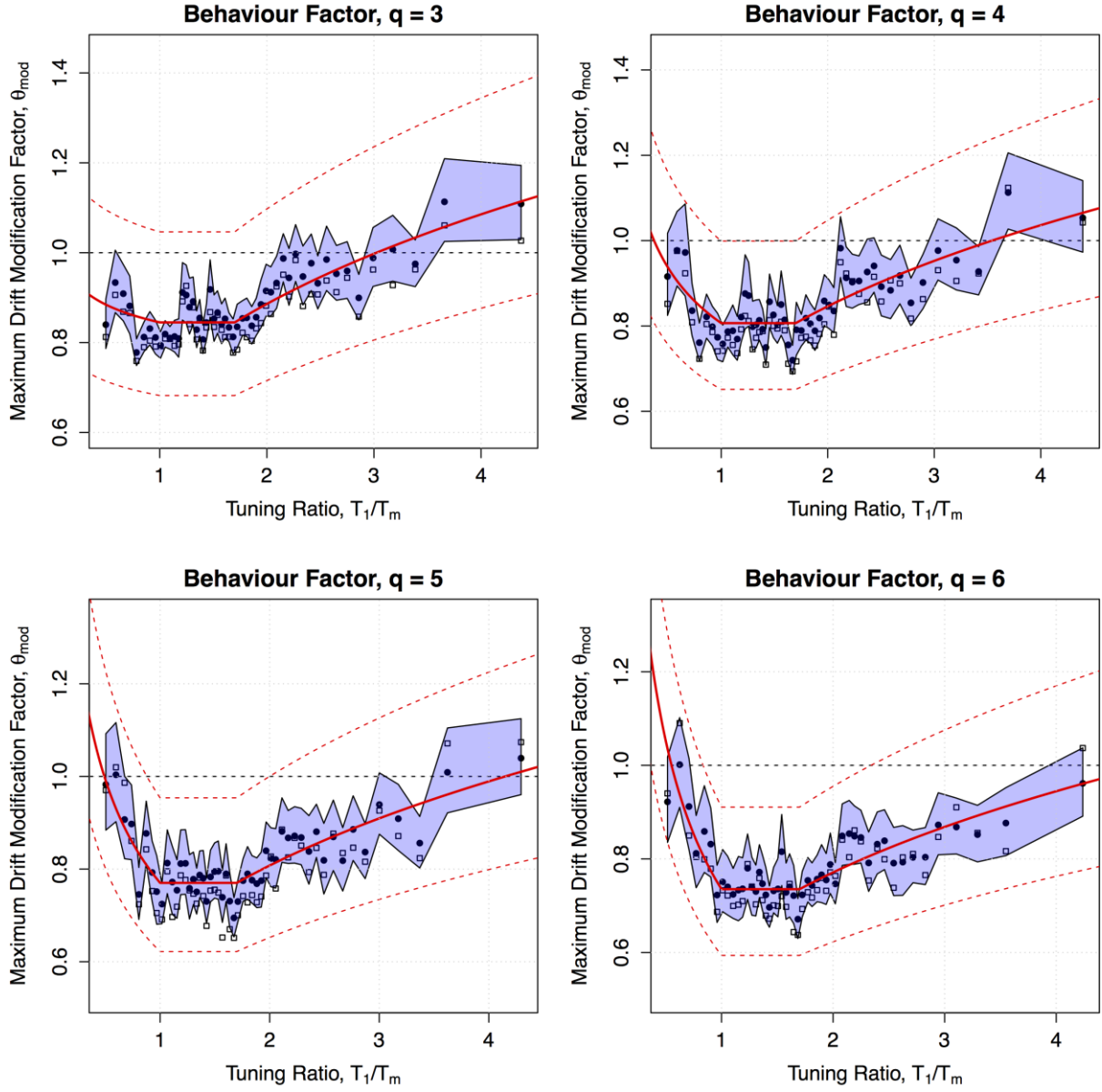


Figure 7: Mean maximum drift modification factor (θ_{mod}) for various period ratios and behaviour factor bins.

