Assessment of Strength Demands in Moment Resisting Frames

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SUMMARY:
This paper investigates the influence of key structural parameters, the level of inelasticity and the frequency content of ground motions on the base-shear demands of multi-storey frames. To represent a wide range of structural characteristics, 39 steel moment-resisting frames are designed using the provisions of Eurocode 8. Suitable frequency content parameters are considered for representing the characteristics of ground motions. To this end, 72 far-field records are identified that represent a wide range of frequency contents. Extensive incremental dynamic analysis (IDA) is employed to simulate four selected levels of inelasticity by scaling the ground-motion records. The results of the parametric investigations demonstrate that the influence of frequency content on the base shear can be captured using suitable parameters as a function of the behaviour factor as well as the relative stiffness and strength characteristics between various storeys within the frames. Finally, the implications of the findings for codified design procedures, with particular reference to European seismic provisions incorporated in Eurocode 8, are discussed.

Keywords: Eurocode, Moment Resisting Frames, Frequency Content, Seismic Design, Strength Demands

1. INTRODUCTION

In order to ensure that a structure obeys capacity design rules and remains stable under seismic excitation, it is imperative that the strength demands imposed on columns do not exceed their capacity. Therefore, in accordance with the capacity design philosophy, columns are designed to be stronger than beams. Conventionally, moments, shear and axial forces for beams are amplified to account for material and structural over-strength in order to obtain design forces for columns. However, various studies (e.g. Pettinga and Priestley, 2005; Medina and Krawinkler, 2005) on the topic have shown that the strength demands on the frame are also amplified significantly due to higher mode effects.

Medina and Krawinkler (2005) studied the influence of various parameters on the amplification of strength demands due to higher modes using generic moment-resisting frames. The amplification of strength demands was found to be mainly dependent on the fundamental period of the frames, the relative intensity (equivalent to the behaviour factor, q, in EC8) and the frequency content of ground motions. Pettinga and Priestley (2005) reported dynamic amplification of base shear for RC frames designed with direct displacement based design using spectrum-compatible ground motions. Furthermore, they recommended a Modified Modal Superposition (MMS) (an improved version of the conventional modal response spectrum analysis method) that accounts for ductility demand in addition to the contribution of shear and moment demands. Subsequently, Priestley et al., (2007) proposed simpler equations that only account for the ductility demand to compute base shear and moment magnification demands. The amplification of moments in columns was recommended from the first storey to a point at 3/4 of the structure's height, as column hinging is allowed in the top storeys of the structure. More recently, Sullivan et al. (2008) suggested a revised equation to calculate base shear using the concept of transitory inelastic modes. The term 'transitory inelastic modes' refers to the modal periods of structure following the formation of a plastic mechanism. Therefore, Eigenvalue analysis of a structure with plastic hinges at anticipated locations is carried out to determine the modal periods. The modal periods, thus obtained, are used subsequently with the conventional response spectrum analysis method, using a square-root-of-sum-of-squares (SRSS) combination rule, in place of elastic response periods (but still using the elastic spectrum).

Based on the literature discussed above, it can be deduced that the studies conducted so far have incorporated the influence of the frequency content of ground motions using variants of the response spectrum analysis method (Pettinga and Priestley, 2005; Sullivan et al., 2008). In other cases, the influence of the frequency content, and other structural properties (fundamental period, for instance) are overlooked in order to develop simpler models based on ductility demands on the structure (Priestley et al., 2007). The objective of this paper is to investigate the influence of frequency content using quantitative measures of the frequency content, various structural properties and the behaviour factor (relative intensity) on the base-shear demands on the structure. Based on the study of Kumar et al., (2011) the mean period, T_m, of the ground-motion records is used to represent the frequency content. The scope of the paper is limited to the investigation of the base-shear only.

In order to investigate the influence of structural characteristics, 39 moment-resisting steel-framed structures are designed in accordance with the Eurocode requirements. The designed frames are subsequently processed to identify the structural parameters that may influence the strength demands of the frame. Incremental dynamic analysis is performed, thereafter, by scaling the records to simulate various behaviour factors using ground motion records that encompass a wide range of frequency content. The next section of the paper, discusses the assumed structural configuration, the design procedure and the modelling of the frames used in the study.

