Influence of Panel Zone Design on Performance of Steel Special Moment Frame Buildings

A Thesis

by

Jones Joju

for the award of the degree of

Master of Science

(By Research)

Department of Civil Engineering

Indian Institute of Technology Madras

December 2019

Influence of Panel Zone Design on Performance of Steel Special Moment Frame Buildings

A Thesis

by

Jones Joju

(Roll No. CE17S006)

for the award of the degree of

Master of Science

(By Research)

Department of Civil Engineering

Indian Institute of Technology Madras

December 2019

Certificate

This is to certify that the thesis titled "*Influence of Panel Zone Design on Performance of Steel Special Moment Frame Buildings*" submitted by *Jones Joju (*Roll No. CE17S006) to Indian Institute of Technology Madras for the award of the degree of **Master of Science (By Research)**, is a bonafide record of the research work done by him under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

Rupen Goswami
Associate Professor
Department of Civil Engineering
Indian Institute of Technology Madras

Abstract

Steel special moment frame (SMF) is a popular lateral load resisting system used in earthquake resistant design of buildings. To facilitate the structural design of SMFs, design codes provides guidelines based on Force-Based Design (FBD) method. FBD relies on *capacity design principles* to contain inelastic action primarily in beams of SMFs during strong earthquake shaking. Therefore, in line with the strength hierarchy required in capacity design, seismic design codes recommend specific guidelines for the design of capacity-protected components in steel SMFs, such as connections, panel zones, and columns. In this study, seismic performance of two study buildings, a 3-storey and a 9-storey, with steel SMF as lateral load resisting system designed as per the current Indian codes [IS 800, 2007; IS 1893 (Part-1), 2016] is evaluated using nonlinear static and dynamic analyses. The performance assessment of the designed buildings indicates undesirable behaviour wherein conventional expectation of resisting strong earthquake ground shaking through ductile flexural plastic actions at beam ends is not realised; excessive yielding of panel zones and column bases are observed instead. This undesirable seismic performance is due to design provisions (a) not considering the expected increase in material yield stress above the minimum specified or characteristic yield strength of the material, (b) underestimating the demand on panel zones, and (c) specifying an inadequate minimum column-to-beam strength ratio (*CBSR*) requirement. Acceptable seismic performance is observed in the two study buildings when redesigned addressing the above mentioned issues.

Additional detailed investigation carried out to better understand panel zone behaviour in steel SMFs reveals that the existing design procedures [*e.g.*, AISC 341, 2016; AISC 360, 2016] can result in significant panel zone distortion. Based on the results, a design procedure is proposed for design of panel zones in steel SMFs. Further, the study investigates the effectiveness of using higher grade of steel for columns in SMFs. It is found that, columns with yield stress of 345 MPa are the best choice among the commonly manufactured steel grades. Finally, the study recommends a minimum value of *CBSR* to be maintained in SMFs to eliminate the use, or limit the thickness, of additional doubler plates in panel zones.

…

Acknowledgements

I express my deepest gratitude and appreciation to my advisor, Professor *Rupen Goswami*, for his tireless guidance and constant support throughout the course of this study. Once I graduate, I will definitely miss those *engaging technical discussions*, *tea time talks*, and obviously the *bleeding pages* that I get after every submission; all which helped me in developing technical competence. Let me keep it simple, "Thank you" for everything.

I am extremely grateful to all my teachers at IIT Madras and NIT Calicut for imparting in me the basics of various subjects. I take this opportunity to specially thank Professor *C.V.R Murty*; not just for his invaluable technical guidance, but also for being a caring mentor to me. I also sincerely thank members of my GTC, Professors, *A. Mehr Prasad* and *Deepak Kumar* for their constructive suggestions and review of Thesis. I just can't finish this section without acknowledging Professors, *S. Arul Jayachandran* and *S. R. Satish Kumar* for enthusiastically sharing their wealth of experience.

From the bottom of my heart, I thank all my lab mates, especially *A. R. Vijayanarayanan* and *M. Sabareesan*. The former in many ways has been more than guide to me. Apart from them, I am deeply indebted to my seniors; *Sunitha P. Menon, Philip Cherian, Tharun John Joseph, Arnav Anuj Kasar, M. Velmurugan, Anilkumar P.M., Rohan Bhasker, S. Krishnachandran, Dixon David, R. Sevugan, K. Prabakaran,* and *S. Harishankar* for their valuable time and advise. Also, my fellow colleagues and friends Venkatesh C., Krishnanunni C.G., Shereena O.A, Jhanabi Basu, Saurabh Tak, Anand J. Nair, and Safeel A.K. supported and helped in many ways: I am thankful to them for making my stay at IIT Madras memorable.

I remain indebted to my parents and brother for their love, affection, care and constant support in all my endeavors...

Jones Joju

…

Table of Contents

…

Blank Page

List of Tables

List of Figures

Figure *Title* *Page*

Figure	*Title*	*Page*

Figure	*Title*	*Page*

Figure	*Title*	*Page*

…

List of Symbols

Symbol	*Description*
b_{bf}	Width of the beam flange
b_{cf}	Width of the column flange
d	Depth of the section
d_b	Depth of the beam
d_c	Depth of the column
d_w	Depth of the section between flanges
g	Acceleration due to gravity
h	Storey height
l_p	Length of plastic hinge
r_y	Radius of gyration about minor axis
t	Thickness of panel zone
t_{bf}	Thickness of beam flange
t_{cf}	Thickness of column flange
t_w	Thickness of web
A_v	Shear area
C_a	Axial load ratio
C_d	Coefficient of displacement amplification
DL	Dead Load
E	Modulus of elasticity
F_y	Characteristic or Minimum Specified yield stress
F_{yb}	Characteristic or Minimum Specified yield stress of beam
F_{yc}	Characteristic or Minimum Specified yield stress of column
F_{ye}	Expected ultimate yield stress
F_u	Characteristic or Minimum Specified ultimate tensile stress
F_{ue}	Expected ultimate tensile stress
G	Shear modulus of elasticity
I	Importance factor
K_e	Elastic stiffness
K_{eff}	Effective stiffness
K_{el}	Elastic stiffness of panel zone
K_i	Post-elastic stiffness
$K_{p\text{-}el}$	Post-elastic stiffness of panel zone
K_{sh}	Strain hardening stiffness of panel zone
L	Centreline span of beams
L_b	Unbraced beam length
L_h	Beam span between beam hinges
L_s	Shear span
M	Moment
M_{bl}	Moment in the left beam at column centreline
M_{bo}	Moment in beam at column centreline

Symbol	*Description*
M_{br}	Moment in the right beam at column centreline
M_c	Ultimate flexural capacity
M_{cfi}	Overstrength moment at the column face
M_{cb}	Moment in the bottom column at beam centreline
M_{cmax}	Maximum moment in column
M_{ct}	Moment in the top column at beam centreline
$M_{Ed,col}$	Overstrength moment demand in column as per Eurocode
$M_{Ed,E}$	Moment demand in columns due to seismic load combinations as per Eurocode
$M_{Ed,g}$	Shear demand in columns due to non-seismic load combinations as per Eurocode
M_{pc}	Moment in column at beam centreline
M_{pri}	Overstrength moment capacity of the beam
M_v	Moment at column centreline due to shear amplification due to plastic hinge formation
M_y	Yield flexural capacity
M_w	Moment magnitude
N	North
$N_{Ed,col}$	Overstrength axial load demand in columns as per Eurocode
$N_{Ed,E}$	Shear demand in columns due to seismic load combinations as per Eurocode
$N_{Ed,g}$	Shear demand in columns due to non-seismic load combinations as per Eurocode
P	Axial load
P_r	Axial load demand in columns, considering overstrength load combinations too
P_y	Tensile nominal yield capacity of column
R	Response Reduction factor
R_i	Reaction at beam support in beam-column subassemblage
R_t	Ratio of expected ultimate tensile stress to characteristic ultimate tensile stress
R_y	Ratio of expected yield stress to yield tensile stress
S_a	Pseudo-spectral acceleration
S_d	Pseudo-spectral displacement
S_{hi}	Distance from column face to beam plastic hinge
T	Time period
T_e	Elatic time period
T_{eff}	Effective time period
V	Base shear
V_{bi}	Overstrength shear at the beam hinge
V_c	Shear in column
V_D	Design base shear
V_E	Equivalent seismic base shear
$V_{Ed,col}$	Overstrength shear demand in columns as per Eurocode
$V_{Ed,E}$	Shear demand in columns due to seismic load combinations as per Eurocode
$V_{Ed,g}$	Shear demand in columns due to non-seismic load combinations as per

