Inelastic Seismic Response Assessment of Moment Resisting Steel Frames

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By

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Abstract

To improve the predictability of structural and non-structural damage of structures for a given hazard scenario, it is essential to identify factors that influence the response and evaluate their contribution. Several studies have therefore focused on the assessment of parameters that influence the inelastic response of structures under seismic loading. However, these studies have in most cases been limited to single degree of freedom (SDF) systems and generic frames with controlled strength and stiffness distribution characteristics. In addition to this, the influence of the frequency content of ground motion on the inelastic response of structures has not been fully explored and utilized. Therefore, this thesis aims to understand the influence of frequency content and key properties of structure, designed to Eurocode provisions, on the inelastic response.

A suitable frequency content measure that can be related to magnitude, distance and site characteristics of an earthquake event, and easily adopted as a design input, is selected from the available literature in order to understand the influence of frequency content. The applicability of the selected parameter is first explored and established by studying the inelastic displacement demand of SDF systems as well as global drift, base shear and maximum storey drift profile of a selected multi-degree freedom (MDF) system, using a suite of 128 far-field ground motion records. Subsequently, incremental dynamic analysis of a large set of moment resisting steel frames designed to Eurocode 8 is conducted using 72 far-field ground motion records. The influence of salient structural properties on the inelastic drift and strength demands and their interaction with frequency content is investigated. Based on extensive parametric studies, regression models are developed as a function of the parameters that influence drift and strength demands of the frames.

Finally, implications of the findings on current seismic design and assessment provisions, with emphasis on the guidelines of Eurocode 8, are discussed. Furthermore, recommendations are proposed for future work that can lead to further improvements in codified procedures.
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Dedication

I would like to dedicate my PhD to my wife, Tulsan, my lovely sons, Gautam and Daksh, and my parents.
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<td>$C_d$</td>
<td>Drift amplification factor proposed in NEHRP (2003)</td>
</tr>
<tr>
<td>$C_i$</td>
<td>Fourier amplitude coefficients</td>
</tr>
<tr>
<td>$C_\mu$</td>
<td>Inelastic displacement ratio for a SDF system with known ductility demand</td>
</tr>
<tr>
<td>DCH</td>
<td>High ductility class proposed in EC8</td>
</tr>
<tr>
<td>DCL</td>
<td>Low ductility class proposed in EC8</td>
</tr>
<tr>
<td>DCM</td>
<td>Medium ductility class proposed in EC8</td>
</tr>
<tr>
<td>DDBD</td>
<td>Direct displacement based design</td>
</tr>
<tr>
<td>EDP</td>
<td>Engineering demand parameter</td>
</tr>
<tr>
<td>FAS</td>
<td>Fourier acceleration spectrum</td>
</tr>
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<td>$f_i$</td>
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</tr>
<tr>
<td>$F_i$</td>
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</tr>
<tr>
<td>$F_{\text{top}}$</td>
<td>Horizontal force for the top storey</td>
</tr>
<tr>
<td>$H$</td>
<td>Total height of MDF system</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$H_i$</td>
<td>Height of $i^{th}$ storey from the base</td>
</tr>
<tr>
<td>IDA</td>
<td>Incremental dynamic analysis</td>
</tr>
<tr>
<td>$m$</td>
<td>Total seismic mass of MDF system</td>
</tr>
<tr>
<td>MDF</td>
<td>Multi-degree of freedom</td>
</tr>
<tr>
<td>$M_{Ed}$</td>
<td>Design moments</td>
</tr>
<tr>
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<tr>
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<td>Storey moment modification factor</td>
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<td>Moment magnitude</td>
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<td>$N$</td>
<td>Number of storeys</td>
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<tr>
<td>$N_{Ed}$</td>
<td>Design axial forces</td>
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<td>PGA</td>
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<td>Behaviour factor</td>
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<td>$q_d$</td>
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<td>$q_u$</td>
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</tr>
</tbody>
</table>
\( R_c \) Closest distance to the fault rupture

\( R \) Force reduction factor

\( S_{ad}(T_c) \) Spectral acceleration of a SDF system for a given elastic period

\( S_c \) Site class

\( S_d(T_1) \) Spectral displacement of a MDF system at fundamental period of the system

SDF Single-degree of freedom

\( S_F \) Scaling factor

\( T_1 \) Fundamental period of a MDF system

\( T_{avg} \) Average spectral period

\( T_c \) Characteristic period

\( T_{ce} \) Characteristic energy period

\( T_e \) Elastic period of a SDF system

\( T_{eff} \) Effective period of a SDF system

\( T_{es} \) Predominant energy period

\( T_g \) Predominant velocity period

\( T_{lp} \) Long period

\( T_m \) Mean period

\( T_n \) Dominant interval period

\( T_o \) Smoothed spectral predominant period

\( T_p \) Predominant period

\( V \) Base shear at a given instant of the pushover analysis
$V_1$ Base shear at the formation of the first yield during the pushover analysis

$V_{b1}^o$ 1st mode base shear obtained from plastic mechanism analysis

$V_{bTIM2}$ Base shears for 2nd transitory inelastic modes

$V_{bTIM3}$ Base shears for 3rd transitory inelastic modes

$V_{bTIMS}$ Base shear obtained with transitory inelastic modal superposition approach

$V_d$ Design base shear

$V_e$ Elastic base shear obtained from response spectrum

$V_{eff}$ Effective base shear

$V_{Ed}$ Design shear forces

$V_{Ed,E}$ Design earthquake shear forces

$V_{Ed,G}$ Design gravity shear forces

$V_p$ Base shear at the formation of the significant yield during the pushover analysis proposed in NEHRP (2003)

$V_Y$ Ultimate base shear obtained from the pushover analysis

$V_{max}$ Maximum base shear obtained from the time history analysis

$V_{i,1}$ Maximum total shear at the i\textsuperscript{th} storey at the formation of the first yield of the frame obtained pushover analysis

$V_{i,max}$ Maximum total shear at the i\textsuperscript{th} storey of the frame obtained from the time history analysis

$V_{mod}$ Base shear modification factor

$V_{st,mod}$ Storey shear modification factor
W  Total seismic weight of MDF system

**Greek Letters**

$\alpha$  Plasticity resistance ratio

$\alpha_{avg}$  Average plasticity resistance ratio

$\alpha_s$  Strain hardening of a SDF system

$\beta_1$  Ratio of the inter-storey drift at the top storey to the maximum drift of the rest of the storeys

$\beta_2$  Ratio of the maximum drift for the upper $1/3^{rd}$ of a given frame to the maximum drift for the lower $2/3^{rd}$ of the frame

$\beta_3$  Ratio of the maximum drift for the upper half of a given frame to the maximum drift for the lower half of the frame

$\beta_d$  Design relative storey stiffness ratio

$\Delta_{1,\text{roof}}$  Roof displacement at the formation of first yield obtained from the pushover analysis

$\Delta_e$  Maximum elastic displacement of SDF system

$\Delta_m$  Maximum inelastic displacement of SDF system

$\Delta f$  Frequency interval

$\Delta_{\text{roof}}$  Maximum roof displacement obtained from time history analysis

$\delta_{\text{mod}}$  Global drift modification factor

$\delta_1$  Corresponding displacement of SDF system at the first yield of MDF system
δy  Yield displacement of SDF system

Γ  Transformation factor for conversion of MDF system to equivalent SDF system

γ  First mode participation factor

γov  Material overstrength

χ  Relative storey drift ratio

λ  Correction factor to account for first mode mass participation proposed in EC8

μ  Ductility demand

μ_{EDR}  Ductility demand, assuming the equal displacement rule

ν  Reduction factor proposed in EC8 to represent smaller/frequent earthquake events

Ω  Beam overstrength factor

Ω_{d}  Design structural overstrength

ψ  Allowable inter-storey drift recommended in EC8

ρ  Beam-to-column stiffness ratio at mid-height of the frame

θ  Stability coefficient proposed in EC8

θ_{1,max}  Maximum inter-storey drift of the frame at the formation of the first yield obtained from pushover analysis

θ_{max}  Maximum inter-storey drift of the frame obtained from time history analysis

θ_{mod}  Maximum drift modification factor

θ_{r}  Global drift
$\omega_v$  Shear magnification factor for response spectrum analysis

$\omega_m$  Moment magnification factor for response spectrum analysis

$\xi_{eq}$  Equivalent damping

$\xi_v$  Viscous damping
Chapter 1

Introduction

1.1. Preamble

Due to severe economic losses incurred in recent earthquakes, particularly the Northridge earthquake in 1994 and the Kobe earthquake in 1995, performance oriented approaches have gained significant attention. The aim of these approaches is to reach an optimum design solution to satisfy pre-defined levels of damage or to provide relatively accurate estimates of damage for existing structural systems, for a given seismic hazard scenario. In order to achieve the main objectives of performance oriented approaches, it is vital to understand the detailed behaviour of structural systems under seismic excitations.

Significant amount of research has therefore been devoted to studying the parameters that affect the inelastic response of structures. Typically, this is examined either by idealizing the Multi-degree of freedom (MDF) system as an equivalent Single degree of freedom (SDF) system or directly studying the inelastic response of MDF systems. Idealization to SDF systems offers a simplified way to study the inelastic response, which may be useful for some studies. However, faithful representation of MDF systems is necessary to capture the influence of various structural characteristics, namely: fundamental period, strength and stiffness distribution, higher mode and P-delta effects. The next two sections aim to introduce the main research developments in the estimation of inelastic response of SDF and MDF systems, and outline the needs for further research.
1.2. Single Degree of Freedom Systems

Investigations on SDF systems subjected to seismic excitations have provided better understanding of the influence of structural parameters such as: fundamental period, ductility/force reduction ratio on the inelastic response (Miranda, 2000; Chopra and Chintanapakdee, 2004; Dwairi et al., 2007). However, it has been concluded that the inelastic response of SDF systems is independent of the earthquake magnitude, distance and site classes for far-field ground motions (Miranda, 2000; Chopra and Chintanapakdee, 2004). As a consequence, relationships for estimation of the inelastic response of SDF systems are typically based on the average results obtained from nonlinear analysis of a large number of earthquake records, which may lead to significant error when these expressions are applied to individual records, as noted by Dwairi et al., (2007). Conversely, recent developments in the area of selection and modification of records for engineering applications have shown that the number of records required to reliably predict the inelastic response of structural systems can be minimized by selecting records whose spectral shapes are most consistent with hazard specific earthquake scenarios (e.g. Baker and Cornell, 2006; Luco and Bazzurro, 2007; Hancock et al., 2008). This development warrants a re-appraisal of the inelastic response of SDF under the influence of ground motions whose spectral shapes represent different frequency-content scenarios. Despite the fact that a spectral shape provides comprehensive information on the frequency content of ground motion, a single parameter that provides a sound indication of frequency content and holds a relationship to parameters that influence the shape of spectrum (magnitude, distance and site characteristics) can prove to be more effective, particularly for the development of simple design models.

Therefore, there exists a need to identify a suitable frequency content indicator, selected based on the literature review, and to study the influence of this indicator on the inelastic response of SDF systems. It is also necessary to explore the significance of such an indicator for the improvement of inelastic response prediction.
1.3. Multi Degree of Freedom Systems

Inelastic response studies of MDF systems encompass a wide range of issues that influence drift and strength demands (generally referred to as engineering demand parameters (EDP)). Recent developments in estimating drift and strength demands, as well as needs for further research, are discussed below:

1.3.1. Drift Demands

Numerous studies have been devoted to the identification of parameters that influence drift demands imposed on structures under seismic excitations (e.g.: Uang and Maarouf, 1994; Medina and Krawinkler, 2004; Medina and Krawinkler 2005; Karavasilis et al.; 2008). However, it is noted that there exists a lack of consensus over the parameters that influence the global and maximum drifts of structures. For example, Medina and Krawinkler (2005), using generic frames, concluded that the maximum storey drift is dependent on the fundamental period and number of storeys of the frame. On the other hand, Karavasilis et al. (2008) found that the maximum storey drift for a given frame is dependent on the number of storeys, beam-to-column stiffness ratio, the average plastic moment capacity ratio between the bottom storey column and the average of the plastic moments of resistance of the beams of all storeys of the frame, and is independent of the fundamental period of the structure. Similarly, inconsistencies can be identified amongst different seismic design codes. EC8 (CEN, 2004), for example, prescribes the equal-displacement approach (i.e. inelastic drifts/displacements are equal to elastic counterparts) to evaluate global and maximum drift. On the other hand, NEHRP (2003) recommends displacement modification factors that are dependent on the behaviour factor of the structural system. Furthermore, it is also observed that the influence of frequency content is not fully incorporated in the models for estimation of global and maximum drifts.

A detailed investigation is therefore required to fully understand the influence of structural parameters on drift demands. Furthermore, there is a need to incorporate the influence of frequency content more comprehensively within assessment and design procedures.
1.3.2. Strength Demands

Various studies (e.g. Pettinga and Priestley, 2005; Medina and Krawinkler 2005) have shown that strength demands on frames amplify significantly due to higher mode effects, when the structure undergoes inelastic behaviour. Furthermore, it has been identified that strength demands are a function of frequency content in addition to other parameters such as level of inelasticity and contribution from different modes of vibration. However, it is noted that the studies conducted so far have incorporated the influence of frequency content of ground motion using various versions of response spectrum analysis method (Pettinga and Priestley, 2005; Sullivan et al., 2008). In other cases, the influence of 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).

In the light of the above discussion, there is a need for models that incorporate the influence of frequency content directly using a suitable frequency content indicator, instead of using response spectrum analysis. Furthermore, the influence of various other structural properties on the strength demands of a structure also needs to be investigated.

1.4. Objectives and Scope

The main purpose of this study is to develop an improved understanding of the influence of frequency content and structural parameters, and their interaction, on the inelastic response of MDF systems, and propose models for improved prediction of drift and strength demands. It is important here to define the MDF systems considered, which may otherwise include a broad range of structures. Moment resisting steel frames designed to satisfy Eurocode provisions are herein adopted to represent MDF systems. Moreover, the study is confined to medium rise frames with 3, 5 and 7 storeys. To this end, 40 steel moment resisting frames (MRFs) are designed. On the other hand, the study of frequency content is restricted to far-field ground motion only.

As a part of this research, the mean period, $T_m$, has been identified as a suitable frequency content indicator. Subsequently, the influence of $T_m$ on the inelastic
displacement ratios for SDF systems and global drift, base shear and maximum storey drift profile of a steel moment resisting frame (MRF) is explored. The study is then extended to include 40 steel MRFs, to examine the influence of structural characteristics and frequency content on global drift, maximum inter-storey drift (referred hereafter as maximum drift), base shear, maximum storey shear and maximum storey moment demands on the frames. Based on extensive parametric studies, factors that influence these demands are identified and subsequently processed to perform regression equations. The results are thereafter compared with existing design provisions, with special emphasis on Eurocode provisions. Finally, modifications are suggested, and discussed, for improved design of steel moment resisting frames.

1.5. Thesis Outline

Chapter 2 of the thesis presents a detailed literature review pertaining to frequency content indicators, as well as inelastic response of SDF and MDF systems. Based on the literature review, a suitable frequency content indicator is selected. Moreover, various seismic provisions relevant to each topic are discussed.

In order to study the influence of structural properties on the inelastic response of structures, a set of frames with a practical range of structural characteristics is required. Chapter 3 discusses the structural configuration and design details of selected moment resisting steel frames designed to Eurocode provisions. Furthermore, the distribution of various structural properties of the designed frames, calculated using the geometry of frames, fundamental principles of mechanics, modal analysis and nonlinear pushover analysis, is presented.

Chapter 4 discusses the influence of frequency content on the inelastic response of SDF and MDF systems subjected to a suite of 128 far-field ground motions. To this end, inelastic displacement ratios of SDF systems with elastic period (T_e) ranging from 0.1 s to 1.0 s are computed for target ductility levels of 2, 4 and 6. On the other hand, a 5-storey moment resisting steel frame is selected from the database of the frames designed to Eurocode 8 (EC8) provisions to represent a typical MDF system. Subsequently, incremental dynamic analysis (IDA) of the frame is conducted for six
levels of ductilities from 1 to 6 to study the influence of frequency content on roof displacement, base shear and maximum drift profile.

The Influence of structural parameters, namely: height, fundamental period of vibration, plastic resistance ratio, beam to column stiffness ratio and relative storey stiffness ratio, and frequency content on global and maximum drift demands is discussed in Chapter 5. To this end, 40 MRFs, designed to satisfy EC8 provisions, are subjected to incremental dynamic analysis by scaling 72 far-field records for four levels of relative intensities (in accordance with the behaviour factor, q, in EC8) of 3, 4, 5 and 6. Based on the parametric studies, parameters that influence the drift demands are identified and used to develop regression models.

The same approach is adopted to study the influence of structural properties and frequency content on base shear, storey shear and storey moment demands of the frames. Chapter 6 discusses the results of the parametric studies carried out to identify the parameters that influence these strength demands, and presents the regression models for their predictions.

Based on the studies conducted on the influence of structural properties on drift demands, presented in Chapter 5, it is identified that the relative storey stiffness (storey stiffness of upper half of frames in relation to lower half) plays a key role in the maximum drift demand exhibited by the frames. In Chapter 7, this parameter is investigated purely from a design perspective. The aim of the study presented in this chapter is to evaluate the design value of this parameter, which would result in a more uniform distribution of storey drift demands within the frames.

In Chapter 8, the prediction relationships for the drift and the strength demands, developed in Chapter 5 and 6, are compared with the previous research studies and the codified provisions, which include the European and the US seismic assessment and design provisions.

Chapter 9 provides a summary of the main conclusions obtained from this research and suggests possible areas for future research work.
Chapter 2

Literature Review

2.1. Background

The literature relevant to various developments in the estimation of inelastic response is reviewed and discussed in this chapter. The review is divided into three segments: the first part provides a detailed discussion of frequency content measures of seismic excitations; the second part outlines developments in the estimation of inelastic response of SDF systems, and the last part focuses on work related to the estimation of drift, ductility and strength demands of MDF systems.

2.2. Frequency Content Measures

Recent developments in the area of selection and matching of records for engineering applications have shown that the number of records required to satisfactorily predict the inelastic response of structural systems can be minimized by matching with spectral shapes for hazard specific earthquake scenarios (Hancock et al., 2008). In other words, the inelastic response of structures is dependent on the shape of the spectrum, which in turn is dependent on the frequency content of ground motion. However, a single parameter that provides a sound indication of frequency content and holds a relationship to parameters that influence the shape of the spectrum (i.e. magnitude, distance and site characteristics) can prove to be more effective, particularly to develop simple design models.
A review of existing frequency indicators is therefore carried out. Nine frequency content indicators are identified from the literature search. The review of these indicators can be divided in two parts: the first consists of five indicators that have been employed in the past to study structural response; the second part introduces the more recently proposed indicators that are available, which are not yet adopted for response studies of SDF or MDF systems.

Conventionally, five indicators have been employed to study structural response: i) Characteristic energy period (Tce), ii) predominant velocity period (Tg), iii) characteristic period (Tc), iv) predominant energy period (Te) and v) long-period (Tlp). Tce is defined as the period at the intersection of two straight lines representing an idealized bilinear energy response spectrum (Akiyama, 1980), and was adopted by Shimazaki and Sozen (1984) to examine the inelastic response of a structure from an energy perspective. The authors proposed an expression to estimate the inelastic response based on the ratio of the Tce of ground motion and the fundamental period (Te) of the structure. Miranda (1991, 1993) proposed Tg (referred to as the predominant period in his study) as the period at which the maximum input energy of a 5% damped linear elastic system is maximum throughout the whole period range. It can also be computed using the linear elastic velocity response spectrum, and defined as the period at which the maximum relative velocity occurs. The study showed that the ratio Te/Tg influences significantly the strength reduction factors of structures built on soft-soil deposits. This concept is already incorporated in FEMA-356 (FEMA, 2000) for very soft soil conditions. Furthermore, this parameter has been used extensively to study the inelastic response of SDF systems built on soft soil to explore the effect of stiffness degradation on the lateral strength demands for known ductility demands (Miranda and Garcia, 2002), and inelastic displacement demands of structures with known lateral strength and stiffness (Garcia and Miranda, 2006). Vidic et al., (1994) highlighted the influence of Tc, referred to as T1 in their study, on the inelastic spectrum. Furthermore, they noted that the parameter varies significantly for different groups of records investigated in the study. Uang and Ahmed (1994) used a set of 8 ground motions to study the influence of Tc on the deflection amplification factors of four MDF structures. Tc was calculated by idealizing the acceleration response spectrum as a bilinear curve and defined as the period at which the two straight lines intersect. Cuesta and Aschheim, (2001) compared the inelastic response
spectra estimates using pulse R-factors with those obtained from other contemporary relationships. They observed that the accuracy of the estimates is influenced by the characteristic period of ground motion and the presence of soft soil deposits. Chopra and Chintanapakdee (2001) studied the inelastic response of SDF systems in the context of the spectral regions for far-field and near-field records. They demonstrated the dependence of the inelastic response on Tc and recommended Tc values of 0.42 s and 0.79 s for far-field and near-field ground motions respectively. This parameter is also adopted in several design codes, such as EC8 (CEN, 2004), to define code spectra. Hutchinson et al. (2002) studied the correlation between the inelastic structural response of a structure and the period of the ground motion using Tes, Tlp and Tc. Tes is defined as the period corresponding to the peak of the input energy spectrum; Tlp corresponds to the dominant spectral ordinate in the long period range, whereas Tc is as defined earlier.

There are several other frequency content indicators that are available but have not been used to infer structural response. Based on the analysis of 306 ground motion records from 20 earthquakes in active plate margin regions, Rathje et al. (1998) proposed three frequency content indicators, namely mean period (Tm), predominant period (Tp), and smoothed spectral predominant period (To). Tm represents the mean of the periods of the Fourier Amplitude Spectrum (FAS) in specified frequency ranges, where the weights are assigned based on the Fourier amplitudes and calculated using the following expression:

$$T_m = \frac{\sum C_i^2 \times f_i}{\sum C_i^2}$$  \hspace{1cm} \text{for } 0.25 \text{ Hz} \leq f_i \leq 20 \text{ Hz, with } \Delta f \leq 0.05 \text{ Hz} \quad (2.1)$$

where $C_i$ is the Fourier amplitude coefficients, corresponding to frequencies, $f_i$, obtained from a discrete fast Fourier transform (FFT) frequencies between 0.25 and 20 Hz, and $\Delta f$ is the frequency interval used in the FFT computation. Tp is defined as the period corresponding to the maximum spectral acceleration calculated for a damping ratio of 5%. To utilizes the 5%-damped acceleration spectrum and averages the periods using weights depending on the relative strength of the spectrum. Only spectral ordinates greater than 1.2 times the peak ground acceleration (PGA) are considered in To, which can be evaluated using following equation:
\[ T_o = \frac{\sum_i T_{ei} \times \ln \left( \frac{S_a(T_{ei})}{PGA} \right)}{\sum_i \ln \left( \frac{S_a(T_{ei})}{PGA} \right)} \quad \text{for } T_i \text{ with } \frac{S_a(T_{ei})}{PGA} \geq 1.2, \text{ with } \Delta \log T_{ei} \leq 0.02 \quad (2.2) \]

Rathje et al. (2004) later proposed another frequency content indicator – the average spectral period \( T_{avg} \); this is similar to \( T_o \), as it uses the 5% damped acceleration spectrum, but the periods are averaged over specified frequency ranges, as shown in Equation 2.3 below:

\[ T_{avg} = \frac{\sum_i T_{ei} \times \left( \frac{S_a(T_{ei})}{PGA} \right)^2}{\sum_i \left( \frac{S_a(T_{ei})}{PGA} \right)^2} \quad \text{for } 0.05 \text{ s} \leq T_{ei} \leq 4 \text{ s}, \text{ with } \Delta T_{ei} \leq 0.05 \text{ s} \quad (2.3) \]

Based on 835 records from 44 earthquake events ranging in magnitude \( M_w \) from 4.9 to 7.6, relationships were proposed for \( T_m, T_o \) and \( T_{avg} \) that had functional terms involving earthquake magnitude, source-to-site distance, site conditions, and rupture directivity. Rathje et al. (2004) carried out a detailed study of the four proposed parameters and concluded that \( T_p \) was not recommended as an indicator due to the large uncertainty in its prediction. The authors recommended the use of \( T_m \), as this parameter is derived from the FAS that provides a direct representation of the amplitudes within an acceleration time history. Moreover, they showed that the indicator is stable and can therefore be predicted reliably. They also demonstrated that it best distinguishes the frequency content of strong ground motions along with its dependence on magnitude, distance and soil condition, as shown in Figure 2-1, plotted using the Equation 2.4 given as under:

\[ \ln(T_m) = c_1 + c_2 \cdot (M_w - 6) + c_3 \cdot R_c + c_4 \cdot S_c + c_5 \cdot S_D + c_6 \cdot \left(1 - \frac{R}{20}\right) \cdot FD \quad (2.4a) \]

for \( 5.0 \leq M_w \geq 7.25 \)

\[ \ln(T_m) = c_1 + c_2 \cdot (7.25 - 6) + c_3 \cdot R_c + c_4 \cdot S_c + c_5 \cdot S_D + c_6 \cdot \left(1 - \frac{R}{20}\right) \cdot FD \quad (2.4b) \]

for \( M_w > 7.25 \)

In the above equations, \( M_w \) is moment magnitude; \( R_c \) is the closest distance to the fault rupture plane (in km); \( S_C \) and \( S_D \) are indicator variables that designate site class;
FD is an indicator variable that designates forward directivity conditions; \( c_1, c_2, c_3, c_4, c_5, \) and \( c_6 \) are regression coefficients. Recently, Bommer et al. (2006) proposed the dominant interval period (\( T_n \)) as a measure of frequency content. This parameter effectively corresponds to a simplified version of the \( T_o \). It is computed as the arithmetic difference between the first and last periods where a spectral acceleration predefined threshold is exceeded. It should be noted that this parameter is not strictly a period and is best interpreted as a bandwidth.

In addition to the frequency content indicators outlined above, other indirect ways are also available to indicate the spectral shape of records. The epsilon (\( \varepsilon \)) value proposed by Baker and Cornell, (2005) is one of the most frequently used parameters for selection of records. It is defined as the number of standard deviations by which an observed logarithmic spectral acceleration differs from the mean logarithmic spectral acceleration of a ground-motion attenuation model. This parameter is particularly useful in the selection of accelerograms to match a given hazard scenario. However, it may not be effective in the context of estimating the response for a given ductility demand. Epsilon is able to distinguish between inelastic responses due to records scaled to a consistent value of \( S_a(T_1) \), but it is non-informative when records are scaled to obtain a particular level of demand (in this all \( S_a(T_1) \) values will differ).

![Figure 2-1: Variation of \( T_m \) with respect to magnitude (\( M_{w} \)), distance and site class (\( S_c \))](image-url)
A survey of the literature reveals that frequency content indicators are typically used to study the structural response of SDF systems and that their application to MDF systems has been very limited. It is also identified that recently proposed parameters have not been tested to study structural response of both SDF and MDF systems. Additionally, it is noted that all of the parameters, with the exception of $T_m$, are derived either from energy or response spectra. Based on the detailed study of Rathje et al. (2004), $T_m$ is selected in this study as the indicator to explore the effect of frequency content on both SDF and MDF systems due to its ability to infer differences in spectral shapes and its dependence on seismological parameters, in addition to other merits highlighted earlier. The following section presents a brief review of current procedures used to study the inelastic response of SDF and MDF systems.

### 2.3. Inelastic Response of SDF Systems

Two parallel concepts have evolved over the years to estimate inelastic deformations of a MDF system: the ‘displacement coefficient method’ and the ‘equivalent linearization’ methods. In the former approach, which is implemented in FEMA-356 (FEMA, 2000), the maximum inelastic deformation is estimated as the product of the elastic deformation of the system and various coefficients $C_0$, $C_1$, $C_2$ and $C_3$ that account for MDF to SDF transformation, inelastic displacement ratios of SDF systems, hysteretic characteristics and P-delta effects respectively. On the other hand, in the second method, which is adopted by ATC-40 (ATC, 1996), FEMA-356 (FEMA, 2000) and Eurocode 8 (CEN, 2004), the maximum inelastic deformation is obtained using an equivalent SDF system with modified stiffness and viscous damping. A detailed comparison of these two methods for estimation of inelastic demands of new and existing structures is provided in FEMA-440 (FEMA, 2005).

Veletsos and Newmark (1960) pioneered the study of the inelastic displacement ratios (Coefficient $C_1$) using elasto-plastic SDF systems subjected to simple pulses and to three earthquake ground motions. It was found that the ratio of elastic to inelastic response of a SDF system is approximately equal to unity except for systems falling in the high frequency range ($T_e < 0.5s$) for which it can be shown that the maximum potential energy stored in elastic and elasto-plastic systems is comparable.
This led to the definition of the well-known ‘equal displacement’ and ‘equal energy’ rules. These observations were later confirmed by other researchers (e.g. Shimazaki and Sozen, 1984; Ye and Otani, 1999). Miranda (2000) performed a study on the inelastic response of elastic perfectly-plastic SDF systems built on firm sites with known displacement ductility using larger sets of seismic input data comprising records from Californian earthquakes with moment magnitudes ranging from 5.8 to 7.7. The results indicated that the ratio of inelastic to elastic deformation depends essentially on the period of vibration of the system and on the level of ductility demand. Limited influence was observed in terms of magnitude, distance and site condition when the shear wave velocity was higher than 180 m/sec. The following equation to calculate inelastic displacement ratio was proposed:

\[ C_\mu = \left[ 1 + \left( \frac{1}{\mu} - 1 \right) \exp\left(12T_e \mu^{-0.8}\right) \right]^{-1} \]  

(2.5)

Where \( C_\mu \) is the ratio of inelastic displacement to elastic displacement for a given ground motion, \( \mu \) is the ductility demand imposed on the SDF system and \( T_e \) is the period of vibration of the system. Chopra and Chintanapakdee (2004) studied the inelastic response of SDF systems with either known ductility or strength, using bilinear non-degrading systems for various levels of post-yield stiffness and using ground motions from earthquakes with moment magnitude ranging between 5.8 to 6.9 and distances between 13 to 60 km. They concluded that, for the dataset of ground motions considered, the median ratio of maximum inelastic to elastic displacements is independent of the earthquake magnitude, distance and site class for far-field ground motions. However, it was shown that the post-yield stiffness has a significant influence on the inelastic response of systems with known ductility. Chopra and Chintanapakdee (2004) proposed the following equation for the computation of inelastic displacement ratio:

\[ C_\mu = 1 + \left[ \left( L_\mu - 1 \right)^{-1} + \left( \frac{a}{\mu^b + c} \right) \left( \frac{T_e}{T_c} \right)^d \right]^{-1} \]  

(2.6a)

In the expression above, \( \mu \) is the displacement ductility demand, \( T_e \) is the elastic period of the SDF system, \( a = 105, b = 2.3, c = 1.9, d = 1.7, T_c \) is the characteristic period of ground motions and \( L_\mu \) is given as:
\[ L_\mu = \frac{\mu}{1 + (\mu - 1)\alpha_s} \]  

(2.6b)

On the other hand, the equivalent linearization approach originates from the work of Jacobsen (1930) that related the force-deformation curve to the damping forces under sinusoidal excitations. The method evolved over the years to include the notion of the period-shift of a system when it undergoes inelastic deformation by using the secant stiffness. The procedure was later improved by Gulkan and Sozen (1970) to be applicable to earthquake loading situations. Kowalsky (1994) used the secant stiffness at maximum deformation to develop an approach for the estimation of the equivalent viscous damping of systems with hysteretic response of the Takeda type. Recently, Dwairi et al., (2007) developed new relationships for equivalent viscous damping for four different types of hysteretic models using 100 ground motions and systems with known ductility. The proposed expression for the elasto-plastic hysteretic model is given below:

\[ \xi_{eq} = \xi_v + C_{EP} \left( \frac{\mu - 1}{\pi \mu} \right) \% \]  

(2.7a)

\[ C_{EP} = 85 + 60(1 - T_{eff}) \quad T_{eff} < 1 \text{s} \]  

(2.7b)

\[ C_{EP} = 85 \quad T_{eff} \geq 1 \text{s} \]  

(2.7c)

In the above equations, \( \xi_{eq} \) refers to equivalent damping; \( \xi_v \) refers to viscous damping; \( \mu \) is the displacement ductility demand, and \( T_{eff} \) is the effective period of the SDF system. Developments in both methods for systems with known ductility have shown that the inelastic response of SDF systems is dependent on the elastic period of the system (effective period in the case of the equivalent linearization method), on the level of ductility, and on the hysteretic behaviour of the system. However, the influence of ground motion characteristics is not fully understood yet. The expressions developed using either the displacement modification or the equivalent linearization approaches are based on the averaged results obtained from nonlinear analysis of a large number of accelerograms. Significant differences between observed and expected response may be anticipated when these expressions are applied to individual accelerograms (Dwairi et al., 2007; Miranda and Ruiz-Garcia, 2002).
These observations are thoroughly tested in the next chapter, by studying the inelastic response of SDF systems exclusively in the light of $T_m$, the selected indicator of frequency content of ground motion.

### 2.4. Inelastic Response of MDF Systems

Commonly used engineering demand parameters for design and assessment purposes include drifts, ductility and strength demands at global and storey levels of structural systems. The review presented hereafter is confined to drift and strength demands. In order to facilitate the discussion, developments in the prediction of each of these EDPs are presented separately along with relevant provisions in various codes.

#### 2.4.1. Drift Demands

Drift demand attract special consideration in the design and assessment of the MDF systems due to their direct correlation with non-structural damage. Typically, global drift and maximum drift are used as drift measures. Global drift, $\theta_r$, can be defined as the maximum roof displacement experienced by the structure under seismic excitations divided by the height of the frame. This parameter reflects the overall performance of the frame, and is typically adopted to calculate global ductility of the frame using global drift at yield from pushover analysis. On the other hand, maximum drift, $\theta_{\text{max}}$, is the maximum drift experienced by the structure during seismic excitation, whereas the inter-storey drift is defined as the relative displacement between adjacent storeys of the frame divided by the vertical distance between the storeys.

Uang and Maarouf (1994) modelled four existing structures that included two steel frames of 2 and 13 storeys with braced and moment resisting systems respectively, and two reinforced concrete buildings of 6 and 10 storeys with column sway and beam sway mechanisms respectively, to study global and maximum drift with the help of eight ground motion records. The study concluded 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. Moreover, it was found to be higher than unity for structures with fundamental periods lower than 0.3 s (2-storey...
steel braced frame), and ranges between 0.7-0.9 for the remaining structures. In contrast, the maximum drift amplification factor, which is the ratio of inelastic to elastic maximum drift for a given ground motion, can be much higher than 1.0 for the estimation of maximum storey drift particularly for frames with a weak first storey. Furthermore, it was concluded that the fundamental period of a structure does not influence the drift amplification factor (except for periods lower than 0.3 s). It is pertinent to mention here that the study included only four frames and a very low number of records, with seven records exhibiting predominant periods ranging between 0.35 to 0.60 s and one record with 1.10 s.

On the other hand, Medina and Krawinkler (2005) used generic frames with stiffness and strength regulated in a way that the first mode profile is a straight line and yielding occurs simultaneously at all storeys under a parabolic lateral load pattern, while gravity loading was considered only to incorporate the P-delta effects, to evaluate the drift demands. The study concluded that the maximum drift is dependent on the fundamental period and number of storeys of the frame, as shown in Equations 2.8, 2.9 and 2.10. It should be mentioned, however, that the scope of the study was confined to generic frames. Moreover, suites of ground motion records did not include large magnitude earthquake events, and were limited to a magnitude range between 6.5 and 6.9.

\[
\theta_{\text{max}} = \theta_r * (0.67 + 1.1T_1) \quad \text{for stiff frames with } T_1 \text{ close to } 0.1N \quad (2.8)
\]

\[
\theta_{\text{max}} = \theta_r * (0.46 + 0.9T_1) \quad \text{for stiff frames with } T_1 \text{ close to } 0.1N \quad (2.9)
\]

\[
\theta_r = \gamma * S_d(T_1)/H \quad (2.10)
\]

Where \(T_1\) is the fundamental mode period; \(\gamma\) is the first mode participation factor; \(S_d(T_1)\) is the spectral displacement at the fundamental period of the frame; and \(N\) is the number of storeys of the frame. The proposed relations shown above are restricted to the frames ranging between \(0.6 \, s \leq T_1 \geq 3.6 \, s\), without significant influence of P-delta effects. The above equations imply that roof drift remains unaltered by the influence of the fundamental period of the structure for periods higher than 0.6 s, whereas maximum drift is dependent on the fundamental period and number of storeys of the frame.
Pettinga and Priestley (2005) studied the inelastic dynamic response of five reinforced concrete frames with 4, 8, 12, 16 and 20 storeys designed with direct displacement based design (DDBD). The study was conducted using artificial accelerograms by matching to the EC8 (CEN, 2004) design spectrum for Soil type-B with a corner period, $T_c$, equal to 0.5 s. The study indicated that frames above 12-storeys showed significant maximum drift amplification at the top storey. The authors proposed to strengthen (stiffen) top storey members by applying additional base shear at the top storey of the frame; a procedure consistent with other studies (Medina, 2004; Paulay and Priestley, 1992) and earlier US and New Zealand seismic design codes (NZS 4203:1992; IBC, 2000).