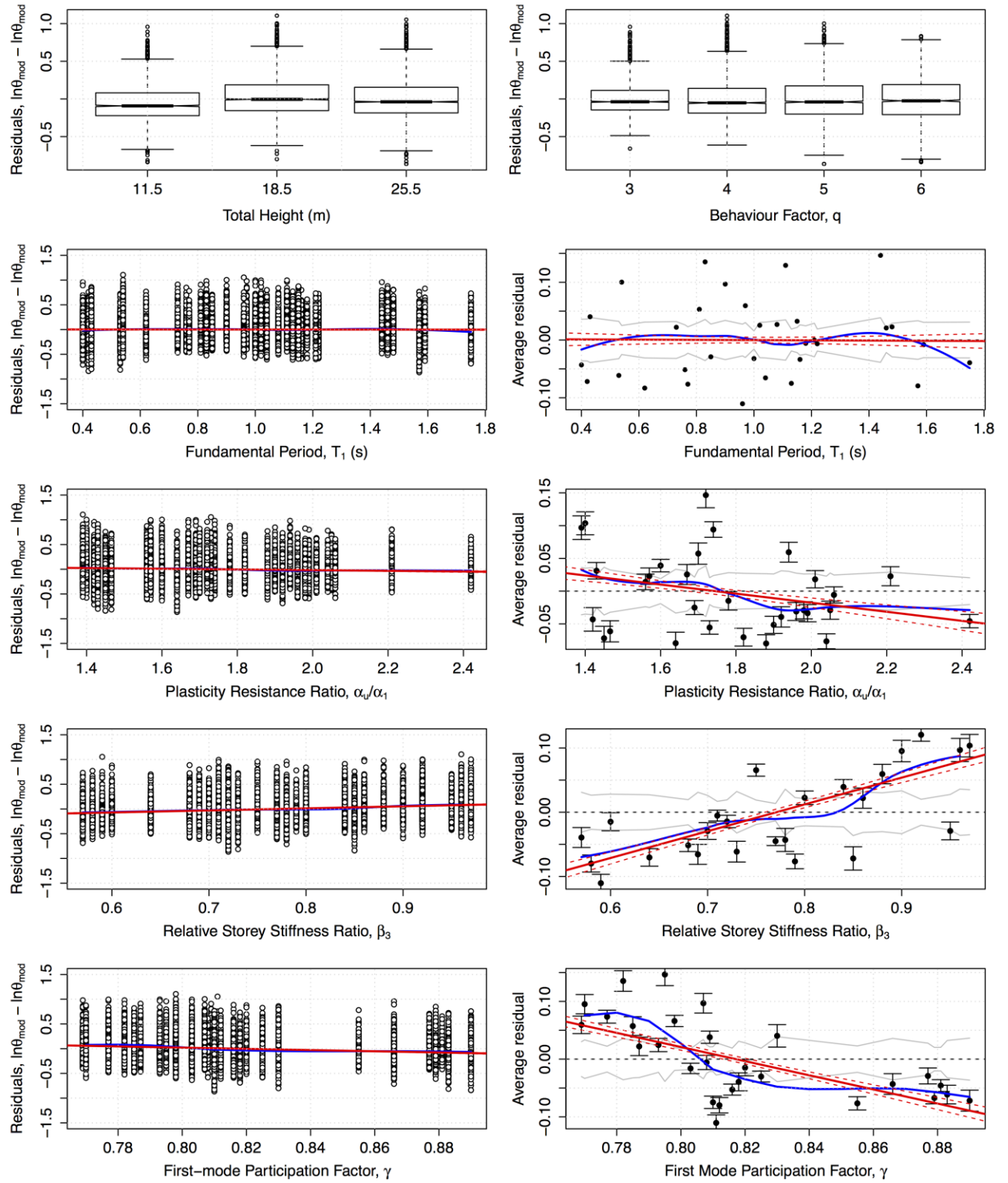


Figure 8: Residuals of the model for θ_{mod} shown in Figure 7 plotted against various structural characteristics.