Symbol	*Description*
	Eurocode
V_g	Shear from gravity loads
V_{pzc}	Panel zone design shear capacity
V_{pzd}	Panel zone design shear demand
V'_{pz}	Shear capacity in the presence of axial load
V_u	Ultimate shear capacity of panel zone
V_y	Yield shear capacity of panel zone
Z	Zone factor as per IS 1893 Part-1 (2016)
Z_{pb}	Nominal plastic section capacity of the beam
Z_{pc}	Nominal plastic section capacity of the column
α	Ratio of neutral axis depth to section depth
β	Shear demand-to-capacity ratio
γ_{ov}	Material Overstrength
γ_{pz}	Shear strain of panel zone
γ_{pzy}	Yield shear strain of panel zone
γ_m	Partial safety factor for material
γ_{M0}	Partial factor of safety for yield
δ	Inter-storey drift
δ_e	Elastic inter-storey drift
ε	Strain
ε_u	Ultimate strain
ε_y	Yield strain
θ	Beam rotation
θ_c	Monotonic pre-capping rotation capacity
$\theta_{inelastic}$	Plastic rotation in beam
θ_p	Plastic rotation capacity
$\theta_{p\ beam}$	Cyclically reduced plastic rotation capacity of beam
θ_{pc}	Monotonic post-capping rotation capacity
$\theta_{p\text{-}monotonic}$	Plastic rotation in beam under monotonic loading
Θ	Stability coefficient as per ASCE 07, 2016
ζ	Damping ratio
ξ	Scaling factor to account for change in yield stress
Δ	Lateral displacement
Δ_d	Lateral displacement of building at design base shear
Δ_e	Lateral displacement demand of buildings estimated using either equal displacement rule or equal energy rule
Δ_c	Lateral displacement capacity of the building
Δ_u	Lateral displacement demand of buildings when it sustain earthquake
Ω	Overstrength factor

…

Blank Page

List of Abbreviations

Abbreviations	*Description*
BMD	Bending Moment Diagram
CBSR	Column-to-Beam Strength Ratio
CJP	Complete Joint Penetration
ERD	Earthquake Resistant Design
FBD	Force Based Design
IDA	Incremental Dynamic Analysis
LLRS	Lateral Load Resisting System
LTB	Lateral Torsional Buckling
MDOF	Multi Degree Of Freedom
PoA	Pushover Analysis
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement
PZ	Panel Zone
PZ-I	Panel Zone Design approach 1
PZ-II	Panel Zone Design approach 2
PZ-III	Panel Zone Design approach 3
SCWB	Strong Column Weak Beam
SDOF	Single Degree Of Freedom
SFD	Shear Force Diagram
SMF	Special Moment Frame
W	Wide flange section
WCSB	Weak Column Strong Beam
WUF-B	Welded Unreinforced Flange Bolted Web

…

Blank Page

Chapter 1
Introduction

1.0 Steel Special Moment Frame

Steel *Special Moment Frame* (SMF) is a popular lateral load resisting system used for seismic applications. These moment frames are called *Special* since additional design and detailing, which enhance the seismic behaviour, are carried out in these frames compared to the less stringently designed *Intermediate* and *Ordinary* moment frames. The additional design and detailing enables SMFs to be used in all regions, especially those of high seismicity. Moment frames have always been a preferred architectural choice, especially for buildings, due to the open space on offer (Figure 1.1); the open space at least in select bays otherwise gets obstructed by diagonal braces or structural walls. But, the primary limitation associated with moment frame is its inherent flexibility compared to its counterparts like braced or shear wall systems. Hence, damage to lateral drift-sensitive non-structural components can be significant in buildings with SMFs, if proper design of non-structural components is overlooked. Further, in general, SMFs are costlier compared to braced frame systems. Despite these disadvantages, steel SMFs continue to be widely used in buildings since its inception in the late 19th century due to its relatively better ductile response compared to other structural systems and the architectural freedom it offers.

Figure 1.1: A building with steel SMF as lateral load resisting system (Ventura County Medical Centre, Ventura) [*Photo credit*: Wheeler and Gray, 2019]

1.1 Force-Based Seismic Design of Steel SMF Buildings

Earthquakes impose lateral displacement demand (Δ_u) on structures (Figure 1.2), which in multi-storey buildings, leads to relative lateral inter-storey displacement demand between the storeys. Hence to safely sustain earthquakes, buildings must have adequate overall lateral deformability (or displacement capacity Δ_c), and each storey must have inter-storey drift capacity to be more than the lateral demand. Nevertheless, for convenience in structural design, the displacement loading due to earthquake shaking on structures is converted into an equivalent lateral seismic force. But the magnitude of this equivalent seismic force (V_E) is usually quite large for severe level of earthquake shaking expected in a region. Thus, the classical *Earthquake Resistant Design Philosophy* allows inelastic actions in normal buildings during the severe level of shaking provided such buildings have sufficient displacement capacity (Δ_c). This allows structures to be designed to have relatively smaller lateral strength capacity (V_D=V_E/R), and leads to relatively economical design. But, to guarantee the required displacement capacity, the inelastic actions developed in such buildings must be *ductile*. Hence, brittle modes of failures are precluded by design and detailing of the buildings; this is ensured through a combination of material and cross section selection, local and global stability checks, and adopting *capacity design principle* in design. Use of capacity design ensures hierarchy in strength to be maintained between all possible brittle and ductile modes of damage in a building to ensure an overall ductile behaviour.

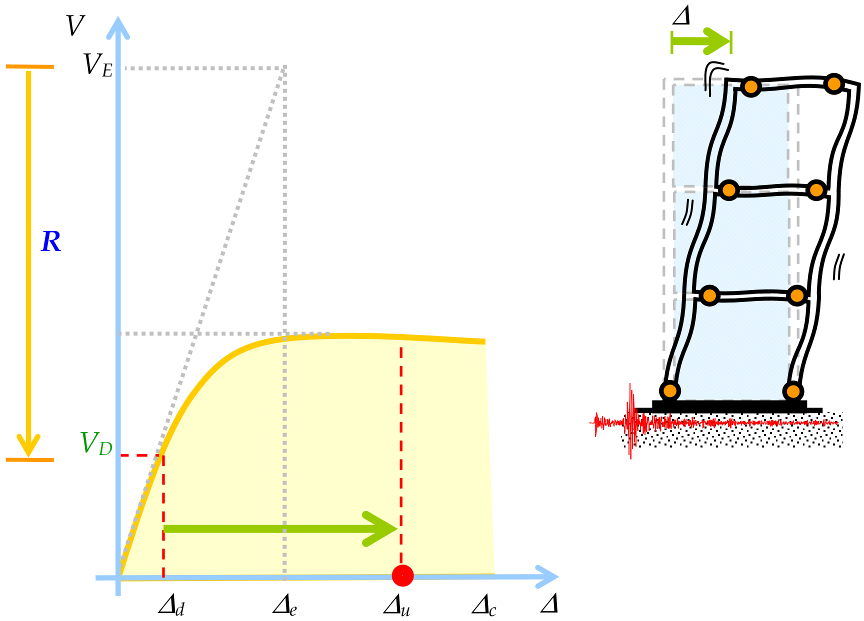


Figure 1.2: Overview of force-based seismic design. V_E, is the equivalent seismic force, V_D the design seismic force, Δ_u the displacement demand, and R the response reduction factor.