Karavasilis et al. (2008) performed nonlinear time history analysis of 72 plane steel moment resisting frames to conduct a parametric study involving the number of storeys (ranging from 3 to 20), number of bays, beam-to-column stiffness ratio at mid-height of the frame ($\rho$), ratio of average plastic moment capacity ratio of the bottom storey column and the average of plastic moments of resistance of the beams of all storeys of the frame ($a_{avg}$), and fundamental period of the frames ranging from 0.53 s to 2.82 s to estimate the influence of these parameters on global and maximum storey drift. The study concluded that the roof drift is not affected by the fundamental period and characteristic period, $T_c$, of ground motion for fundamental periods greater than 0.5 s, and are only dependent on the force reduction factor (behaviour factor) of MDF systems, as in Equations 2.11 and 2.12.

\[
q = 1 + 1.39 \times (\mu - 1) \quad \text{for } \mu \leq 5.8 \quad (2.11)
\]

\[
q = 1 + 8.84 \times (\mu^{0.32} - 1) \quad \text{for } \mu > 5.8 \quad (2.12)
\]

Where $q$ is the force reduction factor, and $\mu$ is calculated as the ratio of maximum roof displacement $\Delta_{\text{max}}$, for a given level of $q$ and roof displacement at first yielding, $\Delta_{1, \text{roof}}$. On the other hand, the maximum storey drifts were found to be dependent on $N$, $\rho$ and $a_{avg}$, and independent of $T_1$ of the structure. This observation is different from that of Medina and Krawinkler (2005). Furthermore, the relationships given in Equations 2.13 and 2.14 were recommended to relate maximum roof drift obtained from the above equations to maximum storey drifts.

\[
\theta_{\text{max}} = \beta / \theta_r \quad (2.13)
\]
Design codes, however, adopt a simplified approach for the calculation of drift demands. EC8 (CEN, 2004) recommends the equal displacement rule for structures with fundamental periods higher than $T_c$. Therefore, the drifts at roof and storey level are calculated from elastic lateral load analysis factored with the displacement behaviour factor ($q_d$) assuming the drift profile remains unchanged. It should be specified here, that EC8 (CEN, 2004) prescribes the lateral load pattern defined by the first mode shape or linear displacement shape. On the other hand, NEHRP (2003) recommends a similar procedure; however, modification factors (Table 2-1, only relevant systems are presented) are dependent on the force reduction factor (R) that in turn is dependent on the type of seismic-force resisting system. Inspection of the relation between R and $C_d$ values recommended by NEHRP (2003) shows that the higher the R value the lower is the corresponding $C_d$. It appears from the table that the EC8 provisions are more conservative than NEHRP provisions. However, this issue cannot be discussed in isolation without taking into the full design procedures of the codes. For instance, NEHRP (2003) recommends a parabolic load pattern that accounts for amplification for of drift at top storey, although the extent to which the load pattern influence on the elastic drift profiles needs to be studied in detail.

**Table 2-1:** Displacement modification factors ($C_d$) and Behaviour factors for various seismic force-resisting systems by NEHRP (2003)

<table>
<thead>
<tr>
<th>Type of seismic force-resisting system</th>
<th>Behaviour factor (R)</th>
<th>$C_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special steel concentrically braced frames</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Ordinary steel concentrically braced frames</td>
<td>5</td>
<td>4.5</td>
</tr>
<tr>
<td>Composite eccentrically braced frames</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>Composite concentrically braced frames</td>
<td>5</td>
<td>4.5</td>
</tr>
<tr>
<td>Special steel moment frames</td>
<td>8</td>
<td>5.5</td>
</tr>
<tr>
<td>Intermediate steel moment frames</td>
<td>4.5</td>
<td>4.0</td>
</tr>
<tr>
<td>Special composite moment frames</td>
<td>8</td>
<td>5.5</td>
</tr>
<tr>
<td>Intermediate composite moment frames</td>
<td>4.5</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Based on the studies cited above, it can be inferred that there exists a lack of consensus over the parameters that influence the global and maximum drifts of a
structure. Furthermore, it can be seen that, in most cases, the set of records used by various researchers do not include large magnitude ground motions that are known to contain significant contributions at long periods of vibration. Similarly, inconsistencies amongst different seismic design codes and the above-discussed studies can be observed. Therefore, there is a need to re-investigate the parameters that influence the global and maximum drift demands of MDF systems.

2.4.2. Strength Demands

Inarguably, displacements and drifts are the direct descriptors of structural and non-structural damage, though the importance of strength demands cannot be disregarded. Strength demands imposed on the structure are to be reduced as a way to introduce inelastic response, in the form of behaviour or force reduction factors. The structure is therefore, subjected to the reduced base shear obtained from spectral information, which is distributed spatially to determine stiffness and strength requirements throughout the structure to satisfy drift and damage limitations. Subsequently, using weak beam strong column philosophy, beams are designed to the reduced loading to undergo inelasticity in the event of a design earthquake. Simultaneously, it is ensured that the structure obeys capacity design rules and remains stable under seismic excitations. Therefore, it is imperative that the strength demands imposed on columns do not exceed their capacity. In the most simplistic case, in which higher modes are ignored completely (as adopted in EC8 (CEN, 2004), if the structure satisfies structural regularity criteria), columns are designed to be stronger than beams. Therefore, moments, shear and axial forces for beams are amplified to account for material and structural overstrength to obtain design forces for columns. However, various studies (e.g. Pettinga and Priestley, 2005; Medina and Krawinkler, 2005) have shown that strength demands on a frame amplify significantly due to higher mode effects. In order to facilitate better understanding of strength demands, the review presented hereafter has been divided into two sub-sections. The first segment discusses the issues relevant to the distribution of base shear over the height of the frame, whereas the second part focuses on the issues relevant to amplification of strength demands due to higher mode effects.
2.4.2.1. Distribution of Strength Demands

The reduced base shear, in traditional force based design, is typically calculated using the spectral acceleration at the fundamental period of the structure. The base shear for lateral force method, using EC8, is determined using the following relationship:

$$V_e = S_a(T_1) \times m \times \lambda$$  \hspace{1cm} (2.15)

In the above equation, $V_e$ is the elastic base shear, $S_a(T_1)$ is the design spectral acceleration at the fundamental period of structure, $m$ is total seismic mass of the structure, and $\lambda$ is correction factor to account for first mode mass participation, recommended as 0.85 for $T_1 \leq 2 \ T_c$ or 1.0 otherwise. Comparable recommendations are proposed by NEHRP (2003), for structures regular in terms of stiffness and strength distribution throughout the height.

Calculation of base shear is followed by distribution of the base shear along the height of the structure for optimum use of ductility capacity of components of the structure, and to achieve optimum storey drift distribution over the height of the structure. The most simplified method is to distribute the base shear using the lateral load method, in accordance to the first mode displacement profile of the structure, as recommended by EC8.

$$F_i = V_d \times \frac{s_i \times m_i}{\sum s_j \times m_j}$$  \hspace{1cm} (2.16)

where $F_i$ is the horizontal force acting on storey $i$, $V_d$ is seismic design base shear, $s_i$ and $s_j$ are the displacement of masses $m_i$, $m_j$ in the fundamental mode shape, and $m_i$, $m_j$ are the storey masses. Thus, the influence of higher mode on the alteration of drift and strength demand distribution is not accounted for. EC8 recommends another expression (Equation 2.17) for the distribution that can be applied if linearly varying displacement profile for the first mode is assumed.

$$F_i = V_d \times \frac{z_i \times m_i}{\sum z_j \times m_j}$$  \hspace{1cm} (2.17)

where $z_i$, $z_j$ are the heights of masses $m_i$, $m_j$ above the level of application of seismic actions.
On the other hand, NEHRP (2003) recommends the variable lateral load patterns depending on the fundamental period of the structure, as given by following equation.

\[ V_i = V_p \times \frac{z_i^k \times m_i}{\sum z_j^k \times m_j} \]  \hspace{1cm} (2.18)

where \( k \) is the exponent related to effective fundamental period of structure, the value of \( k \) varies from 1 to 2; lower limit of \( k \) is used for structures with a fundamental period of 0.5 s or less and 2 for fundamental period of 2.5 s; the value of \( k \) can be obtained through linear interpolation for periods between 0.5 s and 2.5 s.

Medina (2004) proposed an optimum load pattern to achieve uniform ductility at all storeys of the generic frames considered, with the stiffness of members tuned such that the first mode shape is linear and simultaneous yielding of the frame occurs under pre-defined lateral loading. The frames were subjected to 40 ordinary ground motions from a Californian database with moment magnitude between 6.5 and 6.9 and closest distance between 13 and 40 km. A parabolic load pattern was found to be most suitable to achieve uniform ductility distribution, whereas triangular and uniform load patterns were shown to be more effective to limit the storey ductility demands at the bottom storeys. The proposed lateral load pattern comprises of an additional shear force at the top storey, \( F_{\text{top}} \), whereas the parabolic load pattern of NEHRP (2003) is retained. However, the value of the exponent \( k \) and \( F_{\text{top}} \) are dependent on number of storeys, target storey ductility, and fundamental period of structure. Park and Medina (2007) proposed a lateral load pattern as a function of target storey ductility demand and height of a frame, using generic frames with strength distributed to achieve simultaneous yielding in beams and column supports and stiffness distributed in order to obtain a first-mode shape consistent with shear-type building. Similarly, Hajirasouliha and Moghaddam (2009) proposed an optimum lateral load pattern, using shear-building models, as a function of the fundamental period of the structure and target ductility demand to achieve uniform storey ductility demand throughout the frame.

In general, the studies discussed above, have focused on the uniform ductility distribution using generic frames. Furthermore, damage distribution along the height of frames is studied in the context of static lateral load pattern. In addition to this, these studies have not incorporated the influence of frequency content on the damage
distribution. Therefore, there is a need for studies that focus on achieving uniform drift damage along the height of the frames and include the influence of frequency content.

2.4.2.2. Amplification of Strength Demands

Amplification of strength demands on columns due to higher mode dynamic effects have long been recognized. However, traditionally, higher modes effects are considered to be relevant mainly for long period or irregular structures. Subsequently, elastic response spectrum analysis is recommended to compute the contribution of higher modes on amplification of strength demands. For instance, EC8 recommends elastic response spectrum analysis for a structure that either does not satisfy criteria for regular structures, or has a fundamental period greater than $4.0T_c$ or 2.0 s.

However, recent studies have shown that strength demands amplify significantly for short and medium period regular structures as well (Medina and Krawinkler, 2005). Pettinga and Priestley (2005) reported dynamic amplification of base shear for reinforced concrete frames designed through the direct displacement based approach using spectrum compatible ground motions, and recommended dynamic amplification of design column shears and bending moments. The authors compared the results of time-history analysis for a 12-storey frame with the response spectrum method of EC8, and concluded that the EC8 prescribed method severely underestimates envelope of shear force throughout the height of the structure, and recommended the following equations to account for shear and moments amplification.

\[
V_i = \omega_v \left( V_{1i}^2 + \left( \frac{V_{2i}}{\mu} \right)^2 + \left( \frac{V_{3i}}{\mu} \right)^2 + \ldots \right)^{1/2}
\]  

(2.19)

Where:

\[
\omega_v = \left( \frac{\mu}{2} \right)^{0.5} \geq 1.0
\]  

(2.20)

In the above equations, $V_i$ is the resulting shear at a given storey, $i$, evaluated using the SRSS combination of inelastic first mode design shear, $V_{1i}$, with elastic higher mode shears, $V_{2i}$, etc. divided by design ductility level $\mu$. On the other hand, $\omega_v$ is a
ductility-dependent dynamic amplification factor. Similarly, they proposed a height and ductility dependent to factor to magnify column moments, given as:

\[
\omega_m = \sqrt{\mu} - 0.15 \times \left( \frac{H_i}{H} \right) \geq 1.3
\]

(2.21)

In the above equation \(H_i/H\) is the normalized height i.e. the ratio of the height of a storey from the ground to the total height of the structure.

Medina and Krawinkler (2005) also evaluated the influence of higher modes on strength demands of generic moment resisting frames with 40 Californian normal ground motion records, with moment magnitude range between 6.5 and 6.9, and distance range between 13 and 40 km. The study concluded that the code provisions and static pushover analysis underpredict the intensity and distribution of shear and axial forces in columns. Moreover, the moment demands in the columns can be severely high, therefore may lead to column hinging. The amplification of these quantities was found to be mainly dependent on the fundamental period of frames, as well as the intensity and frequency content of ground motions. However, it was found that a conclusive study requires actual frames with multiple bays, actual column strength, and account of strength and stiffness deterioration. Priestley et al., (2007) proposed simpler equation to account for base shear amplification and moment magnification demands on columns, with the relationships presented as follows:

\[
\omega_v = 0.1\mu V_{Eff}
\]

(2.22)

In the above equation, \(\omega_v\) is the dynamic amplification factor that accounts for higher mode amplification of shear; \(V_{Eff}\) is the base shear calculated with effective stiffness at the designed ductility. The column shear is amplified with the above factor, whereas overstrength is accounted for separately, as follows:

\[
\omega_m = 1.15 + 0.13 \left( \frac{\mu}{\alpha} - 1 \right)
\]

(2.23)

In the above equation, \(\alpha\) is the plasticity resistance ratio, whereas \(\omega_m\) and \(\mu\) are as defined earlier. In addition to the above amplification factor, column moments are amplified further to account for overstrength. The moments in columns are amplified from first storey to \(\frac{3}{4}\) point of the height of the structure, 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 mode implies the modal period of the structure after formation of a plastic mechanism; therefore, Eigenvalue analysis of a structure with plastic hinges at anticipated locations is carried out to compute modal periods, which are then used to obtain response accelerations from spectral information and calculate base shear using the following equation:

\[ V_{b_{TIMS}} = \left( V_{b1}^o \right)^2 + \left( V_{b_{TIM2}}^o \right)^2 + \left( V_{b_{TIM3}}^o \right)^2 \ldots \right)^{1/2} \]  

(2.24)

The notation \( V_{b_{TIMS}} \) is the base shear obtained with transitory inelastic modal superposition approach; \( V_{b1}^o \) is the 1st mode base shear obtained from plastic mechanism analysis; \( V_{b_{TIM2}} \) and \( V_{b_{TIM3}} \), are the base shears for the 2nd, 3rd transitory inelastic modes.

Based on the literature review presented above, it can be deduced that the studies conducted so far have incorporated the influence of the frequency content of ground motion using various versions of response spectrum analysis method (Pettinga and Priestley 2005; Sullivan et al., 2008). However, in other cases, the influence of frequency content, and other structural properties (fundamental period, for instance) are overlooked to develop simpler models based on ductility demands on the structure (Priestley et al., 2007). Therefore, there is a need to develop simple models that incorporate the influence of frequency content using frequency content measures.

2.5. Concluding Remarks

Notable developments in the estimation of the inelastic response of SDF and MDF systems have been discussed in this chapter. Based on the literature review, it was identified that the mean period, \( T_m \), can be used as a frequency content indicator to investigate and incorporate the influence of frequency content in SDF and MDF systems. It was also shown that a better understanding of influence of key structural parameters on drift demands is required. Furthermore, there is a need for development of the simple models, which incorporate frequency content, for the prediction of strength demands.
Chapter 3

Design of Typical Steel Moment Resisting Frames

3.1. Introduction

As discussed previously, steel moment resisting frames that comply with EC8 design provisions are employed herein to study the influence of the key structural properties on drift and strength demands. A total of 40 frames of 3, 5 and 7 storeys were selected and designed to represent a wide range of structural characteristics. This chapter discusses the EC8 seismic design provisions adopted for design of the frames. Subsequently, the structural system, loading conditions adopted for the design, and design details of the frames are presented. The distribution of various structural properties, which are evaluated using geometry of the frames, Eigenvalue analysis and pushover analysis, are discussed thereafter.

3.2. EC8 Design Provisions

EC8 offers elastic design, generally adopted for important structures or at low seismicity, as well as inelastic design, commonly adopted for the economic design of structures particularly in areas of moderate and high seismicity. Behaviour factors (i.e. force reduction or modification factor), \( q \), are therefore provided to reduce the code-specified forces resulting from idealised elastic response spectra. EC8 recommends three ductility classes, namely: high, moderate and low, typically referred as DCH, DCM and DCL respectively. The ductility classes, to be selected by the designer
considering the allowable damage to structural and non-structural components, provide an upper limit of behaviour factor that can be adopted for a structure.

To address performance based design issues, the maximum allowable drift, \( d_r \), of the structure under lateral loading is used as a measure of non-structural damage in the code, which are needed to be satisfied for the serviceability earthquake. The limitation on the allowable inter-storey drift, \( \psi \), is dictated by the type of non-structural components installed in the structure, given as: 0.5\%, 0.75\% and 1.0\% for brittle, ductile or non-interfering components, respectively. The maximum design drift, \( d_d \), is then calculated by applying the lateral design base shear. Using the equal displacement rule, \( d_r \) is computed using the expression: \( d_r = d_d \times q_d \), with \( q_d = q \). EC8 provisions can then be represented using the following expression:

\[
d_r \nu \leq \psi h
\]  

(3.1)

In the above equation, \( \nu \) is a reduction factor in the range of 0.4–0.5 to represent smaller/frequent serviceability events, and \( h \) is the storey height.

To ensure stability of the structure and to avoid collapse, second-order effects are considered through the sensitivity coefficient, \( \theta \). This is calculated using following expression:

\[
\theta = \frac{P_{tot} d_r}{V_{tot} h}
\]  

(3.2)

In the above expression, \( P_{tot} \) and \( V_{tot} \) are the total cumulative gravity load and seismic shear applied at the storey under consideration; \( h \) is the inter-storey height; and \( d_r \) is the design inter-storey drift. For \( 0.1 \leq \theta \leq 0.2 \) second-order effects may be ignored. If \( 0.1 < \theta < 0.2 \), the multiplier \( \frac{1}{(1 - \theta)} \) may be used to account for this effect and, in any case, the value of \( \theta \) should not exceed 0.3.

Simultaneously, columns are designed to satisfy 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 following expressions:

\[
M_{Ed} = M_{Ed,G} + 1.1 \gamma_{ov} \Omega M_{Ed,E}
\]  

(3.3)

\[
V_{Ed} = V_{Ed,G} + 1.1 \gamma_{ov} \Omega V_{Ed,E}
\]  

(3.4)
\[ N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov} \Omega N_{Ed,E} \] (3.5)

In the above expression, \( M_{Ed,G}, V_{Ed,G} \) and \( N_{Ed,G} \) refer to the design moments, shear and axial forces, respectively, for the column due to gravity loads, and \( M_{Ed,E}, V_{Ed,E} \) and \( N_{Ed,E} \) represent the design moments, shear and axial forces, respectively, for the column due to lateral seismic loads; \( \gamma_{ov} \) is the material over-strength typically assumed to be 1.25; \( \Omega \) is a beam over-strength factor determined as a minimum of \( \Omega_i = \frac{M_{pl,Rd,i}}{M_{Ed,i}} \) of beams, where \( M_{Ed,i} \) is the design moment in beam ‘i’ and \( M_{pl,Rd,i} \) is the corresponding plastic moment.

### 3.3. Structural Configuration, Loading Conditions and Design Details

Figure 3-1 shows a plan and elevation of the structural system adopted in this study. It 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 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 EC1 (CEN, 2002) and EC3 (CEN, 2005). Dead loads of 1 kN/m² (excluding self weight) and an imposed load of 2 kN/m² 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 sections made of steel grade S275. The same sections are used for the internal and external column 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 the following expression:

\[ F_i = V_d \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \] (3.6)

where, \( F_i \) is the horizontal force acting on storey i; \( V_d \) is the design 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. Member details for the 3, 5 and 7 storey frames are presented in Table 3-1, Table 3-2 and Table 3-3 respectively.

Figure 3-1: Plan and elevation of a typical moment resisting steel frame adopted in the study (elevation shown for the 5-storey frames)
Table 3-1: Member details for 3-storey frames

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Table 3-2: Member details for 5-storey frames

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3.4. Evaluation of Structural Characteristics

The database of designed structures is now processed to evaluate the characteristics of the frames to be used to study their influence on drifts and strength demands. Based on the literature review, various structural characteristics are identified, which may prove to be useful in understanding the inelastic response. These characteristics have been evaluated using Eigenvalue analysis, pushover analysis, geometry of the frame and simple structural analysis principles. It is pertinent to mention that the conventional form of pushover analysis is implemented using the load pattern obtained from the first mode shape of the frame.

To perform nonlinear pushover and incremental dynamic analysis, the designed frames were modelled in OpenSees (2008); the software has been validated extensively. The files developed for the pushover analysis in OpenSees are provided in Appendix-A. Hinges are allowed to form in the beams and columns. A bilinear stress-strain curve for steel with post-yield stiffness of 0.5% is selected to account for material nonlinearity, as shown in Figure 3-2. Vertical loads comprising of dead loads and an allowance of 30% of live loads were applied at mid-span 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 analysis.

![Figure 3-2: Force-displacement curve adopted for the material nonlinearity with post yield stiffness of 0.5%](image-url)
The main characteristics of the frame evaluated in the study are:

I. The total height of each frame, $H$, is obtained directly by adding the storey heights. As described earlier, the structures in the database consist of 3, 5 and 7 storey frames; thus the height of the frames is 11.5 m, 18.5 m and 25.5 m, respectively. The height distribution of the frames is shown in Figure 3-3.

![Figure 3-3: Distribution of height of frames](image)

II. The fundamental period, $T_1$, of each frame is obtained using Eigenvalue analysis. The fundamental period of the structure is chosen to examine its influence on the strength demands, based on the principles of dynamics. The distribution of the fundamental period of frames is shown in Figure 3-4, which ranges from 0.40 sec to 1.75 sec.

III. Plasticity resistance ratio, $\alpha$, (referred as $\alpha_u/\alpha_1$ in EC8), calculated as the ratio of base shear, $V_y$, when the plastic mechanism has developed in the structure to the base shear at the formation of the first plastic hinge in the structure (evaluated from pushover analysis of the frame). This parameter may prove to be useful, bearing in mind the influence of plasticity (typically measured relative intensity or ductility) on the strength demand.
IV. Figure 3-5, Figure 3-6 and Figure 3-7 present pushover curves, which present the global drift (in % of the overall height of the frame) on X-axis and normalized base shear \((V/V_1)\) on the Y-axis. The base shear has been normalized using the base shear corresponding to the formation of the first plastic hinge in the structure, \(V_1\). The distribution of the plasticity resistance ratio of the frames in the database is given in Figure 3-8, and ranges from 1.39 to 2.42.

![Figure 3-4: Distribution of fundamental period of frames](image)

V. The beam-to-column stiffness ratio, \(\rho\), of the frames, calculated for the storey closest to the mid-height of the structure is determined using the expression:

\[
\rho = \frac{\sum(I/l)_b}{\sum(I/l)_c}
\]  

(3.7)

In the above equation, \(I\) and \(l\) are the second moment of inertia and length of the beam or column, respectively, and subscript ‘b’ and ‘c’ refer to beam and column respectively. The distribution of beam-to-column stiffness ratio of the frames is shown in Figure 3-9, and ranges from 0.07 to 0.21.
**Figure 3-5:** Pushover curves for 3-storey frames

**Figure 3-6:** Pushover curves for 5-storey frames
**Figure 3-7:** Pushover curves for 7-storey frames

**Figure 3-8:** Distribution of plasticity resistance ratio ($\alpha$) of frames
VI. The relative storey stiffness ratio is calculated 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 to study the inelastic response, this parameter is proposed in this work to account for variation in the stiffness or strength of the top storeys. This parameter may be of interest, considering that relatively less stiff/strong top storeys may lead to earlier yielding and overall increase in the plasticity of the frame. There can be multiple ways to calculate this parameter to incorporate for the relative stiffness. Three different variations are proposed here: 1) $\beta_1$, calculated as the ratio of the inter-storey drift at the top storey to the maximum drift of the rest of the storeys; 2) $\beta_2$, calculated as the ratio of the maximum drift for the upper 1/3rd of the frame to the maximum drift for the lower 2/3rd of the frame; 3) $\beta_3$, calculated as the ratio of the maximum drift for the upper half of the frame to the maximum drift for the lower half of the frame. For a frame with odd number of total storeys of the frame, upper 1/3rd and 1/2nd of total number of storeys is rounded off to the lower number. For example, for a 7 storey frame, $\beta_2$ is calculated as the ratio of maximum drift of top 2 storeys ($7/3 = 2.66 \approx 2$) to maximum drift of the bottom 3 storeys. Similarly, $\beta_3$ is calculated as the ratio of the maximum drift of the top 3 storeys.
(7/2 = 3.5 ≈ 3) to the maximum drift of the bottom 4 storeys. Figure 3-10, Figure 3-11 and Figure 3-12 provide normalized drift profiles of the 3, 5 and 7 storey frames, respectively, used in this study. The drift profiles presented have been normalized using inter-storey drift of the first storey of the respective frame. The distribution of $\beta_1$, $\beta_2$ and $\beta_3$ of the frames used in the study is given in Figure 3-13, Figure 3-14 and Figure 3-15 respectively.

All the parameters discussed above are presented in tabular form in Table 3-4, Table 3-5 and Table 3-6.

**Figure 3-10:** Normalized drift profile of 3-storey frames obtained from Eigenvalue analysis
Figure 3-11: Normalized drift profile of 5-storey frames obtained from Eigenvalue analysis

Figure 3-12: Normalized drift profile of 7-storey frames obtained from Eigenvalue analysis
**Figure 3-13**: Distribution of $\beta_1$ for the frames

**Figure 3-14**: Distribution of $\beta_2$ for the frames
Figure 3-15: Distribution of $\beta_3$ for the frames

Table 3-4: Structural characteristics of 3-storey frames

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<th>$T_1$ (s)</th>
<th>H (m)</th>
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<th>$\beta_1$</th>
<th>$\beta_2$</th>
<th>$\beta_3$</th>
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Table 3-5: Structural characteristics of 5-storey frames

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3.5. Concluding Remarks

This chapter discussed the EC8 design provisions adopted for the design of steel moment resisting frames. Subsequently, the structural design details, and the evaluation and distribution of structural characteristics were presented for the frames under consideration. The influence of these characteristics on the drift and the strength demands will be investigated in Chapters 5 and 6 respectively.
Chapter 4

Influence of Frequency Content on Inelastic Response

4.1. Introduction

This chapter focuses on examining the effects of frequency content of the ground motion on the inelastic demands imposed on both SDF and MDF systems. As discussed in Chapter 2, the mean period ($T_m$) of ground motion is selected owing to its ability to distinguish between various spectral shapes of ground motion, and its relationship with magnitude, distance and site characteristics. The inelastic displacement demands on SDF systems for target ductility levels are first studied in the light of $T_m$, using a suite of 128 ground motion records. The study is then extended to MDF systems with the help of incremental dynamic analysis by employing the same ground motion ensemble to assess the influence of $T_m$ on various engineering demand parameters (EDPs).

4.2. Inelastic Response of SDF Systems

Due to its simplicity, the displacement coefficient method is adopted in this work to assess the influence of the frequency content of the ground motion on the inelastic response of SDF systems. In effect, $T_m$ represents the average of the response periods that are most prevalent during the strong shaking portion of the record. If the maximum nonlinear response of a SDF system is dependent on the $T_m$ of a given record, then it can be anticipated that the maximum inelastic displacements ($\Delta_m$).
should occur during this strong-shaking phase. This expected behaviour of the SDF system under the influence of $T_m$ is tested for two SDF systems with $T_e$ equal to 0.2 s and 0.9 s, and characterized by a bilinear elasto-plastic hysteresis loop with strain hardening ($\alpha_s$) taken as 3%. The SDF systems are subjected to the HWA015-E component of the 1999 Chi-chi earthquake (Taiwan), recorded at a distance of 51 km in Soil Type D with a PGA of 0.105g and $T_m$ of 0.862 s. The time history of the ground motion is presented in Figure 4-1. The systems are subjected to the ground motions to achieve a target displacement ductility ($\mu$) of 4, using the procedure illustrated in Miranda and Ruiz-Garcia (2002) with the help of the special purpose program SeismoSignal (Seismosoft, 2008). The displacement time-histories of the elastic and inelastic SDF systems with $T_e$ of 0.2 s and 0.9 s are shown in Figure 4-2. It is noted that for both systems, the maximum inelastic ($\Delta_{in}$) displacements occur during the high acceleration amplitudes of the ground motion. Moreover, for the SDF system with $T_e = 0.2$ s, the displacements tend to amplify in comparison with the elastic system ($\Delta_e = 0.13$ cm; $\Delta_{in} = 0.293$ cm); conversely, for the system with $T_e = 0.9$ s the maximum inelastic displacement is lower in comparison with the maximum displacement recorded in the elastic system ($\Delta_e = 5.59$ cm and $\Delta_{in} = 3.46$ cm).

**Figure 4-1**: Acceleration time-history of HWA015-E component of Chi-Chi Taiwan 1999 earthquake, with $T_m$ of 0.862 sec.
Figure 4-2: Comparison of elastic and inelastic displacement time history of a SDF system subjected to Chi-Chi Taiwan earthquake: (a) $T_e = 0.2$ sec and $\alpha=3\%$; (b) $T_e = 0.9$ sec and $\alpha=3\%$
These observations are tested in detail using a large ensemble of ground motions comprising of 128 records from 23 earthquakes recorded in various regions of the world; only one horizontal component from each station is selected (refer to Table 4-1). Condensed information of earthquakes used in the study is presented in Table 4-1; a more detailed table is given in Table B-1(Appendix B). It is also ensured that the time series represent different site classes (according to the NEHRP classification) and magnitude-distance combinations. The distribution of the selected records with respect to magnitude, distance and site classes is presented in Figure 4-3. In order to avoid near-field effects, the records have been selected within 20-80 km (closest distance from fault rupture) for magnitudes higher than 6 and between 0-80 km (closest distance from fault rupture) for magnitudes between 5.5 and 6. Each record is applied to 20 SDF systems with elastic period (T_e) ranging from 0.1 s to 1.0 s at constant interval of 0.05 s. Each analysis is repeated for three levels of target displacement ductility, i.e., μ = 2, 4 and 6. Hence, a total of 7680 analyses are performed. For each inelastic time-history analysis, the period ratio (T_e/T_m) is calculated in addition to the inelastic displacement ratio (C_μ = Δ_m/ Δ_e).

**Figure 4-3:** Distribution of earthquakes with respect to magnitude, distance and site classes, for ground motions used in the study
Table 4-1: 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). In the column for site class, number in the parenthesis indicates number of records for a given site class.

<table>
<thead>
<tr>
<th>Earthquake Name</th>
<th>Magnitude</th>
<th>Mechanism</th>
<th>Number of records</th>
<th>Distance, km (Rmin-Rmax)</th>
<th>Site Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fruili, Italy-03 1976-09-11</td>
<td>5.5</td>
<td>RV</td>
<td>1</td>
<td>20</td>
<td>C(1)</td>
</tr>
<tr>
<td>Point Mugu 1973-02-21</td>
<td>5.65</td>
<td>RV</td>
<td>1</td>
<td>20</td>
<td>D(1)</td>
</tr>
<tr>
<td>Coyote Lake 1979-08-06</td>
<td>5.74</td>
<td>SS</td>
<td>5</td>
<td>9-34</td>
<td>B(1)+C(2)+ D(2)</td>
</tr>
<tr>
<td>Coalinga-05 1983-07-22</td>
<td>5.77</td>
<td>RV</td>
<td>5</td>
<td>11 - 16</td>
<td>C(3)+ D(2)</td>
</tr>
<tr>
<td>Livermore-01 1980-01-24</td>
<td>5.8</td>
<td>SS</td>
<td>5</td>
<td>20.5 - 57.5</td>
<td>C(3)+ D(2)</td>
</tr>
<tr>
<td>Westmorland 1981-04-26</td>
<td>5.9</td>
<td>SS</td>
<td>2</td>
<td>15 -19</td>
<td>C(1)+ D(1)</td>
</tr>
<tr>
<td>Taiwan SMART1(5) 1981-01-29</td>
<td>5.9</td>
<td>RV</td>
<td>4</td>
<td>29 - 32</td>
<td>D(4)</td>
</tr>
<tr>
<td>Whittier Narrows-01 1987-10-01</td>
<td>5.99</td>
<td>RO</td>
<td>8</td>
<td>27 - 56.5</td>
<td>C(4)+ D(4)</td>
</tr>
<tr>
<td>N. Palm Springs 1986-07-08</td>
<td>6.06</td>
<td>RO</td>
<td>6</td>
<td>27 - 52</td>
<td>C(5)+ D(1)</td>
</tr>
<tr>
<td>Morgan Hill 1984-04-24</td>
<td>6.19</td>
<td>SS</td>
<td>6</td>
<td>23 - 31</td>
<td>C(3)+ D(3)</td>
</tr>
<tr>
<td>Parkfield 1966-06-28</td>
<td>6.19</td>
<td>SS</td>
<td>1</td>
<td>63</td>
<td>C(1)</td>
</tr>
<tr>
<td>Coalinga-01 1983-05-02</td>
<td>6.36</td>
<td>RV</td>
<td>13</td>
<td>24 - 44</td>
<td>C(7)+ D(6)</td>
</tr>
<tr>
<td>Friuli, Italy-01 1976-05-06</td>
<td>6.5</td>
<td>RV</td>
<td>2</td>
<td>33 - 49</td>
<td>C(1)+ D(1)</td>
</tr>
<tr>
<td>Imperial Valley-06 1979-10-15</td>
<td>6.53</td>
<td>SS</td>
<td>3</td>
<td>22 - 37</td>
<td>D(3)</td>
</tr>
<tr>
<td>Superstition Hills-02 1987-11-24</td>
<td>6.54</td>
<td>SS</td>
<td>1</td>
<td>22</td>
<td>D(1)</td>
</tr>
<tr>
<td>San Fernando 1971-02-09</td>
<td>6.61</td>
<td>RV</td>
<td>6</td>
<td>23 - 69</td>
<td>C(4)+ D(2)</td>
</tr>
<tr>
<td>Borrego Mtn 1968-04-09</td>
<td>6.63</td>
<td>SS</td>
<td>1</td>
<td>46</td>
<td>D(1)</td>
</tr>
<tr>
<td>Northridge-01 1994-01-17</td>
<td>6.69</td>
<td>RV</td>
<td>15</td>
<td>20 - 78</td>
<td>B(2)+C(7)+ D(6)</td>
</tr>
<tr>
<td>Duzce, Turkey 1999-11-12</td>
<td>7.14</td>
<td>SS</td>
<td>1</td>
<td>26</td>
<td>B(1)</td>
</tr>
<tr>
<td>Landers 1992-06-28</td>
<td>7.26</td>
<td>SS</td>
<td>7</td>
<td>24 - 69</td>
<td>C(2)+ D(5)</td>
</tr>
<tr>
<td>Kocaeli, Turkey 1999-08-17</td>
<td>7.51</td>
<td>SS</td>
<td>5</td>
<td>31 - 67</td>
<td>C(1)+ D(4)</td>
</tr>
<tr>
<td>Chi-Chi Taiwan 1999-09-20</td>
<td>7.62</td>
<td>RV</td>
<td>17</td>
<td>25 - 67</td>
<td>C(7)+ D(10)</td>
</tr>
</tbody>
</table>
In Figure 4-4, the results are plotted for each displacement ductility level in terms of $C_\mu$, against the $T_e/T_m$ ratio. Two power series trend lines are fitted to the data for each displacement ductility level, in order to study the trends of the results; the standard deviation for $\mu = 2, 4$ and $6$ was found to be $0.22, 0.29$ and $0.36$ respectively. The plots clearly show that amplification of the inelastic displacement occurs as the period ratio ($T_e/T_m$) tends to be lower than unity. Moreover, a stronger dependence between the period ratio and $C_\mu$ is evident for higher levels of ductility of the system. It may be justified on the basis that the higher the ductility, the higher the probability that the system yields earlier in the time history and responds inelastically during high amplitudes of acceleration. Consequently, the inelastic displacement demands of SDF systems are dependent on the elastic period, post-yield stiffness, ductility and mean period of the ground motions, therefore confirming the dependence on frequency content. The coefficient of variation (COV) of the inelastic displacement ratios for all records used in this study is plotted against $T_e$ and $T_e/T_m$ in Figure 4-5(a) and Figure 4-5(b) respectively. In general, COV increases with increase in displacement ductility demand as noted in other studies (Miranda, 2000). Moreover, it is observed that the COVs exhibit downward trend for a $T_e/T_m$ ratio lower than unity.

The observation of dependence of the inelastic displacement ratio on the mean period of ground motion illustrated above is in harmony with the observation made by Chopra and Chintanapakdee (2004) that showed its dependence on the $T_e/T_c$ ratio. The dependence of inelastic deformation on $T_c$ or $T_m$ hints to the possible correlation between $T_c$ and $T_m$. $T_c$ is therefore calculated for the ground motion ensemble using the procedure illustrated by Chopra and Chintanapakdee (2004). The plot of $T_c$ versus $T_m$ for all records is shown in Figure 4-6; random effects regression is performed in order to account for intra-event and inter-event variability and to obtain a relationship between the two parameters. The data exhibits a good correlation between the two parameters, with a correlation coefficient of $0.90$. Nevertheless, it must be kept in mind that this observation is based on the ground motion data used in this study; a larger set of data may be required to validate this observation.
Figure 4-4: Plot of $C_\mu$ with respect to $T_e/T_m$ for 2560 SDF systems with $\alpha=3\%$ and (a) $\mu=2$, (b) $\mu=4$ and (c) $\mu=6$
Although the dependence of the inelastic deformation shown in this study is similar to that identified in the work of Chopra and Chintanapakdee (2004), the dependence of $C_\mu$ on $T_e/T_m$, and the relation of $T_m$ with magnitude, distance and site classification can be further used to improve the accuracy of the estimates of inelastic displacement demands of SDF systems; i.e., as $T_m$ is very predictable it can be incorporated into analyses. The results from the nonlinear time history analysis of SDF systems are...
therefore grouped into five $T_m$ ranges as shown in Table 4-2. The median normalized acceleration spectra for each group are shown in Figure 4-7 from which it is clear that $T_m$ is strongly linked to the spectral shape. For each $T_m$ group, the median value of $C_\mu$ is calculated for each pair of $T_e$ and target ductility level to obtain the curves presented in Figure 4-8. The figure also includes the estimated curve (referred to as FF-C) using Equation 4.1 below, proposed by Chopra and Chintanapakdee (2004), considering a value of 0.42 for $T_c$ as in Chopra and Chintanapakdee (2004) for far-field earthquake events.

$$C_\mu = 1 + \left[ (L_\mu - 1)^{-1} + \left( \frac{a}{\mu^b + c} \right) \left( \frac{T_e}{T_c} \right)^d \right]^{-1}$$  \hspace{1cm} (4.1a)

In the expression above, $\mu$ is the displacement ductility demand, $T_e$ is the elastic period of the SDF system, $a = 105$, $b = 2.3$, $c = 1.9$, $d = 1.7$, $T_c$ is the characteristic period of ground motions and $L_\mu$ is given as:

$$L_\mu = \frac{\mu}{1 + (\mu - 1)\alpha}$$  \hspace{1cm} (4.1b)

The curves plotted for three ductility levels shown in Figure 4-8 further clarify the influence of frequency content on the amplitude of inelastic deformation. A trend can be easily identified from the plots. As $T_m$ increases (from Group 1 to Group 5), the amplification of the inelastic deformation for a given $T_e$ increases. Moreover, the system period for which the $C_\mu$ becomes lower than unity increases with increasing $T_m$. These trends become more evident as the ductility of the system increases; the curves flatten for high values of $T_e$ to reach a value of approximately 0.8.