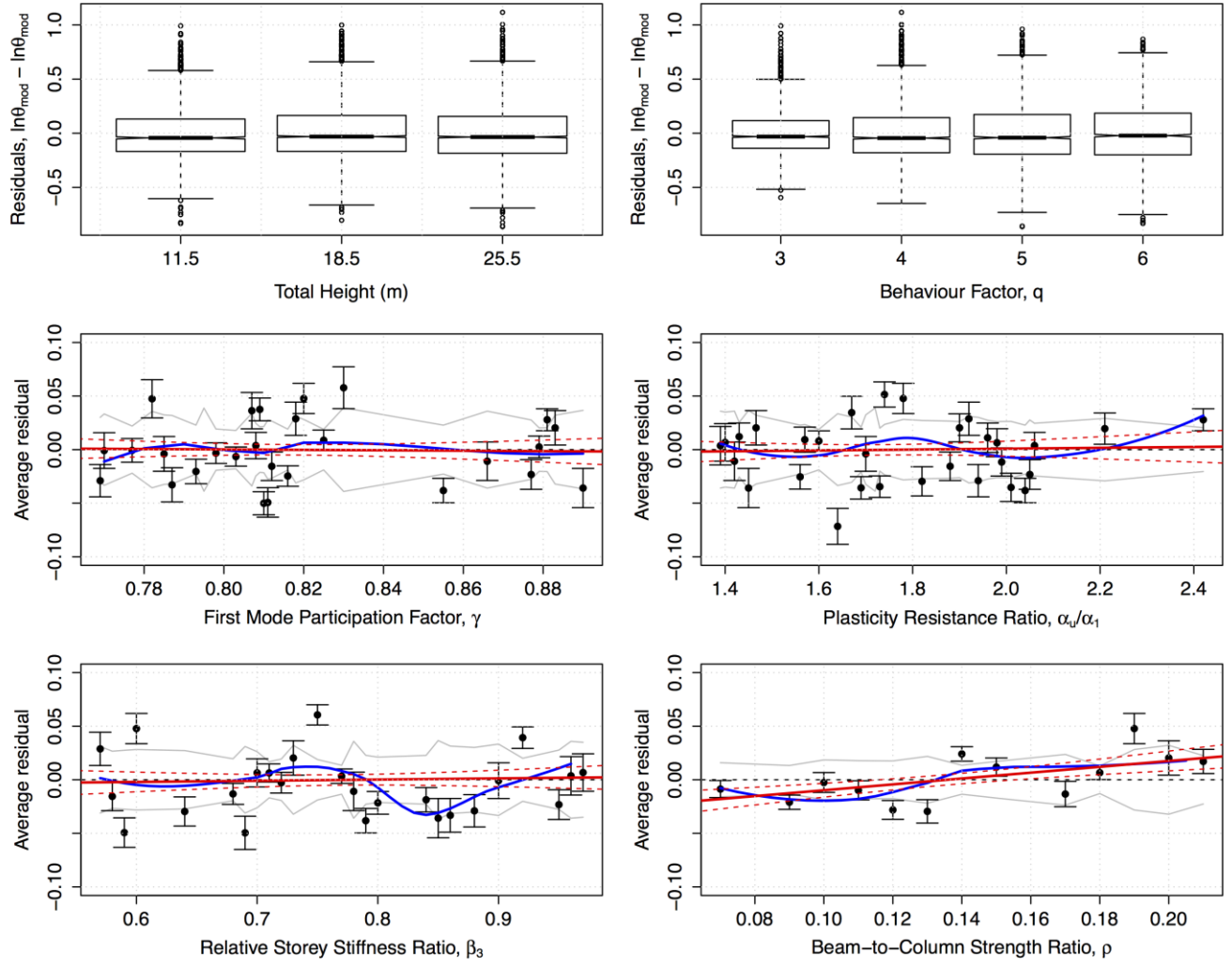


Figure 9: Residual distributions of the maximum drift modification factor (θ_{mod}) for various structural characteristics.