In particular, in case of steel buildings which use SMFs as the lateral load resisting system, capacity design principle is followed for the design of all capacity-protected components, *i.e.*, connections, panel zones, and columns, to contain inelastic action primarily to the beams in the form of ductile flexural hinges. Thus, as a first step, capacity design of (beam-to-column) connections is a must. Once the connections are designed to remain elastic, the performance of steel SMFs then critically depends on the, design and thus the, behaviour of the two remaining capacity-protected elements, namely the panel zone and the column. In line with this strategy, seismic design codes provide guidelines to proportion the capacity-protected components. In India, Indian Standard General Construction in Steel – Code of Practice, commonly known as IS 800 [IS 800, 2007], provides guidelines to design these capacity-protected elements of steel SMFs. It is in this regard, lack of consensus exists on appropriate guidelines for design of panel zones and columns in SMFs. In particular, there exists ambiguity regarding demand and capacity of panel zone to be considered in design to obtain desired seismic response of steel buildings with SMFs. Hence, there need for further research to be undertaken in this area.

Also, use of higher grades of structural steel for columns in SMFs can help to achieve capacity-protection of panel zones and columns with smaller column cross-sections. Eliminating the need of using additional stiffeners in columns and doubler plates in panel zones can have many advantages. As the use of higher grades of structural steel is becoming popular in the construction industry, there is a need to investigate its effect on seismic behaviour of buildings with SMFs.

1.2 Organisation of Thesis

This *thesis* is presented in six Chapters. To begin with, the broad idea and area of work with few insights into the objective of the study is presented in Chapter 1. Thereafter, review of pertinent literature is presented in Chapter 2, with emphasis on (i) seismic design philosophy of steel SMF buildings, (ii) performance of steel SMF buildings in past earthquakes and lessons learned, (iii) panel zone design, (iv) column-to-beam strength ratio requirement, and (v) nonlinear modelling. Subsequently, the identified gap areas, the specific objectives, and the scope of the current study are presented at the end of the Chapter. The analyses and performance of two standard office buildings with steel SMF as lateral load resisting system, designed compliant to the Indian code, are discussed in Chapter 3. Certain shortcomings in behaviour of the designed buildings are identified and possible redesigns are explored towards the end of Chapter 3. In Chapter 4, detailed investigation is presented on the effects of different panel zone designs on seismic behaviour of the two buildings, with focus arriving on specific recommendation for design of panel zones. In Chapter 5, the issue of exploring the effectiveness of using higher grade of steel for columns is investigated. Finally in Chapter 6, a summary of work carried out and the key conclusions drawn are presented. Limitations of present study are highlighted too, along with scope for possible future work in the subject area.

...

Chapter 2

Literature Review

2.0 Introduction

Steel *Special Moment Frames* (SMFs) have been in use for over a century, starting from the late 19th century. The earlier versions of the steel SMFs had thick masonry infills, which enabled them to withstand large lateral forces through composite action. The excellent performance of steel SMFs compared to other lateral load resisting systems in past earthquakes (till 1980's) instilled confidence within structural engineering community regarding superiority of steel SMFs for earthquake resistance. A key component to the lateral resistance of SMFs is the rigid connections. But these rigid connections are generally considered to be costly. Therefore in the 1980's, in an attempt to bring down the cost of SMFs, engineers began to limit the number of bays in which SMFs are used thereby bringing down the number of rigid connections required. This resulted in the use of larger sections, ultimately leading to less redundant structural systems. Further, the discovery of the stable and ductile hysteretic behaviour of the panel zones lead to use of weak panel zones, which enabled to economize the joint design. By 1990's the thick masonry infills had given way to non-structural glass facades and claddings which do not contribute to the lateral resistance of the frames. The entire evolution of steel SMFs from its birth to 1990's seemed all well till the 1994 Northridge and 1995 Kobe earthquakes. The unanticipated brittle damages observed in steel SMFs provided a wake up call to the structural engineering community. The post-Northridge and Kobe research on steel SMFs brought to light many shortcomings, which eventually lead to significant overhaul of the provisions pertaining to the design of steel SMFs. Meanwhile, in India, steel building construction industry did not flourish as did the reinforced concrete building industry. Consequently, the first set of earthquake resistant design provisions for steel buildings were published only recently in India [IS 800, 2007]. Though this is a good beginning, thorough investigation is required still to fully understand the efficacy and implication of the design provisions on seismic behaviour of structures designed using the code. The key issues related to seismic design, analysis, and behaviour of buildings with SMFs as the lateral load resisting system (LLRS) is discussed in this chapter.

2.1 Seismic Design Philosophy for Steel SMF Buildings

For buildings to safely sustain earthquakes of severe shaking intensity, their lateral displacement capacity has to be larger than the corresponding lateral displacement demand. In general, the *beam-sway* mechanism is the desired mechanism for moment frame buildings, as it maximises the lateral displacement capacity of the building by maximising the inelastic energy dissipation capacity [Roeder *et al.*, 1988]. This necessitates designing the columns to have flexural yield capacity greater than the overstrength flexural capacity of beams framing into them, with a margin. Such a design approach, wherein columns are flexurally *stronger* than the beams, is termed as *Strong Column Weak Beam* (SCWB) design [Park and Paulay, 1975]. In contrast, the design approach wherein columns are flexurally *weaker* than the beams is termed *Weak Column Strong Beam* (WCSB) design. In the past, buildings with WCSB exhibited significant inelastic action in columns leading to undesirable behaviour such as (a) curtailed lateral displacement capacity, (b) reduced energy dissipation capacity, and (c) partial/complete collapse of buildings (Figure 2.1) [Schneider *et al.*, 1991; Lee, 1996]. Hence SCWB design philosophy is adopted for SMFs in almost all seismic design codes. Apart from beams, columns, and connections, a fourth element which forms a crucial link in the force flow mechanism in a SMF is the *panel zone*. Panel zone is the region of the beam column joint, delimited by the continuity plates and column flanges. During severe seismic shaking, these panel zones are subjected to shear forces which are many times more than those in the columns. Hence, the design of panel zone plays a central role in overall seismic behaviour of steel SMF buildings.

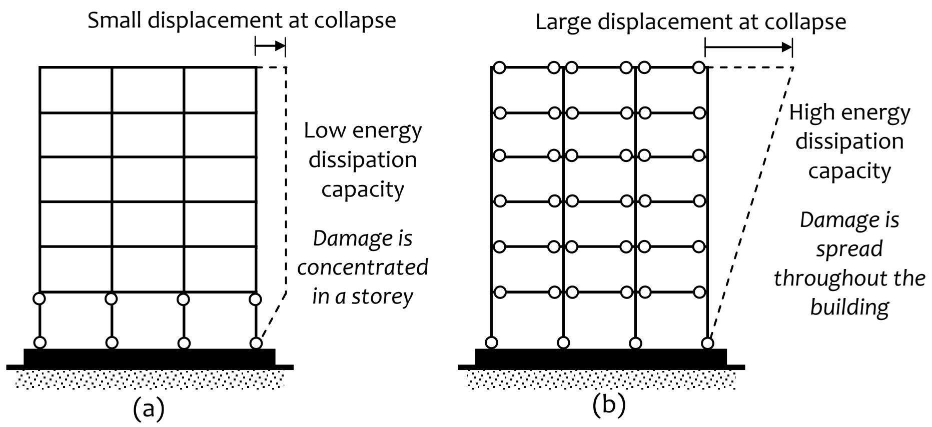


Figure 2.1: (a) Weak Column Strong Beam (WCSB), and (b) Strong Column Weak Beam (SCWB). [Adapted from Murty *et al.*, 2012]

2.1.1 Capacity Design Principle

The SCWB design philosophy is implemented in design practice using *capacity design principle*. The capacity design principle outlines a strength hierarchy required between different components of a SMF meeting at a joint, *i.e.*, between beam, connection, panel zone, and column. These four key components at an exterior beam column joint are shown in Figure 2.2.