The plot of $C_\mu$ obtained from Equation 2 using $T_c$ of 0.42 in conjunction with all groups further clarify the influence of $T_m$ for all ductility levels as shown in Figure 4-8. In general, for elastic periods shorter than 0.5 s, the FF-C curve closely follows the median inelastic displacement ratios obtained from Group 3 records for all ductility levels, while overestimating and underestimating for records of lower and higher $T_m$ groups, respectively. On the other hand, the FF-C curve is conservative for elastic periods longer than 0.5 s for all $T_m$ groups except Group 5 consisting of records with longest mean periods.
Table 4-2: Grouping of ground motions according to $T_m$

<table>
<thead>
<tr>
<th>Group Number</th>
<th>$T_m$ Range</th>
<th>Number of records</th>
<th>Mean $T_m$</th>
<th>1st Quartile (Q1)</th>
<th>3rd Quartile (Q3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>0.17 s - 0.35 s</td>
<td>26</td>
<td>0.29</td>
<td>0.24</td>
<td>0.31</td>
</tr>
<tr>
<td>Group 2</td>
<td>0.35 s - 0.50 s</td>
<td>31</td>
<td>0.42</td>
<td>0.40</td>
<td>0.45</td>
</tr>
<tr>
<td>Group 3</td>
<td>0.50 s - 0.65 s</td>
<td>24</td>
<td>0.59</td>
<td>0.53</td>
<td>0.62</td>
</tr>
<tr>
<td>Group 4</td>
<td>0.65 s - 0.80 s</td>
<td>23</td>
<td>0.72</td>
<td>0.70</td>
<td>0.75</td>
</tr>
<tr>
<td>Group 5</td>
<td>0.80 s - 1.25 s</td>
<td>24</td>
<td>0.93</td>
<td>0.85</td>
<td>1.09</td>
</tr>
</tbody>
</table>

Figure 4-6: Relation between $T_c$ and $T_m$
Figure 4-7: PGA-Normalized acceleration spectra of suites of records used in the study: a) Group 1 ($T_m < 0.35$); b) Group 2 ($T_m$ range 0.35-0.50); c) Group 3 ($T_m$ range 0.50-0.65); d) Group 4 ($T_m$ range 0.65-0.80); e) Group 5 ($T_m > 0.80$) and f) median acceleration spectra of all groups
The results presented in Figure 4-8 confirm that the prediction of inelastic displacement ratios can be improved if the ground motions are explicitly categorized in terms of frequency content, or otherwise according to combinations of magnitude, distance and site class. Previous studies (e.g. Chopra and Chintanapakdee, 2004) have not identified this issue, probably due to two possible reasons: firstly, the database of records used in their study did not include large magnitude earthquakes; secondly, the classification scheme used to study the influence of ground motion characteristics was based either on magnitude and distance combinations or site categories. On the other hand, the frequency content of ground motion is dependent on all three parameters as shown by Rathje et al., (2004). It should be noted that the purpose of highlighting the influence of frequency content on the inelastic response of SDF systems is not to suggest another set of equations, but to emphasize the need to use representative values of parameters that influence the shape of spectrum and subsequently the inelastic response of SDF systems. This can be demonstrated by plotting the FF-C

**Figure 4-8:** $C_\mu$ curves for various $T_m$ groups and compared with equation from Chopra and Chintanapakdee (2004) for far-field (FF-C); a) $\mu=2$; b) $\mu=4$ and c) $\mu=6$
curve using Equation 2 for Groups 2 and 4, using median $T_c$ values for each group (0.38 and 0.75 respectively) along with the median curves for the groups for displacement ductility of 4, as shown in Figure 4-9. The matching of the curves obtained from the analyses in the study with the FF-C curves using representative values improves significantly. It can be noted, however, that the FF-C curve predicts slightly conservative estimates, which can be easily improved.

**Figure 4-9:** $C_\mu$ curves for $T_m$ Groups 2 and 4 compared with equation from Chopra and Chintanapakdee (2004) for far-field (FF-C), for $\mu=4$ using median $T_c$ value for each group.

In the current version of EC8, the inelastic response is based on $T_e$ and $T_c$, and the variation of $T_c$ is dictated solely by the site characteristics. However, it is also dependent on magnitude and distance of the earthquake; this points to the need for improvement in the current version of EC8 to address these issues. The recommendation presented herein, based on the results shown in this section lends strong support to the need pointed out by Bommer and Pinho (2006) for better adaptation of EC8 to meet performance based design objectives. More recently, Bommer et al. (2009) have proposed the use of PGA and peak ground velocity (PGV) in hazard assessment procedures for improved estimate of $T_c$ and the shape of the design spectrum.
4.3. Inelastic Response Assessment of MDF Systems

The study conducted for SDF systems to assess the influence of frequency content characterized by the mean period is extended herein to MDF systems. To this end, a five-storey three-bay moment-resisting frame, B15, is selected from the database of frames presented in Chapter 3. The frame is modelled in OpenSees (2008) using two force-based elements (with seven Gauss points each) per member. Material nonlinearity is considered through the adoption of a bilinear stress-strain curve for steel with post yield stiffness of 0.5%. Vertical loads, which include the dead loads and an allowance of 30% for live loads, are applied at mid-span of the beams and at beam-to-column joints. For the dynamic analysis, a seismic mass of approximately 70 tons is considered at every floor of the frame and a mass of approximately 56 tons is applied at the roof level. Initial-stiffness proportional damping is considered with 2% of viscous damping assigned to the first mode.

Incremental dynamic analysis of the frame is conducted by scaling the records to attain various levels of global ductility assuming:

a) The equal displacement rule (EDR) holds, i.e., the elastic spectral displacement obtained from the SDF system is equal to the inelastic displacement.

b) The structure continues to vibrate in the fundamental mode drift profile as it moves from the elastic to the inelastic domain under the applied ground motion; thereby, the drift profile remains unaltered and the response of the structure is unaffected by higher modes.

The assumptions above are presented to justify scaling of records with respect to spectral response at the fundamental period, and to facilitate the discussion within the context of EC8 that prescribes the equal displacement rule for predicting inelastic displacement demands. However, the assumptions enlisted above may or may not hold true as will be inspected in subsequent sections. Thus, the scaling factor \( S_F \) required for an individual record to attain target displacement ductility \( \mu_{EDR} \) levels can be calculated using Equation 4.2; in the force based concept this quantity can be interpreted as the behaviour factor or force reduction factor.
\[ S_F = \mu_{EDR} \times \frac{\Delta_{1,\text{roof}}}{S_d(T_1)} \]  

(4.2)

Where \( S_d(T_1) \) is the spectral displacement of a given record at the fundamental period of the frame \((T_1 = 1.15 \text{ s})\); \( \Delta_{1,\text{roof}} \) is the displacement at the roof level at the formation of the first yield in the frame obtained from static pushover analysis using a force profile based on the fundamental mode shape of the frame. \( \Gamma \) represents a transformation factor (1.34 for the frame under study) required to compute the roof displacement from the equivalent SDF system using following expression:

\[ \Gamma = \frac{\sum m_i \Delta_i}{\sum m_i \Delta_i^2} \]  

(4.3)

where \( m_i \) is the seismic mass at each storey and \( \Delta_i \) represents the displacement at each storey normalized to the roof displacement for the fundamental mode shape obtained from Eigenvalue analysis. The complete process of determining the scaling factor for a given record is presented schematically in Figure 4-10.

IDA is conducted using the 128 earthquake records utilized earlier in the assessment of the SDF systems. The ground motions are scaled to six levels of ductilities from 1 to 6 using Equation 3. Therefore, a total of 768 nonlinear time-history analyses (NTHA) are conducted. Three response quantities are recorded for each run:

i. Maximum roof displacement \((\Delta_{\text{max}})\)

ii. Maximum base shear \((V_{\text{max}})\)

iii. Maximum drift at each storey of the frame \((\theta_{si,\text{max}})\)

The response quantities are clustered in the \( T_m \) groups considered earlier for SDF systems, and treated statistically to calculate the median, 16\(^{th}\) percentile and 84\(^{th}\) percentile corresponding to each ductility level. A detailed examination of results is presented in the following subsections.
Figure 4-10: (a) First-mode displacement profile of 5-storey MRF along with undeformed shape of the frame; b) corresponding equivalent SDF system of the frame; c) pushover curve of the frame obtained using first mode load pattern suggested by EC8; d) Drift profile of the frame at the formation of the first plastic hinge; e) displacement spectrum of the actual record, scaled to produce $\mu_{EDR} = 4$ corresponding to yield displacement of the frame.
4.3.1. Roof Displacements

Estimation of inelastic displacements is a fundamental aspect of the seismic design process irrespective of the design philosophy implemented. Conventionally, displacement demands for MDF systems are estimated using the inelastic displacements of SDF systems obtained using the procedures discussed previously. As demonstrated earlier, for SDF systems with elastic periods lower than the mean period of the ground motion, the inelastic displacement ratios are significantly higher than unity. On the other hand, for systems with elastic periods higher than the mean period of records, the modification factors reduce to lower than unity for high levels of displacement ductility.

In this section, roof displacements for the 5-storey moment-resisting frame obtained from IDA are investigated and compared with roof displacements estimated with EDR. The results of IDA are assembled in the form of median, 16th percentile and 84th percentile for each Tm-group of records and for corresponding ductility levels. Roof displacements for Groups 1, 3 and 5 are presented in Figure 4-11 along with EDR estimates. The EDR estimates are calculated by linear scaling of the roof displacement at first yielding obtained from static pushover analysis with corresponding displacement ductility. As anticipated, the roof displacements observed for MDF system are not significantly influenced by the frequency content of the ground motion for all groups of records. This is due to the fact that the fundamental period of the structure under study is higher than the mean period of the records and the roof displacement is least affected by higher mode effects. In addition to this, the median roof displacements obtained from IDA decrease with the increase of displacement ductility demands. However, the dispersion of roof displacement for the frame measured in the form of 16th and 84th percentiles exhibit severe dispersion particularly for large levels of ductility demands, whereas EDR predicts conservative estimates of the roof displacements for all Tm-groups.
4.3.2. Dynamic Pushover Curves

Based on the capacity design approach, moment frames should in principle exhibit plastic hinges only in the beams with the exception of the column bases in the bottom storey. However, amplification of base shear and bending moments within the columns as a result of higher mode response of the system may jeopardize this requirement. Whilst the influence of higher modes on base shear is evident for relatively long period as well as irregular structures in the elastic range, this response quantity may be severely influenced by post yielding of the structure. This can also result in significant amplifications in short to medium period structures, as demonstrated by Pettinga and Priestley (2005).

Detailed assessment of this aspect is conducted using IDA results to develop the dynamic pushover curves (DPO) for pairs of $V_{\text{max}}$ and $\mu_{\text{EDR}}$. Median, 16th percentile and 84th percentile curves for each group are presented in Figure 4-12. The influence of frequency content on base shear is manifested from the dynamic pushover moving from Group 1 to Group 5. Dispersion of the base shear for a given ductility level is relatively high for the lowest $T_m$ group (Group 1) that corresponds to the records with $T_m$ in the proximity of second period of vibration of the frame, and reduces for higher

Figure 4-11: Median, 16th percentile and 84th percentile roof displacements from IDA of the frame along with EDR estimates for a) Group 1 ($T_m$ range < 0.35); b) Group 3 ($T_m$ range 0.50-0.65); c) Group 5 ($T_m$ range > 0.80)
$T_m$ groups being minimal for Group 5 ($T_m > 0.8$ sec). A general trend in all the DPO plots is that the dispersion of the data tends to increase for higher levels of displacement ductility.

![Graph showing median, 16th percentile and 84th percentile dynamic pushover curves for different $T_m$ groups.]

**Figure 4-12:** Median, 16th percentile and 84th percentile dynamic pushover curves for a) Group 1 ($T_m$ range < 0.35); b) Group 2 ($T_m$ range 0.35-0.50); c) Group 3 ($T_m$ range 0.50-0.65); d) Group 4 ($T_m$ range 0.65-0.80); e) Group 5 ($T_m$ range > 0.80); f) Static pushover curve (SPO) and median dynamic pushover curves for all $T_m$ groups.

Further insight into this may be gained by comparing the median dynamic pushover curves with static pushover analysis results obtained using the lateral load pattern defined by EC8, which corresponds to the 1st mode displacement profile of the frame as explained earlier in Figure 4-10(c). The figure elucidates the influence of the frequency content on the contribution of higher mode response. A clear influence of the mean period can be observed for Group 1 ($T_m < 0.35$) and Group 2 ($T_m$ 0.35 - 0.50), both being in the vicinity of the higher mode periods of the structure. The
influence of frequency content on the base shear reduces as the mean periods approach the fundamental period of the structure. Though the influence of ductility on base shear amplification can be observed, the influence of mean period cannot be ignored.

4.3.3. Maximum Storey Drift Profiles

Maximum storey drift profiles provide a detailed characterisation of the distribution of drift and ductility demands imposed by the seismic action over the height of the structure. The median values of the maximum drift profiles of the frame using \( \theta_{si, \text{max}} \) are presented in this section, for displacement ductilities of 1 and 4 for each \( T_m \) group along with dispersion measures. It should be recalled that the first mode displacement shape was assumed in the design, and that the corresponding EC8 lateral load pattern was adopted to perform the pushover analysis. The drift profile recorded at the formation of the first plastic hinge is given in Figure 4-10(d).

The profiles presented in Figure 4-13 for \( \mu_{\text{EDR}} \) equal to 1 resemble the first mode drift profile for all \( T_m \)-groups. Nevertheless, slight amplification of the drifts at the top storeys can be observed for low \( T_m \) groups. As the ductility demand increases, the drift profiles modify appreciably as shown in Figure 4-14 for \( \mu_{\text{EDR}} \) equal to 4. Prominent amplification of drifts occurs at the top three storeys particularly for Group 1 (\( T_m < 0.35 \)) and Group 2 (\( T_m 0.35-0.50 \)) records. On the other hand, the maximum drift profile recorded for Group 5 (\( T_m > 0.80 \)) records resembles the profiles observed in the pushover analysis, whereas the median drift profile for Groups 3 and 4 further clarify the transition of higher mode influence from Group 1 to 5. This trend can be demonstrated more clearly, by plotting the ratio of maximum storey drift profile obtained from IDA to the storey drift profile using EDR. The storey drift profile obtained from EDR for a given displacement ductility is simply the drift profile from lateral load analysis as in Figure 4-10(d), multiplied by the corresponding ductility. The ratios of the drift profiles are presented in Figure 4-15 for \( \mu_{\text{EDR}} \) of 4 and 6; the results are presented only for Groups 1, 3 and 5 for clarity. It is evident that EC8 severely overestimates the drifts for the bottom three storeys of the frame for all three groups; the overestimation of drifts decreases for top storeys due to the influence of higher modes that is found to be highest for the low \( T_m \)-group. The dispersion of the
results increases significantly as the ductility demands increase. In general, the dispersion of maximum drifts is relatively higher for the bottom and top storey of the frame. Relatively lower dispersion is observed for Group 5 records with the mean period close to the fundamental period of the frame.

The influence of the mean period on the higher mode response of the structure is described above with the help of maximum storey drift profile. This represents the assemblage of maximum drifts at a storey for a given record; therefore, the maximum drifts for each storey occur at different instants of time. It would be interesting to observe the drift profile of the frame at the instant when the maximum drift at any storey occurs during the time history analysis. To demonstrate this aspect, two records SJB213 and HWA015-E with relatively low and high $T_m$, respectively, are selected (refer to Figure 4-1 and Figure 4-16). SJB213 was recorded for the Coyote Lake earthquake at a distance of 20 km located in Soil Type C with PGA of 0.108g and $T_m$ of 0.473 s, whilst HWA015-E was recorded from the 1999 Chi-chi earthquake (Taiwan) with a $T_m$ of 0.862 s; other characteristics of the ground motion are as described previously. Drift profiles for both records are captured and presented in Figure 4-17 along with the displacement profile at the instant when the maximum drift in the frame occurs. The records are scaled to produce $\mu_{EDR}$ equal to 4. A clear distinction in the response of the frame subjected to both records can be seen in the drift and displacement profiles, the maximum drift for both records occurs during high acceleration amplitudes of the record. The shape of the displacement profiles for both records further illustrates the influence of higher modes. For the SJB213 record, with low $T_m$, the frame responds under the influence of a higher mode, whereas for HWA015-E, with high $T_m$, the drift profile follows the first mode of vibration; the maximum base shear recorded for both records was found to be 899 kN and 1092 kN respectively.
Figure 4-13: Median, 16\textsuperscript{th} percentile and 84\textsuperscript{th} percentile maximum drift profiles for $\mu_{\text{EDR}} = 1$: a) Group 1 ($T_m < 0.35$) b) Group 2 ($T_m$ range 0.35-0.50) c) Group 3 ($T_m$ range 0.50-0.65) d) Group 4 ($T_m$ range 0.65-0.80) e) Group 5 ($T_m > 0.80$).
Figure 4-14: Median, 16th percentile and 84th percentile maximum drift profiles for $\mu_{\text{EDR}} = 4$: a) Group 1 ($T_m < 0.35$) b) Group 2 ($T_m$ range 0.35-0.50) c) Group 3 ($T_m$ range 0.50-0.65) d) Group 4 ($T_m$ range 0.65-0.80) e) Group 5 ($T_m > 0.80$).
Figure 4-15: Comparison of maximum drift profile for each storey for displacement ductility a) $\mu_{EDR} = 4$; b) $\mu_{EDR} = 6$

Figure 4-16: Acceleration time history for SJB213 component of Coyote Lake 1979 earthquake, with $T_m$ of 0.473 s.
Higher-mode response of the MDF system is analysed herein in the light of the mean period of records. The roof displacement, base shear and maximum drift profiles are therefore examined for a given displacement ductility demand for the frame. As discussed earlier, roof displacements are least influenced by frequency content, due to reasons elaborated upon before. On the other hand, the base shear and the maximum storey drift profiles are significantly influenced by frequency content as the displacement ductility demands imposed on the system increase. Pronounced higher-mode effects were noticed for records with the mean period close to the second period of the frame. In general, maximum dispersion was observed for roof displacements that increased with the increase in ductility demands. On the other hand, minimum dispersion was observed for the base shear demands on the frame, and increased with the increase in ductility demands.

**Figure 4-17:** Drift and displacement profile for the frame at the instant when maximum drift in the frame occurs when subjected to SJB213 and HWA015-E records, scaled to produce a displacement ductility $\mu_{\text{EDR}} = 4$.

In the current version of EC8, computation of the base shear is carried out with the help of spectral information at the fundamental period, for structures with the fundamental period smaller than 2 s or 4 times $T_c$. The base shear thus obtained is
distributed throughout the height of the structure according to the first mode displacement profile. The inelastic displacement profile is computed with the assumption that the displacement profile remains unaltered as the structure undergoes inelastic deformations, in conjunction with the equal displacement rule for structures with a fundamental period higher than $T_c$. The results presented in this chapter for MDF systems highlight the deficiencies of current recommendations. It is demonstrated that the roof displacements reduce with the increase in the imposed ductility demands on the MDF system. However, this reduction in the inelastic displacements is typically carried out in conjunction with a modification of the load pattern to account for shifting of drift demands to the top storey of the frame, as noted previously. In addition to this, the influence of higher modes on the amplification of the base-shear demands is of particular concern; this issue is not dealt with in most seismic design codes including EC8. Nevertheless, direct displacement-based design (Priestley et al., 2007) explicitly recognizes the issue and proposes simplified design equations to account for the influence of ductility on the base shear demand. However, the influence of frequency content is not accounted for; a more inclusive design approach is therefore necessary to accommodate this parameter. This would result in significant improvement in the accuracy of response predictions.

4.4. Concluding Remarks

The inelastic response of SDF and MDF systems is studied in this chapter in the light of the mean period, which is a measure of the frequency content of the ground motion.

For SDF systems, the maximum inelastic response is found to be dependent on the $T_c/T_m$ ratio and on the ductility level. It is observed that if ground motions are categorized based on $T_m$, the inelastic response of SDF systems can be predicted more accurately. This is due to the dependence of the frequency content ($T_m$) of ground motion on magnitude, distance and site conditions. Therefore, it offers a suitable replacement for the traditional ground motion groups using magnitude and distance, or site categories alone. These observations become more prominent for higher ductility demand levels.
For MDF systems, it is observed that $T_m$ significantly influences the dynamic base shear and maximum drift profiles of the five storey frame studied, due to higher mode effects. The influence of higher modes on the dynamic base shear increases as the $T_m$ of ground motions approaches the higher periods of the structure. Similar trends are observed for the maximum storey drift profiles, if ground motions are categorized based on $T_m$. The influence of $T_m$ on the contribution of the higher modes increases with increased ductility demand on the structure.

Overall, assessment of the effects of the mean period of ground motion on SDF and MDF systems presented in this chapter shows that $T_m$ appears to be a good parameter for representing the frequency content (in conjunction with the amplitude-based intensity measures), since this parameter is related to magnitude, distance and site conditions. Moreover, the chapter demonstrates that the inelastic response of MDF systems can be improved with the aid of this parameter. This parameter can be used for developing simplified models that incorporate the dynamic characteristics of MDF system and period of ground motion in order to predict drift and ductility demands with improved accuracy.

The chapter stresses the need for improvement in the representation of spectral shapes in EC8 to accommodate various hazard scenarios for improved estimates of the inelastic response of SDF systems. In addition to this, a more sophisticated approach is deemed necessary in EC8 to account for modifications in response parameters due to inelastic response and higher-mode effects.

The latter part of this Chapter has focused on the influence of the frequency content on MDF through a selected typical frame. This discussion is extended in Chapter 5 and 6 to cover the full range of frames designed in Chapter 3 in order to examine in detail the additional influence of structural characteristics on drift and strength demands.
Chapter 5

Evaluation of Drift Demands in Moment Resisting Frames

5.1. Introduction

This chapter investigates the influence of structural properties and frequency content of ground motion on global and maximum drift demand in frames designed to comply with EC8 provisions. A number of structural parameters are considered herein. These parameters are: height, number of storeys, fundamental period, plasticity resistance ratio, beam-to-column stiffness ratio, and level of inelasticity (measured in terms of the behaviour factor ‘q’). Based on the findings of the literature review, it is evident that there is lack of consensus on the parameters that influence drift demands in MDF structures. Moreover, the interaction of these parameters with frequency content is not fully understood. This chapter aims to provide a detailed investigation into the influence of the parameters mentioned above on drift demands. The study also examined the influence of the relative storey stiffness parameter, which has been proposed in this thesis (as discussed in Chapter 3).

The 40 moment resisting steel frames considered in Chapter 3 are used. To study the influence of the above-mentioned parameters, incremental dynamic analysis is employed by scaling the ground motion to simulate four levels of behaviour factors, typically encountered in the seismic design process. To investigate the influence of frequency content, measured in terms of mean period, $T_m$, of ground motion, 72 far-field records are adopted. It should be recalled that in Chapter 4 a total of 128 far-field records were employed to study the influence of frequency content on SDF systems.
and a single MDF system. The number of records is reduced to 72 to decrease the computational effort.

This chapter, hereafter, is divided into five parts. The first segment briefly presents the information relevant to the 72 far-field records used in the study. The second part discusses the modelling of the frames conducted in OpenSees (2008); the procedure adopted to carry out incremental dynamic analysis, and the definitions of parameters investigated in this chapter are described. The third part of this chapter discusses the detailed parametric study carried out to investigate the influence of various parameters on the global and maximum drift demands. Regression modelling of these parameters is presented in fourth segment. Concluding remarks are finally presented in the last section.

5.2. Ground Motions and Frequency Content

To investigate the influence of ground motion frequency characteristics, 72 far-field records from 21 earthquakes that include a wide range of magnitude, distance and soil conditions (according to the NEHRP classification), are identified. The distribution of earthquake records used in this study with respect to magnitude, distance and site class is shown in Figure 5-1. Only one horizontal component from each recording station is selected. In order to limit the study to far-field ground motion, the records were chosen with rupture distances between 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 along with related information is presented in Table 5-1 (detailed information of records is available in Table B-2 (Appendix B)). Figure 5-2 shows 5%-damped acceleration response spectra for the earthquake records used in this study.

5.3. Nonlinear Dynamic Analysis and Parameters Investigated

As discussed before, the frames were modelled in OpenSees (2008) in order to carry out the nonlinear dynamic analysis. The beam and column elements of the frames were represented by two force-based elements (with 7 Gauss points) per
member. A bilinear stress-strain curve for steel with post-yield stiffness of 0.5% was adopted to account for the material nonlinearity. Vertical loads comprising of dead loads and an allowance of 30% for live loads were applied at mid-span 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 analysis. An initial-stiffness proportional damping was considered with 2% viscous damping assigned to the first mode. A typical OpenSees file for incremental dynamic analysis of 3-storey frame is provided in Appendix-C.

![Figure 5-1: Distribution of magnitude, distance and site conditions for the records used in the study](image)

Incremental dynamic analysis of the frames was conducted by scaling the records with respect to the fundamental period of the frames 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 calculated using Equation 5.1 below:
Table 5-1: 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).

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

\[ S_F = q \times \frac{V_1}{S_a(T_1)} \times m \times \gamma \]  \hspace{1cm} (5.1)

Where \( S_a(T_1) \) is the spectral acceleration of a given record at the fundamental period of the frame; \( V_1 \) is the yield base shear corresponding to the formation of the first plastic 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 \( \gamma \) represents the mass participation ratio corresponding to the first mode.

The ground motions are scaled in order to achieve four behaviour factors: 3, 4, 5 and 6, typically encountered in seismic design. For each analysis, the maximum roof displacement, \( \Delta_{\text{max}} \) and the maximum drift, \( \theta_{\text{max}} \), are recorded. The data from each run of analysis is then processed to determine the following:

I. Global drift modification factor, \( \delta_{\text{mod}} \), computed as the ratio of the maximum roof displacement, \( \Delta_{\text{max}} \), recorded from IDA for a given behaviour factor to the product of behaviour factor, \( q \), and the roof yield displacement, \( \Delta_{1,\text{roof}} \).
(obtained from pushover analysis using a force profile based on the fundamental mode shape of the frame). This can be expressed as:

\[ \delta_{\text{mod}} = \frac{\Delta_{\text{max}}}{q \times \Delta_{1,\text{roof}}} \]  \hspace{1cm} (5.2)

II. Maximum drift modification factor, \( \theta_{\text{mod}} \), computed as the ratio of the maximum storey drift, \( \theta_{\text{max}} \), recorded from IDA for a given behaviour factor to the product of behaviour factor, \( q \), and the maximum storey drift at the formation of the first yield, \( \theta_{1,\text{max}} \) (obtained from pushover analysis using a force profile based on the fundamental mode shape of the frame). This can be expressed as:

\[ \theta_{\text{mod}} = \frac{\theta_{\text{max}}}{q \times \theta_{1,\text{max}}} \]  \hspace{1cm} (5.3)

5.4. Parametric Studies

In this section, the data obtained from the incremental dynamic analysis is processed to investigate the influence of various parameters on the global and maximum drift modification factors.

5.4.1. Global drift modification factor (\( \delta_{\text{mod}} \))

The influence of the behaviour factor and period ratio (\( T_1/T_m \)) is examined first by compiling the data in various \( T_1/T_m \) bins for a particular behaviour factor. Mean values of \( \delta_{\text{mod}} \) are then obtained for the respective bin. As shown in Figure 5-3, the mean \( \delta_{\text{mod}} \) for various \( T_1/T_m \) bins is compared for behaviour factors of 3, 4, 5 and 6. It can be observed that the influence of \( T_1/T_m \) on \( \delta_{\text{mod}} \) can be divided into three zones as: short, intermediate and long. The boundaries of these zones lie roughly at \( T_1/T_m \) ratio of 1 and 2.7. It can be noted that for \( T_1/T_m \) lower than 1, \( \delta_{\text{mod}} \) increases as \( T_1/T_m \) decreases. This trend is similar to that observed for the inelastic response of SDF systems in Chapter 4. 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 \( \delta_{\text{mod}} \). Due to the same reason, \( \delta_{\text{mod}} \) increases with the increase in \( q \) in the short \( T_1/T_m \) range. It can be identified from the difference for \( \delta_{\text{mod}} \) from \( T_1/T_m \) of 1 to 0.5. In
the specified $T_1/T_m$, the change of $\delta_{\text{mod}}$ is found to be 0.13 (=0.93-0.80) and 0.34 (=1.13-0.79) for $q$ of 3 and 5 respectively. 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 $\delta_{\text{mod}}$ is found to be negligible. Moreover, it is observed that the increase in $q$, which results in further elongation of the fundamental period, decreases $\delta_{\text{mod}}$ in this range further. In the long $T_1/T_m$ range, $T_m$ of ground motion is relatively closer to the higher mode periods, particularly the second mode period, due to the elongation of the fundamental period. Due to this reason, the $\delta_{\text{mod}}$ increases as $T_1/T_m$ increases. Apparently, it can be noted that, for a given $T_1/T_m$ in this range, the increase of $q$ results in the corresponding decrease in $\delta_{\text{mod}}$. However, if noted carefully that for $q$ of 3 $\delta_{\text{mod}}$ increases from 0.81 to 0.95 for $T_1/T_m$ of 2.7 to 4.2, whereas for $q$ of 5 $\delta_{\text{mod}}$ increases from 0.77 to 0.93 for $T_1/T_m$ of 2.7 to 4.2. As a result, the change $\delta_{\text{mod}}$ from $T_1/T_m$ of 2.7 to 4.2 for $q$ of 3 and 5 is found to be 0.14 and 0.16 respectively. Therefore, $\delta_{\text{mod}}$ remains relatively less affected by $q$ in this range.

Figure 5-3: Mean global drift modification ($\delta_{\text{mod}}$) factor for various period ratios and behaviour factor bins
The influence of the plasticity resistance ratio ($\alpha$) can be studied by dividing the data mainly in 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 1.71. The value of 1.71 is the average plasticity resistance ratio of all the frames used in the study. Mean values of $\delta_{mod}$, for $q$ of 3, 4, 5 and 6, are presented in Figure 5-4. For the short $T_1/T_m$ range, $\delta_{mod}$ decreases with the increase in $\alpha$. On the other hand, for the intermediate and long $T_1/T_m$ ranges, $\delta_{mod}$ increases with the increase in $\alpha$.

Similarly, the influence of other parameters, namely: beam-to-column stiffness ratio ($\rho$), relative storey stiffness ratio ($\beta_1$, $\beta_2$ and $\beta_3$) and height ($H$) on $\delta_{mod}$ is studied by dividing the data into $T_1/T_m$ bins for $q$ of 3, 4, 5 and 6. To study the influence of $\rho$, $\beta_1$, $\beta_2$ and $\beta_3$, the data is further divided into two groups using the average of the parameters for all frames. The average value of $\rho$, $\beta_1$, $\beta_2$ and $\beta_3$ were found to be 0.14, 0.73, 0.79 and 0.86, respectively. Mean value of $\delta_{mod}$ for various $T_1/T_m$ and $q$ bins is plotted for $\rho$, $\beta_1$, $\beta_2$, $\beta_3$ in Figure 5-5, Figure 5-6, Figure 5-7 and Figure 5-8, respectively. It is noted that there is a slight influence of $\rho$ and $\beta_1$ on $\delta_{mod}$ in the intermediate and long $T_1/T_m$ ranges for $q$ of 3, 4 and 5. However, this trend is not consistent for the short $T_1/T_m$ range and $q$ of 6. Moreover, the trend cannot be explained rationally. Therefore, these parameters are excluded in the regression modelling discussed later in this chapter. On the other hand, there is insignificant influence of $\beta_2$ and $\beta_3$ on $\delta_{mod}$ for all $T_1/T_m$ ranges. To study the influence of the height on $\delta_{mod}$, the data is sub-divided into three groups, based on the number of storeys of the frames. As noted previously, the height corresponding to the three frame groups is 11.5 m, 18.5 m and 25.5 m respectively. Figure 5-9 shows that the height of a frame does not influence $\delta_{mod}$.
Figure 5-4: Mean global drift modification factor ($\delta_{\text{mod}}$) for various period ratios, behaviour factor and plasticity resistance ratio bins.
Figure 5-5: Mean global drift modification factor ($\delta_{\text{mod}}$) for various period ratios, behaviour factor and beam-to-column stiffness ratio bins.
Figure 5-6: Mean global drift modification factor ($\delta_{\text{mod}}$) for various period ratios, behaviour factor and relative storey stiffness ratio ($\beta_1$) bins.
Figure 5-7: Mean global drift modification factor ($\delta_{\text{mod}}$) for various period ratios, behaviour factor and relative storey stiffness ratio ($\beta_2$) bins.
Figure 5-8: Mean global drift modification factor ($\delta_{mod}$) for various period ratios, behaviour factor and relative storey stiffness ratio ($\beta_3$) bins.
Figure 5-9: Mean global drift modification factor ($\delta_{mod}$) for various period ratios, behaviour factor and height bins.
It is relevant to evaluate the scatter of $\delta_{\text{mod}}$ with respect to $T_1/T_m$ and $q$ for various parameters used for parametric studies. To this end, coefficient of variation (COV) is calculated for each parameter using the schemes adopted earlier. COVs for $\alpha$ and $\rho$; $\beta_1$, $\beta_2$ and $\beta_3$; and $H$, are plotted in Figure 5-10, Figure 5-11 and Figure 5-12 respectively. In general, it can be observed from the figures that COV increase with the increase of $q$, whereas it remains relatively less influenced by $T_1/T_m$. This observation is similar to that observed for COV of the inelastic displacement ratios of SDF systems, presented in Figure 4-5. Furthermore, it is observed that COV varies roughly from 0.2 to 0.35 as $q$ increase from 3 to 6.

**Figure 5-10:** COVs for the global drift modification factor ($\delta_{\text{mod}}$) plotted against $T_1/T_m$ and $q$ for $\alpha$ and $\rho$. 
Figure 5-11: COVs for the global drift modification factor ($\delta_{\text{mod}}$) plotted against $T_1/T_m$ and $q$ for $\beta_1$, $\beta_2$ and $\beta_3$
Figure 5-12: COVs for the global drift modification factor ($\delta_{\text{mod}}$) plotted against $T_1/T_m$ and $q$ for $H$

5.4.2. Maximum drift modification factor ($\theta_{\text{mod}}$)

Firstly, the influence of the behaviour factor and the period ratio on $\theta_{\text{mod}}$ is studied by compiling the data, obtained from IDA, into various $T_1/T_m$ bins for four behaviour factors of 3, 4, 5 and 6. The mean value of $\theta_{\text{mod}}$, obtained for the respective bin, is shown in Figure 5-13. Based on the trends shown in the figure, the influence of $T_1/T_m$ ratio can be divided into short, intermediate and long ranges, as observed for $\delta_{\text{mod}}$ in the previous section. The boundaries of these ranges can be identified approximately at 1.0 and 1.7. Therefore, the intermediate $T_1/T_m$ range for $\theta_{\text{mod}}$ is relatively shorter than that of $\delta_{\text{mod}}$. For the short $T_1/T_m$ range, the decrease in $T_1/T_m$ ratio results in the increase in $\theta_{\text{mod}}$ and the increase in $q$ results in the increase of $\theta_{\text{mod}}$, as the elongated fundamental period gets closer to $T_m$ of ground motion. The change of $\theta_{\text{mod}}$ from $T_1/T_m$ 1.0 to 0.5 for $q$ of 3 and 5 is found to be 0.08 and 0.23, respectively. For the intermediate $T_1/T_m$ range, $\theta_{\text{mod}}$ remains uninfluenced by $T_1/T_m$ ratio and decreases with the increase of $q$. For the long $T_1/T_m$ range, $\theta_{\text{mod}}$ increases with the increase in
$T_1/T_m$ ratio. On the other hand, $\theta_{mod}$ is found to be less sensitive to $q$. It can be noted from the change in $\theta_{mod}$ from $T_1/T_m$ of 1.7 to 4.2 for $q$ of 3 and 5, which is found to be 0.29 and 0.26 respectively. In general, it is observed that the influence of $T_1/T_m$ and $q$ on $\theta_{mod}$ is found to be similar to that observed for $\delta_{mod}$.

The influence of plasticity resistance ratio ($\alpha$) is studied by using a grouping procedure as adopted in the case of $\delta_{mod}$. Thus, the data is divided using $T_1/T_m$ bins for four behaviour factors and further divided into two groups using $\alpha$ higher and lower than 1.71. Figure 5-14 shows the mean values of $\theta_{mod}$ for the respective groups. It can be observed that the influence of $\alpha$ on $\theta_{mod}$ is not as significant as in the case of $\delta_{mod}$. However, in general, for the short and long $T_1/T_m$ ranges, the trends show that higher values of $\alpha$ produce lower $\theta_{mod}$. For the intermediate $T_1/T_m$ range, $\theta_{mod}$ remains largely unaffected by $\alpha$. Moreover, it is noted that these trends are not consistent for all the behaviour factors.

**Figure 5-13:** Mean maximum drift modification factor ($\theta_{mod}$) for various period ratios and behaviour factor bins
Figure 5-14: Mean maximum drift modification factor ($\theta_{\text{mod}}$) for various period ratios and behaviour factor and plasticity resistance ratio bins.
As before, the data is also arranged in $T_1/T_m$ bins for four behaviour factors and further sub-divided into two groups using $\rho$ higher and lower than 0.14, to study the influence of beam-to-column stiffness ratio at mid-height of the frame. Figure 5-15 show that the mean values of $\theta_{mod}$ for the two groups. It can be observed that $\theta_{mod}$ remains unaffected by $\rho$.