2. STUDY FRAMES AND MODELLING DETAILS

A large set of steel moment-resisting frames, that satisfy the EC3 (CEN, 2005) and EC8 (CEN, 2004) design provisions, is designed. Seismic design of steel moment-resisting frames using EC8 provisions requires the designed structure to satisfy strength, drift and stability criterion for earthquake scenarios. Using the forced-based design concept, the behaviour factor, q, is selected using three ductility classes proposed in EC8. Additionally, the structure is required to satisfy drifts for serviceability as well as the design earthquake scenario that can be implemented using following expression:

$$d_r v = \psi h \tag{1}$$

In this expression, d_r denotes the inter-storey drift; ν is a reduction factor which accounts for the smaller more-frequent earthquakes associated with serviceability, recommended as 0.4–0.5 depending on the importance class; ψ is defined as 0.5, 0.75 and 1.0% for brittle, ductile or non-interfering non-structural components, respectively, and h refers to the inter-storey height. Subsequently, second-order stability effects need to be addressed using the following expression proposed in EC8:

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \tag{2}$$

In the above equation, P_{tot} and V_{tot} are the total cumulative gravity load and seismic shear applied at the storey under consideration; h is again the inter-storey height, and d_r is the design inter-storey drift (the product of elastic inter-storey drift and 'q', using the equal displacement rule). For $\theta \leq 0.1$, second order effects may be neglected. If $0.1 \leq \theta \leq 0.2$, the multiplier $1/(1-\theta)$ needs to be used to account for this effect and, in any case the value of θ should not exceed 0.3. Finally, columns are designed according to the capacity design philosophy to resist flexural, shear and axial demands. The design moments, M_{Ed}, shear forces, V_{Ed}, and axial forces, N_{Ed}, in the columns are calculated using the following expressions:

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E} \tag{3}$$

$$V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E} \tag{4}$$

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{5}$$

In the above expressions, $M_{Ed,G}$, $V_{Ed,G}$ and $N_{Ed,G}$ refer to the design moments, shears and axial forces, respectively, for columns due to gravity loads, and $M_{Ed,E}, V_{Ed,E}$ and $N_{Ed,E}$ represent the design moments, shears and axial forces due to lateral seismic loads; γ_{ov} is the material over-strength, typically assumed to be 1.25; Ω is a beam over-strength factor determined as a minimum of $\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$ of beams, where $M_{Ed,i}$ is the design moment in beam 'i' and $M_{pl,Rd,i}$ is the corresponding plastic moment.

Figure 1 shows plan and elevation views of the structural system adopted in this study that consists of three lateral resisting moment frames, each comprised 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 is assumed to have a separate lateral resisting system. The interior moment frame selected in this study was initially designed for gravity loading according to EC3 (CEN, 2005). A dead load of 1 kN/m^2 (excluding self-weight) and an imposed load of 2 kN/m^2 were considered for the gravity design. 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), and the sections were made of steel grade S275. The same sections are used for the internal and external columns for each storey. Likewise, beam profiles are 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 in plan and elevation. Thus, the lateral load was distributed using following expression:

$$F_i = F_b \frac{s_i . m_i}{\sum s_j . m_j} \tag{6}$$

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, m_j in the fundamental mode shape.

A total of 39 frames, with 3, 5 and 7 storeys, were designed using this procedure (details of the frames can be found in Kumar (2012)). To perform nonlinear pushover and incremental dynamic analysis, the designed frames were modelled in OpenSees (2008). Hinges were allowed to form in the beams and column bases only, while the rest of the columns were modelled with elastic elements. A bilinear stress-strain curve for steel with a post-yield stiffness of 0.5% was selected to account for the material nonlinearity. Vertical loads comprised of dead loads and an allowance for 30% of the live loads was applied at the mid-spans of the beams and at beam-to-column joints. A seismic mass of 70 tons was considered at every floor of the frame and a mass of approximately 56 tons was applied at the roof level for the dynamic analyses. Initial-stiffness proportional damping was considered with 2% viscous damping assigned to the first mode.

3. EVALUATION OF STRUCTURAL CHARACTERISTICS

The database of the frames was processed to evaluate the characteristics of the frames whose influence on the strength demands was considered. The chosen characteristics of the frames, described below, were obtained using Eigenvalue analysis, pushover analysis, the frame geometry and simple structural analysis principles. It is pertinent to mention that conventional pushover analysis is implemented using a load pattern obtained from the first mode shape of the frame. The characteristics of the frame evaluated in the study are as follows:

1. First mode period, T_1, of the frame, obtained using Eigenvalue analysis. The elastic

fundamental period is chosen to study its influence on the strength demands, based on principles of dynamics. The distribution of the fundamental periods of the frames is shown in **Figure 2**, where it can be seen that these range from 0.40 to 1.75 seconds.