Capacity design principle for steel SMF envisions the inelastic action sustained during a severe earthquake shaking to be contained primarily in the beam end regions. The beams transfer the forces generated through the connections to the panel zone. In general, the connections, often designed as *capacity protected elements*, are expected to behave elastically, the exception being few connection configurations which are designed specifically to dissipate energy (*e.g.*, sideplate moment connection). Similarly, the panel zones are supposed to be designed as capacity protected element too, *i.e.*, panel zones should remain elastic during strong seismic shaking. However, current provisions of various design codes [AISC 341, 2016; CEN, 2005] allow limited yielding of panel zones. Detailed discussion regarding panel zone behaviour will be presented later in **Section 2.4**. Finally, the columns are also supposed to be designed as capacity protected elements. However, limited yielding of column is allowed too compromising the aim of capacity protection of columns to *avoid storey mechanism* only, and not to entirely eliminate yielding of columns.

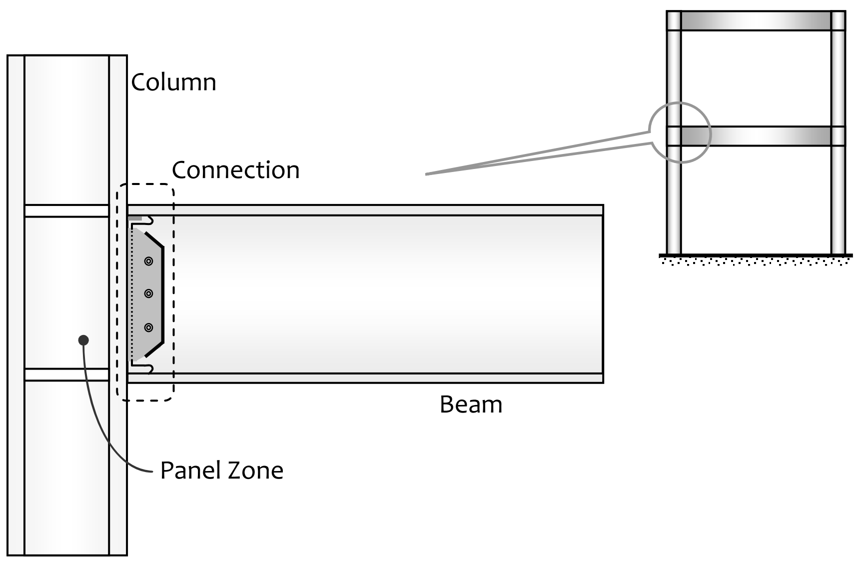


Figure 2.2: Components of a steel SMF at an exterior beam column joint

As a result, the order of importance (from higher to lower), and therefore, the strength hierarchy of the components of a steel SMF are generally listed as be (i) connections, (ii) panel zones, (iii) columns, and finally (iv) beams. The different possible modes of energy dissipation in steel SMFs are shown in Figure 2.3, of which the beam sway mechanism as shown in Figure 2.3(a) is the most preferred energy dissipation mechanism, where only the beams yield. The fixed column bases too yield, since yielding of base is required to form the collapse mechanism. The mode of energy dissipation shown in Figure 2.3(b) is similar to beam sway mechanism. However, limited yielding of panel zone is also allowed. The current American seismic provisions [AISC 341, 2016] targets this collapse mechanism. The collapse mechanisms shown in Figure 2.3(c) and (d), are unacceptable modes of damage for steel SMFs. When the yielding is concentrated in the panel zone as shown in Figure 2.3(c), the connections are subjected to strain demands potent enough to result in brittle fracture of connecting welds. Similarly, when yielding is concentrated in columns, the potential of storey mechanism formation is high. Additionally the ductility of the column is curtailed due to the presence of axial loads. Thus, the intent of the whole capacity design principles for steel SMFs is to avoid collapse mechanisms shown in Figure 2.3(c) and (d), and ensure the occurrence of mechanisms shown in Figure 2.3 (a) or (b).

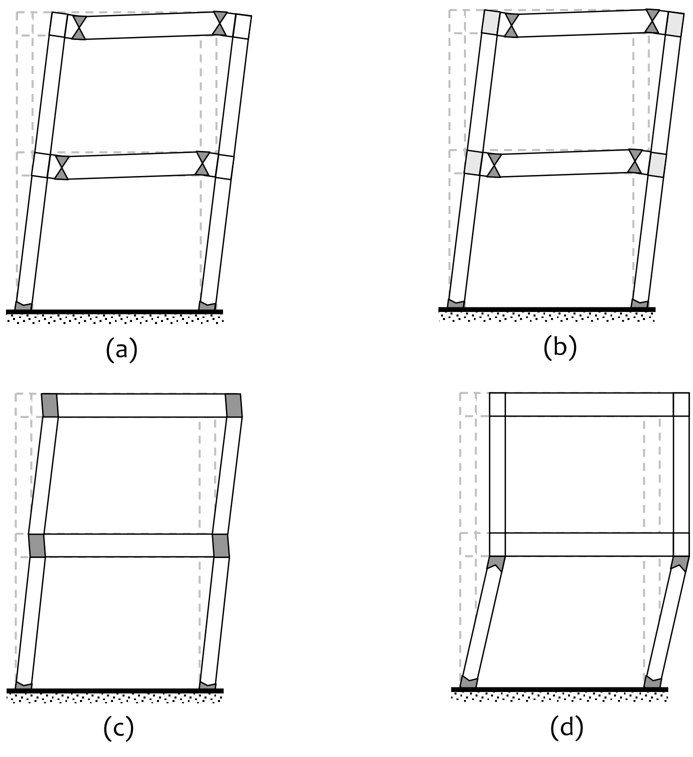


Figure 2.3: Possible seismic energy dissipation modes in steel moment frames: (a) beam - sway mechanism, (b) beam-sway mechanism with minor panel zone yielding, (c) weak panel zone mechanism, and (d) storey mechanism

2.2 Behaviour of Steel SMFs in Past Earthquakes

Prior to the 1994 Northridge earthquake, steel SMFs were regarded highly for their seismic resistance. But, the 1994 Northridge earthquake exposed many shortcomings of steel SMFs which let to undesirable seismic performance of the system. Exactly a year later, the Kobe earthquake confirmed that, the limitations of steel SMFs were not limited to American steel industry; even the Japanese steel industry had similar shortcomings. These two events drew global attention towards the need for further research to upgrade the seismic behaviour of steel SMFs.

Even before the 1994 Northridge earthquake, the 1985 Michoacan earthquake, where the Pino Suarez building collapsed did ring warning bells. Excessive local buckling was observed throughout the building coupled with isolated occurrence weld fractures. However, the collapse was attributed primarily to the local site amplification due to the presence of soft clays, which resulted in excessive seismic demands for which the building was not designed for. Similarly, steel SMFs did come unscathed in the 1989 Loma Prieta earthquake, simply because the damages sustained were not discovered. Thus, the image of steel SMFs as a superior structural system to resist earthquakes was kept intact [Gioncu and Mazzolani, 2002].

The Japanese and American practice for the design of steel SMFs has several differences. The Japanese steel SMF buildings are, in general, stiffer and more redundant compared to their American counterparts. The reasons for this include (i) every bay of a building is generally constructed as SMF in Japan while only select peripheral frames are designed as lateral load resisting SMF in American practice, (ii) while box columns are predominantly used in Japanese constructions, heavy wide flange sections are commonly used in America. An in-depth review of the evolution of steel construction industry and performance of steel SMFs in past earthquakes in both these countries is well documented in literature [Gioncu and Mazzolani, 2002; Goswami, 2007; Hamburger and Malley, 2009]. The following section briefly presents the behaviour of steel SMFs in the 1994 Northridge and 1995 Kobe earthquakes.