Figure 5-15: Mean maximum drift modification factor ($\theta_{mod}$) for various period ratios and beam-to-column strength ratio bins
The Influence of the relative storey stiffness ratio is now studied using three definitions (β₁, β₂ and β₃), as discussed earlier. The data is divided using T₁/Tₘ bins for four behaviour factors and further sub-divided into two groups using β₁ higher and lower than 0.73; β₂ higher and lower than 0.79; and β₃ higher and lower than 0.86. Figure 5-16 shows the mean values of θ_{mod}, compared for β₁ higher and lower than 0.73. It can be observed that θ_{mod} is sensitive to β₁ for the long T₁/ Tₘ range only. This trend is more pronounced for q of 3 and reduces as q increases. Similarly, Figure 5-17 compares the mean values of maximum θ_{mod} for β₂ higher and lower than 0.79. The comparison shows that θ_{mod} is quite sensitive to β₂, and increases with the increase in the storey stiffness ratio. In other words, the relatively less stiff storeys at the upper 1/3rd of the frame lead to an increase in the maximum drift demands. Comparison of mean values of θ_{mod}, for β₃ higher and lower than 0.86, is presented in Figure 5-18. It can be observed that the increase in β₃ results in an increase in θ_{mod} for all period ratios. Based on the study, it can be concluded that θ_{mod} is sensitive to β₁, β₂ and β₃; however, the sensitivity is more pronounced for β₂ and β₃. Moreover, either of the two parameters can be used to represent the correlation.

To investigate the influence frame height (H) on θ_{mod}, the data is divided into T₁/Tₘ bins for four behaviour factors and three more categories based on the heights of the frames used in the study, as shown in Figure 5-19. It is noted that θ_{mod} increases as the frame height increases from 11.5 m (3-storey frame) to 18.5 m (5-storey frame), and does not increase further for the 7-storey frame with a height of 25.5 m. It can be concluded that, although there is some influence of height on θ_{mod} for 3 and 5 storey frames, it does not display a consistent trend, which can be taken into account in subsequent prediction modelling.
Figure 5-16: Mean maximum drift modification factor ($\theta_{\text{mod}}$) for various period ratios and relative storey stiffness ratio ($\beta_1$) bins.
Figure 5-17: Mean maximum drift modification factor ($\theta_{\text{mod}}$) for various period ratios and relative storey stiffness ratio ($\beta_2$) bins
Figure 5-18: Mean maximum drift modification factor ($\theta_{mod}$) for various period ratios and relative storey stiffness ratio ($\beta_3$) bins
Figure 5-19: Mean maximum drift modification factor ($\theta_{mod}$) for various period ratios and height bins
COVs are now calculated to study the scatter for $\theta_{\text{mod}}$ with respect to $T_1/T_m$ and $q$ for various parameters using the schemes adopted earlier. COVs for $\alpha$ and $\rho$; $\beta_1$, $\beta_2$ and $\beta_3$; and $H$, are plotted in Figure 5-20, Figure 5-21 and Figure 5-22 respectively. From the figures it is observed that for $T_1/T_m$ lower than 2 (approximately), COV for $\theta_{\text{mod}}$ increases with the increase of $q$ and remains less influenced by $T_1/T_m$. On the other hand, for $T_1/T_m$ greater than 2 (approximately), COV tends to decrease with the increase in $T_1/T_m$ for $q$ of 6 and it tends increase with the increase in $T_1/T_m$ for $q$ of 3, while the other values of $q$ show mixed trends. In general, it is observed that COVs fluctuate from 0.2 to 0.35.

**Figure 5-20:** COVs for the maximum drift modification factor ($\theta_{\text{mod}}$) plotted against $T_1/T_m$ and $q$ for $\alpha$ and $\rho$
Figure 5-21: COVs for the maximum drift modification factor ($\theta_{\text{mod}}$) plotted against $T_1/T_m$ and $q$ for $\beta_1$, $\beta_2$ and $\beta_3$
Based on the parametric studies described in the previous section, prediction models for the global and maximum drift modification factors are proposed and discussed in this section.

5.5.1. Global drift modification factor (\(\delta_{\text{mod}}\))

In the previous section, it was shown that the parameters that influence \(\delta_{\text{mod}}\) are: \(T_1/T_m\), \(q\) and \(\alpha\). In order to simplify the model, \(q_\mu\), which is simply the ratio of \(q\) and \(\alpha\), can be introduced in the model to replace \(q\) and \(\alpha\). Regression analysis is carried out to fit the data using MATLAB, which results in the following model for predicting \(\delta_{\text{mod}}\):

\[
\delta_{\text{mod}} = \exp\left(b_1 + b_2 q_\mu + b_3 q_\mu \sigma_1 + b_4 q_\mu \sigma_2 \right)
\]  

(5.4)
\[ \sigma_1 = \frac{1}{1 + \exp[-b_5(\log(T_1/T_m) - \log (4))]} \]  
\[ \sigma_2 = \frac{\exp[-b_6 q_{\mu}(\log(T_1/T_m) - \log (0.75))]}{[1 + \exp[-b_6(\log(T_1/T_m) - \log (0.75))]]} \]

The regression coefficients for the above equations are presented in Table 5-2.

**Table 5-2: Regression coefficient for the global drift modification ($\delta_{\text{mod}}$) factor**

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
<th>$b_5$</th>
<th>$b_6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.152</td>
<td>0.198</td>
<td>-0.14</td>
<td>-0.115</td>
<td>-7.836</td>
<td>-2.072</td>
</tr>
</tbody>
</table>

The predictions of $\delta_{\text{mod}}$, using the above set of equations, are now plotted for $q_{\mu}$ of 2, 4 and 6 and $T_1/T_m$ range of 0.5 to 4.0 in Figure 5-23. Similarly, 3D depiction of the model, representing the above equations, is presented in Figure 5-24. Figure 5-25 plots the residuals from the model against the parameters used in the study. In general, the residuals do not exhibit significant trends.

The model can now be tested by comparing its predictions with the actual data. The data obtained from incremental dynamic analysis is mainly distributed in various $T_1/T_m$ bins and further sub-divided into four ranges of $q_{\mu}$: i) $q_{\mu} < 2$; ii) $2 \leq q_{\mu} < 3$; iii) $3 \leq q_{\mu} < 4$; iv) $q_{\mu} \geq 4$. The model is simulated using the above equation with $T_1/T_m$ ratio, and mean of $q_{\mu}$ pertaining to the four $q_{\mu}$ ranges (which were found to be 1.73, 2.51, 3.43 and 4.20 respectively). Figure 5-26 shows the plots of mean, 25th, 50th and 75th percentile of the actual data, and the model predictions. It can be observed that the model performs reasonably well for all $T_1/T_m$ ratios and $q_{\mu}$. 

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Figure 5-23: Model predictions for the global drift modification factor ($\delta_{\text{mod}}$) for $q_\mu$ of 2, 4 and 6

Figure 5-24: 3D view of the regression relationship for the global drift modification factor ($\delta_{\text{mod}}$)
Figure 5-25: Plots of residuals for the global drift modification factor ($\delta_{\text{mod}}$) model
Figure 5-26: Comparison of the model predictions with the mean, 25th, 50th and 75th percentile (bottom, intermediate and top levels of the error bars respectively) of the data obtained from IDA for the global drift modification factor ($\delta_{\text{mod}}$) for various $T_1/T_m$ and $q_\mu$. 
5.5.2. Maximum drift modification factor ($\theta_{\text{mod}}$)

Based on the parametric studies carried out in Section 5.4.2, it was identified that $\theta_{\text{mod}}$ is significantly influenced by $T_1/T_m$, $q$ and $\beta_2$ (or $\beta_3$), whereas it is moderately influenced by $\alpha$. Moreover, it was noted that the frame height ($H$) showed some influence on $\theta_{\text{mod}}$, however, the influence of $H$ was found to be inconsistent. Hence, the equation is modelled using $T_1/T_m$, $q$, $\alpha$ and $\beta_3$. It should be noted that, although $\alpha$ showed moderate influence on $\theta_{\text{mod}}$, it is included for the sake of consistency with the previous model for $\delta_{\text{mod}}$. In order to simplify the model, $q_{\mu}$, is used to replace $q$ and $\alpha$, as conducted previously for modelling of $\delta_{\text{mod}}$. Moreover, $\beta_3$ is used to model the variation of the relative storey stiffness of the frame. As discussed before, it should be noted that $\beta_2$ can be used as well for the same purpose.

\[ \theta_{\text{mod}} = \exp \left\{ \left( b_1 + b_2 q_{\mu} + (b_3 q_{\mu} + b_4 \beta_3)\sigma_1 + (b_5 q_{\mu} + b_6 / \beta_3)\sigma_2 \right) \right\} \]  \hspace{1em} (5.7)

\[ \sigma_1 = \frac{1}{1 + \exp\left\{ - (b_7 \beta_3) (\log(T_1/T_m) - b_8) \right\}} \]  \hspace{1em} (5.8)

\[ \sigma_2 = \frac{\exp\left\{ - b_9 q_{\mu} (\log(T_1/T_m) - b_{10}) \right\}}{1 + \exp\left\{ - b_9 q_{\mu} (\log(T_1/T_m) - b_{10}) \right\}} \]  \hspace{1em} (5.9)

The regression coefficients for the above equations are presented in Table 5-3. The predictions of $\theta_{\text{mod}}$, using the above set of equations, are now plotted for $q_{\mu}$ of 2, 4 and 6, $T_1/T_m$ range of 0.4 to 4.0 and $\beta_3$ of 0.86 in Figure 5-27. Similarly, 3D depiction of the model, representing the above equations, is presented in Figure 5-28. Figure 5-29 shows the residuals of the model plotted against the various parameters used in this study, which, in general, do not indicate any significant trends.

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
<th>$b_5$</th>
<th>$b_6$</th>
<th>$b_7$</th>
<th>$b_8$</th>
<th>$b_9$</th>
<th>$b_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.152</td>
<td>0.198</td>
<td>-0.14</td>
<td>-0.115</td>
<td>-7.836</td>
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<td>-0.14</td>
<td>-0.115</td>
<td>-7.836</td>
<td>-2.072</td>
</tr>
</tbody>
</table>
**Figure 5-27:** Model predictions for the maximum drift modification factor ($\theta_{\text{mod}}$) for $q_\mu$ of 2, 4 and 6

**Figure 5-28:** 3D view of the model for the maximum drift modification factor ($\theta_{\text{mod}}$)
Subsequently, the model is tested for its predictive capabilities by comparing with the data obtained from IDA. The comparison is carried out for two cases: i) $T_1/T_m$ and $q_\mu$; ii) $T_1/T_m$, $q_\mu$, and $\beta_3$. For the first case, classification of data is carried out by distributing it in various $T_1/T_m$ bins and further sub-dividing into four ranges of $q_\mu$: i) $q_\mu < 2$; ii) $2 \leq q_\mu < 3$; iii) $3 \leq q_\mu < 4$; iv) $q_\mu \geq 4$. The model is applied using an average $q_\mu$ for each bin, calculated as 1.73, 2.51, 3.43 and 4.20, along with average value of $\beta_3$.

**Figure 5-29:** Plots for residuals for the maximum drift modification factor ($\theta_{\text{mod}}$) model
for all frames (found to be 0.86). Figure 5-30 shows the plots of mean, 25th, 50th and 75th percentile of actual data, and model predictions. It can be observed that the model performs very well.

**Figure 5-30:** Comparison of the model predictions with the mean, 25th, 50th and 75th percentile (bottom, intermediate and top levels of the error bars respectively) of the data obtained from IDA for the maximum drift modification factor ($\theta_{\text{mod}}$), for various $T_{1}/T_{m}$ and $q_{\mu}$

For the second case, the model predictions can be tested for the influence of $\beta_{3}$, by further sub-dividing the data into two groups for $\beta_{3}$ greater and lower than 0.86.
Model predictions are calculated through varying $T_1/T_m$ ratio and average $q_\mu$ for each bin, calculated as 1.73, 2.51, 3.43 and 4.20. An average value of $\beta_3$ for each group ($\beta_3 \leq 0.86$ and $\beta_3 > 0.86$), found to be 0.78 and 0.93 respectively, is used. Plots of mean, 25$^{th}$, 50$^{th}$ and 75$^{th}$ percentile of actual data and model predictions are shown in Figure 5-31 and Figure 5-32. It can be observed that the model predictions generally fit well with the data obtained from IDA.

\textbf{Figure 5-31}: Comparison of the model predictions with the mean, 25$^{th}$, 50$^{th}$ and 75$^{th}$ percentile (bottom, intermediate and top levels of the error bars respectively) of the data obtained from IDA for the maximum drift modification factor ($\theta_{mod}$), for various $T_1/T_m$, $q_\mu$ and $\beta_3$ ($\leq 0.86$)
Figure 5-32: Comparison of the model predictions with the mean, 25th, 50th and 75th percentile (bottom, intermediate and top levels of the error bars respectively) of the data obtained from IDA for the maximum drift modification factor ($\theta_{\text{mod}}$), for various $T_1/T_m$, $q_\mu$ and $\beta_3 (> 0.86)$. 

$q_\mu < 2, \beta_3 > 0.86$

$2 \leq q_\mu < 3, \beta_3 > 0.86$

$q_\mu \geq 4, \beta_3 > 0.86$
5.6. Concluding remarks

This chapter has investigated the influence of structural characteristics, level of inelasticity and ground motion frequency content on the global and maximum drift demands in frames designed to EC8.

It was shown that the global drift modification factor ($\delta_{\text{mod}}$), which is the ratio of the maximum global drift obtained from the nonlinear dynamic analysis to the product of the global drift at the formation of the first yield using pushover analysis and behaviour factor (q), was dependent on the period ratio ($T_1/T_m$), q and plasticity resistance ratio ($\alpha$). Furthermore, the parametric study showed that the influence of $T_1/T_m$ on $\delta_{\text{mod}}$ can be divided in three ranges, namely: short, intermediate and long. It was also noted that the influence of q and $\alpha$ on $\delta_{\text{mod}}$ is also dependent on the $T_1/T_m$ range. Based on the parametric studies, a regression model was developed for prediction of $\delta_{\text{mod}}$ as a function of $T_1/T_m$ and $q_\mu$ ($= q/\alpha$). The parameter $q_\mu$, defined as the ratio of q and $\alpha$, was used to simplify the model. The predictions of the model showed that in the short range ($T_1/T_m \leq 1$, approximately) $\delta_{\text{mod}}$ increased with the decrease in $T_1/T_m$. For $T_1/T_m$ equal to 0.5, $\delta_{\text{mod}}$ was found to be 0.92 and 1.2 for $q_\mu$ of 2 and 6 respectively. In the intermediate range ($1 \leq T_1/T_m \leq 2.7$, approximately), $\delta_{\text{mod}}$ was independent of $T_1/T_m$ and was found to be 0.80 and 0.62 for $q_\mu$ of 2 and 6 respectively. In the long range ($T_1/T_m > 2.7$, approximately), $\delta_{\text{mod}}$ increased with the increase in $T_1/T_m$ and was found to be 0.83 and 0.76 for $q_\mu$ of 2 and 6 respectively, for $T_1/T_m$ of 3.5.

On the other hand, the maximum drift modification ($\theta_{\text{mod}}$), defined as the ratio of the maximum drift obtained from the nonlinear dynamic analysis to the product of the maximum drift at the formation of the first yield using pushover analysis and behaviour factor (q), was found to be strongly dependent on $T_1/T_m$, q and relative storey stiffness ratio (expressed as $\beta_3$) and moderately dependent on $\alpha$ and the frame height (H). Furthermore, it was noted that the influence of $T_1/T_m$ on $\theta_{\text{mod}}$ can be divided in three ranges, as in the case of $\delta_{\text{mod}}$. A regression model was developed for prediction of $\theta_{\text{mod}}$ as a function of $T_1/T_m$, $q_\mu$ and $\beta_3$. The influence of H on was $\theta_{\text{mod}}$ ignored, whereas $\alpha$ was included in order to retain consistency with the model for $\delta_{\text{mod}}$. For an average value of 0.86 for $\beta_3$, the predictions of the model showed that in the short range ($T_1/T_m \leq 1$, approximately) $\theta_{\text{mod}}$ increased with the decrease in $T_1/T_m$. 
For $T_1/T_m$ equal to 0.5, $\theta_{\text{mod}}$ was found to be 0.92 and 1.28 for $q_\mu$ of 2 and 6 respectively. In the intermediate range ($1 \leq T_1/T_m \leq 1.7$, approximately), $\theta_{\text{mod}}$ was found to be relatively less dependent on $T_1/T_m$ and it decreased from 0.88 to 0.68 for $q_\mu$ of 2 and 6 respectively, for $T_1/T_m$ of 1.5. In the long range ($T_1/T_m > 1.7$, approximately), $\theta_{\text{mod}}$ increased with the increase in $T_1/T_m$ and was found to be 0.80 and 1.03 for $q_\mu$ of 2 and 6 respectively, for $T_1/T_m$ of 3.5.

In the next chapter the influence of the frequency content and structural parameters on the base shear, distribution of the storey shear along the frame height and distribution of the storey moment demands along the frame height, is investigated in detail.
Chapter 6

Assessment of Strength Demands

6.1. Introduction

As discussed in the literature review presented in Chapter 2, previous studies have shown that the main parameters that influence strength demands are: the fundamental period ($T_1$), level of inelasticity and frequency content. Furthermore, it was noted that there is a need to develop simple models that incorporate both structural properties and frequency content information to predict strength demands. This chapter aims to demonstrate the influence of these parameters on strength demands in frames designed to comply with EC8 provisions. The level of inelasticity in the frame is assessed using the behaviour factor ($q$) and the plasticity resistance ratio ($\alpha$). In addition to the above mentioned parameters, the relative storey stiffness ratio parameter is also included in the parametric study. This parameter may prove useful, considering that a relatively low stiffness at the top storeys of the frame may lead to earlier yielding of the top storeys, which in turn may lead to an increase in the overall inelasticity in the frame. To evaluate the influence of these parameters, the procedure adopted previously for the evaluation of drift demands is repeated in this chapter. Thus, the 40 moment resisting steel frames discussed in Chapter 3 are subjected to incremental dynamic analysis for four levels of $q$ using the 72 far-field records discussed in Chapter 5. Similarly, the nonlinear models adopted in the previous chapter for the evaluation of drift demands are employed.

This chapter is divided into four parts. The first part provides the definitions of parameters investigated. The second part describes detailed parametric studies carries
out to investigate the influence of various parameters on strength demands. Regression modelling of these parameters is presented in the third part of the chapter. Concluding remarks are finally presented in the last part.

### 6.2. Parameters Investigated

Incremental dynamic analysis (IDA) is conducted by scaling the records with respect to the spectral acceleration corresponding to the fundamental period with respect to four behaviour factors: 3, 4, 5 and 6, as was the case in Chapter 5. From each dynamic analysis, the maximum base shear ($V_{\text{max}}$), maximum storey shear ($V_{\text{i,max}}$) and maximum storey moment ($M_{\text{i,max}}$) are obtained. These quantities can be defined as follows:

1. $V_{\text{max}}$, is the maximum of the total base shear, which is the sum of the shears at all supports of the frame.

2. $V_{\text{i,max}}$, is the maximum of the total shear at the $i^{\text{th}}$ storey of the frame; where the total shear at a given storey is obtained as the sum of shears in all columns of the storey.

3. Since the column moments for a given storey vary from one end to another end, the sum of the maximum moments (observed from the nonlinear dynamic analysis) at the top and the bottom end of the columns is calculated separately and the higher of the two is chosen to be $M_{\text{i,max}}$ for the $i^{\text{th}}$ storey. This procedure is adopted assuming that the column section (design) remains uniform for a particular storey of the frame; therefore, the higher moment will govern the design. Furthermore, it should be noted that the maximum moments in the columns for a given storey may not occur at the same time instant.

In the same way, the base shear, storey shear, and storey moment are calculated from the pushover analysis at the formation of the first yield in the structure to compute the corresponding modification factors (to be discussed later). The definitions are given as follows:
1. Base shear at yield, $V_1$, is computed as the sum of the shears at all supports of the frame at the first yield.

2. Storey shear at yield, $V_{i,1}$, is the total shear at $i^{th}$ storey, calculated as the sum of shears in all columns of the storey at the first yield.

3. Storey moment, $M_{i,1}$, is the highest of the total moment at both ends of the columns at the $i^{th}$ storey at the first yield, which are calculated as the sum of the moments at the given end of all the columns for the given storey.

Using the quantities obtained from dynamic and pushover analysis, the strength demand modification factors, discussed below, are computed:

1. Base shear modification factor, $V_{mod}$, computed as the ratio of maximum base shear, $V_{max}$, recorded from IDA for a given behaviour factor to the product of plasticity resistance ratio, $\alpha$, and base shear at yield, $V_1$ (obtained from pushover analysis using a force profile based on the fundamental mode shape of the frame). This can be expressed as:

$$ V_{mod} = \frac{V_{max}}{\alpha \times V_1} \quad (6.1) $$

2. Storey shear modification factor, $V_{st,mod}$, computed as the ratio of maximum storey shear $V_{i,max}$, at the $i^{th}$ storey, registered from IDA for a given behaviour factor to the product of $\alpha$ and $V_{i,1}$ (obtained from pushover analysis using a force profile based on the fundamental mode shape of the frame). This can be expressed as:

$$ V_{st,mod} = \frac{V_{i,max}}{\alpha \times V_{i,1}} \quad (6.2) $$

3. Storey moment modification factor, $M_{st,mod}$, computed as the ratio of the maximum storey moment $M_{i,max}$, at the $i^{th}$ storey, obtained from IDA for a given behaviour factor to the product of $\alpha$ and $M_{i,1}$ (obtained from pushover analysis using a force profile based on the fundamental mode shape of the frame). This can be expressed as:
\[ M_{st,mod} = \frac{M_{i,max}}{\alpha \times M_{i,y}} \]  

(6.3)

6.3. Parametric studies

In this section, the data obtained from the incremental dynamic analysis is processed to examine the influence of various parameters on the base shear, storey shear and storey moment modification factors.

6.3.1. Base shear modification factor (\(V_{mod}\))

The influence of period ratio (\(T_1/T_m\)) and \(q\) on \(V_{mod}\) is studied first by compiling the data in various \(T_1/T_m\) bins for a particular \(q\). Subsequently, the mean value of the factor is evaluated for the respective bin. Mean \(V_{mod}\) for various \(T_1/T_m\) bins are compared for behaviour factors of 3, 4, 5 and 6, as shown in Figure 6-1. Based on the general trends, the influence of 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 behaviour factor of 3; however, the extent of this zone reduces as a behaviour factor increases. Moreover, it is noted that for the short \(T_1/T_m\) range, \(V_{mod}\) increases with the decrease in \(T_1/T_m\). This behaviour can be attributed to the increase of inelasticity of the structure as a result of the increase in ductility demands (global drift demands) due to the short period effect (as discussed in Chapter 5). In the intermediate \(T_1/T_m\) range, the influence of 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 the increase in \(q\) results in the increase of \(V_{mod}\) for \(T_1/T_m\) ranges.

The influence of \(\alpha\) can be assessed by dividing the data in 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 1.71. It should be noted that the value of 1.71 is the average plasticity resistance ratio of all the frames used in the study. The mean values of \(V_{mod}\), for \(q\) of 3, 4, 5 and 6, are assembled in Figure 6-2. It can be observed that the increase in \(\alpha\) 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 \(\alpha\) results in lower \(V_{mod}\).
The influence of the relative storey stiffness ratio using three definitions ($\beta_1$, $\beta_2$ and $\beta_3$) is now examined. The data is divided using $T_1/T_m$ bins for four behaviour factors and further sub-divided into two groups using average values of $\beta_1$, $\beta_2$ and $\beta_3$ for all the frames in the study (found to be 0.73, 0.79 and 0.86 respectively). Hence, the data is further divided into $\beta_1$ higher and lower than 0.73, $\beta_2$ higher and lower than 0.79 and $\beta_3$ higher and lower than 0.86. Mean values of $V_{mod}$ are firstly compared for $\beta_1$ higher and lower than 0.73 for four behaviour factors, as shown in Figure 6-3. It can be noted that $V_{mod}$ increases with the increase of relative storey stiffness ratio (expressed as $\beta_1$). In other words, it may be inferred that 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 6-4 and Figure 6-5 compare mean values of $V_{mod}$ for $\beta_2$ higher and lower than 0.79 and $\beta_3$ higher and lower than 0.86, respectively. It can be noted that the trends are similar to those observed for $\beta_1$; an increase in $\beta_2$ or $\beta_3$ results in the increase of $V_{mod}$. Furthermore, it can be noted that $V_{mod}$ is relatively more sensitive to $\beta_3$ then $\beta_1$ and $\beta_2$.

**Figure 6-1:** Comparison of the mean base shear modification factor ($V_{mod}$) for various period ratios for behaviour factors of 3, 4, 5 and 6
Figure 6-2: Comparison of mean base shear modification factor ($V_{mod}$) for various period ratios with $\alpha \leq 1.71$ and $\alpha > 1.71$
Figure 6-3: Comparison of mean base shear modification factor ($V_{mod}$) for various period ratios with $\beta_1 \leq 0.73$ and $\beta_1 > 0.73$
**Figure 6-4**: Comparison of mean base shear modification factor ($V_{mod}$) for various period ratios with $\beta_2 \leq 0.79$ and $\beta_2 > 0.79$
Figure 6-5: Comparison of mean base shear modification factor ($V_{mod}$) for various period ratios with $\beta_3 \leq 0.86$ and $\beta_3 > 0.86$
In order to study the scatter of $V_{\text{mod}}$ with respect to $T_1/T_m$ and $q$ for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$, COVs are computed using the data classification schemes developed earlier for the parametric studies, presented in Figure 6-6. It can be observed from the figure that COV for $V_{\text{mod}}$ remains relatively unaffected by $q$ and increases with the increase of $T_1/T_m$. Furthermore, it is noted that COV for $V_{\text{mod}}$ varies approximately from 0.05 to 0.25 for variation of $T_1/T_m$ from 0.3 to 4.2. It is interesting to note that the scatter of data (measured in terms of COV) is lesser for the base shear demands in comparison to the global and maximum drift demands (noted in Chapter 5).

**Figure 6-6:** COV for the base shear modification factor ($V_{\text{mod}}$) for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$
6.4. Storey shear modification factor (V_{st,mod})

To understand the influence of various parameters on the distribution of storey shear modification factors (V_{st,mod}) over the height of the frames, the results are distributed in various normalized heights, H_i/H, (height to storey from the base, H_i, divided by the total height of the frame, H) bins, four behaviour factors (3, 4, 5 and 6), and three T_1/T_m bins (to represent short, intermediate and long T_1/T_m ranges) with the following ranges: 1) T_1/T_m ≤ 1; 2) 1 < T_1/T_m ≤ 2, and 3) T_1/T_m > 2. To investigate the influence of other parameters, the data is further divided based on the respective parameter under consideration.

The influence of T_1/T_m and q is first assessed using the basic classification scheme based on H_i/H, q and T_1/T_m. The mean V_{st,mod} for three T_1/T_m bins with respect to H_i/H for q of 3, 4, 5 and 6 are plotted in Figure 6-7. It can be noted that V_{st,mod} increases along the height of the frame. Moreover, it is observed that for H_i/H lower than 0.75, the factor increases significantly for T_1/T_m greater than 2 in comparison to the other two T_1/T_m ranges. On the other hand, for H_i/H greater than 0.75, the factor increases consistently as T_1/T_m increases. In other words, the influence of the period ratio is more significant at the top storeys of the frame. This trend can be attributed to the higher contribution of shear from the second mode at the top storeys of the frame.

The influence of q on V_{st,mod} is examined by comparing the mean values of the factor for q of 3 and 5, and 4 and 6 for three period ratio bins separately, as shown in Figure 6-8. It is noted that an increase in q (relative intensity) results in a consistent increase in V_{st,mod} over the height of the frame.
Figure 6-7: Comparison of mean storey shear modification factor ($V_{st,mod}$) for various normalized heights for $T_1/T_m \leq 1$, $1 < T_1/T_m \leq 2$ and $T_1/T_m > 2$ (different vertical scales are used in the plots above)
Figure 6-8: Comparison of mean storey shear modification factor ($V_{st,\text{mod}}$) for various normalized heights for behaviour factors of 3 and 5, and 4 and 6 (different vertical scales are used in the plots above).
The influence of $\alpha$ is now assessed by further classifying the data into two groups based on the mean $\alpha$ of all frames used in the study (i.e. 1.71). Thus, the data is classified into two groups consisting of $\alpha$ lower than 1.71 and higher than 1.71. Figure 6-9 plots the mean value of the factor for $q$ of 3 and 5 respectively for three period ratio bins. It can be noted that the increase in $\alpha$ results in a relative decrease in $V_{st,mod}$ due to lower overall inelasticity of the frame, as discussed for the base shear in the previous section.

Similarly, the influence of the relative storey stiffness ratio is observed by classifying the data into two groups based on the mean relative storey stiffness ratio of all frames used in the study (found to be 0.73, 0.79 and 0.86 for $\beta_1$, $\beta_2$ and $\beta_3$ respectively). The mean value of $V_{st,mod}$ based on $\beta_1$, $\beta_2$ and $\beta_3$ for three $T_1/T_m$ bins and $q$ of 3 and 5 is plotted with respect to $H_i/H$ in Figure 6-10, Figure 6-11 and Figure 6-12, respectively. In general, it is observed that the storey shear modification factor is most sensitive to $\beta_3$. Furthermore, it is noted that the increase in relative storey stiffness ratio results in an increase of $V_{st,mod}$.

COVs are computed for $V_{st,mod}$ with respect to $H_i/H$ for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$ and three $T_1/T_m$ groups, using the schemes adopted previously for the parametric studies. Figure 6-13, Figure 6-14 and Figure 6-15 present COVs against $H_i/H$ for $T_1/T_m \leq 1$, $1 < T_1/T_m \leq 2$ and $T_1/T_m > 2$ respectively. In general, it is observed that COV remain constant for $H_i/H$ lower than 0.75; whereas, for $H_i/H$ greater than 0.75, it increases with an increase in $H_i/H$. On the other hand, COV increases with the increase in $T_1/T_m$. It is also noted that COV for $V_{st,mod}$ are insensitive to $q$ for $T_1/T_m \leq 1$ and $1 < T_1/T_m \leq 2$; whereas, for $T_1/T_m > 2$, COV increases moderately with the increase in $q$. Furthermore, it is noted that for $H_i/H$ lower than 0.75 maximum COV ranges approximately from 0.05 to 0.15, whereas, for $H_i/H$ greater than 0.75 maximum COV ranges from approximately from 0.15 to 0.30.
Figure 6-9: Comparison of mean storey shear modification factor ($V_{st,mod}$) for various normalized heights with $\alpha \leq 1.71$ and $\alpha > 1.71$ for behaviour factors of 3 and 5 (different vertical scales are used in the plots above)
Figure 6-10: Comparison of mean storey shear modification factor ($V_{st,mod}$) for various normalized heights with $\beta_1 \leq 0.73$ and $\beta_1 > 0.73$ for behaviour factors of 3 and 5 (different vertical scales are used in the plots above)
Figure 6-11: Comparison of mean storey shear modification factor ($V_{\text{st,mod}}$) for various normalized heights with $\beta_2 \leq 0.79$ and $\beta_2 > 0.79$ for behaviour factors of 3 and 5 (different vertical scales are used in the plots above)
Figure 6-12: Comparison of mean storey shear modification factor \( (V_{st,mod}) \) for various normalized heights with \( \beta_3 \leq 0.86 \) and \( \beta_3 > 0.86 \) for behaviour factors of 3 and 5 (different vertical scales are used in the plots above)
Figure 6-13: COV for the storey shear modification factor ($V_{st,mod}$) for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$ for $T_1/T_m \leq 1$
Figure 6-14: COV for the storey shear modification factor ($V_{st,mod}$) for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$ for $1 < T_1/T_m \leq 2$
Figure 6-15: COV for the storey shear modification factor ($V_{st,mod}$) for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$ for $T_1/T_m > 2$

6.5. Storey moment modification factor ($M_{st,mod}$)

The influence of various parameters on the distribution of the storey moment modification factor ($M_{st,mod}$) over the height of the frames is examined by classifying the data into normalized height ($H_i/H$) bins (as conducted for $V_{st,mod}$). Similarly, the data is categorized in three $T_1/T_m$ bins ($T_1/T_m \leq 1$; $1 < T_1/T_m \leq 2$, and $T_1/T_m > 2$) and four behaviour factors (3, 4, 5 and 6).
Based on the above classification scheme, mean $M_{st,mod}$ for three $T_1/T_m$ bins for $q$ of 3, 4, 5 and 6 separately, are plotted with respect to $H_i/H$ in Figure 6-16. Generally, the trends are similar to those observed for $V_{st,mod}$. It is observed from the plots that $M_{st,mod}$ increases along the height of the frame. Furthermore, it is noted that for $H_i/H$ lower than 0.75, the factor increases significantly for $T_1/T_m$ greater than 2 when compared to the other two period ratio ranges. For $H_i/H$ greater than 0.75, the factor increases consistently as $T_1/T_m$ increases due to contribution from higher modes, particularly from the second mode of vibration.

![Figure 6-16](image.png)

**Figure 6-16:** Comparison of mean storey moment modification factor ($M_{st,mod}$) for various normalized heights for $T_1/T_m \leq 1$, $1 < T_1/T_m \leq 2$ and $T_1/T_m > 2$ (different vertical scales are used in the plots above)
The influence of $q$ on $M_{st,mod}$ can be studied more easily by comparing the mean values of $M_{st,mod}$ for $q$ of 3 and 5, and 4 and 6, for three $T_1/T_m$ bins separately, as shown in Figure 6-17. It can be observed from the plots that an increase in $q$ (relative intensity) results in a consistent increase in $M_{st,mod}$.

The influence of $\alpha$ is also assessed by further classifying the data into two groups comprising of $\alpha$ lower than 1.71 and higher than 1.71 (as conducted previously). Figure 6-18 compares the mean value of $M_{st,mod}$ for $q$ of 3 and 5 for three $T_1/T_m$ bins. The plots show that the increase in the plasticity resistance ratio results in a relative decrease in $M_{st,mod}$ due to the same reasons discussed in previous sections.

Likewise, the influence of the relative storey stiffness ratio is studied by classifying the data into two groups based on the mean relative storey stiffness ratio of all frames used in the study (using the classification of data adopted for $V_{st,mod}$). The mean value of $M_{st,mod}$ based on $\beta_1$, $\beta_2$ and $\beta_3$ for three $T_1/T_m$ bins and behaviour factor of 3 is plotted with respect to $H_i/H$ in Figure 6-19, Figure 6-20 and Figure 6-21 respectively. In general, the trends show that $M_{st,mod}$ is relatively more sensitive to $\beta_3$ in comparison to $\beta_1$ and $\beta_2$. It is noted that the increase in relative storey stiffness ratio results in an increase of storey moment modification factor.

In order to study the scatter of $M_{st,mod}$, COVs are computed for with respect to $H_i/H$ for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$ and three $T_1/T_m$ groups, using the data bins adopted in the parametric studies. Figure 6-22, Figure 6-23 and Figure 6-24 present COVs against $H_i/H$ for $T_1/T_m \leq 1$, $1 < T_1/T_m \leq 2$ and $T_1/T_m > 2$ respectively. In general, it is observed that COV is moderately dependent on $H_i/H$ for $H_i/H$ lower than 0.75; whereas, for $H_i/H$ greater than 0.75, it increases significantly with an increase in $H_i/H$. On the other hand, it is noted that COV increases with the increase in $T_1/T_m$, particularly for $H_i/H$ greater than 0.75. COV for $M_{st,mod}$ is found to be dependent on $q$ for and $1 < T_1/T_m \leq 2$ and $T_1/T_m > 2$ and increases with the with the increase in $q$. In contrast, for $T_1/T_m \leq 1$, COV is found to be insensitive to $q$. Furthermore, it is noted that for $H_i/H$ lower than 0.75 COV ranges between 0.05 and 0.20 approximately. For $H_i/H$ equal to 1, COV is found to be as high as 0.40 and as low as 0.20.
Figure 6-17: Comparison of storey moment modification factor ($M_{s,\text{mod}}$) for various normalized heights for behaviour factors of 3 and 5, and 4 and 6 for: a) $T_1/T_m \leq 1$; b) $1 < T_1/T_m \leq 2$; c) $T_1/T_m > 2$ (different vertical scales are used in the plots above)
Figure 6-18: Comparison of mean storey moment modification factor (M_{st,mod}) for various normalized heights with α ≤ 1.71 and α > 1.71 for behaviour factors of 3 and 5 (different vertical scales are used in the plots above)
Figure 6-19: Comparison of mean storey moment modification factor ($M_{st,mod}$) for various normalized heights with $\beta_1 \leq 0.73$ and $\beta_1 > 0.73$ for behaviour factors of 3 and 5 for $T_1/T_m \leq 1$; $1 < T_1/T_m \leq 2$; $T_1/T_m > 2$ (different vertical scales are used in the plots above)
Figure 6-20: Comparison of mean storey moment modification factor ($M_{st,mod}$) for various normalized heights with $\beta_2 \leq 0.79$ and $\beta_2 > 0.79$ for behaviour factors of 3 and 5 for $T_1/T_m \leq 1$; $1 < T_1/T_m \leq 2$; $T_1/T_m > 2$ (different vertical scales are used in the plots above)
Figure 6-21: Comparison of mean storey moment modification factor ($M_{st,mod}$) for various normalized heights with $\beta_3 \leq 0.86$ and $\beta_3 > 0.86$ for behaviour factors of 3 and 5 for $T_1/T_m \leq 1$; $1 < T_1/T_m \leq 2$; $T_1/T_m > 2$ (different vertical scales are used in the plots above)
**Figure 6-22:** COV for the storey moment modification factor ($M_{st,mod}$) for $\alpha$, $\beta_1$, $\beta_2$ and $\beta_3$ for $T_1/T_m \leq 1$
Figure 6-23: COV for the storey moment modification factor (\(M_{sl,mod}\)) for \(\alpha\), \(\beta_1\), \(\beta_2\) and \(\beta_3\) for \(1 < \frac{T_1}{T_m} \leq 2\)
**Figure 6-24**: COV for the storey moment modification factor (\(M_{st,mod}\)) for \(\alpha\), \(\beta_1\), \(\beta_2\) and \(\beta_3\) for \(T_1/T_m > 2\)
6.6. Prediction Models

Based on the parametric studies described in the previous section, regression modelling of the base shear modification factor, storey shear modification factor, and storey moment modification factor is carried out in MATLAB (2011), as discussed hereafter.