2. Plasticity resistance ratio, $= \alpha_u/\alpha_1$, calculated as the ratio of the base shear when the plastic mechanism has developed to the base shear at the formation of the first plastic hinge. This is evaluated from a pushover analysis of each frame. This parameter may prove to be useful, bearing in mind the influence of plasticity (typically measured in terms of relative intensity or ductility) on the strength demand. The distribution of the plasticity resistance ratio of the frames in the database is given in **Figure 2**, and ranges from 1.39 to 2.42.

3. Relative storey stiffness ratio, calculated from the first-mode drift profile obtained from Eigenvalue analysis. This parameter is introduced to account for the relative stiffness or strength of the top storeys. This may be of interest, considering that less stiff, or weaker top storeys may result in earlier yielding and increase the overall extent of plasticity in the frame. There are multiple ways to define a parameter that reflects the relative stiffness. Three options are considered here: 1) β_1: calculated as the ratio of the inter-storey drift at the top storey to the maximum inter-storey drift over the rest of the storeys; 2) β_2: calculated as the ratio of the maximum inter-storey drift in the upper 1/3rd of the frame to the maximum inter-storey drift for the lower 2/3rd of the frame; 3) β_3: calculated as the ratio of the maximum inter-storey drift for the upper half of the frame to the maximum inter-storey drift for the lower half. The distributions of β_1, β_2 and β_3 for the frames used in the study are shown in Figure 2.

4. GROUND MOTIONS AND FREQUENCY CONTENT MEASURES

To investigate the influence of the frequency content of ground motions, the mean period, T_m, is chosen based on the study of Kumar et al. (2011). The mean period, T_m, proposed by Rathje et al. (1998) and Rathje et al. (2004), is 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 Equation 7:

$$T_m = \frac{\sum_i C_i^2 \times \frac{1}{f_i}}{\sum_i C_i^2} \quad \text{for } 0.25 \text{ Hz} \leq f_i \leq 20 \text{ Hz, with } \Delta f \leq 0.05 \text{ Hz} \qquad (7)$$

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 reflect natural variations in the frequency content, 72 far-field records from 21 earthquakes that include a wide range of magnitude, distance and soil conditions (according to the NEHRP classification), are selected. **Figure 3** shows the distribution of earthquake records used in this study with respect to magnitude, distance and site class. It needs to be specified that only one horizontal component from each recording is selected. Since the study is limited to far-field ground motions, the records have been chosen to have rupture distances between 0-80 km (closest distance from fault rupture) for magnitudes between 5.5 and 6 and within the range of 20-80 km for magnitudes greater than 6.