2.2.1 Northridge Earthquake (1994)

The Northridge earthquake was a moderate earthquake with a moment magnitude 6.7. However, it took place directly beneath an urban settlement causing 57 fatalities and the economic loss was reported to be around 30 billion US dollars, the largest natural disaster till then in US history [EERI, 1995]. To the credit of steel SMFs, none of them collapsed, and seemed to have survived the earthquake pretty well. However two weeks later, detailed investigations revealed the uncharacteristic performance of steel SMFs [Bertero *et al.*, 1994; Tremblay *et al.*, 1995; Mahin, 1998; Miller, 1998; FEMA-355E, 2000]. Rather than dissipating the seismic energy through formation of ductile flexural plastic hinges in beams, premature brittle fracture in and around the beam column connection was widely reported. The most common among the damage modes was premature fracture around the bottom flange groove weld at the beam column connection. A popular pre-Northridge beam column connection detail, the Welded Unreinforced Flange-Bolted Web (WUF-B) connection, is shown in Figure 2.4. Complete Joint Penetration (CJP) welds were used to connect the beam flanges to the columns, and beam webs were attached to the column using bolted shear tabs. The damages observed classified into 8 types are shown in Figure 2.5. The most common fractures were of type 1 and 2, where crack formed at the CJP weld to column interface. In types 3, 4, 5 and 6, cracks initiated at the root of the CJP groove weld and propagated into the column. Types 5 and 6 damage was characterized by cracks propagating all through the thick column flange. Few cases of types 7 and 8 damage modes were also reported. The reasons behind the premature cracks are attributed to a wide variety of material defects and design flaws, including [Engelhardt, 1997]:

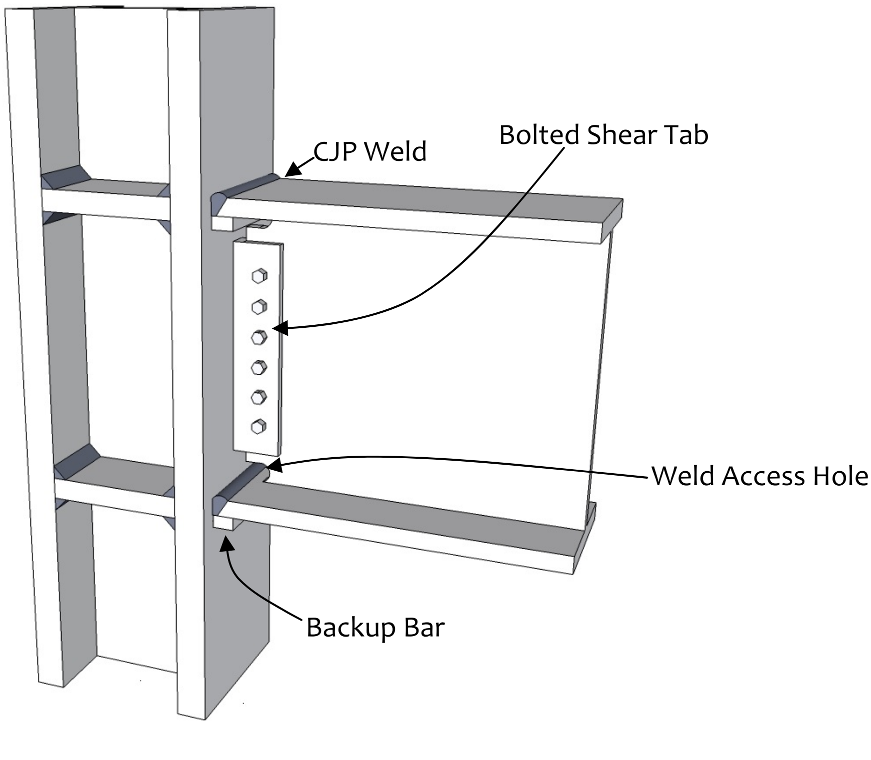


Figure 2.4: Pre-Northridge Welded Unreinforced Flange-Bolted Web (WUF-B) connection

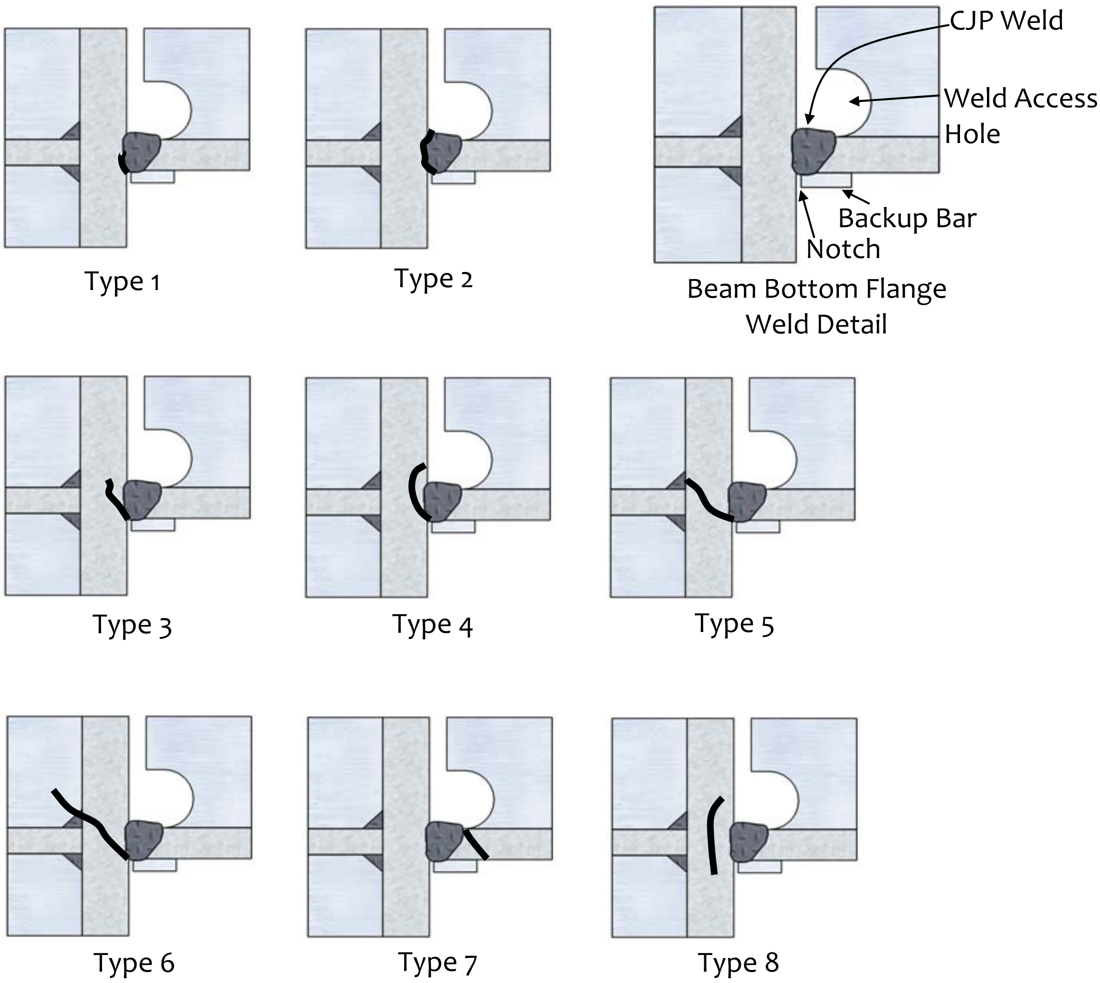


Figure 2.5: Typical damages found at or near the bottom flange CJP weld in strong-axis beam-column connections during the 1994 Northridge earthquake.

1. *Material Yield Stress:* The actual beam yield stresses were significantly larger than the minimum specified (or characteristic) yield stress of the material. Typical steel SMF construction used beams and columns with minimum specified yield stresses of 250 MPa (36 ksi) and 345 MPa (50 ksi) respectively. However, tests done on the damaged structures due to the earthquake revealed that, the actual yield stress of the lower grade steel (250 MPa or 36 ksi) used for the beams was close to the yield stress of columns. This unanticipated increase of the beam yield stress lead to under designing of connections, panel zones, and columns. Further, the stocky columns with large flange thickness (>38 mm) generally had actual yield stress close to the minimum specified yield value.
2. *Weld Toughness:* The Charpy V-Notch (CVN) tests conducted on the weld metal which failed during the Northridge earthquake revealed that the energy absorption capacity was in the range of 10-15 J at room temperature, which do not meet the

current code requirement of 27J at -18^{0}C. This coupled with the fact that, the earthquake struck the city in the early hours of the day, when the temperature was low, contributed to premature brittle fracture of the welds.