6.6.1. Base shear modification factor ($V_{\text{mod}}$)

In previous sections, it was shown that the parameters that influence $V_{\text{mod}}$ are: $T_1/T_m$, $q$, $\alpha$ and $\beta_3$ ($\beta_1$ or $\beta_2$). In order to simplify the model, $q_\mu$, which is simply the ratio of $q$ and $\alpha$, can be introduced in the model to replace $q$ and $\alpha$. $\beta_3$ is preferred over $\beta_1$ and $\beta_2$, considering the relatively higher sensitivity of this parameter on $V_{\text{mod}}$.

The following model is obtained based on the regression analysis of the data:

$$V_{\text{mod}} = \exp \left[ b_1 \beta_3 + b_2 q_\mu + \left( \frac{b_3}{q_\mu} + b_4 \beta_3 \frac{b_5}{\sigma_1} \right) \sigma_1 + \left( b_6 q_\mu + b_7 \beta_3 \right) \sigma_2 \right]$$

(6.1)

$$\sigma_1 = \frac{1}{1 + \exp\{ -b_8 (\log(T_1/T_m) - \log(2.3)) \}}$$

(6.2)

$$\sigma_2 = \frac{1}{1 + \exp\{ -b_9 (\log(T_1/T_m) - \log(0.80)) \}}$$

(6.3)

The regression coefficients for the above equations are presented in Table 6-1. The predictions of $V_{\text{mod}}$, using the above set of equations, are now plotted for $q_\mu$ of 2, 4 and 6, $T_1/T_m$ range of 0.5 to 4.0 and $\beta_3$ of 0.86 in Figure 6-25. 3D representation of the model, generated using $\beta_3$ of 0.86, is shown in Figure 6.26. Figure 6-27 shows the residuals of the model plotted against the various parameters used in this study, which, in general, do not indicate any significant trends.

Table 6-1: Regression coefficient for the base shear modification factor ($V_{\text{mod}}$)

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
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<tbody>
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Figure 6.25: Model predictions for the base shear modification factor ($V_{\text{mod}}$) for $q_\mu$ of 2, 4 and 6

Figure 6.26: The base shear modification factor ($V_{\text{mod}}$) presented as a function of $q_\mu$ and $T_1/T_m$ using an average value of $\beta_3$ of 0.86
The model is subsequently tested by carrying out a comparison between its predictions and the data obtained from IDA. The data obtained from IDA is mainly distributed in various $T_1/T_m$ ratio bins and further sub-divided in four ranges of $q_\mu$: i) $q_\mu < 2$; ii) $2 \leq q_\mu < 3$; iii) $3 \leq q_\mu < 4$; iv) $q_\mu \geq 4$. The model is simulated using the above equation with the $T_1/T_m$ ratio, and mean of $q_\mu$ pertaining to the four $q_\mu$ ranges that were found to be 1.73, 2.51, 3.43 and 4.20 respectively. Figure 6-28 shows the

Figure 6-27: Residual for base shear modification factor ($V_{\text{mod}}$) model plotted against various parameters used in the study
plots of mean, 25th, 50th and 75th percentile of the actual data for: i) $q_\mu$ less than 2; ii) $q_\mu$ ranging between 3 and 4, and corresponding model predictions. It can be observed that model predictions fit the data reasonably well.

Figure 6-28: Comparison of model predictions with mean, 25th, 50th and 75th percentile base shear modification factor ($V_{mod}$) (different vertical scales are used in the plots above)
Similarly, the model can be tested for the variation in $\beta_3$. Thus, the data is further sub-divided in two groups based on average $\beta_3$ of all frames, as applied in the parametric study. Figure 6-29 and Figure 6-30 depict the mean, 25th, 50th and 75th percentile of actual data with $\beta_3$ larger and smaller than 0.86, and the corresponding model prediction using $\beta_3$ of 0.93 and 0.80 (mean corresponding to both groups of $\beta_3$). It can be noted that the predictions generally correlate well with the data.

**Figure 6-29:** Comparison of model predictions with mean, 25th, 50th and 75th percentile base shear modification factor ($V_{\text{mod}}$) for $\beta_3 \leq 0.86$ (different vertical scales are used in the plots above)
Figure 6-30: Comparison of model predictions with mean, 25th, 50th and 75th percentile base shear modification factor ($V_{mod}$) for $\beta_3 > 0.86$ (different vertical scales are used in the plots above)
6.6.2. Storey shear modification factor ($V_{st,mod}$)

Based on the parametric studies conducted earlier, $V_{st,mod}$ can be modelled as a function of: $T_1/T_m$, $q$, $\alpha$, $\beta_3$ and $H_i/H$. In order to simplify the model, $q_\mu$ is used to replace $q$ and $\alpha$.

$$V_{st,mod} = \exp\left[\left(\frac{\sigma_1}{\sigma_2} + \sigma_3\right)\right]$$

(6.4)

$$\sigma_1 = b_1 + b_2 q_\mu + b_3 \beta_3 + b_4 \left[\frac{(H_i/H)^2}{\beta_3} + b_5 (H_i/H)\right]$$

(6.5)

$$\sigma_2 = 1 + \exp\left[-b_6 \left(\frac{T_1}{T_m} - b_7\right)\right]$$

(6.6)

$$\sigma_3 = b_8 + q_\mu \log\left(\frac{T_1}{T_m}\right)$$

(6.7)

The regression coefficients for the above equations are presented in Table 6-2. The predictions of $V_{st,mod}$ using the above set of equations, are now plotted for $q_\mu$ of 2, 4 and 6, $T_1/T_m$ range of 0.5 to 4.0, an average value of $\beta_3$ of 0.86 and $H_i/H$ of 0.33, 0.66 and 1.0 in Figure 6-31. Figure 6-32 presents the three-dimensional representation of $V_{st,mod}$ for the roof storey (i.e. $H_i/H = 1$) using the above equations, by varying the $T_1/T_m$ and $q_\mu$ while using $\beta_3$ of 0.86. Figure 6-33 shows the variation of $V_{st,mod}$ along the height of the frames. The plot is developed by varying $T_1/T_m$ and $H_i/H$, while $q_\mu$ is taken as 3, and with an average $\beta_3$ for all frames. Figure 6-34 plots the residuals of the model against various parameters used in the study; it can be observed that there are no significant trends in the residual plots.

**Table 6-2: Regression coefficient for the storey shear modification factor ($V_{st,mod}$)**

<table>
<thead>
<tr>
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</thead>
<tbody>
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</tbody>
</table>
Figure 6-31: The storey shear modification factor ($V_{st,mod}$) plotted against $T_1/T_m$ for $H_i/H$ of 0.33, 0.66 and 1.0 (different vertical scales are used in the plots above)
Figure 6-32: Storey shear modification factor ($V_{st,mod}$) at roof ($H_r/H = 1$) presented as a function of $q_\mu$ and $T_1/T_m$ using an average value of $\beta_3 = 0.86$.

Figure 6-33: Storey shear modification factor ($V_{st,mod}$) for $q_\mu = 3$ presented as a function of $H_r/H$ and $T_1/T_m$ using an average value of $\beta_3 = 0.86$. 
The model predictions can now be compared with those obtained from the dynamic analysis. The comparison is carried out for two cases: 1) $q_\mu$ and $T_1/T_m$; 2) $q_\mu$, $T_1/T_m$ and $\beta_3$. For the first case, the data is categorized in four groups based on $q_\mu$ ($q_\mu < 2; 2 \leq q_\mu < 3; 3 \leq q_\mu < 4; q_\mu \geq 4$), and three $T_1/T_m$ groups ($T_1/T_m \leq 1; 1 < T_1/T_m \leq 2, \text{ and } T_1/T_m > 2$) in addition to normalized height bins. Model estimates are calculated using the average of the parameters relevant to each group. Therefore, an average $q_\mu$ of 1.73, 2.51, 3.43 and 4.20 is used respectively for four groups of $q_\mu$, and an average $T_1/T_m$ of 0.75, 1.5 and 2.8 respectively is adopted for three $T_1/T_m$.
groups along with an average value of $\beta_3$ for all frames (0.86). Figure 6-35 shows the comparison of plots of mean, 25th, 50th and 75th percentile of actual data for $q_\mu$ less than 2, and $q_\mu$ ranging between 3 and 4, with the corresponding model predictions. It can be observed that the model predictions generally fit well with the data.

Figure 6-35: Comparison of model predictions with mean, 25th, 50th and 75th percentile storey shear modification factor ($V_{st,mod}$) for a) $q_\mu < 2$ and $T_1/T_m \leq 1$; b) $q_\mu < 2$ and $1 < T_1/T_m \leq 2$; c) $q_\mu < 2$ and $T_1/T_m > 2$; d) $3 \leq q_\mu < 4$ and $T_1/T_m \leq 1$; e) $3 \leq q_\mu < 4$ and $1 < T_1/T_m \leq 2$; f) $3 \leq q_\mu < 4$ and $T_1/T_m > 2$ (different vertical scales are used in the plots above)
For the second case, the data classification from the first case is modified by further sub-dividing the data in two groups based on $\beta_3$ (as adopted in the parametric study). Figure 6-36 plots the comparison of mean, 25th, 50th and 75th percentile of actual data for $q_\mu$ less than 2 (for $\beta_3$ less than or equal to 0.86 and $\beta_3$ greater than 0.86). In general, the predictions of the model are in agreement with the data. However, it is observed that the model over predicts $V_{st,\text{mod}}$ for $T_1/T_m$ lower than unity for $q_\mu$ less than 2 and $\beta_3$ less than or equal to 0.86.

**Figure 6-36:** Comparison of model predictions with mean, 25th, 50th and 75th percentile storey shear modification factor ($V_{st,\text{mod}}$) for $q_\mu < 2$ with a) $T_1/T_m \leq 1$ and $\beta_3 \leq 0.86$; b) $1 < T_1/T_m \leq 2$ and $\beta_3 \leq 0.86$; c) $T_1/T_m > 2$ and $\beta_3 \leq 0.86$; d) $T_1/T_m \leq 1$ and $\beta_3 > 0.86$; e) $1 < T_1/T_m \leq 2$ and $\beta_3 > 0.86$; f) $T_1/T_m > 2$ and $\beta_3 > 0.86$ (different vertical scales are used in the plots above)
6.6.3. Storey moment modification factor \( (M_{\text{st,mod}}) \)

Based on the parametric studies, \( M_{\text{st,mod}} \) is modelled as a function of: \( T_1/T_m \), \( q \), \( \alpha \), \( \beta_3 \) and \( H_i/H \). In order to simplify the model, \( q_{\mu} \) is used to replace \( q \) and \( \alpha \).

\[
M_{\text{st,mod}} = \exp\left(\frac{\sigma_1}{\sigma_2} + \sigma_3\right) \quad (6.8)
\]

\[
\sigma_1 = b_1 + b_2 q_{\mu} + b_3 \beta_3 + b_4 (H_i/H)^2 + b_5 (H_i/H) \times \beta_3^{b_6} \quad (6.9)
\]

\[
\sigma_2 = 1 + \exp\left(-b_7 \left(\frac{T_1}{T_m} - b_8\right)\right) \quad (6.10)
\]

\[
\sigma_3 = b_9 + q_{\mu} \log\left(\frac{T_1}{T_m}\right) \quad (6.11)
\]

The regression coefficients for the above equations are presented in Table 6-3. The predictions of \( M_{\text{st,mod}} \), using the above set of equations, for \( q_{\mu} \) of 2, 4 and 6, \( T_1/T_m \) range of 0.5 to 4.0, an average value of \( \beta_3 \) of 0.86 and \( H_i/H \) of 0.33, 0.66 and 1.0 are plotted in Figure 6-37. The three-dimensional representation of \( M_{\text{st,mod}} \) for the roof storey (i.e. \( H_i/H = 1 \)) is generated using the above equations by varying \( T_1/T_m \) and \( q_{\mu} \) while using an average value of \( \beta_3 \) for all frames (0.86), as shown in Figure 6-38. Similarly, variation of \( M_{\text{st,mod}} \) along the height of the frames is developed by varying \( T_1/T_m \) and \( H_i/H \) for \( q_{\mu} \) equal to 3, and using the average \( \beta_3 \) for all frames, as shown in Figure 6-39. Residuals of the model against various parameters used in the study are plotted in Figure 6-40. It is noted that the residuals do not exhibit any significant trends.

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Table 6-3: Regression coefficient for the storey moment modification \( (M_{\text{st,mod}}) \) factor
Figure 6-37: The storey moment modification factor ($M_{st,mod}$) plotted against $T_1/T_m$ for $H_i/H$ of 0.33, 0.66 and 1.0 (different vertical scales are used in the plots above)
Figure 6-38: Storey moment modification factor ($M_{st,mod}$) at roof ($H/H = 1$) presented as a function of $q_\mu$ and $T_1/T_m$ using an average value of $\beta_3 = 0.86$

Figure 6-39: Storey moment modification factor ($M_{st,mod}$) for $q_\mu = 3$ presented as a function of $H/H$ and $T_1/T_m$ using an average value of $\beta_3 = 0.86$
Figure 6-40: Residual for storey moment modification factor ($M_{st,mod}$) model plotted against various parameters used in the study

The model predictions are now compared with the data obtained from IDA. The approach adopted for the comparison of $M_{st,mod}$ is identical to that adopted for $V_{st,mod}$. Thus, the comparison has been conducted for two cases; the details of data classification and the parameters adopted for model prediction are as provided in the previous section. For the first case, Figure 6-41 shows the comparison of plots of mean, 25th, 50th and 75th percentile of actual data for $q_{\mu}$ less than 2, and $q_{\mu}$ ranging between 3 and 4, with corresponding model predictions. For the second case, Figure 6-42 plots the comparison of mean, 25th, 50th and 75th percentile of actual data for $q_{\mu}$ less than 2 for $\beta_3$ less than or equal to 0.86 and $\beta_3$ greater than 0.86. In general, it is
observed that the model for prediction are less accurate for $T_1/T_m$ less than or equal to 1, whereas it performs reasonably well for the $T_1/T_m$ greater than 1. Furthermore, it is noted that the model prediction for $M_{s,t,mod}$ are relatively less accurate in comparison to $V_{mod}$ and $V_{s,mod}$.

Figure 6-41: Comparison of model predictions with mean, 25th, 50th and 75th percentile storey moment modification factor ($M_{s,t,mod}$) for a) $q_\mu < 2$ and $T_1/T_m \leq 1$; b) $q_\mu < 2$ and $1 < T_1/T_m \leq 2$; c) $q_\mu < 2$ and $T_1/T_m > 2$; d) $3 \leq q_\mu < 4$ and $T_1/T_m \leq 1$; e) $3 \leq q_\mu < 4$ and $1 < T_1/T_m \leq 2$; f) $3 \leq q_\mu < 4$ and $T_1/T_m > 2$ (different vertical scales are used in the plots above)
Figure 6-42: Comparison of model predictions with mean, 25th, 50th and 75th percentile storey moment modification factor ($M_{st,mod}$) for $q_\mu < 2$ with a) $T_1/T_m \leq 1$ and $\beta_3 \leq 0.86$; b) $1 < T_1/T_m \leq 2$ and $\beta_3 \leq 0.86$; c) $T_1/T_m > 2$ and $\beta_3 \leq 0.86$; d) $T_1/T_m \leq 1$ and $\beta_3 > 0.86$; e) $1 < T_1/T_m \leq 2$ and $\beta_3 > 0.86$; f) $T_1/T_m > 2$ and $\beta_3 > 0.86$ (different vertical scales are used in the plots above)
6.7. Concluding Remarks

In this chapter, the influence of various structural parameters, level of inelasticity and frequency content of ground motion records on base shear, storey shear and storey moment demands of the frames is examined and evaluated in terms of the modification factors. In essence, the modification factors proposed in this chapter compare the strength demands obtained from the nonlinear dynamic analysis with the strength demands obtained from the approximated pushover analysis.

Base shear modification factor \( (V_{\text{mod}}) \), defined as the ratio of the maximum base shear obtained from the dynamic analysis to the product of the base shear at the first yield using pushover analysis with the plasticity resistance ratio \( (\alpha) \), was found to be dependent on period ratio \( (T_1/T_m) \), behaviour factor \( (q) \), \( \alpha \) and relative storey stiffness ratio (expressed as \( \beta_3 \)). It was noted that the influence of \( T_1/T_m \) can be divided in three ranges: short, intermediate and long, as noted in the case of \( \delta_{\text{mod}} \) and \( \theta_{\text{mod}} \). Furthermore, it was noted that, for all \( T_1/T_m \) ranges, the increase in \( q \) or \( \beta_3 \) resulted in an increase in \( V_{\text{mod}} \), whereas an increase in \( \alpha \) resulted in the decrease of \( V_{\text{mod}} \). Subsequently, a regression model was developed as a function of \( T_1/T_m \), \( q_{\mu} (= q/\alpha) \) and \( \beta_3 \). Using the regression model, for an average value of \( \beta_3 (= 0.86) \) and \( q_{\mu} \) of 4, \( V_{\text{mod}} \) was found to be 1.2, 1.15, 1.55 for \( T_1/T_m \) of 0.5, 1.0 and 4.0 respectively.

Storey shear modification factor \( (V_{\text{st,mod}}) \) for a given storey, defined as the ratio of the maximum storey shear obtained from the dynamic analysis to the product of the maximum storey shear at the first yield using pushover analysis with \( \alpha \), was found to be dependent on \( T_1/T_m \), \( q \), \( \alpha \), \( \beta_3 \) and the normalized height \( (H_i/H) \). From the observed that \( V_{\text{st,mod}} \) increased significantly as \( H_i/H \) approached to 0.75. The increase in \( V_{\text{st,mod}} \) was more prominent for long \( T_1/T_m \) range due to large contribution from higher modes. Moreover, it was observed that \( V_{\text{st,mod}} \) for a given storey increased with an increase in \( q \) or \( \beta_3 \) and decreased with the increase of \( \alpha \). A regression model was developed as a function of \( T_1/T_m \), \( q_{\mu} \), \( \beta_3 \) and \( H_i/H \). Using the regression model, for an average value of \( \beta_3 (= 0.86) \) and \( q_{\mu} \) of 4, \( V_{\text{st,mod}} \) was found to be 1.15 and 1.40 for \( T_1/T_m \) of 0.5 and 4.0 respectively for \( H_i/H \) of 0.33; 1.10 and 1.50 for \( T_1/T_m \) of 0.5 and 4.0 respectively for \( H_i/H \) of 0.66; and 1.15 and 4.0 for \( T_1/T_m \) of 0.5 and 4.0 respectively for \( H_i/H \) of 1.0.
Storey moment modification factor \((M_{st,mod})\) for a given storey, defined as the ratio of the maximum storey moment obtained from the dynamic analysis to the product of the maximum storey moment at the first yield using pushover analysis with \(\alpha\), was found to be dependent on \(T_1/T_m\), \(q\), \(\alpha\), \(\beta_3\) and \(H_i/H\). Furthermore, it was observed that \(M_{st,mod}\) for a given storey increased with an increase in \(q\) or \(\beta_3\) and decreased with the increase of \(\alpha\), as observed for \(V_{st,mod}\). Subsequently, a regression model was developed for prediction of \(M_{st,mod}\) as a function of \(T_1/T_m\), \(q_{\mu}\), \(\beta_3\) and \(H_i/H\). Using the regression model, for an average value of \(\beta_3\) (= 0.86) and \(q_{\mu}\) of 4, \(M_{st,mod}\) was found to be 1.40 and 1.90 for \(T_1/T_m\) of 0.5 and 4.0 respectively, for \(H_i/H\) of 0.33; 1.50 and 2.20 for \(T_1/T_m\) of 0.5 and 4.0 respectively, for \(H_i/H\) of 0.66; and 1.90 and 4.0 for \(T_1/T_m\) of 0.5 and 4.0 respectively, for \(H_i/H\) of 1.0.

The implication of the findings in this chapter on the current European and US provisions are discussed in Chapter 8. The next chapter focuses on the evaluation of the design relative storey stiffness ratio to achieve uniform drift demands at the upper and lower half of the frame for a given fundamental period, behaviour factor and frequency content scenario.
Chapter 7

Evaluation of Design Relative Storey Stiffness

7.1. Introduction

In Chapter 5, it was demonstrated that the distribution of stiffness along the height of the frame, which was measured in terms of the relative storey stiffness, plays an important role in the resulting maximum drift demands of the frame. It was shown that relatively stiffer top storeys result in reducing the maximum drift demands and vice versa. Furthermore, it was demonstrated that, among the various definitions proposed to determine the relative storey stiffness, the maximum drift demands were more sensitive to the relative storey stiffness ratio, $\beta_3$. This parameter is evaluated using the maximum storey drift (obtained from the first mode shape obtained from Eigenvalue analysis) of the upper and lower half of the frame. This leads to an important question from a design perspective, as to what should be the design value of the relative storey stiffness parameter, which would result in more uniform distribution of drift demand for a given frame and frequency content scenario. Therefore, this chapter focuses on evaluating the design relative storey stiffness, $\beta_{d}$, of this parameter for a given frequency content, behavior factor (q) and fundamental period ($T_1$) of structure that would result in approximately equal drift demands at the upper and lower half of the frame.

To this end, three frames of 3, 5 and 7 storeys are identified, as reference frames, from the database of moment resisting steel frames discussed in Chapter 3. Four variations (including the reference frame) of relative storey stiffness of each frame are obtained by modifying the stiffness of the upper half of the frames. Furthermore, the
The seismic mass of these frames is scaled to produce six fundamental period scenarios of 0.5 s and 1.0 s for 3 storey frames; 0.75 s and 1.25 s for 5 storey frames, and 1.0 s and 1.5 s for 7 storey frames. A total of 72 far-field ground motion records used earlier in Chapters 5 and 6, with mean periods ranging from 0.31 to 0.98, are classified in three groups to develop three frequency content scenarios. IDA is conducted to scale the ground motion in order to develop four behavior factors of 3, 4, 5 and 6. Thus, in total, there are 288 (6 x 4 x 3 x 4) scenarios, which is a product of 6 fundamental periods, 4 relative storey stiffness ratios, 3 frequency contents and 4 behavior factors.

For each case, i.e. for a frame with a given $T_1$, $q$ and frequency content, $\beta_d$ is evaluated. Subsequently, a simple expression is proposed for estimating the relative stiffness ratio, required to achieve uniform storey drifts, as a function of these parameters. Finally, the application of this parameter within force based design is proposed.

### 7.2. Description of Frames

Three MRFs of 3, 5 and 7 storey frames namely: A02, B12 and C11 are identified from the database of frames discussed in Chapter 3. In order to obtain variation of relative storey stiffness, the stiffness of the upper storeys of the reference frame is modified. Four variations (including the reference frame) are obtained for each case. The design details of the frames used in this study are shown in Table 7-1, Table 7-2 and Table 7-3. Normalized drift profiles ($\theta_i/\theta_{max}$) of the frames, obtained from Eigenvalue analysis, are presented in Figure 7-1, Figure 7-2 and Figure 7-3. Table 7-4, Table 7-5 and Table 7-6 present the structural characteristics of the 3, 5 and 7 storey frames, respectively. The structural characteristics include $\beta_3$, $T_1$, second mode period ($T_2$) and $\alpha$ of frames. It is interesting to note that the variation of $\beta_3$ of the frame does not significantly influence the $T_1$. For example, $T_1$ of the 3-storey frame changes from 0.42 s to 0.45 s as the relative storey stiffness changes from 0.73 to 1.02. This trend can be used effectively in the design of frames, as discussed later on.

The fundamental period of the frames is modified by scaling the seismic masses of the frames in order to understand the influence of $T_1$ on $\beta_d$; two variations of $T_1$ are attained for each frame. These variations are decided based on the variation of $T_1$ of
frames used earlier in the study (as discussed in Chapter 3). The fundamental periods of the 3, 5 and 7 storeys are modified to 0.5 s and 1.0 s; 0.75 s and 1.25 s, and 1.0 s and 1.5 s, respectively. Therefore, in total, 24 frames (12 x 2) are designed to evaluate $\beta_d$.

**Table 7-1**: Design details of the 3-storey frames

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<td>IPE550</td>
<td>HEM550</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>IPE550</td>
<td>HEM550</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floor</th>
<th>R710</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>IPE300</td>
<td>HEB300</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>IPE400</td>
<td>HEB550</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>IPE500</td>
<td>HEB550</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>IPE550</td>
<td>HEB550</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>IPE550</td>
<td>HEM550</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>IPE550</td>
<td>HEM550</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>IPE550</td>
<td>HEM550</td>
<td></td>
</tr>
</tbody>
</table>
Figure 7-1: Normalized drift profile of the 3-storey frames

Figure 7-2: Normalized drift profile of the 5-storey frames
Figure 7-3: Normalized drift profile of the 7-storey frames

Table 7-4: Structural properties of the 3-storey frames. Solid box in the table indicates the reference frame

<table>
<thead>
<tr>
<th>Structural Properties</th>
<th>Frame ID</th>
<th>R37</th>
<th>R38</th>
<th>R39</th>
<th>R310</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta_3 )</td>
<td></td>
<td>0.73</td>
<td>0.79</td>
<td>0.92</td>
<td>1.02</td>
</tr>
<tr>
<td>( T_1 )</td>
<td></td>
<td>0.42</td>
<td>0.43</td>
<td>0.44</td>
<td>0.45</td>
</tr>
<tr>
<td>( T_2 )</td>
<td></td>
<td>0.12</td>
<td>0.12</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>( \alpha )</td>
<td></td>
<td>1.43</td>
<td>1.43</td>
<td>1.37</td>
<td>1.43</td>
</tr>
</tbody>
</table>
Table 7-5: Structural properties of the 5-storey frames. Solid box in the table indicates the reference frame

<table>
<thead>
<tr>
<th>Structural Properties</th>
<th>Frame ID</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R57</td>
</tr>
<tr>
<td>$\beta_3$</td>
<td>0.70</td>
</tr>
<tr>
<td>$T_1$</td>
<td>0.94</td>
</tr>
<tr>
<td>$T_2$</td>
<td>0.3</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1.49</td>
</tr>
</tbody>
</table>

Table 7-6: Structural properties of the 7-storey frames. Solid box in the table indicates the reference frame

<table>
<thead>
<tr>
<th>Structural Properties</th>
<th>Frame ID</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R77</td>
</tr>
<tr>
<td>$\beta_3$</td>
<td>0.71</td>
</tr>
<tr>
<td>$T_1$</td>
<td>1.1</td>
</tr>
<tr>
<td>$T_2$</td>
<td>0.36</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1.69</td>
</tr>
</tbody>
</table>

7.3. Frequency Content Scenarios

To incorporate the influence of frequency content, 72 far-field ground motion records are employed. It would be difficult to evaluate $\beta_d$ for all the records used in the study. Therefore, ground motion are records have been classified into three groups (pertinent information shown in Table 7-7) to develop three scenarios. The acceleration spectra of 72 records for three groups are shown in Figure 7-4.
Table 7-7: Grouping of ground motions according to $T_m$

<table>
<thead>
<tr>
<th>Group Number</th>
<th>$T_m$ Range</th>
<th>Number of records</th>
<th>Mean $T_m$</th>
<th>1st Quartile (Q1)</th>
<th>3rd Quartile (Q3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>0.32 s - 0.50 s</td>
<td>24</td>
<td>0.41</td>
<td>0.38</td>
<td>0.45</td>
</tr>
<tr>
<td>Group 2</td>
<td>0.52 s - 0.70 s</td>
<td>24</td>
<td>0.60</td>
<td>0.55</td>
<td>0.64</td>
</tr>
<tr>
<td>Group 3</td>
<td>0.71 s – 0.98 s</td>
<td>24</td>
<td>0.80</td>
<td>0.75</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Figure 7-4: PGA-Normalized acceleration spectra of suites of records used in the study: a) Group 1 ($T_m$ range: 0.32-0.50); b) Group 2 ($T_m$ range: 0.52-0.70); c) Group 3 ($T_m$ range: 0.71-0.98) and d) median acceleration spectra of all groups
7.4. Evaluation of Design Relative Storey Stiffness ($\beta_d$)

This section discusses the procedure employed to evaluate $\beta_d$. To this end, each frame is subjected to IDA using the 72 records for four behavior factors. Nonlinear modelling of the frames is carried out in OpenSees (2008) as described previously in Chapter 5. IDA is conducted by scaling the records with respect to $T_1$ of the frames to attain various levels of relative intensity (as adopted previously in Chapters 5 and 6). The scaling factor, $S_F$, required for an individual record to attain a given $q$ is calculated using Equation 5.1. The ground motions are scaled in order to achieve four behaviour factors: 3, 4, 5 and 6. For a given frame, the maximum drift at each storey of the frame, $\theta_{si, \text{max}}$, is recorded for each analysis.

Median values of these parameters are evaluated for each frequency content scenario ($T_m$ groups) and behaviour factor. Median values are used subsequently to develop $\theta_{si, \text{max}}$ and maximum drift modification factors ($\theta_{\text{mod}}$), which is computed using the following equation:

$$\theta_{\text{mod}} = \frac{\theta_{\text{max}}}{q \times \theta_{1, \text{max}}}$$

(7.1)

In the above equation, $\theta_{\text{max}}$ is the maximum inter-storey drift recorded from IDA for a given behaviour factor; $\theta_{1, \text{max}}$ is the maximum inter-storey drift obtained from pushover analysis using a force profile based on the fundamental mode shape of the frame; and $q$ is the behaviour factor.

Subsequently, the median value of the maximum drift profile and $\theta_{\text{mod}}$ is evaluated corresponding to each frequency content scenario and $q$, for a given frame. Using the median maximum drift profile for each scenario, the relative storey drift ratio, $\chi_e$, is calculated. This parameter can be defined as the ratio between the maximum storey drift for the upper and lower half of the frame, obtained from the dynamic analysis. Thus, for uniform drift demands at upper and lower half of the frame, the relative storey drift ratio should be equal to unity.

The aforementioned procedure is repeated for four variations of a relative stiffness parameter, $\beta_3$, for a given $T_1$, number of storeys ($N$) and frequency content scenario,
to obtain $\chi$. Using four sets of $\beta_3$ and $\chi$, interpolation is applied to find $\beta_d$, for a relative storey drift ratio, $\chi$, of unity.

This procedure is demonstrated for a 7-storey frame with $T_1$ of 1.0 s and for a $q$ of 4. The median $\theta_{st,max}$ for R77, R78, R79 and R710 with $\beta_3$ of 0.71, 0.79, 0.90 and 1.0, respectively, is plotted for three frequency content scenarios, namely: Group 1, Group 2 and Group 3 in Figure 7-5, Figure 7-6 and Figure 7-7, respectively. The median maximum drift profiles are normalized using the maximum drift at the bottom storey of the frame. It can be observed that an increase in the relative storey stiffness ratio results in increase in the drift demands in the upper half of the frame. Furthermore, it is noted that this increase in drift demands of upper storeys of the frame is also a function of the frequency content scenario. Drift demands at upper storeys increase significantly for Group-1 records with median $T_m$ of 0.41 s, while it reduces for Group-2 and Group-3 with median $T_m$ of 0.60 s and 0.80 s, respectively. On the other hand, it is observed that the drift demands at the lower half of the frame generally remain stable, and are less influenced by the variation in $\beta_3$.

Using median maximum drift profiles, $\chi$ can be computed as the ratio of the maximum drift demands for the upper half of the frame with maximum drift demand for the lower half of the frame. The resulting $\chi$ is now plotted against $\beta_3$ in Figure 7-8. It can be observed that $\chi$ increases significantly due to increase in $\beta_3$. Furthermore, it is also important to note that unnecessary increase in the stiffness of top storeys, leading to a low $\beta_3$, results in very low drift demands at the upper half of the frame. This is particularly true for the Group-3 frequency content scenario with relatively high $T_m$ records. Using this plot, $\beta_d$ can be evaluated by setting $\chi$ as unity. Thus, $\beta_d$ for the given frame (with 7-storey; period of 1.0 s and behavior factor of 4) is found to be 0.83, 0.88 and 0.99 for Group 1, Group 2 and Group 3, respectively. Maximum drift modification factor can be obtained subsequently using $\beta_d$, as shown in Figure 7-9. The maximum drift modification factors are evaluated for the whole frame using Equation 7.1, which are found to be 0.76, 0.79 and 0.83 for Group 1, Group 2 and Group 3 frequency content scenarios, respectively.
Figure 7-5: Median of the normalized maximum drift profile for the 7-storey frame for Group-1 frequency content scenario (with median $T_m$ of 0.41 s)

Figure 7-6: Median of the normalized maximum drift profile for the 7-storey frame for Group-2 frequency content scenario (with median $T_m$ of 0.60 s)
**Figure 7-7:** Median of the normalized maximum drift profile for the 7-storey frame for Group-3 frequency content scenario (with median $T_m$ of 0.80 s)

**Figure 7-8:** Comparison of influence of relative storey stiffness ratio, $\beta_3$, on relative storey drift ratio, $\chi$, for three frequency content scenarios for four 7-storey frames
7.5. Sensitivity analysis and Modelling

This section discusses the influence of the $T_1$, $T_m$ and $q$, on the design relative storey stiffness ratio. Subsequently, a prediction model for $\beta_d$ is proposed.

$\beta_d$ is evaluated using the procedure discussed in the previous section, and plotted against the period ratio ($T_1/T_m$) for $q$ of 3, 4, 5 and 6 respectively in Figure 7-10, Figure 7-11, Figure 7-12 and Figure 7-13. Polynomial trend lines are used to facilitate a better understanding. The Y-intercept of the trend line is set to one, assuming that there would be no effect of higher modes as the $T_1/T_m$ ratio approaches zero. It is noted that, for all behaviour factors, $\beta_d$ decreases with an increase in $T_1/T_m$. This trend is predictable, considering that an increase in $T_1/T_m$ results in increased higher mode effects in the upper storeys; hence, a lower $\beta_d$ (stiffer upper storeys) is required to counter the effect.
**Figure 7-10:** Plot of design relative storey drift ratio, $\beta_d$, against $T_1/T_m$ ratio for q of 3

\[ \beta_d = -0.021(T_1/T_m)^2 - 0.018(T_1/T_m) + 1 \]

$R^2 = 0.83$

**Figure 7-11:** Plot of design relative storey drift ratio, $\beta_d$, against $T_1/T_m$ ratio for q of 4

\[ \beta_d = -0.012(T_1/T_m)^2 - 0.038(T_1/T_m) + 1 \]

$R^2 = 0.735$
Figure 7-12: Plot of design relative storey drift ratio, $\beta_d$, against $T_1/T_m$ ratio for q of 5

$$\beta_d = -0.0007(T_1/T_m)^2 - 0.069(T_1/T_m) + 1$$

$R^2 = 0.609$

Figure 7-13: Plot of design relative storey drift ratio, $\beta_d$, against $T_1/T_m$ ratio for q of 6

$$\beta_d = 0.004(T_1/T_m)^2 - 0.088(T_1/T_m) + 1$$

$R^2 = 0.49$
In order to understand the influence of $q$ on $\beta_d$, polynomial trend lines obtained earlier in Figure 7-10, Figure 7-11, Figure 7-12 and Figure 7-13 for the four behaviour factors are plotted together in Figure 7-14. In general, it is noted that $\beta_d$ decreases with an increase in $q$. However, this behaviour is slightly inconsistent and less significant.

**Figure 7-14**: Polynomial trend lines for behaviour factor of 3, 4, 5 and 6

Based on the sensitivity analysis shown earlier, it is noted that $\beta_d$ is strongly dependent on $T_1/T_m$ and weakly dependent on $q$. This parameter can be modelled in two possible ways: 1) modelling it as a function of period ratio and behaviour factor; 2) modelling it as a function of period ratio only. The second option is preferred in order to ensure simplicity of the model (as shown in Figure 7-15). Moreover, a linear model is proposed as shown in Equation 7.2. Residuals of the model are plotted against $T_1/T_m$ and $q_\mu$ in Figure 7-16.

$$\beta_d = 1 - 0.0714 \left( \frac{T_1}{T_m} \right)$$  \hspace{1cm} (7.2)
Figure 7-15: Modelling design relative storey stiffness ratio ($\beta_d$) as a function of period ratio ($T_1/T_m$)

Figure 7-16: Residuals of model against: a) $T_1/T_m$; b) $q_\mu$
The design relative storey stiffness obtained from Equation 7.2 can be used in conjunction with the models presented in Chapters 5 and 6 for predicting the maximum drift and strength demands of the structure. However, a simpler model can be developed using the maximum drifts for a given $\beta_d$, as shown in Figure 7-9. Regression analysis is carried out to fit the data using MATLAB, to obtain the following model to predict $\theta_{\text{mod}}$:

$$\theta_{\text{mod}} = \exp\left(b_1 + b_2 q_\mu + b_3 q_\mu \sigma_1 + b_4 q_\mu \sigma_2\right)$$

$$\sigma_1 = \frac{1}{1 + \exp\left(-b_5 (\log(T_1/T_m) - \log(4))\right)}$$

$$\sigma_2 = \frac{\exp\left[-b_6 q_\mu \left(\log(T_1/T_m) - \log(0.75)\right)\right]}{1 + \exp\left(-b_6 (\log(T_1/T_m) - \log(4))\right)}$$

The regression coefficients in the above equations are presented in Table 7-8. It should be noted here that $q_\mu$, which is simply the ratio of $q$ and $a$, is employed as conducted previously to simplify the model. It was noted previously from Table 7-1, Table 7-2 and Table 7-3 that $a$ of frames is relatively insensitive to modification of $\beta_3$. Therefore, average values of 1.41, 1.47 and 1.62 are used to calculate $q_\mu$ for four variations of the 3, 5 and 7 storey frames respectively.