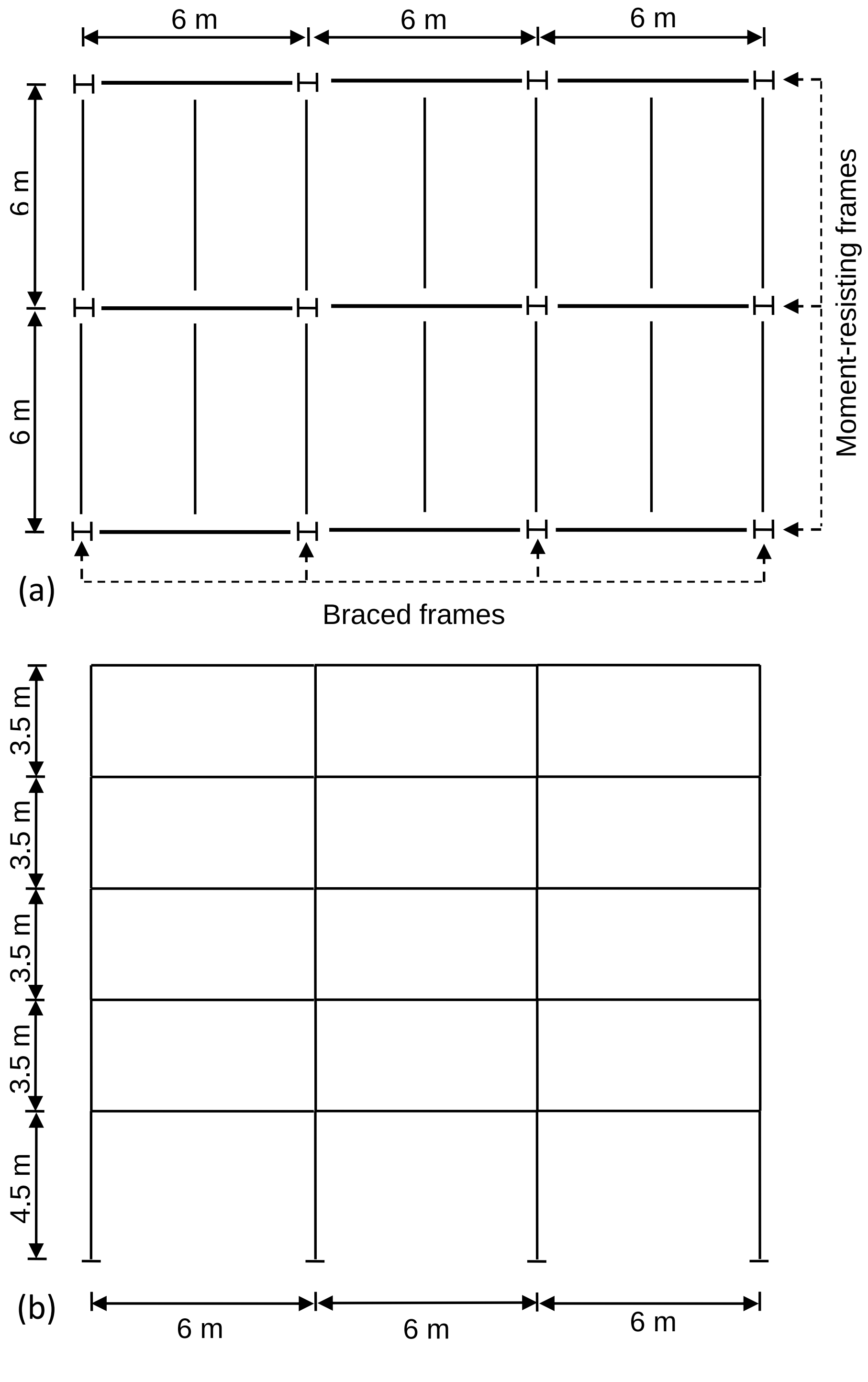

Figure 1. Moment-resisting steel frame adopted in the study: a) plan view; b) elevation view