3. *Weld Backing:* The weld backup bars were often left in place after groove welding at the beam flange to column connection location. These backup bars created notches, which acted as stress raisers contributing to brittle fracture of the connections.
4. *Shear Distortion of Panel Zone:* Weak panel zones lead to excessive shear distortion of panel zones and the subsequent kinking of column flanges imposed additional strain demands on the CJP welds contributing to brittle fractures [El-Tawil *et al.*, 1999].
5. *Connection Design:* The unreinforced connections, especially the WUF-B connection configuration, lead to overstressing of the beam flanges. The beam webs, which were bolted to shear tab which connects beam to column, did not participate in shear transfer thereby forcing the beam flanges to transfer the entire beam force.
6. *Poor Workmanship and Quality Control:* Investigations reported frequent instances of poor welding quality and lack of proper inspection of the connections.

2.2.2 Kobe Earthquake (1995)

The 1995 Kobe earthquake recorded a moment magnitude of 6.9. Even though Japan in the past had experienced stronger earthquakes, the ground motions recorded were larger than the previously recorded ones [Tremblay *et al.*, 1996; Nakashima *et al.*, 1998]. Similar to the Northridge earthquake, the Kobe earthquake struck a thickly populated urban habitat, resulting in deaths of around 6000 people and caused economic loss of over 100 billion US dollars [EERI, 1995]. Thus, the damage caused by Kobe earthquake was multiple times more than that caused by the Northridge earthquake. Also, unlike in Northridge, a large number of steel buildings collapsed; about 1,247 steel framed buildings were damaged, of which 286 steel buildings collapsed [AIJ, 1995]. The primary cause behind the extensive damage is attributed to the unprecedented levels of ground shaking, for which the buildings were not designed for. However, the modern steel SMFs were not able to generate the amount of ductility, they were supposed to develop. Unlike, the Northridge steel SMFs, limited amount of yielding was observed in frames prior to brittle fracture modes. The factors which curtailed the ductility of steel frames include [Tremblay *et al.*, 1996]:

1. *Material*: Similar to the US practice, prior to 1995, the Japanese steel mills consistently produced steels whose actual yield stress was higher than the minimum specified

yield stress. Lack of quality control issues related to weld metal and welding procedure was also widely responsible for the premature brittle fracture. Further, over 70 percent of steel buildings were built 35 years before the earthquake and was reported to have undergone material deterioration.

2. *Connection Detailing*: The typical column-tree connection configuration used in Japanese SMFs is shown in Figure 2.6. If backing bars were the primary cause of producing notch effect in American connections, it was the runoff tabs which acted as stress raisers in Japanese connections. This is because the Japanese practice generally uses box columns, which make the beam flange edges as the regions of high stress intensity as illustrated in Figure 2.7. Further, the design flaws related to weld access hole detailing were also responsible for stress concentrations, eventually leading to brittle fractures. Fillet weld used in connections were also reported to have undergone brittle fractures, without any indication of inelasticity in beams.

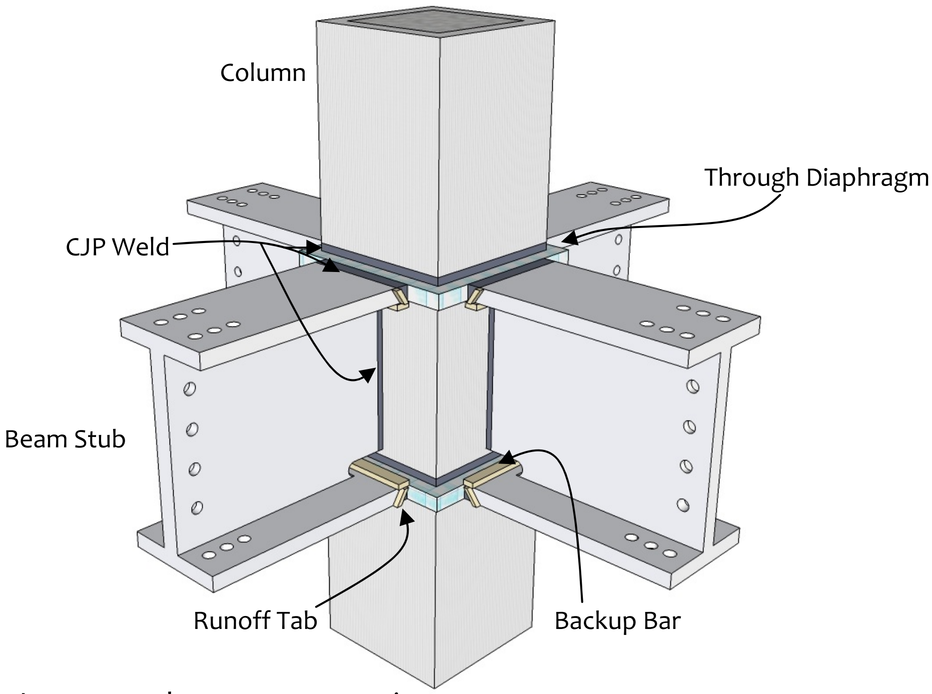


Figure 2.6: Japanese column tree connection

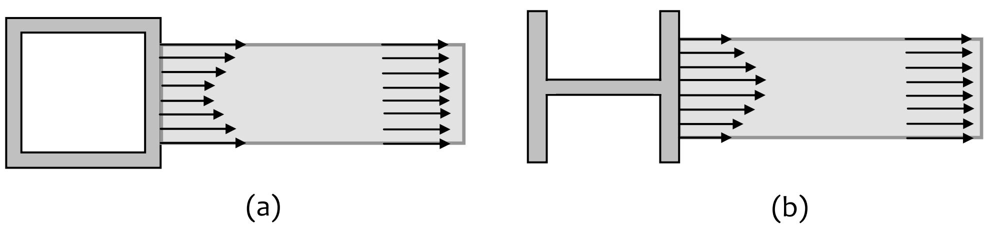


Figure 2.7: Stress profile in the beam flange at the column face: (a) box column, and (b) wide flange column.

2.3 Post Northridge and Kobe Era

The unanticipated brittle modes of failure observed in the 1994 Northridge and the 1995 Kobe earthquakes directed the efforts of global research community to fix the issues which culminated in the unsatisfactory performance of steel SMF buildings. While, the thrust of American research fraternity was to develop better connection design strategies, the Japanese counterparts focussed on improving the material quality. In US, a multimillion dollar research initiative, the SAC Steel Project, was formed to investigate the performance of welded steel SMFs and to suggest reliable design and retrofit methods. The key findings and recommendations of the research that followed are presented in following sections [SAC, 1995; Malley 1998; FEMA-350, 2000; FEMA-351, 2000; FEMA-352, 2000; FEMA-353, 2000; FEMA-355D, 2000; FEMA-355E, 2000]

2.3.1 Material

As discussed earlier, the uncertainty in material yield strength lead to under designing of connections and columns in SMFs. For instance, in Japan, the SM490 steel, which is designated to have a nominal yield (F_y) and tensile (F_u) stresses of 325 MPa and 490 MPa, respectively, had mean yield and tensile stresses of 376 MPa and 536 MPa respectively. Similarly in US, the ASTM A36 grade steel, which is supposed to have a nominal yield stress (F_y) of 250 MPa (36 ksi), was reported to have average yield stress of 339 MPa (49 ksi). To address the uncertainty in material strength, Japan developed new line of steel designated as SN, which has lower and upper limits for yield and tensile strengths, an upper bound for yield to tensile strength ratio (0.8), and more stringent minimum toughness requirement. The Americans, rather than introducing new grade of steel, by statistical analysis established the bounds of strengths for the existing grades of steel and mandated minimum toughness requirement. All steels used in seismic lateral load resisting systems are required to have CVN toughness not less than 27 J at 21^0C [AISC 341, 2016]. Also, since higher grade steels in general have lower toughness. AISC 341 limits the minimum specified yield stress to 345 MPa of steel to be used in locations where inelasticity is expected to develop. Further, two new parameters were introduced to account for the variation of material yield stress in design. They are the R_y and R_t factors which are defined as:

$$R_y = \frac{F_{ye}}{F_y} \text{ , and} \tag{2.1}$$

$$R_t = \frac{F_{ue}}{F_u}, \tag{2.2}$$

where, F_{ye} and F_{yu} are the expected material yield and tensile stresses while F_y and F_u are the minimum specified values of the respective quantities. Further, the ratio of tensile strength to yield strength is limited to 0.85. Thus, while designing capacity protected elements, the demand needs to be estimated based on the expected yield and tensile strengths of the yielding elements rather than using the nominal strengths. For the purpose, R_y *and* R_t factors need to be found out experimentally. However, in the absence of experimental data, R_y *and* R_t factors for standard grades of steel provided in the AISC Seismic Provisions [AISC 341, 2016] shall be used. In line with this development, most of the seismic steel codes today formally recognise the anticipated increase in material yield stress. However, the Indian code [IS 800, 2007] does not.