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
<th>$b_5$</th>
<th>$b_6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.304</td>
<td>-0.104</td>
<td>0.295</td>
<td>0.311</td>
<td>2.04</td>
<td>0.647</td>
</tr>
</tbody>
</table>

The data obtained from IDA is distributed in various $T_1/T_m$ bins and further subdivided four ranges of $q_\mu$: i) $q_\mu < 2$; ii) $2 \leq q_\mu < 3$; iii) $3 \leq q_\mu < 4$; iv) $q_\mu \geq 4$. The model is used to predict $\theta_{\text{mod}}$ using the above equation with $T_1/T_m$, and mean of $q_\mu$ pertaining to the four $q_\mu$ ranges, which were found to be 1.98, 2.65, 3.32 and 3.98 respectively. Figure 7-17 shows the plots of mean, 25th, 50th and 75th percentile of actual data, and model predictions. It can be observed that the model generally performs well. Residuals of the model are plotted against period ratio ($T_1/T_m$) and $q_\mu$ in Figure 7-18.
Figure 7-17: Comparison of the model predictions with mean, 25th, 50th and 75th percentile of the data obtained from IDA for maximum drift modification factor ($\theta_{\text{mod}}$) for various $T_1/T_m$ and $q_\mu$. 
Concluding Remarks

As discussed earlier the current version of Eurocode does not propose any restrictions to achieve a certain level of stiffness or strength at the top storeys of the frame, unlike US code that proposes a parabolic load pattern. However, in US provisions, the lateral load pattern is a function of height and fundamental period of structure; hence the influence of frequency content of ground motion is ignored.

The study presented in this chapter has examined and illustrated the role of the relative stiffness of the upper half of the frame with respective the bottom half, fundamental period (\(T_1\)), behaviour factor (\(q\)) and frequency content of records on the distribution of the maximum drift demands along the height of the frame. Subsequently, design relative storey ratio (\(\beta_d\)), is proposed a parameter to achieve uniform drift demands at the upper and lower half of a given frame. Furthermore, a simple model is presented to evaluate the parameter for a given \(T_1\), \(q\) and \(T_m\). This parameter can be used effectively either to achieve uniform drift demands at the top and bottom storeys of the frame or to limit the top storey drift demands in relation to bottom storeys. In other words, it can be used as an optimizing or limiting parameter.
This parameter can be accommodated easily in the current code provisions as a design check. Thus, the strength and stiffness can be distributed initially using the lateral load pattern prescribed by a given code. The relative storey stiffness ratio of the designed frame can be checked using the displacement profile from Eigenvalue analysis, and compared with the design value using Equation 7.2. Subsequently, it can be modified to match the design value of the parameter by modifying the stiffness of the upper half of the frame. This modification will not affect the overall design of the frame, considering that a modification in relative storey stiffness does not affect the $T_1$ and $\alpha$ significantly. On the other hand, models for prediction of maximum drift demands proposed by EC8 and US codes need to be revised, as was discussed in Chapter 5. A simpler expression is proposed in this chapter for frames which are designed to satisfy design relative storey stiffness ratio evaluated in this chapter.

The next chapter discusses the implications of the findings of the drift and strength demands, discussed in Chapters 5, 6 and 7, on the current European and US seismic assessment and design provisions.
Chapter 8

Implications on Seismic Design and Assessment Provisions

8.1. Introduction

This chapter aims to investigate the applications and implications of the predictive relationships developed in this thesis on seismic design and assessment procedures and provisions. The chapter is divided into three parts. The first part reviews the predictive models developed in this thesis for the drift and strength demands. The second part aims to compare the proposed predictive relationships for drift and strength demands with existing methods, with emphasis on European and US provisions. In the third part of the chapter, modifications are suggested for the existing design provisions.

8.2. Review of Predictive Models

This section presents a brief review of the predictive models developed in this thesis. It comprises two sub-sections, which are devoted to the drift demands and strength demands of frames, respectively.

8.2.1. Drift Demands

The drift demands play a key role in the seismic design and assessment process. In this thesis, two drift demands are addressed, namely: the global drift ($\theta_r$) and maximum drift ($\theta_{\text{max}}$). The global drift serves as an indicator for second order effects
and global ductility that, in turn, can be correlated to the structural damage. On the other hand, the maximum drift, which refers to the maximum inter-storey drift for a given structure, serves as an indicator for non-structural damage.

The drift demands are studied in terms of modification factors. The global drift modification factor ($\delta_{\text{mod}}$) is defined as the ratio of the maximum roof displacement obtained from the nonlinear time history analysis ($\Delta_{\text{max}}$) to the product of the roof displacement at the formation of the first yield in the frame ($\Delta_{1,\text{roof}}$) and behaviour factor ($q$), shown in Equation 8.1. Similarly, the maximum drift modification factor ($\theta_{\text{mod}}$) is defined as the ratio of the maximum drift obtained from the nonlinear time history analysis ($\theta_{\text{max}}$) to the product of the maximum drift at the formation of the first yield in the frame ($\theta_{1,max}$) and behaviour factor ($q$), shown in Equation 8.2.

$$\delta_{\text{mod}} = \frac{\Delta_{\text{max}}}{q \times \Delta_{1,\text{roof}}} \quad (8.1)$$

$$\theta_{\text{mod}} = \frac{\theta_{\text{max}}}{q \times \theta_{1,max}} \quad (8.2)$$

Based on the parametric study conducted in Chapter 5, the following predictive relationships were proposed for $\delta_{\text{mod}}$ (shown in Equations 8.3 to 8.5) and $\theta_{\text{mod}}$ (shown in Equations 8.6 to 8.8). The coefficients for the set of equations for $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ are given in Table 8-1 and Table 8-2 respectively.

$$\delta_{\text{mod}} = \exp\left(b_1 + b_2 q_{\mu} + b_3 q_{\mu} \sigma_1 + b_4 q_{\mu} \sigma_2\right) \quad (8.3)$$

$$\sigma_1 = \frac{1}{1 + \exp\left(-b_5 \left(\log(T_1/T_m) - \log(4)\right)\right)} \quad (8.4)$$

$$\sigma_2 = \frac{\exp\left[-b_6 q_{\mu} \left(\log(T_1/T_m) - \log(0.75)\right)\right]}{1 + \exp\left(-b_6 \left(\log(T_1/T_m) - \log(4)\right)\right)} \quad (8.5)$$
Table 8-1: Regression coefficients for the global drift modification factor (δ_{mod})

<table>
<thead>
<tr>
<th>b_1</th>
<th>b_2</th>
<th>b_3</th>
<th>b_4</th>
<th>b_5</th>
<th>b_6</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.152</td>
<td>0.198</td>
<td>-0.14</td>
<td>-0.115</td>
<td>-7.836</td>
<td>-2.072</td>
</tr>
</tbody>
</table>

θ_{mod} = \exp\{b_1 + b_2 q_\mu + (b_3 q_\mu + b_4 \beta_3)\sigma_1 + (b_5 q_\mu + b_6 / \beta_3)\sigma_2\} \quad (8.6)

\sigma_1 = 1/[1 + \exp\{-(b_7 \beta_3)(\log(T_1/T_m) - b_9)\}] \quad (8.7)

\sigma_2 = \frac{\exp\{-b_9 q_\mu (\log(T_1/T_m) - b_{10})\}}{1 + \exp\{-b_9 q_\mu (\log(T_1/T_m) - b_{10})\}} \quad (8.8)

Table 8-2: Regression coefficients for the maximum drift modification factor (θ_{mod})

<table>
<thead>
<tr>
<th>b_1</th>
<th>b_2</th>
<th>b_3</th>
<th>b_4</th>
<th>b_5</th>
<th>b_6</th>
<th>b_7</th>
<th>b_8</th>
<th>b_9</th>
<th>b_{10}</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.152</td>
<td>0.198</td>
<td>-0.14</td>
<td>-0.115</td>
<td>-7.836</td>
<td>-2.072</td>
<td>-0.14</td>
<td>-0.115</td>
<td>-7.836</td>
<td>-2.072</td>
</tr>
</tbody>
</table>

In order to predict θ_{roof}, using Equations 8.1 and 8.3 to 8.5 presented above, the following parameters are required: behaviour factor (q), roof displacement at the first yield (Δ_{1,roof}), ultimate behaviour factor (q_{\mu}), fundamental period (T_1), and mean period (T_m). On the other hand, for the prediction of θ_{max}, using Equations 8.2 and 8.6 to 8.8, the following parameters are required: q, maximum drift in the frame at the first yield (θ_{1,max}), q_\mu, T_1, T_m and the relative storey stiffness ratio (β_3). The above mentioned parameters are evaluated using various methods discussed in detail subsequently, from the seismic assessment and design perspectives.

8.2.1.1. Seismic assessment

From a seismic assessment perspective, the above mentioned parameters can be evaluated relatively easily using the readily available structural configuration of the existing frame and the seismic hazard scenario.

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Using the existing structural design and loadings, Eigenvalue analysis can be performed to evaluate $T_1$, $\beta_3$ (using the drift profile for the first mode) and the first-mode mass participation ratio ($\gamma$) (to be used subsequently for the calculation of $q$).

The seismic hazard scenario is typically represented in terms of the elastic response spectrum, which is used to evaluate the corner period ($T_c$) and the spectral acceleration at fundamental period $S_a (T_1)$. The relationship between $T_m$ and $T_c$ (developed in Chapter 4), as shown in Equation 8.9, can be employed subsequently to predict $T_m$.

$$T_m = 1.05 T_c$$ (8.9)

In order to evaluate $q$, Equation 8.10 can be used, which is the ratio of the elastic base shear ($V_e$) and the base shear at the first yield ($V_1$); in this case, $V_e$ is calculated as the product of $S_a (T_1)$, total seismic mass of the structure ($m$) and $\gamma$. $q_\mu$ can be calculated as the ratio of the $q$ and the plasticity resistance ratio ($\alpha$), using Equation 8.11; whereas $\alpha$ is defined as the ratio of the ultimate base shear ($V_y$) to $V_1$, as shown in Equation 8.12.

$$q = \frac{S_a (T_1) \times m \times \gamma}{V_1} = \frac{V_e}{V_1}$$ (8.10)

$$q_\mu = \frac{S_a (T_1) \times m \times \gamma}{V_y} = \frac{V_e}{V_y} = \frac{q}{\alpha}$$ (8.11)

Whereas,

$$\alpha = \frac{V_y}{V_1} = \frac{q_\mu}{q}$$ (8.12)

Therefore, in order to calculate $q$ and $q_\mu$, the following parameters are required: $S_a (T_1)$, $m$, $\gamma$, $V_1$ and $V_y$. Parameters other than $V_1$ and $V_y$ have been discussed above already. In case of seismic assessment of existing structures, pushover analysis is a viable option, and it can be utilized to determine $V_1$ and $V_y$. Similarly, pushover analysis can be used to estimate $\Delta_{1,\text{roof}}$ and $\theta_{1,\text{max}}$, as illustrated in Figure 8.1.
Figure 8.1: a) Elastic response spectrum for a given seismic hazard scenario used for calculation of $S_a(T_1)$ and $T_c$; b) Pushover curve plotted for base shear against the roof displacement for a given frame for calculation of $V_1$, $V_y$ and $\Delta_{1,\text{roof}}$; c) Inter-storey drift profile of a 3-storey frame at the formation of the first yield used for determining $\theta_{1,\text{max}}$. 

\[
\alpha = \frac{V_y}{V_1} \\
\Delta_{1,\text{roof}} = V_e = S_a(T_1) \times m \times \gamma \\
q = \frac{V_e}{V_1} \\
q_u = \frac{V_e}{V_y}
\]
8.2.1.2. Seismic design

For seismic design, a similar procedure is adopted as discussed earlier for evaluating the following parameters for the prediction of drift demands: $T_1$, $\gamma$, $T_m$, $S_a(T_1)$ and $m$. However, it should be noted that due to the iterative nature of the seismic design procedure, pushover analysis is not a feasible option to evaluate $V_1$, $V_y$, $\Delta_{1,\text{roof}}$ and $\theta_{1,\text{max}}$. To overcome this difficulty, basic principles of mechanics can be employed to estimate these parameters.

It should be noted that in the design process, $V_e$ is obtained from the elastic response spectrum and this quantity is then reduced with $q$ recommended by code provisions (as in the case of EC8, for instance) to obtain the reduced design base shear ($V_d$). In an ideal situation, for EC8, $V_d$ should be equal to $V_1$ (this issue is discussed further in Section 8.3.1.2). 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. In order to obtain $V_1$, $V_d$ is distributed vertically using a load pattern proposed by the design code, such as the following equation proposed by EC8.

$$F_i = V_d \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \quad (8.13)$$

where, $F_i$ is the horizontal force acting on storey $i$; $m_i$ 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.

Subsequently, the application of the lateral forces along the frame height is used to evaluate the moment demands in beams and columns. On the other hand, the moment capacity of the beams can be computed. The ratio of the moment demands due to applied loads to the moment capacity of the beams is calculated. The minimum of these ratios (referred as the design overstrength, $\Omega_d = V_1/V_d$) can be used to scale $V_d$ to calculate $V_1$. Subsequently, $\Delta_{1,\text{roof}}$ and $\theta_{1,\text{max}}$ can be computed by distributing $V_1$ along the frame height using Equation 8.13. On the other hand, the mechanism of plastic hinge formation can be used to estimate $V_y$ attained by the frame under lateral loading. From the study of the formation of the plastic mechanism, it can be observed that $V_y$ is attained when hinges form at the supports of the bottom storey columns, as shown by Elghazouli (2010). Therefore, $V_y$ can be determined approximately using...
the moment capacity and point of contra-flexure of the bottom storey columns, when a given frame is subjected to lateral loading along with gravity loads. Subsequently, \(\alpha\) can be computed using the ratio of \(V_y\) and \(V_1\). The parameters discussed above can now be used for the prediction of \(\delta_{\text{mod}}\) using Equations 8.1 and 8.3 to 8.5.

The prediction of \(\theta_{\text{mod}}\) requires an additional parameter \(\beta_3\). In the case of seismic design, the designer has a choice to modify the relative storey stiffness ratio by modifying the stiffness of the upper half of the frame, which will result in uniform drift demands for the frame. Therefore, in Chapter 7, the design relative storey stiffness ratio \((\beta_d)\) was modelled for a given period ratio \((T_1/T_m)\), as shown below:

\[
\beta_d = 1 - 0.0714 \left( \frac{T_1}{T_m} \right) \quad (8.14)
\]

Subsequently, \(\theta_{\text{mod}}\) can either be predicted using Equations 8.2 and 8.6 to 8.8 by replacing \(\beta_3\) with \(\beta_d\) along with the other parameters, or using a simpler model developed in Chapter 7 as shown in Equations 8.15 to 8.17. The regression coefficients for the model are provided in Table 8-3.

\[
\theta_{\text{mod}} = \exp(b_1 + b_2 q_0 + b_3 q_0 \sigma_1 + b_4 q_0 \sigma_2) \quad (8.15)
\]

\[
\sigma_1 = \frac{1}{1 + \exp[-b_5 (\log(T_1/T_m) - \log(4))]} \quad (8.16)
\]

\[
\sigma_2 = \frac{\exp[-b_6 q_0 (\log(T_1/T_m) - \log(0.75))]}{[1 + \exp[-b_6 (\log(T_1/T_m) - \log(4))]]} \quad (8.17)
\]

**Table 8-3:** Regression coefficients for the maximum drift modification factor \((\theta_{\text{mod}})\) using the design relative storey stiffness parameter \((\beta_d)\)

<table>
<thead>
<tr>
<th>(b_1)</th>
<th>(b_2)</th>
<th>(b_3)</th>
<th>(b_4)</th>
<th>(b_5)</th>
<th>(b_6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.304</td>
<td>-0.104</td>
<td>0.295</td>
<td>0.311</td>
<td>2.04</td>
<td>0.647</td>
</tr>
</tbody>
</table>
8.2.2. Strength Demands

The strength demands imposed on the columns play an important role in the seismic design and assessment of structures. In this thesis, three strength demand parameters were studied, namely: the base shear, storey shear and storey moment demands. The strength demands were studied in terms of modification factors defined below:

The base shear modification factor \( V_{\text{mod}} \) is defined as the ratio of the maximum base shear obtained from the nonlinear dynamic analysis \( V_{\text{max}} \) to the product of \( V_1 \) and \( \alpha \), as shown in Equation 8.18. The storey shear modification factor \( V_{st,\text{mod}} \) is defined as the ratio of maximum storey shear \( V_{i,\text{max}} \) at the \( i^{th} \) storey, registered from nonlinear dynamic analysis to the product of \( \alpha \) and the storey shear at the \( i^{th} \) storey at the formation of the first yield \( V_{i,1} \), as shown in Equation 8.19. The storey moment modification factor \( M_{st,\text{mod}} \) is defined as the ratio of the maximum storey moment \( M_{i,\text{max}} \) at the \( i^{th} \) storey, registered from nonlinear dynamic analysis to the product of \( \alpha \) and the storey moment at the \( i^{th} \) storey at the formation of the first yield \( M_{i,1} \), as shown in Equation 8.20. It should be noted that the maximum storey moment is calculated by adding up the moments at the top and bottom of the columns separately, and the higher of the two is selected.

\[
V_{\text{mod}} = \frac{V_{\text{max}}}{\alpha \times V_1} \quad (8.18)
\]

\[
V_{st,\text{mod}} = \frac{V_{i,\text{max}}}{\alpha \times V_{i,1}} \quad (8.19)
\]

\[
M_{st,\text{mod}} = \frac{M_{i,\text{max}}}{\alpha \times M_{i,1}} \quad (8.20)
\]

The predictive relationships, formulated for \( V_{\text{mod}} \), \( V_{st,\text{mod}} \) and \( M_{st,\text{mod}} \) are presented in the equations 8.21 to 8.23, 8.24 to 8.27 and 8.28 to 8.31, respectively. The regression coefficients for the prediction of \( V_{\text{mod}} \), \( V_{st,\text{mod}} \) and \( M_{st,\text{mod}} \) are provided in Table 8-4, Table 8-5 and Table 8-6, respectively.

\[
V_{\text{mod}} = \exp \left[ b_1 \beta_3 + b_2 q_\mu + \left( \frac{b_3}{q_\mu} + b_4 \beta_3 \right) \sigma_1 + \left( b_6 q_\mu + b_7 \beta_3 \right) \sigma_2 \right] \quad (8.21)
\]
\[
\sigma_1 = \frac{1}{[1 + \exp\{-b_8(\log(T_1/T_m) - \log(2.3))\}]}
\]

(8.22)

\[
\sigma_2 = \frac{1}{[1 + \exp\{-b_9(\log(T_1/T_m) - \log(0.80))\}]}
\]

(8.23)

\[
V_{st,mod} = \exp\left\{\left(\frac{\sigma_1}{\sigma_2}\right) + \sigma_3\right\}
\]

(8.24)

\[
\sigma_1 = b_1 + b_2q_\mu + b_3\beta_3 + b_4\left[\left(\frac{H_i/H}{\beta_3}\right)^2 + b_5(H_i/H)\right]
\]

(8.25)

\[
\sigma_2 = 1 + \exp\left\{-b_6\left(\frac{T_1}{T_m} - b_7\right)\right\}
\]

(8.26)

\[
\sigma_3 = b_6 + q_\mu \ln\left(\frac{T_1}{T_m}\right)
\]

(8.27)

\[
M_{st,mod} = \exp\left(\left(\frac{\sigma_1}{\sigma_2}\right) + \sigma_3\right)
\]

(8.28)

\[
\sigma_1 = b_1 + b_2q_\mu + b_3\beta_3 + b_4(H_i/H)^2 + b_5(H_i/H) * \beta_3^{b_6}
\]

(8.29)

\[
\sigma_2 = 1 + \exp\left(-b_6\left(\frac{T_1}{T_m} - b_7\right)\right)
\]

(8.30)

\[
\sigma_3 = b_6 + q_\mu \ln\left(\frac{T_1}{T_m}\right)
\]

(8.31)

Table 8-4: Regression coefficients for the base shear modification factor ($V_{mod}$)

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
<th>$b_5$</th>
<th>$b_6$</th>
<th>$b_7$</th>
<th>$b_8$</th>
<th>$b_9$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.2863</td>
<td>-0.0376</td>
<td>-0.8997</td>
<td>12.3583</td>
<td>0.9786</td>
<td>0.0997</td>
<td>-16.788</td>
<td>0.6922</td>
<td>-0.4689</td>
</tr>
</tbody>
</table>
Table 8-5: Regression coefficients for the storey shear modification factor ($V_{st,mod}$)

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
<th>$b_5$</th>
<th>$b_6$</th>
<th>$b_7$</th>
<th>$b_8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1.2588</td>
<td>0.1785</td>
<td>1.5593</td>
<td>2.4484</td>
<td>-2.1628</td>
<td>1.0403</td>
<td>1.7488</td>
<td>-0.0201</td>
</tr>
</tbody>
</table>

Table 8-6: Regression coefficients for the storey moment modification factor ($M_{st,mod}$)

<table>
<thead>
<tr>
<th>$b_1$</th>
<th>$b_2$</th>
<th>$b_3$</th>
<th>$b_4$</th>
<th>$b_5$</th>
<th>$b_6$</th>
<th>$b_7$</th>
<th>$b_8$</th>
<th>$b_8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.4941</td>
<td>0.2419</td>
<td>0.1608</td>
<td>0.8546</td>
<td>0.1767</td>
<td>7.2261</td>
<td>0.81</td>
<td>1.3687</td>
<td>0.0333</td>
</tr>
</tbody>
</table>

For the prediction of $V_{mod}$, the parameters required are: $T_1$, $T_m$, $q_{μ}$, $β_3$, $V_1$ and $α$ of the frame. For the prediction of $V_{st,mod}$ and $M_{st,mod}$, the parameters required are: $T_1$, $T_m$, $q_{μ}$, $β_3$, $V_{i,1}$, $M_{i,1}$, $α$ and the normalized height ($H_i/H$) of the frame. All the other parameters, except $V_{i,1}$, $M_{i,1}$ and $H_i/H$, have been discussed in detail in the previous section from the seismic assessment and design perspectives. $V_{i,1}$ and $M_{i,1}$ can be evaluated using pushover analysis in the case of seismic assessment and using principles of mechanics, by noting respectively the total storey shear and moment at the formation of the first yield in the frame; on the other hand, $H_i/H$ can be evaluated using the geometry of the frame.

8.3. Comparative Studies

This section provides a number of comparisons related to the methodology and predictive relationships proposed in this thesis for the drift and strength demands. Particular emphasis is given to design provisions in Europe and the US.

8.3.1. Drift Demands

This section is divided into two parts. The first part compares the drift demands with previous research studies, discussed in the literature review, and the second part provides comparison with code provisions.
8.3.1.1. **Comparison with other research studies**

In the literature review, it was shown that there is a lack of consensus on the parameters that influence the global and maximum drift demands. For comparison, the studies conducted on steel frames by Uang and Maarouf (1994) and Karavasilis et al., (2008) have been used, whereas the studies of Pettinga and Priestley (2005) and Medina and Krawinkler (2005), conducted on reinforced concrete frames and generic frames respectively, have been omitted as these studies are not directly comparable. However, it should be noted that Medina and Krawinkler (2005) concluded that the maximum drift demand is dependent on global drift ($\theta_{r}$), $T_1$ and the number of storeys ($N$), and Pettinga and Priestley (2005) demonstrated that the maximum drift demand is a function of the stiffness at the top storeys. The models developed in this thesis are compared with the studies of Uang and Maarouf (1994) and Karavasilis et al., (2008) as follows:

1. Uang and Maarouf (1994) used four frames, in total, to study the influence of various parameters on $\delta_{mod}$ and $\theta_{mod}$. For two steel frames (comprising of one moment resisting frame (MRF) with $T_1 = 2.1$ s and one braced frame (BF) with $T_1 = 0.3$ s), $\delta_{mod}$ and $\theta_{mod}$ were found to be dependent on the degree of inelasticity (measured in terms of $q_\mu$). For $q_\mu$ of 2 to 5, $\delta_{mod}$ decreased from 0.8 to 0.7 for the MRF and increased from 0.85 to 1.25 for the BF. On the other hand, for the same range of $q_\mu$, $\theta_{mod}$ was found to increase from 1.1 to 1.6 for the MRF, and from 1.05 to 1.25 for the BF. In order to perform a meaningful comparison, $\delta_{mod}$ have been computed using Equations 8.3 to 8.5 for $q_\mu$ of 2 and 4, $T_m$ of 0.55 s and 0.80 s, and $T_1$ range from 0.2 s to 2.2 s, whereas $\theta_{mod}$ has been evaluated using the same values of $q_\mu$, $T_m$ and $T_1$ with $\beta_3$ of 0.86 (average of all frames used in this thesis). $T_m$ of 0.55 s is used to correspond with the average $T_m$ of 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 in that study was found to be 0.52 s, which was converted to $T_m$ using Equation 8.9. The comparison for $\delta_{mod}$ and $\theta_{mod}$ is presented in Figure 8-2 and Figure 8-3.
Figure 8-2: Comparison of $\delta_{\text{mod}}$ predictions using the model developed in thesis with Uang and Maarouf (1994), referred to as ‘UM’, for $q_\mu$ of 2 and 4
In general, it is observed that $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ predicted by the models are reasonably close to that of Uang and Maarouf (1994) except for $q_{\mu}$ of 4 for the MRF, considering the uncertainties involved in the inelastic seismic response and a very few number of data points. Moreover, it is observed that $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ increase with an increase in $q_{\mu}$ for the BF, which is a similar trend to that shown by the model proposed in this thesis. However, Uang and Maarouf (1994) concluded that $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ is insensitive to $T_1$.
unless $T_1/T_c < 0.3$. In contrast, the model proposed in this study shows that $\delta_{mod}$ and $\theta_{mod}$ are sensitive to $T_1$ and $T_m$ for $T_1/T_m < 1$ (in the short $T_1/T_m$ range) and for $T_1/T_m > 2.7$ and $T_1/T_m > 1.7$ respectively for $\delta_{mod}$ and $\theta_{mod}$ (in the long $T_1/T_m$ range). One possible reason for this discrepancy may be due to a very low number of frames (4) and low number of records (8), which mostly consisted of relatively short period records, used by Uang and Maarouf (1994).

2. Karavasilis et al., (2008) concluded that $\delta_{mod}$ is a function of $q$ (shown in Equation 8.32), whereas $\theta_{max}$ is a function of $\theta_r$, $N$, beam-to-column stiffness ratio ($\rho$) and average plasticity resistance ratio ($\alpha_{avg}$), as shown in Equation 8.33. It should be noted that Equation 8.33 relates $\theta_{max}$ as a function of $\theta_r$ and other parameters, instead of proposing a direct relationship for $\theta_{mod}$. The relationship is based on an assumption that $\theta_{max}$ can be computed directly from $\theta_r$. This assumption is applicable to frames which typically exhibit a linear displacement profile for the first mode of vibration, and where the height of each storey is the same. In other words, $\theta_{max}$ is equal to $\theta_r$. However, the frames used in this thesis do not exhibit a linear displacement profile. As a result, the relationship proposed by Karavasilis et al., (2008) for $\theta_{max}$ cannot be transformed to equate with $\theta_{mod}$. Thus, the comparison of this relationship with the model proposed in this thesis cannot be carried out.

$$\delta_{mod} = \frac{(q + 0.39)}{1.39q} \quad (8.32)$$

$$\theta_{max} = \left(1.0 - 0.193 * (N - 1)^{0.54} * \rho^{0.144} * \alpha_{avg}^{-0.19}\right) / \theta_r \quad (8.33)$$

The comparison of $\delta_{mod}$ using Equation 8.32 and the model proposed in the thesis is presented in Figure 8-4 for $q$ of 3 and 6 and $T_1/T_m$ range of 0.4 to 4. It should be noted that the models presented in the thesis are a function of $q_{\mu}$. In order to convert $q$ to $q_{\mu}$, a value of $\alpha$ of 1.71 (an average of all the frames used in the study) is employed. It is noted from the figure that the predictions of Karavasilis et al., (2008) model, referred to as ‘KM’ in the figure, are reasonably close to the prediction of the model proposed in this thesis in the intermediate $T_1/T_m$ range. However, the model severely underpredicts $\delta_{mod}$ in the short and long $T_1/T_m$ ranges. The reason for this mismatch between the models is the same as observed for Uang and Maarouf (1994). The database of
records used by Karavasilis et al. (2008) comprised of 30 far-field ground motion records. 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.

**Figure 8-4:** Comparison $\delta_{mod}$ predictions using the model developed in thesis with Karavasilis et al., (2008), referred to as ‘KM’, for q of 3 and 6

### 8.3.1.2. Comparison with Code provisions

European and US provisions provide separate methods for the estimation of drift demands for design and assessment purposes. Therefore, the discussion below is divided with respect to seismic assessment and design.

**Seismic Assessment**

For seismic assessment purposes, European and US provisions recommend the displacement coefficient method, as described in Annex B of EC8 (CEN, 2004) as well as FEMA-356 (FEMA, 2000) and FEMA-440 (ATC, 2005), as shown in
Equation 8.34. This method proposes four coefficients: $C_0$, $C_1$, $C_2$ and $C_3$ to account respectively for conversion from a SDF to a MDF system, the inelastic displacement ratio, the pinching effect, the stiffness and the strength degradation, and the P-delta effects in order to predict the roof displacements.

$$\Delta_{\text{max}} = C_0 C_1 C_2 C_3 S_d(T_{\text{eff}}) \frac{T_{\text{eff}}^2}{4\pi^2} g$$  \hspace{1cm} (8.34)

In the above equation $S_d(T_{\text{eff}})$ is the elastic spectral acceleration of the SDF system corresponding to $T_{\text{eff}}$, $g$ is the acceleration of gravity, and $T_{\text{eff}}$ is defined using the following expression:

$$T_{\text{eff}} = T_1 \frac{K_i}{\sqrt{K_e}}$$  \hspace{1cm} (8.35)

In the above equation, $T_1$ is the elastic period; $K_i$ is the initial stiffness corresponding to the elastic period, and $K_e$ is the effective stiffness. EC8 and US provisions provide two different approaches to calculate the effective stiffness using the pushover curve of the MDF system. Using these approaches the actual pushover curve for the MDF system is converted into an idealized bilinear curve.

Using the US approach, the effective stiffness is calculated by the line passing through the origin and $0.6V_y$, whereas the post yield stiffness is approximated as the horizontal line passing through $V_y$. The yield displacement of the idealized SDF system, $\delta_y$, is found as the intersection of the effective stiffness and post yield stiffness line, as demonstrated in Figure 8-5.

EC8, on the other hand, uses the equal energy approach. Therefore, the area under the actual pushover curve is equated with the area under the idealized bilinear pushover curve, as shown in Figure 8-6. Using this approach, $\delta_y$ can be calculated using the following equation:

$$\delta_y = 2 \left( \delta_m - \frac{E_m}{V_y} \right)$$  \hspace{1cm} (8.36)

In this equation, $\delta_m$ is the displacement of the SDF system at the formation of the plastic mechanism; $E_m$ is the area under the actual pushover curve; and $V_y$ is the maximum base shear of the idealized bilinear curve.
**Figure 8-5:** Bilinear pushover curve for an idealized SDF system using US provisions (FEMA-356 and FEMA-440)

**Figure 8-6:** Bilinear pushover curve for an idealized SDF system using EC8 provisions (Annex-B)
Since a non-deteriorating behaviour is used in this thesis and the pushover curves do not exhibit a negative post yield stiffness, the coefficients $C_2$ and $C_3$ can be ignored. The above equation can therefore be re-written as:

$$
\Delta_{max} = C_0 C_1 S_a T_{eff}^2 \frac{T_{eff}^2}{4\pi^2} g
$$

(8.37)

The elastic roof displacement can be calculated using:

$$
\Delta_{elastic} = C_0 S_a T_{eff}^2 \frac{T_{eff}^2}{4\pi^2} g
$$

(8.38)

Since the roof displacement (global drift) modification factor, $\delta_{mod}$, is defined as the ratio of the inelastic roof displacement from ground motion (estimated using Equation 8.37) to the elastic roof displacement from the ground motion (determined using Equation 8.38), $\delta_{mod}$ is equal to $C_1$, which is defined as follows in EC8 and US provisions:

$$
\delta_{mod} = C_1 = 1.0 \quad T_{eff} \geq T_c
$$

(8.39)

$$
\delta_{mod} = C_1 = \frac{1}{q_\mu} \left[ 1.0 + (q_\mu - 1) \frac{T_c}{T_{eff}} \right] \quad T_{eff} < T_c
$$

(8.40)

Whereas,

$$
q_\mu = \frac{S_a(T_{eff}) W_Y}{V_Y}
$$

(8.41)

In the above equation, $W$ is the total weight of the MDF system; the other parameters in the equations have been defined previously.

The method is used to predict the modification factor for the global displacement/drift of MDF systems. The same modification factor is used to predict the maximum drifts assuming that the higher modes do not influence the maximum drift demands. In other words, $\delta_{mod}$ and $\theta_{mod}$ proposed by both EC8 and US provisions are equal.

It should be noted that the predictive relationships for drift demands, developed in this thesis, are a function of $T_1$ and $T_m$, whereas the relationships proposed by the
European and US provisions are a function of $T_{\text{eff}}$ and $T_c$. Therefore, in order to perform a comparative study, the relationships between $T_1$ and $T_{\text{eff}}$, as well as $T_m$ and $T_c$, are required. The relationship for the latter was developed previously in Chapter 4, and presented in Equation 8.9. The relationship between $T_1$ and $T_{\text{eff}}$ can be developed using the methods proposed by EC8 and US provisions, as described before. To this end, $T_{\text{eff}}$ is calculated for all of the frames using the pushover curves presented in Chapter 3. $T_{\text{eff}}$ is then plotted against the corresponding $T_1$, as shown in Figure 8-7 and Figure 8-8 using US and European provisions, respectively. A linear trend line is used to identify the relationship between $T_1$ and $T_{\text{eff}}$. It is found that $T_{\text{eff}}$ is only 1% higher than $T_1$ if the US provisions is employed, whereas $T_{\text{eff}}$ is 30% higher than $T_1$ if the EC8 procedure is used.

![Figure 8-7: Relationship between $T_1$ and $T_{\text{eff}}$ (calculated using US provisions)](image)

$T_{\text{eff}} = 1.01T_1$

$R^2 = 0.99$
The relationships proposed in European and US provisions (presented in Equations 8.39 and 8.40) can now be modified as a function of $T_c$ and $T_1$, as presented in the following equations:

**US Provisions:**

$$
\delta_{mod} = C_1 = 1.0 \quad T_1 \geq 0.94T_m
$$

$$
\delta_{mod} = C_1 = \frac{1}{q_\mu} \left[ 1.0 + (q_\mu - 1) \frac{T_m}{1.06T_1} \right] \quad T_1 < 0.94T_m
$$

**EC8 Provisions:**

$$
\delta_{mod} = C_1 = 1.0 \quad T_1 \geq 0.73T_m
$$

$$
\delta_{mod} = C_1 = \frac{1}{q_\mu} \left[ 1.0 + (q_\mu - 1) \frac{T_m}{1.365T_1} \right] \quad T_1 < 0.73T_m
$$
The comparison is now performed for the global and the maximum drift demands, using $q_\mu$ of 2, 4 and 6 and for $T_1/T_m$ ratios from 0.4 to 4.0. Figure 8-9 and Figure 8-10 show the comparison of the global drifts predicted by the relationships developed in this study using the US and EC8 provisions respectively. It can be observed that for short period ratios ($T_1/T_m < 0.94$ and $T_1/T_m < 0.73$, for US and EC8 models respectively), the trends for $\delta_{\text{mod}}$ exhibit similar patterns for the models proposed in this thesis and using the code provisions. However, there is a significant difference in the magnitude of the modification factors. Furthermore, it is noted that the EC8 predictions are slightly lower than those in the US due to the higher effective periods obtained using the EC8 provisions. On the other hand, for the higher period ratios, the code provisions adopt the simplistic approach of the equal displacement rule, whereas the models in this study show that $\delta_{\text{mod}}$ decreases with the increase in the behaviour factor with a slight influence from the period ratio.

Similarly, the predictions are compared for the maximum drift modification factor ($\theta_{\text{mod}}$). $\theta_{\text{mod}}$ is computed using $q_\mu$ of 2, 4 and 6, $T_1/T_m$ ratio from 0.4 to 4.0, and $\beta_3$ of 0.86 for models developed in this thesis. The comparison of the model predictions with the US and EC8 provisions are shown in Figure 8-11 and Figure 8-12 respectively. The trends are similar to those observed for $\delta_{\text{mod}}$. However, it is noted that the difference between the codified and model predictions is less pronounced, particularly for the EC8 predictions. Nevertheless, the predictions of the code provisions still overestimate $\theta_{\text{mod}}$ significantly.

The key reason for the significant overestimation of the global and maximum drift modification factor using the code provisions is that the factors proposed by the codes are based on the nonlinear dynamic analysis of SDF systems (as discussed in detail in Chapter 4). Therefore, it is presumed that the SDF system can faithfully represent the behaviour of a MDF system. This assumption may be true for a MDF system which is designed to form simultaneous hinges in the beams and the column bases. Moreover, it is assumed that higher mode effects do not play a significant role in the modification of the maximum drift demands. However, as shown by the results, it can be concluded that this assumption is not applicable to the frames used in this study and this leads to the overprediction of drift demands.
Figure 8-9: Comparison of the global drift modification factor ($\delta_{\text{mod}}$) prediction using the models proposed in this thesis and US provisions

Figure 8-10: Comparison of the global drift modification factor ($\delta_{\text{mod}}$) prediction using the models proposed in this thesis and EC8 provisions
Figure 8-11: Comparison of the maximum drift modification factor (θ_{mod}) prediction using the models proposed in this thesis and US provisions

Figure 8-12: Comparison of the maximum drift modification factor (θ_{mod}) prediction using the models proposed in this thesis and EC8 provisions
**Seismic Design**

For the seismic design of frames, European and US provisions prescribe behaviour factors that are dependent on the selected ductility class and structural system respectively. The base shear applied to 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 the first yield of the structure (\( V_1 \)) and to the base shear at the formation of the significant yield (\( V_p \)) for EC8 and US provisions, respectively. It should be noted that the significant yield may differ from the first yield, as it refers to the formation of the 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 large behaviour factors proposed by the codes coupled with the effects of material and design overstrength.