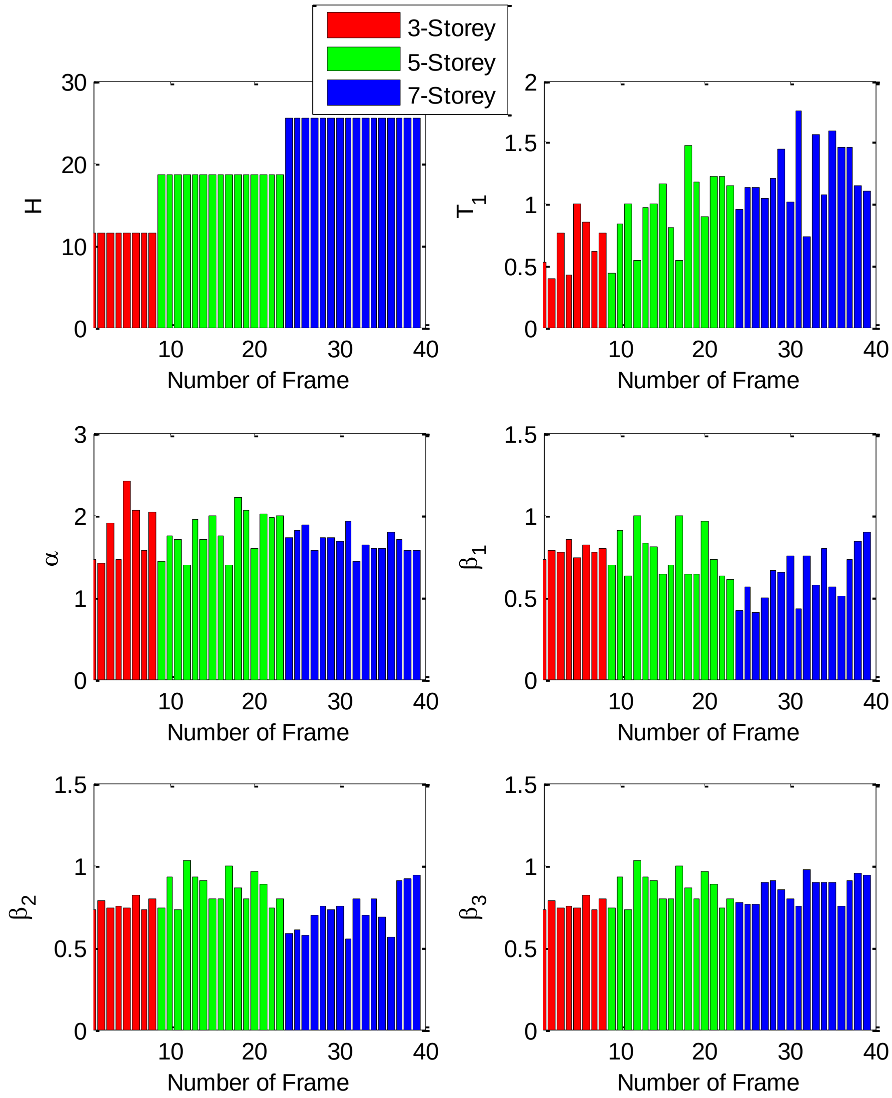

Figure 2. Distribution of structural properties of the frames used in the study

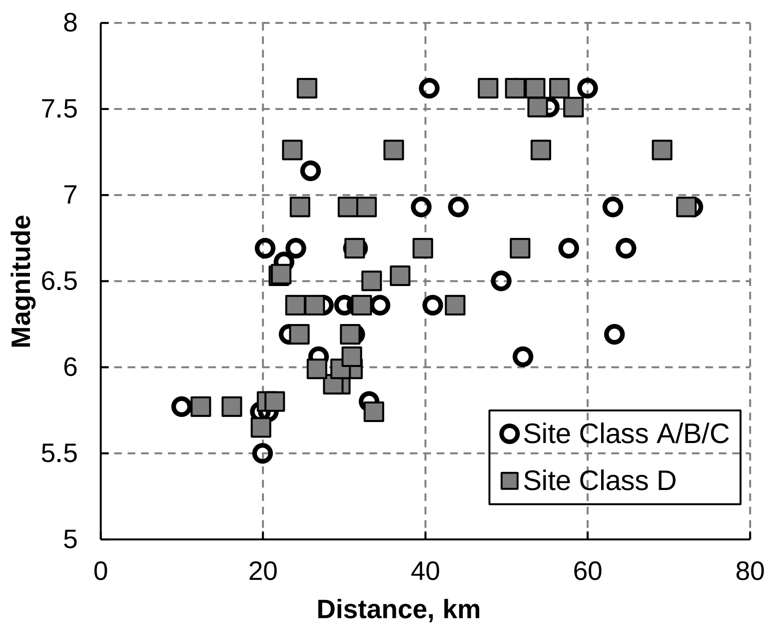

Figure 3. Distribution of magnitude, distance and site conditions for the earthquake records used in the study

5. NONLINEAR DYNAMIC ANALYSIS

Incremental dynamic analysis of the frames is conducted by scaling the records in order to attain various levels of relative intensity (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 calculated using Equation 8.

$$S_F = q \times \frac{V_1}{S_a(T_1) \times m \times \gamma} \tag{8}$$

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 the first hinge in the frame obtained from static pushover analysis using a force profile based on the fundamental mode shape of the frame; m is the seismic mass of the structure; and γ represents the mass participation ratio corresponding to the first mode.

From each dynamic analysis, the maximum base shear (V_{max}) is obtained, which is defined as the sum of the shears at all supports of the frame. On the other hand, the base shear at yield is calculated from the pushover analysis at the formation of the first hinge, and is defined as the sum of the shears at all supports of the frame in that instance.

Using the quantities obtained from the dynamic and pushover analyses, the base-shear modification factor V_{mod} can be computed as the ratio of the maximum base shear, V_{max}, recorded from IDA for a given behaviour factor to the product of the plasticity resistance ratio, α, and the base shear at yield, V_1. This can be expressed as follows:

$$V_{mod} = \frac{V_{max}}{\alpha \times V_1} \tag{9}$$

6. PARAMETRIC ASSESSMENT

In this section, the results obtained from the incremental dynamic analyses are presented in order to examine the influence of various parameters on the base-shear modification factors.