Typically self-shielded Flux Core Arc Welding (FCAW) is used for connection welding in US, while Gas Metal Arc Welding (GMAW) is practiced in Japan. Though GMAW is costly, it provides greater toughness compared to FCAW [Nakashima *et al.*, 2000a]. E70T-4 electrode, which was commonly used for welding in FCAW procedure in the pre-Northridge era, is reported to have CVN values less than 27 J at -28^{0}C which is the current requirement [Kaufman *et al.*, 1996; Xue *et al.*, 1996]. Hence, post-Northridge E70TG-K2 and E71T-8 electrodes, which meet the toughness requirement, replaced the E70T-4 electrodes for seismic applications in the US. More importantly, weld inspection and quality control regulations in both the countries were made stringent to consistently deliver good quality welds.

2.3.2 Connection Design

As brittle fractures at or near the connections was a common feature of SMFs damaged in the Northridge and the Kobe earthquakes, development of new connection design strategies gained traction, especially in the US. The failure of the connections was primarily due to the concentration of stresses at the beam flange CJP weld. Thus the post-Northridge connection design strategies were aimed at shifting the plastic hinge location from the connection region to the beams. The two key ideas adopted to shift the plastic hinge location revolved around (i) strengthening of connections, and (ii) weakening of framing beams [Bruneau *et al.*, 1998]. Strengthening of connections was accomplished generally by using reinforced connections using cover plates, rib plates, diaphragms, side plates, or haunches. However, the strengthening strategies do have few limitations which are: (i) increased beam moments at column face, which necessitates the use of larger column section to accommodate increased panel zone shear demand and to maintain a minimum Column-to-Beam Strength Ratio (*CBSR*), and (ii) increased plastic rotation demands in the beams. Despite these limitations, a host of connection designs were proposed and was shown to behave as required through experiments. The second strategy, *i.e.*, weakening of the framing members resulted in the development of Reduced Beam Section (RBS) connections, popularly known as dog-bone connections. Here, the beam flange width is reduced at a distance from the column face, thereby forcing plastic hinge formation at that location. However, this required additional lateral bracings to prevent premature web buckling and Lateral Torsional Buckling (LTB) modes of failures to occur before yielding of beam flange in compression. A host of connections, which became popular in the post-Northridge era, are shown in Figure 2.8.

Another, important development in the post-Northridge era pertaining to connection design is the introduction of full scale experiments to ensure connection performance. Further, any connection to be used in SMFs is required to undergo inelastic deformations corresponding to a joint rotation of 0.04 radians without significant degradation of strength and stiffness [IS 800, 2007, AISC 341, 2016]. To ascertain joint rotation capacity of new connection configurations, full scale cyclic experimental tests are mandated by seismic codes. In the absence of full scale experiments, which are costly, the designer is forced to select a prequalified connection, whose design guidelines are provided by certain seismic codes [AISC 358, 2016].

2.3.3 Additional Developments

The post-Northridge era also saw Performance Based Seismic Design (PBPD) gaining momentum [SEOAC, 1995; Bertero, 1997]. As computational power became cheaper and faster, nonlinear analysis of structures became popular. The large amount of full scale structural testing of steel structural components which followed the Northridge earthquake aided in developing and validating reliable nonlinear response of components predicted through nonlinear analysis of structural systems. Thus, nonlinear analysis provides valuable insight into the inelastic behaviour of building during seismic shaking, and has become a dependable tool to assess the seismic performance. Despite the considerable progress made in understanding the behaviour of steel SMFs, there are few aspects which still create a difference in opinion in the research community. Panel zone design criteria and minimum Column to Beam Strength Ratio (*CBSR*) requirement are the most prominent among them. Hence, the forthcoming sections dive deep into the progress made on these two aspects of design.

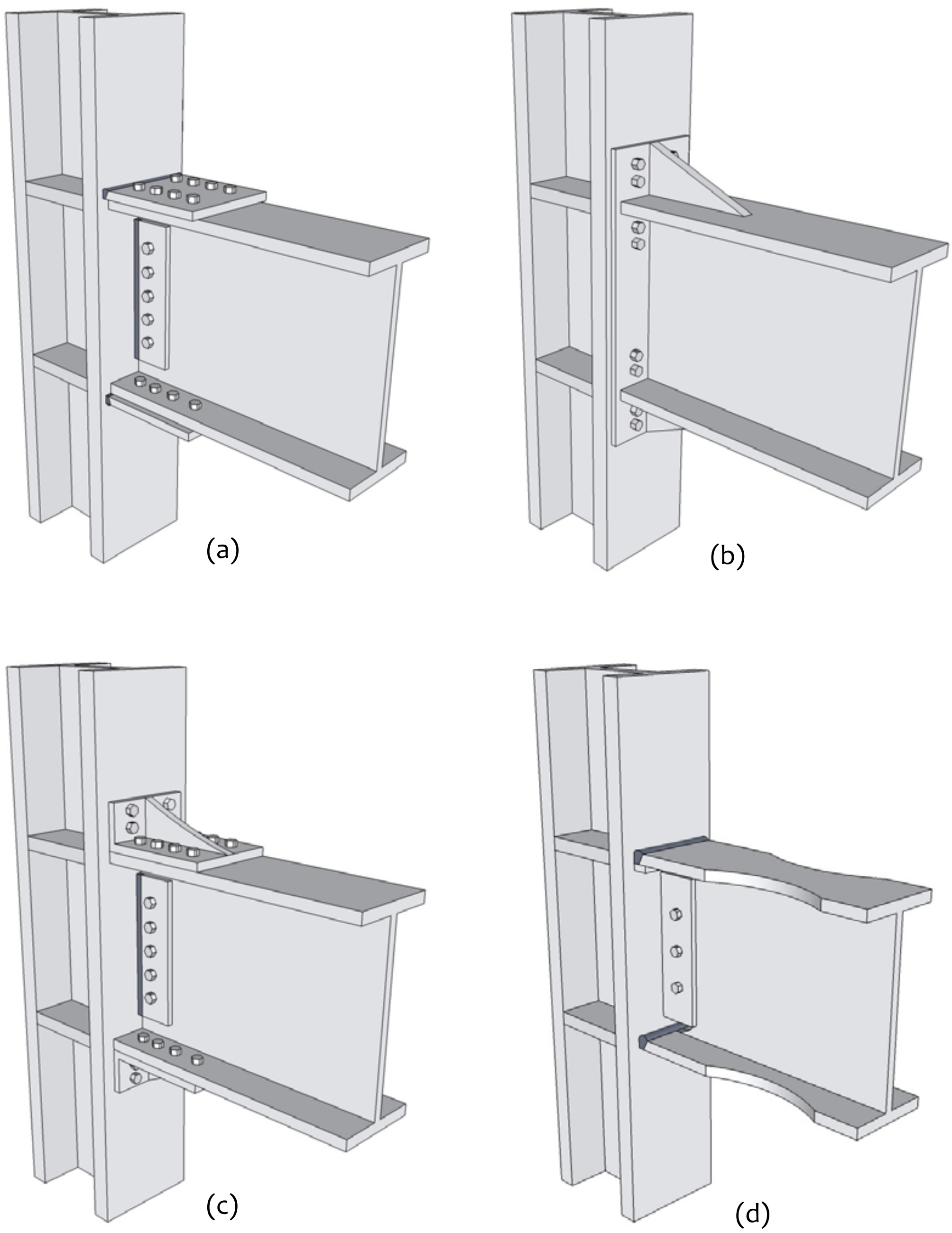


Figure 2.8: Improved connection design strategies: (a) bolted flange plate connection, (b) bolted end plate connection, (c) Kaiser bolt connection, and (d) dogbone connection.