For the estimation of drift demands, EC8 uses the equal displacement rule. In other words, the global and maximum drift modification factors for determining the inelastic drift demands are taken as unity, as shown in Figure 8-13. The US provisions, on the other hand, propose a factor (referred to as the seismic drift amplification factor, \( C_d \)) dependent on the behaviour factor (referred to as the force reduction factor, \( R \)), as shown in Figure 8-14. 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 either equal to or lower than the corresponding behaviour factor.

In order to compare the drift demand predictions using EC8 and US provisions with the predictive relationships developed in this study, it can be assumed that \( V_d \) is equal to \( V_1 \). Moreover, it is assumed that the difference between \( V_1 \) and \( V_p \) is not significant, therefore can be ignored. Thus, the definition of global and maximum drift modification factor for EC8 and US provisions can be evaluated using Equation 8.46 and Equation 8.47 respectively, as follow:

\[
\theta_{mod} = \delta_{mod} = 1 \quad \text{(for EC8)} \tag{8.46}
\]

\[
\theta_{mod} = \delta_{mod} = \frac{C_d}{R} \quad \text{(for US)} \tag{8.47}
\]
Figure 8-13: Evaluation of design base shear and prediction of maximum roof displacement using EC8

Figure 8-14: Evaluation of design base shear and prediction of maximum roof displacement using US Provisions
Thus, based on US provisions, $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ for OMF, IMF and SMF are found to be 0.86, 0.89 and 0.69 respectively. Considering that the models developed in this thesis for $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ are a function of $q_{\mu}$, whereas the US code based recommendation are based on $q$. Therefore, to achieve consistency, the mean value of the plasticity ratio for all frames used in the study (1.71) can be used to scale $q_{\mu}$ for the relevant comparison. For example, to predict the model results for $R = q = 4.5$ (for IMF), $q_{\mu}$ of 2.63 ($=4.5/1.71$) can be used. Furthermore, to evaluate $\theta_{\text{mod}}$, the mean value of $\beta_3$ for all frames (found to be 0.86) is used.

Figure 8-15 plots the prediction of the model developed earlier for $\delta_{\text{mod}}$ for behaviour factors (force reduction factor) of 3.5, 4.5 and 8.0, and the corresponding factors proposed by EC8 and US provisions. It can be noted that EC8 provisions are notably higher than the predictions of the model for all cases except for the short $T_1/T_m$ range for $q$ of 8. On the other hand, US provisions are less conservative when compared to EC8 provisions. However, when compared with the model predictions, it is observed that the modification factor proposed in US provisions is higher in the intermediate ranges of period ratio. For $T_1/T_m$ range in particular, the US suggestions are much smaller than the model predictions. The difference between model predictions and US provisions, in this range, generally increases with the increase in $q$. For long $T_1/T_m$ range, the model predictions are higher for $q$ of 8 and lower for $q$ of 3.5 and 4.5.

Similarly, predictions of the model for $\theta_{\text{mod}}$ for $q$ factors of 3.5, 4.5 and 8.0, and corresponding factors proposed by EC8 and US provisions are plotted in Figure 8-16. In comparison with EC8 provisions, the model predictions are generally much lower for all cases except for the short $T_1/T_m$ range for $q$ of 8. On the other hand, US provisions are relatively close to the model predictions for $q$ factor of 3.5 and 4.5. However, in general, the model predictions for these two behaviour factors are on the low side for intermediate $T_1/T_m$ range, and on the high side for short and long $T_1/T_m$ ranges. On the other hand, $\theta_{\text{mod}}$ proposed in US provisions for $q$ of 8 are much smaller than the model predictions for both short and long $T_1/T_m$ ranges. Furthermore, it is observed that the difference in the model predictions and US provisions is much larger for the short $T_1/T_m$ range. In contrast, the factors obtained from the model and US provisions are close for a brief intermediate period range.
Figure 8-15: Comparison of the model estimates for global drift modification ($\delta_{mod}$) with EC8 and US provisions

Figure 8-16: Comparison of the model estimates for maximum drift modification ($\theta_{mod}$) with EC8 and US provisions
8.3.2. Strength Demand

This section is devoted to the discussion of strength demand prediction methods adopted in European and US codes. These are then compared with the predictions of the relationships proposed in this thesis.

8.3.2.1. Seismic Assessment

Whilst codes do not have a clear guidance for determining the actual strength demand, this can be estimated using pushover analysis. As a result, the strength demands imposed on the frame are typically larger than $V_d$, $V_I$ and $V_p$ due to structural overstrength.

There are two possible ways to compare the predictions of strength demands using pushover with the predictions of the relationships developed in this thesis. In the first method, it can be assumed that the strength demands for a given level of ductility are simply a product of the strength demand under consideration (base shear, storey shear or storey moment) at first yield and the plasticity resistance ratio (obtained using the first principle of mechanics, discussed in detail previously). In other words, it is an approximation of pushover. Whilst this may be accurate for determining the base shear, the actual redistribution of the shear and moment demands over the height of the frame due to sequential hinging of beams and columns are ignored. It needs to be recalled that this assumption forms the basis of calculation of the strength demand modification factors. The second method, on the other hand, requires the comparison to be carried out for each frame for a given seismic hazard scenario. Using this procedure, the global drift demands of the frame are first estimated using the predictive relationships discussed in the previous section. The drift demands, thus obtained, serve as the target drift for the pushover analysis. Subsequently, the pushover analysis is conducted up to the target drift and the strength demands are noted at the target drift, which can then be compared with the predictions using the models developed in the thesis.

Taking into consideration the complexity of the second method, the first method is selected to demonstrate a general comparison between the pushover estimates and the relationships for the strength demands. The base shear demands are calculated using Equations 8.21 to 8.23 for $T_1/T_m$ of 0.4 to 4; $\beta_3$ of 0.86 and $q_{\mu}$ of 2 and 4, as shown in
Figure 8-17 along with the pushover approximation. The storey shear and moment demands are evaluated for normalized heights of 0.5 and 1 i.e. at the mid-height and top of the frame using Equations 8.24 to 8.27 and 8.28 to 8.31, respectively.

It can be observed that the base shear demand is severely underpredicted by the approximate pushover analysis. Moreover, it is noted that the base shear demands are highly dependent on the period ratio and the behaviour factor. The storey shear and moment demands presented in Figure 8-18 and Figure 8-19 exhibit similar trends. The storey shear and moment demands predicted by the models are significantly higher than those predicted by the pushover analysis. The results are more pronounced at the top of the frame. Furthermore, the comparison shows that the pushover analysis results are unable to incorporate the influence of the behaviour factor and the mean period ratio on the strength demands.

![Figure 8-17: Comparison of the base shear modification factor \(V_{\text{mod}}\) using the model developed in this thesis with pushover approximation](image)

**Figure 8-17:** Comparison of the base shear modification factor \(V_{\text{mod}}\) using the model developed in this thesis with pushover approximation
Figure 8-18: Comparison of the storey shear modification factor \((V_{st,mod})\) using the model developed in this thesis with pushover approximation.

Figure 8-19: Comparison of the storey moment modification factor \((M_{st,mod})\) using the model developed in this thesis with pushover approximation.
8.3.2.2. Seismic Design

European and US design provisions require the frame to satisfy capacity design principles. Therefore, the columns have to be designed to actions which are consistent with the ultimate capacity of the frames.

In case of EC8, shear, moment and axial demands on columns, as shown in Equations 8.48, 8.49 and 8.50 respectively, are computed from the lateral load analysis using $V_d$ along with gravity loading.

\[
V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}\Omega V_{Ed,E} \tag{8.48}
\]

\[
M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega M_{Ed,E} \tag{8.49}
\]

\[
N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{8.50}
\]

In the above expressions, $M_{Ed,G}$, $V_{Ed,G}$ and $N_{Ed,G}$ refer to the design moments, shear and axial forces, respectively, for the column due to gravity loads, and $M_{Ed,E}$, $V_{Ed,E}$ and $N_{Ed,E}$ represent the design moments, shear and axial forces, respectively, for the column due to lateral seismic loads; $\gamma_{ov}$ is the material overstrength typically assumed to be 1.25; $\Omega$ 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.

The underlying concept of the equations presented above is to evaluate the strength demands of the columns at the formation of the first hinge. As shown earlier, $V_d$ tends to be lower than $V_1$ due to material and design overstrength. The contribution of these factors towards overstrength is addressed using $\gamma_{ov}$ and $\Omega$. However, it can be noted that $\Omega$ is calculated as the ratio of the plastic moment capacity of the beam to the applied design moment, which leads to overestimation of the overstrength due to double counting of the moments due to gravity loading. To this end, Elghazouli (2010) proposed a modified overstrength factor, $\Omega_{mod}$, to address the design overstrength of the beam. It is defined as a minimum of $\Omega_i = (M_{pl,Rd,i} - M_{Ed,G,i})/M_{Ed,i}$ of the beams, where $M_{Ed,i}$ is the design moment in beam ‘i’; $M_{Ed,G,i}$ is the design gravity moment in beam ‘i’ and $M_{pl,Rd,i}$ is the corresponding plastic moment.
US provisions, on the other hand, use a direct approach to satisfy the capacity design rules and require that at all the beam to column intersections, the summation of moment capacity of column, $\Sigma M_{Rc}$, is equal to or greater than the summation of the moment capacity of beams, $\Sigma M_{Rb}$, as given in following expression:

$$\Sigma M_{Rc} \geq \Sigma M_{Rc}$$ (8.51)

Subsequently, the shear and axial demands on the columns are computed using the plastic moment capacity of the beams framing at given beam-to-column joint.

It is noted that EC8 and US codes ignore the increase in the strength demands on the column due to sequential hinging of beams and columns and higher mode effects. To account for the redistribution due to sequential hinging, Elghazouli (2010) recommended inclusion of the plasticity resistance ratio as an additional factor to amplify the shear, moment and axial demands on columns, as shown in Equations 8.52, 8.53 and 8.54, respectively.

$$V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov} \Omega_{mod} \alpha V_{Ed,E}$$ (8.52)

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov} \Omega_{mod} \alpha M_{Ed,E}$$ (8.53)

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov} \Omega_{mod} \alpha N_{Ed,E}$$ (8.54)

It can be noted that if the factors contributing to the material and design overstrength (i.e. $1.1\gamma_{ov} \Omega_{mod}$) are ignored, the above equation will mimic the pushover analysis approximation adopted in the previous section. Hence, the comparison of the strength demands carried out in the previous section from the assessment perspective is also applicable to the design provisions. Therefore, it can be concluded that the current code provisions for the prediction of strength demands appear to be grossly unconservative.

8.4. Application in Codified Design

In the previous section, the comparison of the models developed in this thesis highlighted the inaccuracies of the current codified provisions in predicting drift and strength demands. This section aims to propose several modifications to the existing codified provisions for seismic design, as enumerated below:
1. As identified earlier, the behaviour factors proposed by current code provisions tend to be on the high side. Moreover, due to material and design overstrength, \( V_d \) is found to be higher than \( V_1 \). In order to apply the models proposed in this thesis, it is a pre-requisite to estimate \( V_1 \) and corresponding drifts and strength demands on the structure appropriately. Evaluation of \( V_1 \) can be carried using the procedure discussed in detail in Section 8.2.

2. If a designer’s intention is to achieve uniform drift demands at the upper and lower half of the frame, then \( \beta_d \) can be used to evaluate the required relative stiffness of the frame. Subsequently, Equations 8.1 and 8.3 to 8.5, and Equations 8.2 and 8.6 to 8.8 or 8.2 and 8.15 to 8.17 can be used to predict the global and maximum drift demands, respectively. It should be noted that the modification of the stiffness of the upper half, required to achieve the desired relative stiffness, should be carried out in the final step of the design. On the other hand, if a designer does not intend to achieve uniform drift demands then \( \beta_3 \) of the designed frame can be calculated and inserted in Equations 8.1 and 8.3 to 8.5, and Equations 8.2 and 8.6 to 8.8 for the global and maximum drifts, respectively.

3. For the strength demands, it should be noted that the models proposed in this thesis do not include the distribution of the axial demands on the columns along the height of the frames. Therefore, there are two options for including the strength demands in the existing codified provisions, as discussed below:

   a. The maximum base shear is calculated using Equations 8.18 and 8.21 to 8.23. On the other hand, the maximum storey shear can be evaluated using Equations 8.19 and 8.24 to 8.27. The maximum storey shear can be used subsequently to calculate maximum storey moments and axial demands using simple structural analysis. However, it should be noted that this is an approximate method.

   b. An additional study on the influence of various parameters on the axial demands of the columns is conducted and predictive relationships are developed, in future. In such case, the storey shear
and moment demands can be evaluated using Equations 8.19 and 8.24 to 8.27, and Equations 8.20 and 8.28 to 8.31, respectively; on the other hand, the axial demands are computed using additional equations obtained from separate future work.

8.5. Concluding Remarks

This chapter provided a detailed comparison of the predictive relationships developed in this thesis for drift and strength demands, with particular emphasis on the current European and US seismic design and assessment provisions.

The models developed in this study for the prediction of the drift demands were first compared with the findings of Uang and Maarouf (1994) and Karavasilis et al. (2008). The comparative study showed that previous studies have not fully incorporated the influence of the frequency content, fundamental period and higher mode effects on the drift demand, and that the records used by both studies mainly consisted of short period records.

For the prediction of drift demands for seismic assessment purposes, the US and EC8 provisions recommend the displacement modification method, which employs pushover analysis in conjunction with idealized SDF system to represent the MDF system. The models proposed for the prediction of the drift demands are a function of the effective period, corner period of the response spectrum and behaviour factor for short period ratios, as well as the equal displacement rule for the remaining period ratios. The comparison of drift demands using the models developed herein with codified seismic assessment provisions showed that the predictions using the US and EC8 provisions significantly overestimate the global and the maximum drift demands of the frames. Furthermore, it was noted that the code provisions and the models of this thesis exhibit similar trends for the short $T_1/T_m$ ratios ($T_1/T_m \leq 0.94$ and $T_1/T_m \leq 0.73$ for the US and EC8 provisions, respectively), but differ in the magnitude of global and maximum drift demands. EC8 predictions are found to be relatively closer to the models of the thesis, due to relatively larger effective period obtained in EC8.

For the remaining period ratios, the code provisions adopt a simplistic approach for the equal displacement rule. This is found to be inconsistent with the models developed herein, which show that the global drift is a function of the period ratio,
behaviour factor and the plasticity resistance ratio, while the maximum drift is a function of the relative storey stiffness ratio in addition to the three parameters required for global drift. Moreover, this approach leads to an overestimation of the drift demands for these period ratios.

On the other hand, for the prediction of drift demands for seismic design, the US and EC8 provisions propose the drift amplification factor and the equal displacement rule, respectively. It is noted that the factors proposed in the US provisions are a function of the behaviour factor. For the short period ratios \(T_1/T_m \leq 0.75\) and \(T_1/T_m \leq 0.80\) approximately for the global and the maximum drift demands respectively, the US provisions significantly underestimate the global and maximum drift demands in comparison to the models developed herein. The magnitude of the difference between the code and the models is relatively smaller in the case of maximum drift demands. On the other hand, comparison of the EC8 prediction with the model shows that, in general, the EC8 predictions overestimate the drift demands for lower behaviour factors (less than 4.5) and underestimates these for the higher behaviour factors (\(q = 8\)). For the intermediate period ratios \(0.80 \leq T_1/T_m \leq 2.80\) and \(0.80 \leq T_1/T_m \leq 2.50\) approximately for the global and the maximum drift demands respectively), it is noted that for large behaviour factors (\(q = 8\)) the predictions of drift demands using the US provisions are very close to the predictions of the models developed herein. For the lower behaviour factors, the US provisions underestimate the drift demands, particularly the global drift demands. On the other hand, EC8 provisions overestimate the drift demands significantly for all behaviour factors. For the longer period ratios \(T_1/T_m \geq 2.8\) and \(T_1/T_m \geq 2.5\) approximately for the global and the maximum drift demands respectively), the US provisions underpredict the drift demands for all behaviour factors in comparison to the models developed herein. On the other hand, the EC8 provisions overpredict the drift demands for all behaviour factors.

For strength demands, both US and EC8 code provisions propose the pushover analysis for the seismic assessment of the frames. It is shown that the pushover predictions severely underestimate the base shear, the storey shear and the storey moment demands on the frame. Furthermore, it is noted that the pushover analysis is unable to incorporate the effects of the period ratio and the behaviour factor on the strength demands.
For the seismic design of the frames, the US and EC8 code provisions require that the strength demands are calculated at the formation of the plastic hinges in the beams. It is noted that the strength demands, determined using this procedure, are found to be lower than those predicted by the pushover analysis. Therefore, the code provisions for strength demands for seismic design are severely underestimated in comparison with the models developed herein.

Based on the comparative study performed in this chapter, it is shown that various modifications are required in codified procedures in order to improve the prediction of drift and strength demands for seismic design and assessment purposes.
Chapter 9

Closure

9.1. Summary and Conclusions

9.1.1. General

This thesis focused on understanding of influence of the frequency content of far-field ground motions and structural characteristics on the inelastic response of moment resisting steel frames (MRFs). To this end, a database of 40 MRFs comprising of 3, 5 and 7 storeys, and designed using European provisions (EC3 and EC8), was developed. The structural characteristics of the designed frames were then determined using Eigenvalue analysis, the geometry of the structure and pushover analysis. The structural characteristics that were evaluated include: the total height of the frame (H), the fundamental period (T₁), the plasticity resistance ratio (α), the beam to column stiffness ratio (ρ) and three variations of the relative storey stiffness ratio (β₁, β₂ and β₃).

On the other hand, to understand the influence of frequency content of the ground motion, a suitable indicator was chosen based on a literature review. Using the study of Rathje et al. (2004), the mean period (Tₘ) of ground motion was selected. It was noted that parameter can be related to the magnitude of the earthquake event, the source-to-site distance and the site conditions. Furthermore, it was observed that, for far-field ground motion, Tₘ increases with the increase in magnitude and the source-to-site distance, and decreases with the increase in shear wave velocity of the site (relatively higher Tₘ values for soil sites over rock sites).
The influence of frequency content measured as $T_m$ was first investigated on the inelastic response of SDF systems to develop an initial understanding. Summary and conclusions pertinent to the inelastic response of SDF systems are discussed in the next section.

9.1.2. Inelastic Response of SDF Systems

Typically, the study of the inelastic response of SDF systems is carried out using either the displacement modification method or the equivalent linearization method. The former approach was chosen in this study owing to its simplicity. This method requires the evaluation of the inelastic displacement ratio, which can be defined as the maximum displacement of a SDF system in elastic behaviour to the maximum displacement of the same SDF system through inelastic response subjected to a given ground motion. This approach can be used either for systems with known strength demands or for systems with known ductility demands; the former was selected in this study. To this end, 20 SDF systems with elastic periods ranging from 0.1 to 1.0 s with an interval of 0.05 s were selected. For inelastic time history analysis, a bilinear hysteretic curve with post-yield stiffness of 3% was selected. The SDF systems were then subjected to 128 far-field ground motions, representing a wide range of magnitude, distance and site condition scenarios, for target ductilities of 2, 4 and 6. The study showed that the inelastic displacement ratio of a given SDF system is a function of the elastic period ($T_e$), $T_m$ and the ductility demand ($\mu$). It was observed that for short period ratios with ($T_e/T_m$) lower than unity, the inelastic displacement ratio increased with the decrease in $T_e/T_m$, and increased with the increase in $\mu$. For $T_e/T_m$ higher than unity, the inelastic displacement ratio was found to be insensitive to $T_e/T_m$ and $\mu$. It is pertinent to mention here that previous studies, conducted by Miranda (2000) and Chopra and Chintanapakdee (2004), concluded that the inelastic displacement ratios of SDF systems are insensitive to the characteristics of the far-field ground motion. However, this study revealed that the inelastic displacement ratios are influenced by the frequency content of the far-field ground motion. The previous studies were unable to capture this trend due to two reasons: i) the ground motion used in these did not include large magnitude earthquakes; ii) grouping of the ground motions was either based on magnitude-distance or site categories. Therefore, in order to predict the inelastic displacement ratios of a SDF system accurately for a
given seismic hazard scenario, it is vital to use the representative value of $T_m$, which is simultaneously dependent on magnitude, distance and site conditions.

Based on the understanding developed from the inelastic response for SDF systems, the study was then extended to MDF systems, which is discussed in the next section.

### 9.1.3. Inelastic Response of MDF Systems

The inelastic response of MDF systems was studied in two stages. In the first stage, the influence of $T_m$ on the roof displacements ($\Delta_{\text{max}}$), the base shear ($V_{\text{max}}$) and the maximum drift profile ($\theta_{\text{si, max}}$) of a single steel MRF was studied. In the second stage, the influence of $T_m$ and various structural parameters on drift and strength demands of the 40 steel MRFs was investigated.

In order to study the influence of $T_m$ on a single MDF system, a steel MRF was selected from the database (with fundamental period, $T_1$, of 1.15 s) to understand the influence of $T_m$ on the roof displacements ($\Delta_{\text{max}}$), the base shear ($V_{\text{max}}$) and the maximum drift profile ($\theta_{\text{si, max}}$). To this end, incremental dynamic analysis (IDA) was conducted by subjecting the system to 128 far-field ground motions scaled to six levels of ductility demand assuming the equal displacement rule ($\mu_{\text{EDR}}$). The results showed that $V_{\text{max}}$ and $\theta_{\text{si, max}}$ were significantly affected by $T_m$. It was observed that $V_{\text{max}}$ increased with the decrease in $T_m$ and increase in $\mu_{\text{EDR}}$, due to the higher mode effects. Furthermore, it was noted that the maximum drift demands on the top storeys increased with the decrease in $T_m$ and increase in $\mu_{\text{EDR}}$, exhibiting pronounced influence of the higher mode effects. $\Delta_{\text{max}}$, on the other hand, was found to be less influenced by the $T_m$ of the ground motion.

Based on the study of the inelastic response of SDF systems and the selected MDF system, it was established that the frequency content of the ground motion significantly influences the inelastic response. To understand the influence of the structural characteristics of the MRFs on the inelastic response and their interaction with $T_m$, the study was extended to the set of MRFs designed using European provisions. The inelastic response parameters studied can be categorized broadly as the drift and the strength demands of the frames. Within the drift demands, the study was limited to the global drift and maximum drift demands. On the other hand, within
the strength demands, the study was restricted to the base shear, storey shear and storey moment demands. The evaluation of the inelastic response was conducted using IDA by scaling 72 far-field records to four levels of relative intensity. It is pertinent to mention that the total number of records was reduced to rationalise the computational effort of nonlinear dynamic analysis. The records were scaled using the 5%-damped elastic acceleration at $T_1$ and the acceleration at the formation of the first yield in the frame to mimic the behaviour factors ($q$) proposed in the seismic design codes. Moreover, the acceleration at the first-yield was chosen, because the parameter could be evaluated easily using the simple principles of mechanics without the need for pushover analysis. The observations and conclusions related to drift and strength demands are discussed subsequently, as follows:

9.1.3.1. Drift Demands

The drift demands, that include the global and the maximum drift demands, were computed in terms of the modification factors referred to as the global modification factor ($\delta_{\text{mod}}$) and the maximum drift modification factor ($\theta_{\text{mod}}$), respectively. For a given frame and a ground motion record, $\delta_{\text{mod}}$ was defined as the ratio of the maximum global drift noted from the nonlinear dynamic analysis to the product of the global drift at the formation of first yield obtained using pushover analysis and $q$. Similarly, for a given frame and a ground motion record, $\theta_{\text{mod}}$ was defined as the ratio of the maximum inter-storey drift noted from the nonlinear dynamic analysis to the product of the maximum inter-storey drift at the formation of first yield obtained using pushover analysis and $q$. Based on the parametric study carried out using IDA results, it was noted that $\delta_{\text{mod}}$ is a function of $T_1/T_m$, $q$ and $\alpha$, whereas $\theta_{\text{mod}}$ was found to be dependent on $T_1/T_m$, $q$, $\alpha$ and $\beta_3$. Based on the trends, the influence of $T_1/T_m$ on $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ was divided in three zones, namely: short, intermediate and long. It was observed that the short $T_1/T_m$ range ends roughly at $T_1/T_m$ of 1 for both $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$, whereas the long period range starts approximately at $T_1/T_m$ of 2.7 and 1.7 for both $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ respectively. It was noted that, for $T_1/T_m$ in the short range, $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ increased with the decrease in $T_1/T_m$. For $T_1/T_m$ in the intermediate range, $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ was not influenced by $T_1/T_m$. For $T_1/T_m$ in the long range, $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ increased with the increase in $T_1/T_m$ due to higher mode effects. Furthermore, it was noted that the increase in $q$, which represents the level of inelasticity, increased $\delta_{\text{mod}}$
and $\theta_{\text{mod}}$ in the short range and decreased $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ in the intermediate and long $T_1/T_m$ ranges. On the other hand, it was noted that the increase in $\alpha$, which represents the delay in the formation of the plastic mechanism, exhibited trends opposite to those noted for $q$. Thus, an increase of $\alpha$ decreased $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ in the short $T_1/T_m$ range and it increased $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$ in the intermediate and long $T_1/T_m$ ranges. The influence of $\alpha$ was found to be more prominent for $\delta_{\text{mod}}$ than $\theta_{\text{mod}}$. It was observed that $\delta_{\text{mod}}$ was not influenced by $\beta_3$, unlike $\theta_{\text{mod}}$. It was observed that the increase in $\beta_3$ results in an increase of $\theta_{\text{mod}}$ for all $T_1/T_m$, due to the relatively lower stiffness of the top storeys of the frame for higher $\beta_3$. Based on the parametric study, predictive relationships were developed for $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$.

**9.1.3.2. Strength Demands**

Similarly, a parametric study of the strength demands of the frames, which includes the base shear, the storey shear and the storey moment, was conducted. These response quantities were studied in the context of modification factors. For a given frame and a ground motion record, the base shear modification factor, $V_{\text{mod}}$, was calculated as the ratio of the maximum base shear obtained from the nonlinear dynamic analysis to the product of the base shear recorded at the first yield using the pushover analysis and $\alpha$. For a given storey of a frame and a given record, the storey shear modification factor, $V_{\text{st,mod}}$, was defined as the ratio of the maximum storey shear for the given storey obtained from the nonlinear dynamic analysis to the product of the storey shear recorded at the first yield using the pushover analysis and $\alpha$, whereas the storey shear was defined as the sum of shear in all columns for a given instant during the pushover or dynamic analysis. For a given storey of a frame and a given record, the storey moment modification factor, $M_{\text{st,mod}}$, was defined as the ratio of the maximum storey moment for the given storey obtained from the nonlinear dynamic analysis to product of the storey moment recorded at the first yield using the pushover analysis and $\alpha$. Since the moment demands vary from the top to bottom ends of a column, for a given storey, the sum of moments was obtained separately for both column ends and the higher of the two was selected as the storey moment. Based on the parametric study of the results obtained from IDA, it was found that $V_{\text{mod}}$ is dependent on $T_1/T_m$, $q$, $\alpha$ and $\beta_3$. On the other hand, $V_{\text{st,mod}}$ and $M_{\text{st,mod}}$ were found to be dependent on $T_1/T_m$, $q$, $\alpha$, $\beta_3$ and the normalized height ($H/H_i$). It was noted that the
influence of $T_1/T_m$ on $V_{\text{mod}}$ could also be divided in three zones: short, intermediate and long, as in the case of $\delta_{\text{mod}}$ and $\theta_{\text{mod}}$. For $V_{\text{mod}}$, the intermediate range covered between $T_1/T_m$ of 1 and 1.7, whereas the short and long ranges were found to be lower and higher than 1 and 1.7 respectively, which is similar to that observed for $\theta_{\text{mod}}$. On the other hand, the increase in either $q$ or $\beta_3$ resulted in the increase of $V_{\text{mod}}$ for all the period ratios. In contrast, the increase in $\alpha$ would result in the relative decrease of $V_{\text{mod}}$ for all the period ratios. The influence of $T_1/T_m$, $q$, $\alpha$ and $\beta_3$ on $V_{\text{st,mod}}$ and $M_{\text{st,mod}}$ was found to be similar to that observed for $V_{\text{mod}}$. Moreover, $V_{\text{st,mod}}$ and $M_{\text{st,mod}}$ were found to be dependent on $H/H_i$. It was noted that $V_{\text{st,mod}}$ and $M_{\text{st,mod}}$ increased with the increase in $H/H_i$; the magnification became more prominent for $H/H_i$ greater than 0.75 and reached a maximum at $H/H_i$ of 1.

9.1.3.3. Design relative storey stiffness parameter

Based on the parametric study of $\theta_{\text{mod}}$, as discussed earlier, it was identified that the increase in $\beta_3$ resulted in a simultaneous increase of $\theta_{\text{mod}}$ and increase in the drift demands at the top storeys. Noting that this parameter can be altered easily in the design process, a study was conducted to evaluate the design value of the relative storey stiffness ratio ($\beta_d$) that would result in a uniform distribution of the drift demands along the height of the designed frame. To this end, 24 MRFs with six variations of $T_1$ and four variations of $\beta_3$ were subjected to three frequency content scenarios. Each frequency content scenario comprised of 24 far-field ground motions. Furthermore, the ground motions were scaled to simulate four behaviour factors. For each scenario, the relative storey drift ratio ($\chi$), defined as the ratio of the maximum drift for the upper half of the frame to the maximum drift for the lower half of the frame, was evaluated. For a given $T_1$, $T_m$ and $q$, $\beta_d$ was calculated by interpolating for $\chi$ and defining $\beta_d$ as the value for which $\chi$ is equal to unity. $\beta_d$ was then subjected to parametric study with respect to $T_1$, $T_m$ and $q$. It was concluded that the parameters that significantly influence $\beta_d$ are $T_1$ and $T_m$. It was observed that $\beta_d$ required for the frame to develop uniform drift demands decreased with the increase in $T_1/T_m$. Furthermore, a simple relationship was proposed for the prediction of $\beta_d$ as a function of $T_1/T_m$.  

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The next section summarizes the conclusions from the comparative study of the model developed in this thesis with the codified provisions for the prediction of the drift and strength demands.

**9.1.4. Comparative studies and design considerations**

Finally, the predictive models developed in this thesis were compared with European and the US seismic design and assessment provisions for the drift and strength demands.

For the prediction of the drift demands of a given structure for seismic assessment, both provisions employ the displacement modification method, which, in turn, uses the inelastic displacement ratio \( C_1 \), as defined earlier. The models proposed by the code provisions for the computation of \( C_1 \) are obtained from the dynamic analysis of a large number of SDF systems. Therefore, the code provisions rely on the assumption that a SDF system can be used for the inelastic response prediction of MDF system. In the models proposed by the provisions, \( C_1 \) is a function of \( T_{\text{eff}}, T_c \) and \( q \), for \( T_{\text{eff}}/T_c \) lower than 1; whereas \( C_1 \) is taken to be unity, for \( T_{\text{eff}}/T_c \) greater than 1. The models proposed by the code provisions were modified as a function of \( T_1/T_m \) to replace \( T_{\text{eff}}/T_c \) in order to carry out the comparisons with the relationships for the drift demands proposed in the thesis. It was noted that the European and the US codes propose different methods for the calculation of \( T_{\text{eff}} \). Using these methods, \( T_{\text{eff}} \) of the frames was calculated for all the MRFs used in the study and a relationship between \( T_{\text{eff}} \) and \( T_1 \) was developed. \( T_{\text{eff}} \) was found to be 1.30\( T_1 \) and 1.01\( T_1 \) for European and US provisions, respectively. Using the 128 far-field ground motion records, \( T_m \) was found to be consistently about 5% higher than \( T_c \). Subsequently, the comparison of the drift demands between the seismic assessment models proposed by the European and the US provisions and the predictive relationships showed that the models proposed by the codes severely overpredict both the global and the maximum drift demands for all period ratios and behaviour factors. However, the European code predictions were found to be closer to that of the predictions using the relationships proposed in this thesis due to the higher \( T_{\text{eff}} \) obtained using the method proposed in the European provisions. On the other hand, for seismic design, the codes propose simple models for the estimation of the drift demands. The European provisions propose the equal displacement rule, whereas the US provisions propose the model as
a function of q. It was noted, however, that the US provisions provided reasonable predictions for the intermediate $T_1/T_m$ range. It was shown that the models proposed by the codes are too simplistic, therefore are unable to capture the influence of various parameters on the drift demands of the frames.

For the prediction of the strength demands, European and US provisions propose the use of pushover analysis for assessment purposes. On the other hand, for seismic design, codes recommend the evaluation of the strength demands at the formation of plastic hinges in the beams. The comparison of code predictions for assessment and design with the predictive relationships for strength demands proposed in this thesis showed that the code provisions severely underestimate the strength demands.

9.2. Recommendations for Future Research

This thesis focused primarily on understanding of influence of frequency content and structural characteristics on the drift and strength demands using steel MRFs with 3, 5 and 7 storeys incorporating non-deteriorating hysteretic behaviour. An initial part of the work was also devoted to the study of various factors influencing the inelastic behaviour of SDF systems with bilinear hysteretic behaviour. This study can serve as a basis for research in various relevant topics, including the following:

- In this thesis, the study of the influence of frequency content on SDF and MDF systems was carried out using far-field ground motions. A similar study can be conducted using the records with near-field effects.
- The investigation into the influence of the characteristics of ground motion was limited to frequency content. It would be interesting to explore the influence of the duration of ground motion on the inelastic response of the structures, which may play a key role in the case of structures with deteriorating hysteretic behaviour.
- The inelastic response studies of SDF systems were performed using the displacement coefficient method. A significant influence of frequency content (expressed in terms of $T_m$) on the inelastic displacement ratios was noted. However, it is observed that the models that employ the equivalent linearization method do not incorporate the influence of the frequency
content. Therefore, there is a need to revisit the equivalent linearization models and evaluate the influence of the frequency content on hysteretic damping.

- The drift and strength demands, in this thesis, were considered mainly from a force-based design perspective; as a result the predictive relationships were developed as a function of the behaviour factor. Similar studies can be carried out from the perspective of displacement based design, and the predictive relationships can be proposed as a function of the global ductility demands.

- The assessment of the influence of frequency content and structural characteristics should be extended to the axial demands of columns, for a more accurate prediction of these effects compared to approximate indirect approaches.

- The studies presented in this thesis were limited to medium rise steel MRFs. The methodology adopted in this study can be extended to other MRFs (higher than 7-storey frames) and other structural systems including: reinforced concrete frames, braced frames, and others.
References


ASCE/SEI (2005). *ASCE 7-05 - Minimum design loads for buildings and other structures*, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA.