The influence of the period ratio (T_1/T_m) and q on V_{mod} is studied first by grouping the data into various T_1/T_m bins for a particular q. Subsequently, the mean value of the factor is evaluated for the respective bin. Mean V_{mod} values for various T_1/T_m bins are compared for behaviour factors of 3, 4, 5 and 6, as shown in Figure 4. Based on the general trends, the influence of the period ratio can be divided into three T_1/T_m ranges (short, intermediate and long). The intermediate T_1/T_m range lies roughly between a T_1/T_m ratio of 1 and 1.7 for a behaviour factor of 3; however, the extent of this zone reduces as the behaviour factor increases. The short and long regions lie below and above this intermediate range, respectively. It is noted that for the short T_1/T_m range, V_{mod} increases with decreasing values of T_1/T_m. This behaviour can be attributed to the increase in inelasticity of the structure as a result of the increase in ductility demands (global ductility demands) due to the 'short-period effect', as discussed in Kumar *et al.* (2012). The 'short-period effect' refers to the increase in the global ductility demands, for T_1/T_m less than unity, as the elongated fundamental period of the structure is closer to the T_m of the ground motion. In the intermediate T_1/T_m range, the influence of the period ratio on V_{mod} is negligible. In the long T_1/T_m range, V_{mod} increases as T_1/T_m increases due to the influence of higher-mode effects. Furthermore, it is observed that an increase in q results in the increase of V_{mod} for all values of T_1/T_m.

The influence of α can be assessed by dividing the data into various T_1/T_m bins for a certain behaviour factor, and performing the comparison by further subdividing the data with respect to plasticity resistance ratios higher and lower than 1.74. Here, it should be noted that the seemingly arbitrary value of 1.74 is the average plasticity resistance ratio over all the frames used in the study. The mean values of V_{mod}, for q of 3 and 5, are presented in Figure 5. It can be observed that the increase in α results in a decrease in V_{mod} and vice versa. Considering that the high plasticity resistance ratio for a given frame means lower overall inelasticity in the frame, the high α corresponds to lower V_{mod}.

The influence of the relative storey stiffness ratio using three definitions (β_1, β_2 and β_3) is then examined by dividing the data using T_1/T_m bins for four behaviour factors and further sub-dividing the data in these bins into two groups using average values of β_1, β_2 and β_3 for all the frames in the study (found to be 0.71, 0.78 and 0.84 respectively). Hence, the data is further divided into groups corresponding to β_1 higher and lower than 0.71, β_2 higher and lower than 0.78 and β_3 higher and lower than 0.84. Mean values of V_{mod} are firstly compared for β_1 higher and lower than 0.71 in Figure 6 for q of 3 and 5. It can be noted that V_{mod} increases with an increase of the relative storey stiffness ratio (expressed as β_1). In other words, it may be inferred that a higher relative storey stiffness ratio (softer top storey in relation to bottom storeys) results in earlier yielding of the top storeys, which consequently increases the overall inelasticity in the frame, and results in a higher V_{mod}. Similarly, Figure 7 and Figure 8 compare mean values of V_{mod} for β_2 higher and lower than 0.78 and β_3 higher and lower than 0.84, respectively, again for q of 3 and 5. It can be noted that the trends are similar to those observed for β_1; an increase in β_2 or β_3 results in the increase of V_{mod}. Furthermore, it can be noted that V_{mod} is more sensitive to β_3 than β_1 or β_2.

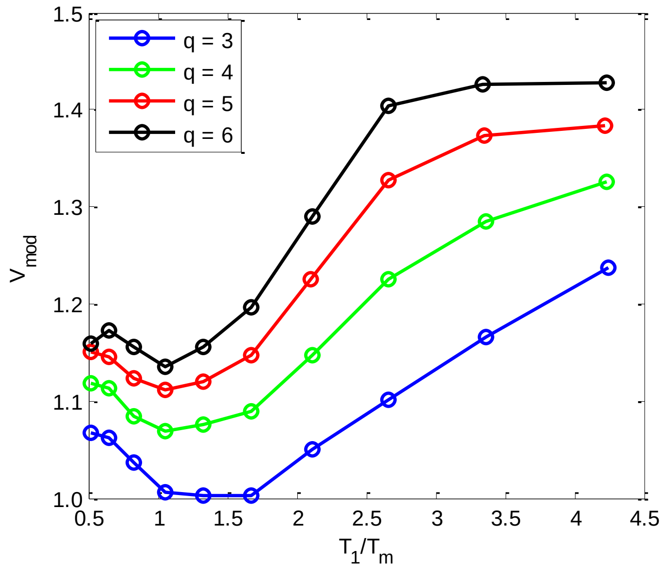

Figure 4. Comparison of the mean base shear modification factor (V_{mod}) for various period ratios for behaviour factors of 3, 4, 5 and 6