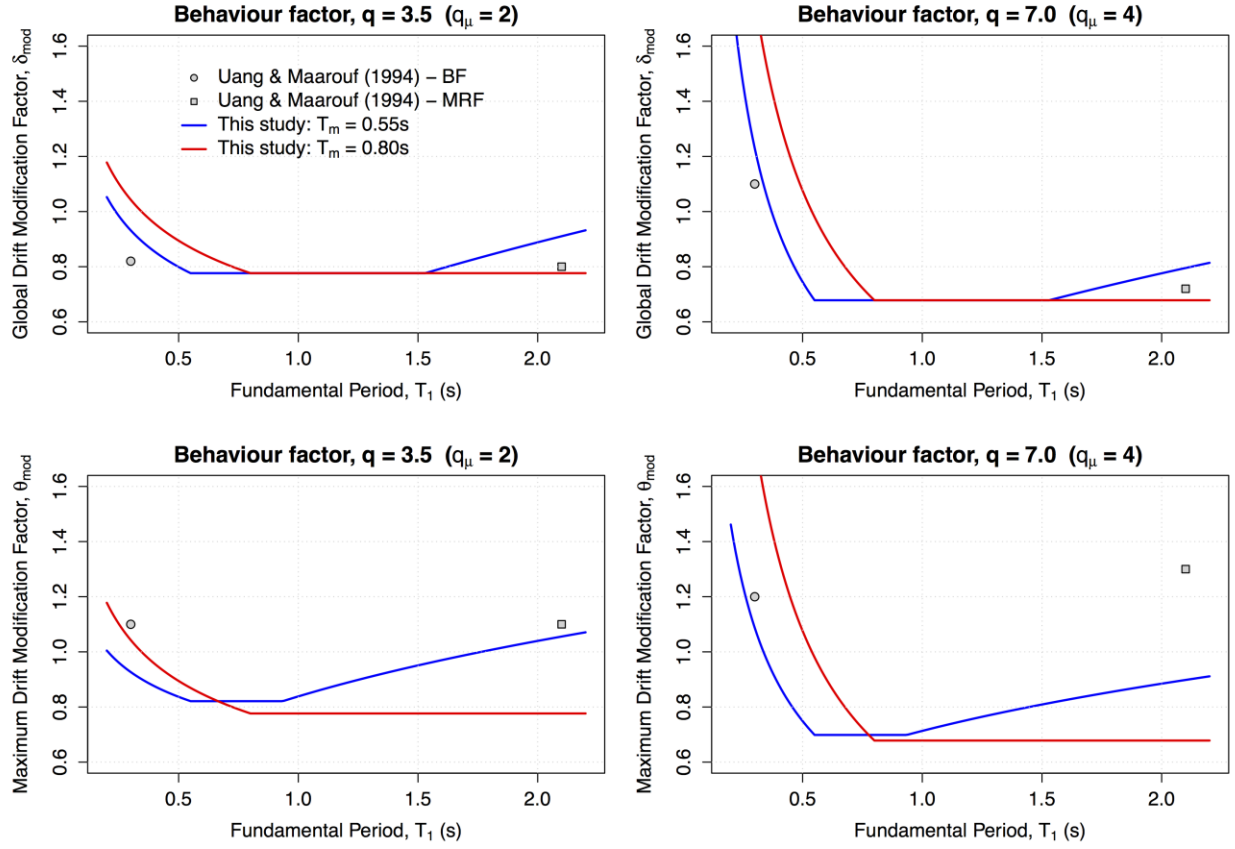


Figure 10: Comparison of δ_{mod} (upper panels) and θ_{mod} (lower panels) predictions using the models developed in this study with the results of Uang and Maarouf (1994) for q_μ of 2 (left) and 4 (right).