SeismoSoft (2008) - *SeismoSignal v.3.2.0*. [www.seismosoft.com](http://www.seismosoft.com)


Appendix A

In order to model the frames in OpenSees for pushover analysis, five tcl files are required, as enumerated below:

1. Pushover.tcl
2. 3Storey_FBE.tcl
3. Analyze_Static.Push.tcl
4. Units_(mks).tcl
5. Wsection.tcl

The above stated files for a 3-storey steel moment resisting frame are presented below:

1. Pushover.tcl

    puts "-- Uniaxial Inelastic Material, Fiber RC-Section, Nonlinear Model --"
    puts "-- Uniform Earthquake Excitation --"

    source 3Storey_FBE.tcl
    source Analyze_Static.Push.tcl
1. 3Storey_FBE.tcl

#**********************----------------**************************
#
# Model of 3 Storey Steel Frame Designed to EC8 Frame ID 1
# Modeled by Kumar, M., Sept 15 2010
#
#************ BASIC STEPS **********************
#************ SET UP **********************

wipe;                        # clear memory of all past model definitions
model BasicBuilder -ndm 2 -ndf 3;   # Define the model builder, ndm = dimension, ndf=dofs

#************INCLUDE TCL FILES TO BE CALLED LATER***************

source Units_(mks).tcl;         # Units definition
source Wsection.tcl;            # procedure to define fiber W section

#************ SET DIRECTORIES ***************

set dataDir DATA;               # set up name of data directory (can remove this)
file mkdir $dataDir;             # create data directory

#************ DEFINE GEOMETRY OF FRAME ***************

set LBay [expr 6*$m];            # Bay Length
set HBSCol [expr 4.5*$m];        # Length of Bottom Storey Columns
set HOSCol [expr 3.5*$m];        # Length of Other Storeys Columns
set NStory 3;                    # number of stories above ground level (can be modified)
set NBay 3;                      # number of bays (max 9) can be modified
set NElem 1;
set tmpNElem $NElem;
set TmpNBay $NBay;

set Coord [open $dataDir/Coord.out w]
#************** DEFINE NODAL COORDINATES OF FRAME ****************

set X 0;
set Y 0;
set CurrStory $HBSCol;
set NLHeight 0;
set InitNID 100;

for {set Pier 1} {$Pier < [expr $NBay+2]} {incr Pier 1} {

    set InitNID [expr 100*$Pier]
    set X [expr $LBay*($Pier-1)]
    set nodeID $InitNID;
    node $nodeID $X $Y;

    for {set level 0} {$level < $NStory} {incr level 1} {
        if {$level > 0} {set Currstory $HOSCol} else {set Currstory $HBSCol}
        if {$level > 0} {set NLHeight [expr $HBSCol+($level-1)*$HOSCol]}
        for {set ElmCount 1} {$ElmCount < $NElem} {incr ElmCount 1} {
            set Y [expr ($NLHeight+ $ElmCount*($Currstory/$NElem))]
            set nodeID [expr $InitNID + ($level*10)+$ElmCount]
            node $nodeID $X $Y;
            #puts $Coord "$nodeID $X $Y"
        }
    }
}

set Y 0;
set NLHeight 0;
}

set X 0;
set Y $HBSCol;
for {set Pier 1} {$Pier < [expr $NBay+2]} {incr Pier 1} {

    set InitNID [expr (1000+100*$Pier)]

}
set NBLength [expr $LBay*($Pier-1)]

for {set level 1} {$level < [expr $NStory+1]} {incr level 1} {
    set InitNID [expr ((1000*$level)+(100*$Pier))]
    set Y [expr $HBSCol+($level-1)*$HOSCol]
    if {$Pier == [expr $NBay+1]} {set tmpNElem 1}
    for {set ElmCount 0} {$ElmCount < $tmpNElem} {incr ElmCount 1} {
        set X [expr ($NBLength+ $ElmCount*($LBay/$tmpNElem))]
        set nodeID  [expr $InitNID + $ElmCount]
        node $nodeID $X  $Y;
        #puts $Coord "$nodeID $X $Y"
    }
    set NBLength [expr $LBay*($Pier-1)];
}

#Specify support nodes
set iSupportNode ""
set level 0
for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
    set nodeID [expr $pier*100]
    lappend iSupportNode $nodeID
}

#************ BOUNDARY CONDITIONS ************

#There is no loop here so if more than 3 bays One has to input Manually!
fix 100 1 1 1;
fix 200 1 1 1;
fix 300 1 1 1;
fix 400 1 1 1;
#*********** MATERIAL PROPERTIES ***********

set Fy [expr 275*$MPa]
set Es [expr 200000.0*$MPa];  # Steel Young's Modulus
set Hiso 0
set Hkin 1005;  #0.5% Post Yield Stiffness
set matIDhard 1
uniaxialMaterial Hardening $matIDhard $Es $Fy $Hiso $Hkin

#*********** ELEMENT PROPERTIES ***********

#********** Beam sections: IPE360 **********

set d [expr 0.360*$m];  # depth
set bf [expr 0.160*$m];  # flange width
set tf [expr 0.0127*$m];  # flange thickness
set tw [expr 0.008*$m];  # web thickness
set nfdw 16;  # number of fibers along dw
set nftw 2;  # number of fibers along tw
set nfbf 16;  # number of fibers along bf
set nftf 4;  # number of fibers along tf
Wsection 1 $matIDhard $d $bf $tf $tw $nfdw $nftw $nfbf $nftf

#********** Beam sections: IPE400 **********

set d [expr 0.4*$m];  # depth
set bf [expr 0.180*$m];  # flange width
set tf [expr 0.0127*$m];  # flange thickness
set tw [expr 0.008*$m];  # web thickness
set nfdw 16;  # number of fibers along dw
set nftw 2;  # number of fibers along tw
set nfbf 16;  # number of fibers along bf
set nftf 4;  # number of fibers along tf
Wsection 2 $matIDhard $d $bf $tf $tw $nfdw $nftw $nfbf $nftf
********** Column sections: HE400B **********

set d [expr 0.4*$m]; # depth
set bf [expr 0.300*$m]; # flange width
set tf [expr 0.024*$m]; # flange thickness
set tw [expr 0.0135*$m]; # web thickness
set nfdw 16; # number of fibers along dw
set nftw 2; # number of fibers along tw
set nfbf 16; # number of fibers along bf
set nftf 4; # number of fibers along tf
Wsection 3 $matIDhard $d $bf $tf $tw $nfdw $nftw $nfbf $nftf

************DEFINE ELEMENTS ************

# set up geometric transformations of element
# separate columns and beams, in case of P-Delta analysis for columns

set IDColTransf 1; # all columns
set IDBeamTransf 2; # all beams
#set ColTransfType Corotational; # options, Linear PDelta Corotational
set ColTransfType Corotational;
geomTransf $ColTransfType $IDColTransf ; # only columns can have PDelta effects (gravity effects)
geomTransf Linear $IDBeamTransf

# Define Beam-Column Elements
set np 7; # number of Gauss integration points (np=2 for linear distribution ok)

# COLUMNS

set CollID 4;
set NLHeight 0;
set InitNID 100;
set Y 0;
for {set Pier 1} {$Pier < [expr $NBay+2]} {incr Pier 1} {

set initNID [expr 100*$Pier]

for {set level 0} {$level < $NStory} {incr level 1} {

if {$level > 1} {set CoID 3} else {set CoID 3}

for {set ElmCount 1} {$ElmCount < [expr $NElem+1]} {incr ElmCount 1} {

if {$ElmCount == 1} {set nodeI [expr (1000*$level+(100*$Pier))]} else {set nodeI [expr $initNID + ($level*10)+($ElmCount-1)]}

if {$ElmCount == $NElem} {set nodeJ [expr ((1000*($level+1))+(100*$Pier))]} else {set nodeJ [expr $initNID + ($level*10)+($ElmCount)]}

set elemID [expr ($initNID+($level*10)+$ElmCount)];
puts $Coord "$elemID $nodeI $nodeJ $CoID $level $Pier";
element nonlinear BeamColumn $elemID $nodeI $nodeJ $np $CoID $IDColTransf
#element dispBeamColumn $elemID $nodeI $nodeJ $np $CoID $IDColTransf

}
}

# BEAMS

set BeamID 2;

for {set Pier 1} {$Pier < [expr $NBay+1]} {incr Pier 1} {

for {set level 1} {$level < [expr $NStory+1]} {incr level 1} {

if {$level > 2} {set BeamID 1} else {set BeamID 2}
#if {$level > 5} {set BeamID 3}

}
for {set ElmCount 1} {$ElmCount < [expr $NElem+1]} {incr ElmCount 1} {
    if {$ElmCount == 1} {set nodeI [expr (1000*$level+(100*$Pier))]} else {set nodeI [expr (1000*$level+(100*$Pier)+($ElmCount-1))]} if {$ElmCount == $NElem} {set nodeJ [expr (1000*$level+(100*($Pier+1)))]} else {set nodeJ [expr (1000*$level+(100*$Pier))+($ElmCount)]}
    set elemID [expr $nodeI+1]; puts $Coord "$elemID $nodeI $nodeJ $BeamID $level $Pier";
    element nonlinearBeamColumn $elemID $nodeI $nodeJ $np $BeamID $IDBeamTransf
    #element dispBeamColumn $elemID $nodeI $nodeJ $np $BeamID $IDBeamTransf
}

**********ASSIGN NODAL MASSES ************

for {set level 1} {$level <= $NStory-1} {incr level 1} {
    for {set pier 1} {$pier < [expr ($NBay+2)]} {incr pier 1} {
        set nodeID [expr ($level*1000)+($pier*100)]
        if {$pier == 1 || $pier == 4} {mass $nodeID 11650 1e-9 0} else {mass $nodeID 23300 1e-9 0}
        puts "$pier $nodeID";
    }
}

mass 3100 9270 1e-9 0;
mass 3200 18540 1e-9 0;
mass 3300 18540 1e-9 0;
mass 3400 9270 1e-9 0;
#************ASSIGN GRAVITY LOADS ************

set LExtCol 57150;
set LIntBmCol 114300;

pattern Plain 101 Linear {
for {set level 1} {$level <= $NStory} {incr level 1} {
    if {$level == 3} {
        set LExtCol 45450;
        set LIntBmCol 90900;
    }
    for {set pier 1} {$pier < [expr ($NBay+2)]]} {incr pier 1} {
        set nodeID [expr ($level*1000)+($pier*100)]
        if {$pier == 1 || $pier == 4} {load $nodeID 0 -$LExtCol 0} else {load $nodeID 0 -$LIntBmCol 0}
        if {$pier < 4} {
            if {$NElem == 1} {
                set elemID [expr (($level*1000)+($pier*100))+1]
                eleLoad -ele $elemID -type -beamPoint -$LIntBmCol 0.5;
            } else {
                set nodeID [expr (($level*1000)+($pier*100))+($NElem/2)]
                load $nodeID 0 -$LIntBmCol 0;
            }
        }
    }
}
}
#**********ASSIGN CONTROL NODES**********

set IDctrlNode [expr ($NStory)*1000+100];  # node where displacement is read
set IDctrlDOF 1;  # degree of freedom of displacement read

#**********LATERAL LOAD DISTRIBUTION**********

set iFj(2) 1;
set iFj(1) 0.94;
set iFj(0) 0.49;

# create node and load vectors for lateral-load distribution in static analysis
set iFPush ""
set iNodePush ""

for {set level 1} {$level <= [expr $NStory]} {incr level 1} {
    set FPush $iFj([expr ($level-1)]);  # lateral load coefficient
    set nodeID [expr ($level*1000)+ 100]
    lappend iNodePush $nodeID
    lappend iFPush $FPush
}
puts "Model Built"

#**********RECORDERS**********

#********** Recorders for displacement and drift ***********

recorder Node -file $dataDir/DFree.out -time -node 3400 -dof 1 disp;
recorder Node -file $dataDir/RBase.out -node 100 200 300 400 -dof 1 reaction;
recorder Drift -file $dataDir/Hist_Drift_Level1.out -time -iNode 400 -jNode 1400 -dof 1 -perpDirn 2;
recorder Drift -file $dataDir/Hist_Drift_Level2.out -time -iNode 1400 -jNode 2400 -dof 1 -perpDirn 2;
recorder Drift -file $dataDir/Hist_Drift_Level3.out -time -iNode 2400 -jNode 3400 -dof 1 -perpDirn 2;
recorder Element -file $dataDir/Col_StsStr_1.out -time -ele 101 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_2.out -time -ele 101 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_3.out -time -ele 201 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_4.out -time -ele 201 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_5.out -time -ele 301 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_6.out -time -ele 301 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_7.out -time -ele 401 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_8.out -time -ele 401 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_9.out -time -ele 111 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_10.out -time -ele 111 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_11.out -time -ele 211 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_12.out -time -ele 211 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_13.out -time -ele 311 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_14.out -time -ele 311 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_15.out -time -ele 411 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_16.out -time -ele 411 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_17.out -time -ele 121 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_18.out -time -ele 121 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_19.out -time -ele 221 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_20.out -time -ele 221 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_21.out -time -ele 321 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_22.out -time -ele 321 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_23.out -time -ele 421 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Col_StsStr_24.out -time -ele 421 section 7 fiber 0.2 0.2 stressStrain

#********* Recorders for stress and strain in beams *************

recorder Element -file $dataDir/Bm_StsStr_1.out -time -ele 1101 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_2.out -time -ele 1101 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_3.out -time -ele 1201 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_4.out -time -ele 1201 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_5.out -time -ele 1301 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_6.out -time -ele 1301 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_7.out -time -ele 2101 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_8.out -time -ele 2101 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_9.out -time -ele 2201 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_10.out -time -ele 2201 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_11.out -time -ele 2301 section 1 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_12.out -time -ele 2301 section 7 fiber 0.2 0.2 stressStrain
recorder Element -file $dataDir/Bm_StsStr_13.out -time -ele 3101 section 1 fiber 0.18 0.18 stressStrain
recorder Element -file $dataDir/Bm_StsStr_14.out -time -ele 3101 section 7 fiber 0.18 0.18 stressStrain
recorder Element -file $dataDir/Bm_StsStr_15.out -time -ele 3201 section 1 fiber 0.18 0.18 stressStrain
recorder Element -file $dataDir/Bm_StsStr_16.out -time -ele 3201 section 7 fiber 0.18 0.18 stressStrain
recorder Element -file $dataDir/Bm_StsStr_17.out -time -ele 3301 section 1 fiber 0.18 0.18 stressStrain
recorder Element -file $dataDir/Bm_StsStr_18.out -time -ele 3301 section 7 fiber 0.18 0.18 stressStrain

#******** Recorders for element internal forces ************

recorder Element -file $dataDir/Fel1.out -time -ele 101 localForce;
recorder Element -file $dataDir/Fel2.out -time -ele 201 localForce;
recorder Element -file $dataDir/Fel3.out -time -ele 301 localForce;
recorder Element -file $dataDir/Fel4.out -time -ele 401 localForce;
recorder Element -file $dataDir/Fel5.out -time -ele 111 localForce;
recorder Element -file $dataDir/Fel6.out -time -ele 211 localForce;
recorder Element -file $dataDir/Fel7.out -time -ele 311 localForce;
recorder Element -file $dataDir/Fel8.out -time -ele 411 localForce;
recorder Element -file $dataDir/Fel9.out -time -ele 121 localForce;
recorder Element -file $dataDir/Fel10.out -time -ele 221 localForce;
recorder Element -file $dataDir/Fel11.out -time -ele 321 localForce;
recorder Element -file $dataDir/Fel12.out -time -ele 421 localForce;
#************ Gravity-analysis parameters ************

set Tol 1.0e-8;                     # convergence tolerance for test
variable constraintsTypeGravity Plain;    # default;
if { [info exists RigidDiaphragm] == 1} {
    if {$RigidDiaphragm=="ON"} {
        variable constraintsTypeGravity Lagrange;     # if rigid diaphragm is on
    }
};                                   # if rigid diaphragm exists
constraints $constraintsTypeGravity ;     # how it handles boundary conditions
# renumber dof's to minimize band-width (optimization), if you want to
numberer RCM;
# how to store and solve the system of equations in the analysis (large model: try UmfPack)
system BandGeneral ;
# determine if convergence has been achieved at the end of an iteration step
test NormDispIncr $Tol 6 ;
algorithm Newton;                    # updates tangent stiffness at every iteration
set NstepGravity 10;                # apply gravity in 10 steps
set DGravity [expr 1./$NstepGravity]; # first load increment;
integrator LoadControl $DGravity;     # determine the next time step for an analysis
analysis Static;                    # define type of analysis static or transient
analyze $NstepGravity;               # apply gravity

# maintain constant gravity loads and reset time to zero
loadConst -time 0.0
set eigenvalues [eigen frequency 3]
set Fr [lindex $eigenvalues 0]
set OmegaOPS [expr sqrt($Fr)]
set TP [expr (6.283/$OmegaOPS)]
puts "Final Time period from OpenSees = $TP sec"
close $Coord;
2. **Analyze.Static.Push.tcl.tcl**

```
# --------------------------------------------------------------------------------------------------
# Modelled by Silvia Mazzoni & Frank McKenna, 2006
# Execute this file after you have built the model, and after you apply gravity

# characteristics of pushover analysis
set Dmax [expr 0.5*$m];    # maximum displacement of pushover. push to 10% drift.
set Dincr [expr 0.005];    # displacement increment.

# -- STATIC PUSHOVER/CYCLIC ANALYSIS
# create load pattern for lateral pushover load coefficient when using linear load pattern
pattern Plain 200 Linear {;
# define load pattern
# these vectors have been defined with the structure
foreach FPush $iFPush NodePush $iNodePush {;
load $NodePush  $FPush 0.0 0.0
}
};    # end load pattern

# display deformed shape:
set ViewScale 5;
# display deformed shape, the scaling factor needs to be adjusted for each model
DisplayModel2D DeformedShape $ViewScale ;
# a window to plot the load vs. nodal displacement
recorder plot $dataDir/DFree.out ForceDisp 910 10 400 400 -columns 2 1;

# ----------------------------------------------- PERFORM STATIC PUSHOVER ANALYSIS
# -------- set up analysis parameters
     # constraintsHandler,DOFnumberer,systemofequations,convergenceTest,solutionAlgorithm
source LibAnalysisStaticParameters.tcl;
     #set fmt1 "%s Pushover analysis: CtrlNode %.3i, dof %.1i, Disp=%.4f %s";  # format
# for screen/file output of DONE/PROBLEM analysis
     # --------------------------------------------- first analyze command -------------------------
set Nsteps [expr int($Dmax/$Dincr)];    # number of pushover analysis steps
# this will return zero if no convergence problems were encountered
```
set ok [analyze $Nsteps];

# ----------------------------------if convergence failure-------------------------

set Tol 1e-6;

if {$ok != 0} {
    # if analysis fails, we try some other stuff; performance is slower inside this loop
    set Dstep 0.0;
    set ok 0
    while {$Dstep <= 1.0 && $ok == 0} {
        set controlDisp [nodeDisp $IDctrlNode $IDctrlDOF]
        set Dstep [expr $controlDisp/$Dmax]
        set ok [analyze 1]
        # if analysis fails, we try some other stuff
        # performance is slower inside this loop
        global maxNumIterStatic;  # max no. of iterations performed before "failure to converge" is ret'd
        if {$ok != 0} {
            puts "Trying Newton with Initial Tangent .."
            test NormDispIncr $Tol 2000 0
            algorithm Newton -initial
            set ok [analyze 1]
            test $testTypeStatic $TolStatic $maxNumIterStatic 0
            algorithm $algorithmTypeStatic
        }
        if {$ok != 0} {
            puts "Trying Broyden .."
            algorithm Broyden 8
            set ok [analyze 1]
            algorithm $algorithmTypeStatic
        }
        if {$ok != 0} {
            puts "Trying NewtonWithLineSearch .."
            algorithm NewtonLineSearch 0.8
            set ok [analyze 1]
            algorithm $algorithmTypeStatic
        }
    }
if {$ok != 0 } {
    puts "Failed"
    #puts [format $fmt1 "PROBLEM" $IDctrlNode $IDctrlDOF [nodeDisp $IDctrlNode $IDctrlDOF] $LunitTXT]
} else {
    puts "Successful"
    #puts [format $fmt1 "DONE" $IDctrlNode $IDctrlDOF [nodeDisp $IDctrlNode $IDctrlDOF] $LunitTXT]
}
3. **source Units_(mks).tcl**

```tcl
puts "Units Definition Started"
set m 1.; # Define Basic Unit - Length
set kg 1.; # Define Basic Unit - Mass
set sec 1.; # Define Basic Unit - Time
set rad 1.; # Define Basic Unit - Radians
set N [expr $kg*$m/pow($sec,2)]; # Define Engineering Units - Newton
set kN [expr $N*1000]; # Define Engineering Units - Kilo Newton
set Pa [expr $N/pow($m,2)]; # Define Engineering Units - Pascals
set MPa [expr $Pa*1000000]; # Define Engineering Units - Mega Pascal
set cm [expr $m/100]; # Define Engineering Units - Centimeter
set mm [expr $m/1000]; # Define Engineering Units - Milimeter
set g [expr 9.81*$m/pow($sec,2)]; # Gravitational Acceleration
set PI [expr 2*asin(1.0)]; # PI constant 3.14159
```

4. **Wsection.tcl**

```tcl
proc Wsection { secID matID d bf tf tw nfdw nftw nfbf nftf } {
    # Wsection $secID $matID d bf tf tw nfdw nftw nfbf nftf
    # create a standard W section given the nominal section properties
    # written: Remo M. de Souza
    # date: 06/99
    # modified: 08/99 (according to the new general modelbuilder)
    # input parameters
    # secID - section ID number
}```
# matID - material ID number
# d  = nominal depth
# tw = web thickness
# bf = flange width
# tf = flange thickness
# nfdw = number of fibers along web depth
# nftw = number of fibers along web thickness
# nfbf = number of fibers along flange width
# nftf = number of fibers along flange thickness

set dw [expr $d - 2 * $tf]
set y1 [expr -$d/2]
set y2 [expr -$dw/2]
set y3 [expr  $dw/2]
set y4 [expr  $d/2]

set z1 [expr -$bf/2]
set z2 [expr -$tw/2]
set z3 [expr  $tw/2]
set z4 [expr  $bf/2]

section fiberSec $secID { 
    #                     nfIJ   nfJK    yI  zI    yJ  zJ    yK  zK    yL  zL
    patch quadr $matID $nfbf $nftf   $y1 $z4   $y1 $z1   $y2 $z1   $y2 $z4
    patch quadr $matID $nftw $nfdw   $y2 $z3   $y2 $z2   $y3 $z2   $y3 $z3
    patch quadr $matID $nfbf $nftf   $y3 $z4   $y3 $z1   $y4 $z1   $y4 $z4
}
}
Appendix B

Table B-1: Catalogue of earthquakes used for inelastic response assessment SDF and MDF systems in Chapter 4. In the column for mechanism of earthquake (RV = Reverse, SS = Strike slip, and RO = Reverse Oblique).

<table>
<thead>
<tr>
<th>Earthquake Name</th>
<th>Magnitude</th>
<th>Mechanism</th>
<th>Record ID</th>
<th>R (km)</th>
<th>Preferred NEHRP</th>
<th>T_m (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fruili, Italy-03 1976-09-11</td>
<td>5.5</td>
<td>RV</td>
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<th>Tm (s)</th>
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Appendix C

In order to model the frames in OpenSees for incremental dynamic analysis, the following tcl files are required, as enumerated below:

1. THA_1EPM_FB.tcl
2. Add_Global_Variables.tcl
3. 3Storey_FBE_THA.tcl
4. Analyze.Dynamic.EQ.Uniform.tcl
5. Units_(mks).tcl
6. Wsection.tcl

Units_(mks).tcl and Wsection.tcl are provided already in Appendix A. The remaining four files for a 3-storey steel moment resisting frame are presented as follows:

1. THA_1EPM_FB.tcl

    puts " -- -----------------------------------------------"
    puts " -- Uniaxial Inelastic Material, Fiber RC-Section, Nonlinear Model --"
    puts " -- Uniform Earthquake Excitation --"

    source Add_Global_Variables.tcl

    set GMID [expr 10+10*0];
    set DurID [expr 13+10*0];

    set GMF1 [expr 15+10*0];
    set GMF2 [expr 16+10*0];
    set GMF3 [expr 17+10*0];
set GMF4 [expr 18+10*0];
set GMF5 [expr 19+10*0];

set CntGM 1;

for {set CntGM 1} {$CntGM < 76} {incr CntGM} {

set GMFacts [read [open ScalingFactor.txt]]

set GMfile [lindex $GMFacts $GMID]

set GMF(1) [lindex $GMFacts $GMF1]
set GMF(2) [lindex $GMFacts $GMF2]
set GMF(3) [lindex $GMFacts $GMF3]
set GMF(4) [lindex $GMFacts $GMF4]
set GMF(5) [lindex $GMFacts $GMF5]

set TmaxAns [lindex $GMFacts $DurID]

set j 1;
# set up name of data directory (can remove this)
set dataDir $GMfile;
file mkdir $dataDir;  # create data directory

for {set j 1} {$j < 6} {incr j} {

set GMfact $GMF($j)
puts "Factor is $GMfact"

source 3Storey_4NP_FBE.tcl
source Ex6.genericFrame2D.analyze.Dynamic.EQ.Uniform.tcl

set k [expr $k+1];
}

puts "Ground motion $GMfile Finito!"

set k 1;
set j 1;

set GMID [expr $GMID+10];

set GMF1 [expr $GMF1+10];
set GMF2 [expr $GMF2+10];
set GMF3 [expr $GMF3+10];
set GMF4 [expr $GMF4+10];
set GMF5 [expr $GMF5+10];

set DurID [expr $DurID+10];

}

puts "Ground motion $GMfile Finito!"
2. Add_Global_Variables.tcl

# A global Variable to control file names for the targeted drift levels

variable k 1;
set GMfile "FOC-NS" ; # ground-motion filenames
set TmaxAns 0;
set ok 0;

3. 3Storey_FBE_THA.tcl

#**************************************************
# Model of 3 Storey Steel Frame Designed to EC8
# Modeled by Kumar, M., Sept 15 2010
#
#
#************** BASIC STEPS ****************
#************** SET UP ******************

wipe; # clear memory of all past model definitions
model BasicBuilder -ndm 2 -ndf 3;

#**********INCLUDE TCL FILES TO BE CALLED LATER***************

source Units_(mks).tcl; # Units definition
source Wsection.tcl; # procedure to define fiber W section

#********** SET DIRECTORIES **********

# set up name of data directory (can remove this)
set dataDir $GMfile;
file mkdir $dataDir; # create data directory
set GMdir "./GMfiles/"; # ground-motion file directory
#************ DEFINE GEOMETRY ****************
set LBay [expr 6*$m];          # Bay Length
set HBSCol [expr 4.5*$m];      # Length of Bottom Storey Columns
set HOSCol [expr 3.5*$m];      # Length of Other Storeys Columns
set NStory 3;                  # number of stories above ground level
set NBay 3;                    # number of bays (max 9)
set NElem 1;
set tmpNElem $NElem;
set TmpNBay $NBay;

set Coord [open $dataDir/Coord.out w]

#************ DEFINE NODAL COORDINATES***************

set Y 0;
set CurrStory $HBSCol;
set NLHeight 0;
set InitNID 100;

for {set Pier 1} {$Pier < [expr $NBay+2]} {incr Pier 1} { 
set InitNID [expr 100*$Pier]
set X [expr $LBay*($Pier-1)]
set nodeID $InitNID;
node $nodeID $X $Y

for {set level 0} {$level < $NStory} {incr level 1} { 
  if {$level > 0} {set Currstory $HOSCol} else {set Currstory $HBSCol}
  if {$level > 0} {set NLHeight [expr $HBSCol+($level-1)*$HOSCol]}
  for {set ElmCount 1} {$ElmCount < $NElem} {incr ElmCount 1} { 
    set Y [expr ($NLHeight+ $ElmCount*($Currstory/$NElem))]
    set nodeID [expr $InitNID + ($level*10)+$ElmCount]
    node $nodeID $X $Y;
  }
}
set Y 0;
set NLHeight 0;
}

set X 0;
set Y $HBSCol;
for {set Pier 1} {$Pier < [expr $NBay+2]} {incr Pier 1} {

set InitNID [expr (1000+100*$Pier)]
set NBLength [expr $LBay*(Pier-1)]

for {set level 1} {$level < [expr $NStory+1]} {incr level 1} {
    set InitNID [expr ((1000*$level)+(100*$Pier))]
    set Y [expr $HBSCol+($level-1)*$HOSCol]
    if {$Pier == [expr $NBay+1]} {set tmpNElem 1}
    for {set ElmCount 0}  {$ElmCount < $tmpNElem} {incr ElmCount 1} {
        set X [expr ($NBLength+ $ElmCount*($LBay/$tmpNElem))]
        set nodeID [expr $InitNID + $ElmCount]
        node $nodeID $X $Y;
    }
    set NBLength [expr $LBay*(Pier-1)];
}
}

#define support nodes where ground motions are input
set level 0
for {set pier 1} {$pier <= [expr $NBay+1]} {incr pier 1} {
    set nodeID [expr spier*100]
    #puts $nodeID
    lappend iSupportNode $nodeID
}
# BOUNDARY CONDITIONS There is no loop here so if more than 3 bays One has to input Manually!

fix 100 1 1 1;
fix 200 1 1 1;
fix 300 1 1 1;
fix 400 1 1 1;

#************* DEFINE MATERIAL PROPERTIES****************

set Fy [expr 275*$MPa]
set Es [expr 200000.0*$MPa]; # Steel Young's Modulus
set Hiso 0
set Hkin 1005; #0.5% Post Yield Stiffness
set matIDhard 1
uniaxialMaterial Hardening $matIDhard $Es $Fy $Hiso $Hkin

#***** ELEMENT properties ******

#******** Beam sections: IPE360 ********

set d [expr 0.360*$m]; # depth
set bf [expr 0.160*$m]; # flange width
set tf [expr 0.0127*$m]; # flange thickness
set tw [expr 0.008*$m]; # web thickness
set nfdw 16; # number of fibers along dw
set nftw 2; # number of fibers along tw
set nfbf 16; # number of fibers along bf
set nftf 4; # number of fibers along tf
Wsection 1 $matIDhard $d $bf $tf $tw $nfdw $nftw $nfbf $nftf

#******** Beam sections: IPE400 ********

set d [expr 0.4*$m]; # depth
set bf [expr 0.180*$m]; # flange width
set tf [expr 0.0127*$m]; # flange thickness
set tw [expr 0.008*$m]; # web thickness
set nfdw 16; # number of fibers along dw
set nftw 2; # number of fibers along tw
set nfbf 16;  # number of fibers along bf
set nftf 4;   # number of fibers along tf
Wsection 2 $matIDhard $d $bf $tf $tw $nfdw $nftw $nfbf $nftf

#********** Column sections: HE400B **********
set d [expr 0.4*$m];  # depth
set bf [expr 0.300*$m]; # flange width
set tf [expr 0.024*$m]; # flange thickness
set tw [expr 0.0135*$m]; # web thickness
set nfdw 16;        # number of fibers along dw
set nftw 2;         # number of fibers along tw
set nfbf 16;        # number of fibers along bf
set nftf 4;         # number of fibers along tf
Wsection 3 $matIDhard $d $bf $tf $tw $nfdw $nftw $nfbf $nftf

set IDColTransf 1;  # all columns
set IDBeamTransf 2; # all beams
set ColTransfType Corotational;
geomTransf $ColTransfType $IDColTransf
geomTransf Linear $IDBeamTransf

# Define Beam-Column Elements
set np 7;
set ColID 4;
set NLHeight 0;
set InitNID 100;
set Y 0;

for {set Pier 1} {$Pier < [expr $NBay+2]} {incr Pier 1} {

set initNID [expr 100*$Pier]

for {set level 0} {$level < $NStory} {incr level 1} {

if {$level > 1} {set ColID 3} else {set ColID 3}
for {set ElmCount 1} {$ElmCount < [expr $NElem+1]} {incr ElmCount 1} {
    if {$ElmCount == 1} {set nodeI [expr (1000*$level+(100*$Pier))]} else {set nodeI [expr $initNID + ($level*10)+($ElmCount-1)]}
    if {$ElmCount == $NElem} {set nodeJ [expr ((1000*$level+1)+(100*$Pier))]} else {set nodeJ [expr $initNID + ($level*10)+($ElmCount)]}
    set elemID [expr ($initNID+($level*10)+$ElmCount)];
    puts $Coord "$elemID $nodeI $nodeJ $ColID $level $Pier";
    element nonlinearBeamColumn $elemID $nodeI $nodeJ $np $ColID $IDColTransf
}
}

set BeamID 2;

for {set Pier 1} {$Pier < [expr $NBay+1]} {incr Pier 1} {

    for {set level 1} {$level < [expr $NStory+1]} {incr level 1} {
        if {$level > 2} {set BeamID 1} else {set BeamID 2}

        for {set ElmCount 1} {$ElmCount < [expr $NElem+1]} {incr ElmCount 1} {
            if {$ElmCount == 1} {set nodeI [expr (1000*$level+(100*$Pier))]} else {set nodeI [expr (1000*$level+(100*$Pier))+($ElmCount-1)]}
            if {$ElmCount == $NElem} {set nodeJ [expr (1000*$level+(100*($Pier+1)))]} else {set nodeJ [expr (1000*$level+(100*$Pier))+($ElmCount)]}
            set elemID [expr $nodeI+1];
            puts $Coord "$elemID $nodeI $nodeJ $BeamID $level $Pier";
            element nonlinearBeamColumn $elemID $nodeI $nodeJ $np $BeamID $IDBeamTransf
        }
    }
}
}
#************ ASSIGN NODAL MASSES ****************

set MFact 1.0;

for {set level 1} {$level <= $NStory-1} {incr level 1} {
    for {set pier 1} {$pier < [expr ($NBay+2)]} {incr pier 1} {
        set nodeID [expr ($level*1000)+($pier*100)]
        if {$pier == 1 || $pier == 4} {mass $nodeID [expr 11650*$MFact] 1e-9 0}
        else {mass $nodeID [expr 23300*$MFact] 1e-9 0}
    }
}

mass 3100 [expr 9270*$MFact] 1e-9 0;
mass 3200 [expr 18540*$MFact] 1e-9 0;
mass 3300 [expr 18540*$MFact] 1e-9 0;
mass 3400 [expr 9270*$MFact] 1e-9 0;

set LExtCol 57150;
set LIntBmCol 114300;

pattern Plain 101 Linear {
    for {set level 1} {$level <= $NStory} {incr level 1} {
        if {$level == 3} {
            set LExtCol 45450;
            set LIntBmCol 90900;
        }
        for {set pier 1} {$pier < [expr ($NBay+2)]} {incr pier 1} {
            set nodeID [expr ($level*1000)+($pier*100)]
            if {$pier == 1 || $pier == 4} {load $nodeID 0 -[$LExtCol 0] else {load
$nodeID 0 -$LIntBmCol 0}
            if {$pier < 4} {
                if {$NElem == 1} {
                    set elemID [expr (($level*1000)+($pier*100))+1]
                    eleLoad -ele $elemID -type -beamPoint -$LIntBmCol 0.5;
                } else {
                    // Additional code
                }
            }
        }
    }
}
set nodeID [expr (($level*1000)+($pier*100))+($NElem/2)]
load $nodeID 0 -$LIntBmCol 0;
}
}
}

set IDctrlNode [expr ($NStory)*1000+100];
set IDctrlDOF 1;

#************LATERAL LOAD DISTRIBUTION ***************

set iFj(2) 1;
set iFj(1) 0.94;
set iFj(0) 0.49;

# create node and load vectors for lateral-load distribution in static analysis
set iFPush ""
set iNodePush ""
for {set level 1} {$level <= [expr $NStory]} {incr level 1} {
    set FPush $iFj([expr ($level-1)])  # lateral load coefficient
    set nodeID [expr ($level*1000)+ 100]
    lappend iNodePush $nodeID
    lappend iFPush $FPush
    #puts "$iNodePush $iFPush"
}

puts "Model Built"
#***************RECORDERS ********************
recorder EnvelopeDrift -file $dataDir/MaxNodeDrift_Level1_$k.out -iNode 400 -jNode 1400 -dof 1 -perpDirn 2;
recorder EnvelopeDrift -file $dataDir/MaxNodeDrift_Level2_$k.out -iNode 1400 -jNode 2400 -dof 1 -perpDirn 2;
recorder EnvelopeDrift -file $dataDir/MaxNodeDrift_Level3_$k.out -iNode 2400 -jNode 3400 -dof 1 -perpDirn 2;

recorder EnvelopeNode -file $dataDir/DFree_$k.out -node 3100 -dof 1 disp;
recorder Node -file $dataDir/RBase_$k.out -node 100 200 300 400 -dof 1 reaction;

recorder EnvelopeElement -file $dataDir/Fel1_$k.out -ele 101 localForce;
recorder EnvelopeElement -file $dataDir/Fel2_$k.out -ele 201 localForce;
recorder EnvelopeElement -file $dataDir/Fel3_$k.out -ele 301 localForce;
recorder EnvelopeElement -file $dataDir/Fel4_$k.out -ele 401 localForce;
recorder EnvelopeElement -file $dataDir/Fel5_$k.out -ele 111 localForce;
recorder EnvelopeElement -file $dataDir/Fel6_$k.out -ele 211 localForce;
recorder EnvelopeElement -file $dataDir/Fel7_$k.out -ele 311 localForce;
recorder EnvelopeElement -file $dataDir/Fel8_$k.out -ele 411 localForce;
recorder EnvelopeElement -file $dataDir/Fel9_$k.out -ele 121 localForce;
recorder EnvelopeElement -file $dataDir/Fel10_$k.out -ele 221 localForce;
recorder EnvelopeElement -file $dataDir/Fel11_$k.out -ele 321 localForce;
recorder EnvelopeElement -file $dataDir/Fel12_$k.out -ele 421 localForce;

#***************GRAVITY ANALYSIS PARAMTERS ***************
set Tol 1.0e-8;  # convergence tolerance for test
variable constraintsTypeGravity Plain;  # default;
if { [info exists RigidDiaphragm] == 1} {
    if {$RigidDiaphragm=="ON"} {
        variable constraintsTypeGravity Lagrange;
    }  # if rigid diaphragm is on
};  # if rigid diaphragm exists
constraints $constraintsTypeGravity ;  # how it handles boundary conditions
numberer RCM;

system BandGeneral ; # (large model: try UmfPack)
test NormDispIncr $Tol 6 ;
algorithm Newton;

set NstepGravity 10; # apply gravity in 10 steps
set DGravity [expr 1./$NstepGravity]; # first load increment;
integrator LoadControl $DGravity;
analysis Static;
analyze $NstepGravity; # apply gravity

# maintain constant gravity loads and reset time to zero
loadConst -time 0.0

set eigenvalues [eigen frequency 3]
set Fr [lindex $eigenvalues 0]
set OmegaOPS [expr sqrt($Fr)]
set TP [expr (6.283/$OmegaOPS)]
puts "Final Time period from OpenSees = $TP sec"

close $Coord;
4. Add_Global_Variables.tcl

# --------------------------------------------------------------------------------------------------
#                                  Silvia Mazzoni & Frank McKenna, 2006
# execute this file after you have built the model, and after you apply gravity

# source in procedures
source ReadSMDfile.tcl; # procedure for reading GM file and converting it to proper format
source ReadPEERNGAFile.tcl

# Uniform Earthquake ground motion (uniform acceleration input at all support nodes)
set GMdirection 1; # ground-motion direction

# set up ground-motion-analysis parameters
set DtAnalysis [expr 0.01*$sec]; # time-step Dt for lateral analysis
set TmaxAnalysis [expr $TmaxAns *$sec];
#puts $TmaxAnalysis

# ----------- set up analysis parameters
source LibAnalysisDynamicParameters.tcl;

# ----------- define & apply damping
# RAYLEIGH damping parameters, Where to put M/K-prop damping, switches (http://opensees.berkeley.edu/OpenSees/manuals/usermanual/1099.htm)
# D=alphaM*M + betaKcurr*Kcurrent + betaKcomm*KlastCommit + betaKinit*Kinitial
set xDamp 0.02; # damping ratio
set alphaM 0;
set betaKcomm 0;
#set betaKinit 0;
set betaKcurr 0;
#set MpropSwitch 0.0;
#set KcurrSwitch 0.0;
#set KcommSwitch 1.0;
set KinitSwitch 1.0;
set nEigenI 1;  # mode 1
set lambdaI [eigen 1];  # eigenvalue analysis for nEigenI modes
set omegal [expr pow($lambdaI,0.5)];
set T1 [expr (6.283/$omegal)]
set betaKinit [expr $KinitSwitch*2.*$xDamp/($omegal)];

# define damping
rayleigh $alphaM $betaKcurr $betaKinit $betaKcomm;  # RAYLEIGH DAMPING

# the following commands are unique to the Uniform Earthquake excitation
set IDloadTag 400;  # for uniformSupport excitation

# Uniform EXCITATION: acceleration input
set inFile $GMdir/$GMfile.at2
set outFile $GMdir/$GMfile.g3;
ReadPEERNGAFile $inFile $outFile dt;
set GMfatt [expr $g*$GMfact];
set AccelSeries "Series -dt $dt -filePath $outFile -factor  $GMfatt"; # time series information
pattern UniformExcitation $IDloadTag $GMdirection -accel  $AccelSeries ;

set Nsteps [expr int($TmaxAnalysis/$DtAnalysis)];
set ok [analyze $Nsteps $DtAnalysis 0.001 0.01 4];
set Tol 1.0e-8

if {$ok != 0} {
puts "analysis Failed"
}

puts "Ground Motion Done. End Time: [getTime]"