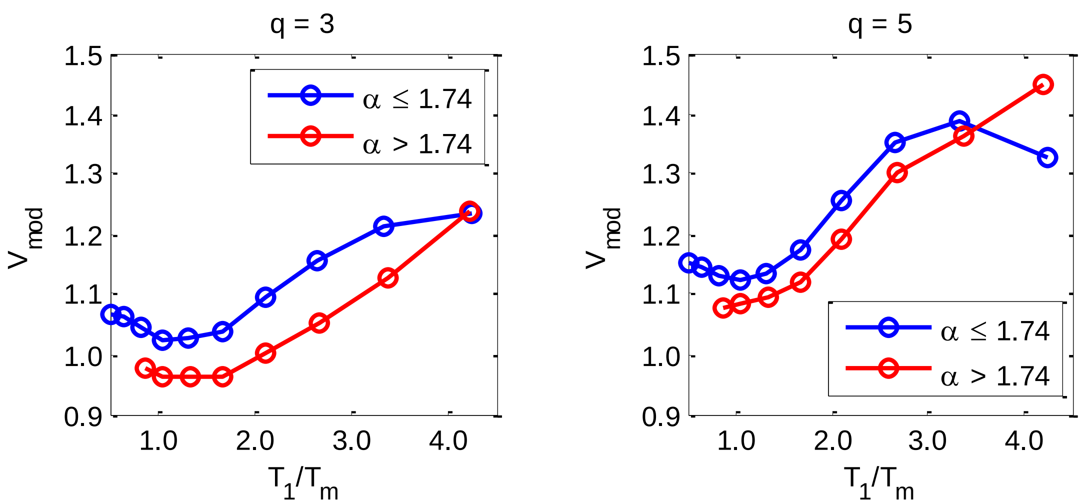

Figure 5. Comparison of mean base shear modification factor (V_{mod}) for various period ratios with $\alpha \leq 1.74$ and $\alpha > 1.74$

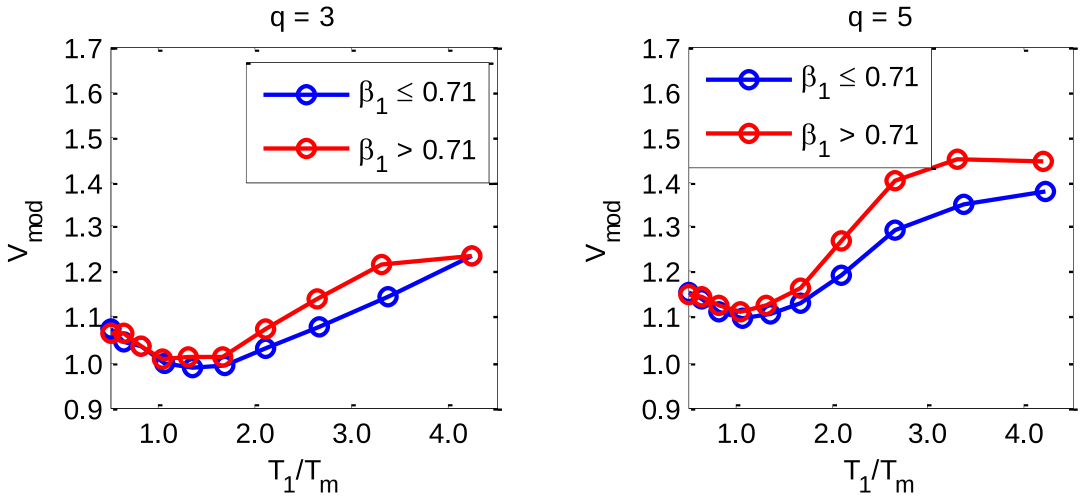

Figure 6. Comparison of mean base shear modification factor (V_{mod}) for various period ratios with $\beta_1 \leq 0.71$ and $\beta_1 > 0.71$