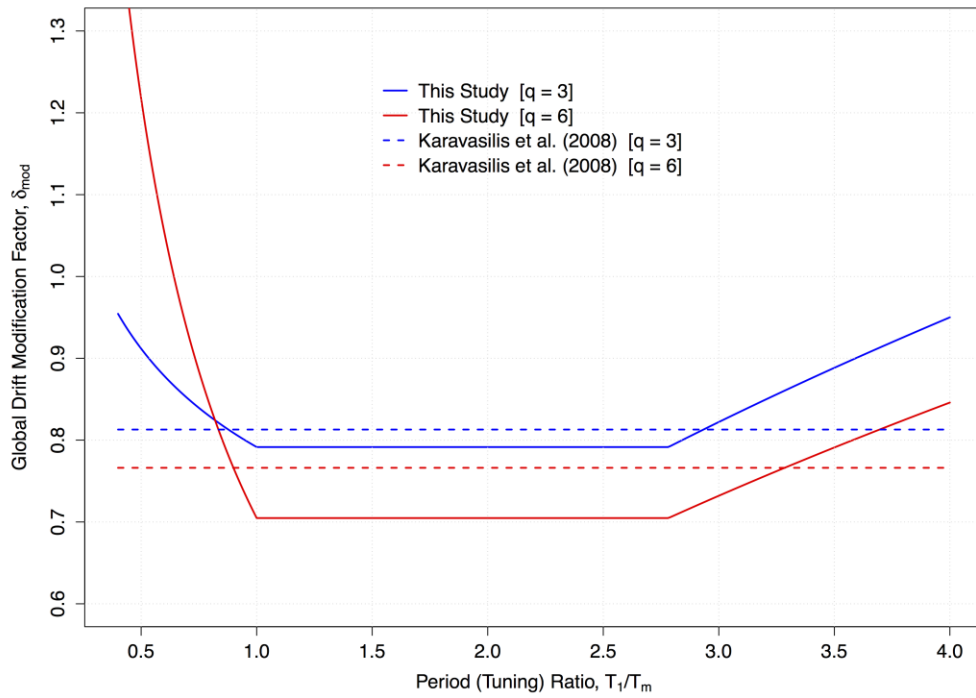


Figure 11: Comparison of δ_{mod} predictions using the model developed in study and that of Karavasilis et al. (2008) for q of 3 and 6.

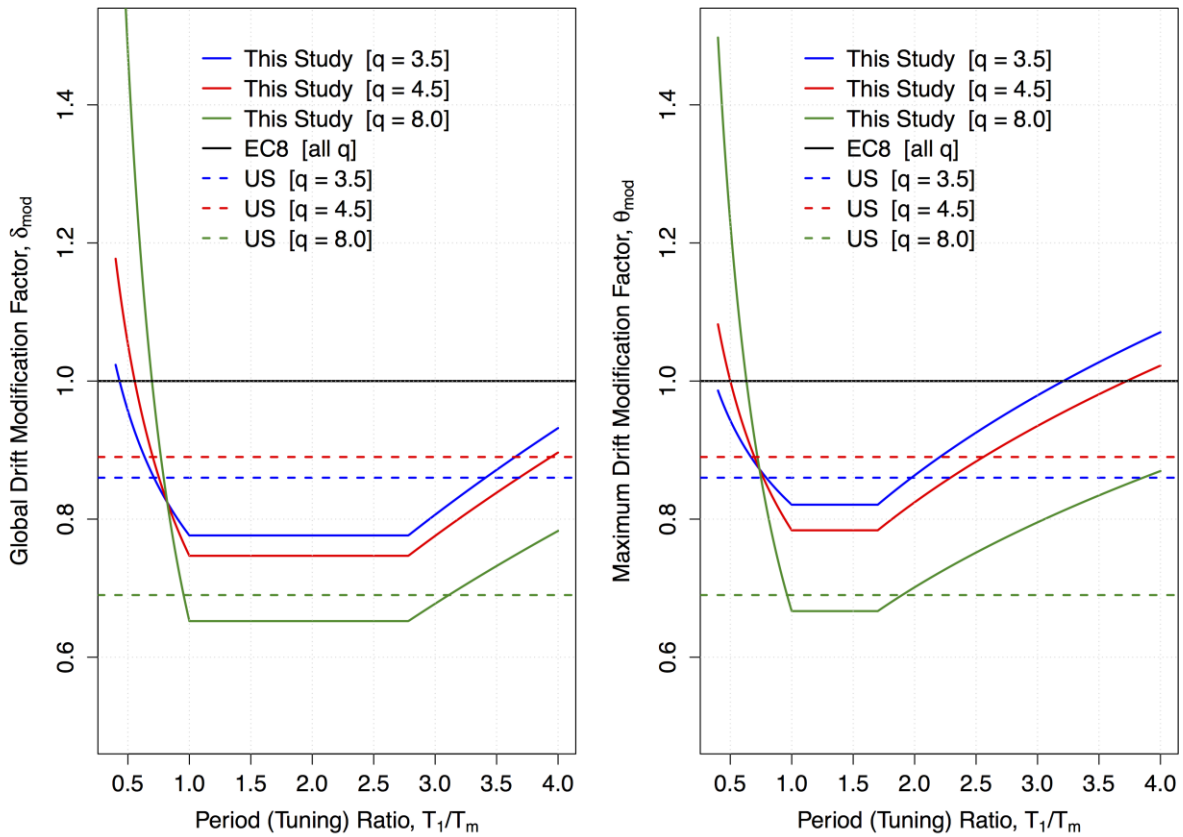


Figure 12: Comparison of the model estimates for global drift modification factor (δ_{mod}) and the maximum drift modification factor (θ_{mod}) with EC8 and US provisions