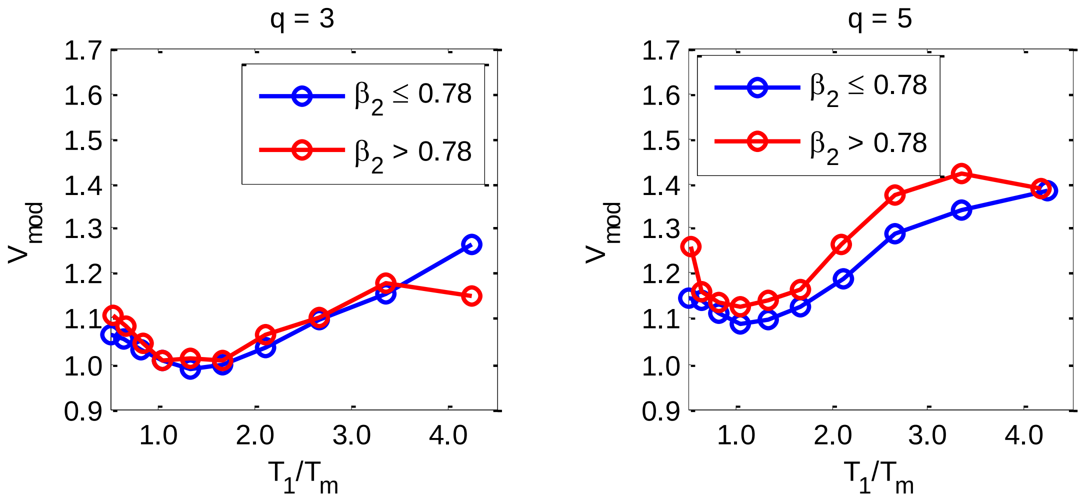

Figure 7. Comparison of mean base shear modification factor (V_{mod}) for various period ratios with $\beta_2 \leq 0.78$ and $\beta_2 > 0.78$

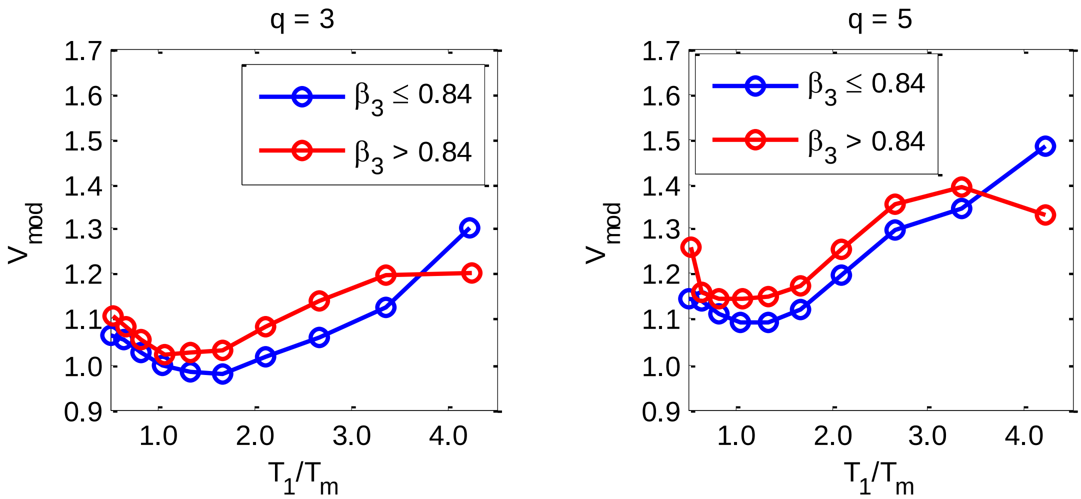

Figure 8. Comparison of mean base shear modification factor (V_{mod}) for various period ratios with $\beta_3 \leq 0.84$ and $\beta_3 > 0.84$

7. CONCLUSION

In this paper, the influence of various structural parameters, the level of inelasticity and the frequency content of ground-motion records on the base shear of steel moment-resisting frames is examined. It is concluded that the base-shear demands are a function of the period ratio, the behaviour factor, the plasticity resistance ratio and the relative storey stiffness ratio.

In the existing EC8 seismic design process, the reduced design base shear (V_d) is obtained by dividing the elastic base shear (V_e), obtained from the elastic response spectrum, with the behaviour factor (q) recommended by code provisions. In an ideal situation, V_d should be equal to V_1. However, in most cases, V_d tends to be lower than V_1 due to the large behaviour factors proposed by design codes coupled with the effect of material and design overstrength. On the other hand, for seismic assessments of moment resisting frames, conventional pushover analysis is typically recommended. Thus, for typical cases, the ultimate base shear, V_u (= V_1 x α), is assumed to be the maximum base shear imposed on the structure. However, the study presented in this paper highlights that the maximum base-shear demands are significantly higher than V_u in most cases due to the contributions of higher-mode effects that are not captured within current EC8 provisions and the pushover analysis. Consequently, the strength demands determined according to the provisions of EC8 may represent notable underestimations. Approaches for improving the predictions of strength demands to incorporate higher-mode effects should therefore be considered.

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