2.4 Seismic Design of Panel Zones

Panel zone is the region of the column delimited by continuity plates and column flanges, whose behaviour is central to the lateral resistance of moment frames. Steel SMF derives its lateral resistance by transferring bending moments from beams to columns. This transfer of bending moment generates significant shear force demands in the panel zones, as illustrated in Figure 2.9. Even though panel zones are subjected to flexural and axial deformations, experiments have demonstrated that those deformations are small compared to the shear deformation; the shear force demand in panel zone can be 4-6 times higher than that in the column during seismic shaking [Fielding and Huang, 1971; Krawinkler *et al.* 1971; Bertero *et al.* 1973]. Hence, it is the shear deformation which controls the response of panel zone, and therefore shear design criterion govern panel zone behaviour. Additionally, the panel zone slenderness need to limited to preclude shear buckling [Brandonisio *et al.*, 2011].

2.4.1 Panel Zone Shear Capacity

The panel zone shear capacity is primarily dependent on the panel zone shear capacity and column flange plastic moment capacity. However, influence of continuity plate, connection components, and the connecting beams make the actual panel zone capacity estimation a challenging task. Despite this, simple mechanics based models have been proposed by various researchers which can reasonably capture panel zone behaviour [Krawinkler, 1978; Tsai and Popov, 1988; Kim and Engelhardt, 1995; Kim *et al.* 2015; Kasar *et al.*, 2017]. All the above models are similar to each other except that they have a slight difference in the representation of post-elastic behaviour. A reasonably

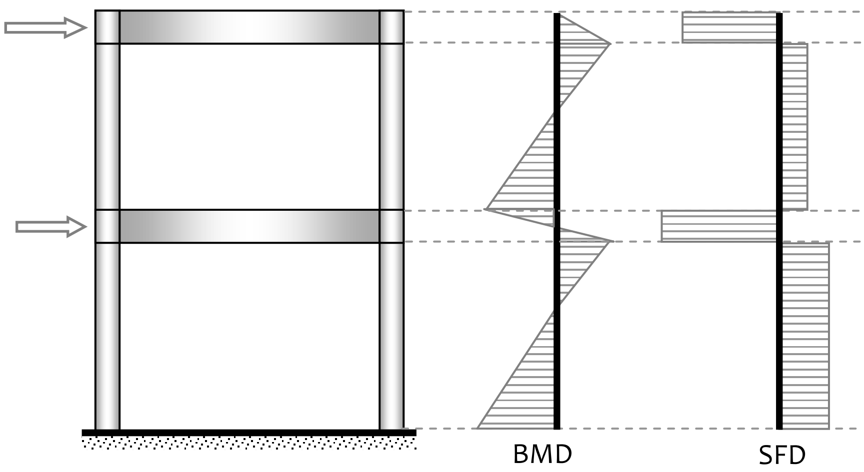


Figure 2.9: Bending moment diagram (BMD) and shear force diagram (SFD) along length of a column in a moment frame under lateral load.

accurate shear yielding capacity, V_y, of the panel zone, based on simple mechanics, is given as [Krawinkler, 1978],

$$V_y = \frac{F_y(0.95d_c)t}{\sqrt{3}}, \tag{2.3}$$

where, F_y is the yield stress, d_c is the depth of column, t is the thickness of panel zone. The corresponding yield shear distortion (γ_{ypz}) is,

$$\gamma_{ypz} = \frac{F_y}{\sqrt{3}G}, \tag{2.4}$$

where, G is the shear modulus of elasticity. The ultimate shear capacity (V_u) of the panel zone is,

$$V_u = 0.55F_y d_c t\left(1 + \frac{3.45b_c t_{cf}^2}{d_b d_c t}\right), \tag{2.5}$$

where, b_c is the breadth of the column, t_{cf} the thickness of column flange, and d_b the depth of the beam. The shear distortion corresponding to V_u is taken as four times the yield strain, γ_{ypz}.. The post elastic stiffness, $K_{p\text{-}el}$ is given by

$$K_{p-el} = \frac{1.09Gb_c t_{cf}^2}{d_b}. \tag{2.6}$$

The final strain hardening stiffness, K_{sh}, of panel zone is reported to be around 0.01-0.02 times K_{el} (elastic stiffness of panel zone). Although panel zones can sustain shear distortion as high as 0.08 radians, panel zone distortion need to be limited to avoid column kinking and fracture of welds at the connection [PEER-ATC, 2010]. *PEER-ATC* recommends shear distortion to be limited to 0.02 radians (~$8\gamma_y$ for steel with yield stress 345 MPa). The overall shear distortion response of panel zone is depicted in Figure 2.10.

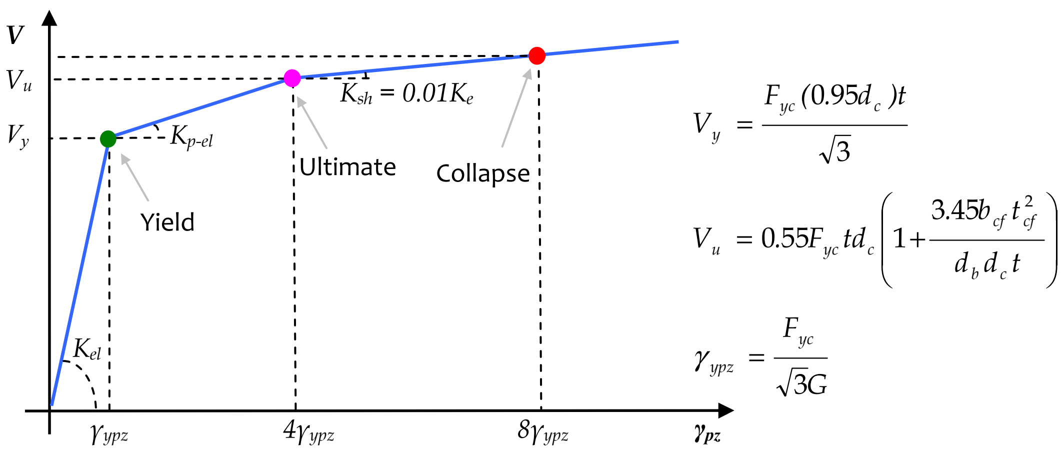


Figure 2.10: Shear-distortion response of panel zone [Krawinkler, 1978].

The tri-linear response of panel zone depicted in Figure 2.10 is derived considering contribution of column flanges towards panel zone shear capacity, strain hardening of web in the post-elastic range, the effect of axial load on shear capacity, and assuming column kinking to occur at around $4\gamma_{ypz}$. Although this predicts the panel zone response reasonably well, actual panel zone response observed in experiments deviate slightly. Yielding of panel zones initiates around $0.8V_y$ (V_y as defined in Eq. (2.3)). Thereafter, stiffness of the panel zone gradually drops. More refined panel zone models to capture this gradual change in the panel zone stiffness have also been proposed in the past [Challa, 1992]. Also, a more refined model was proposed recently [Kim *et al.* 2015], which gives insight into the variation of panel zone shear capacity and kinking rotation for varying axial load ratios, as shown in Figure 2.11. The panel zone shear capacity remains largely unaffected till an axial load ratio of 0.6. Also, there is significant drop in the shear distortion capacity with increasing axial load.

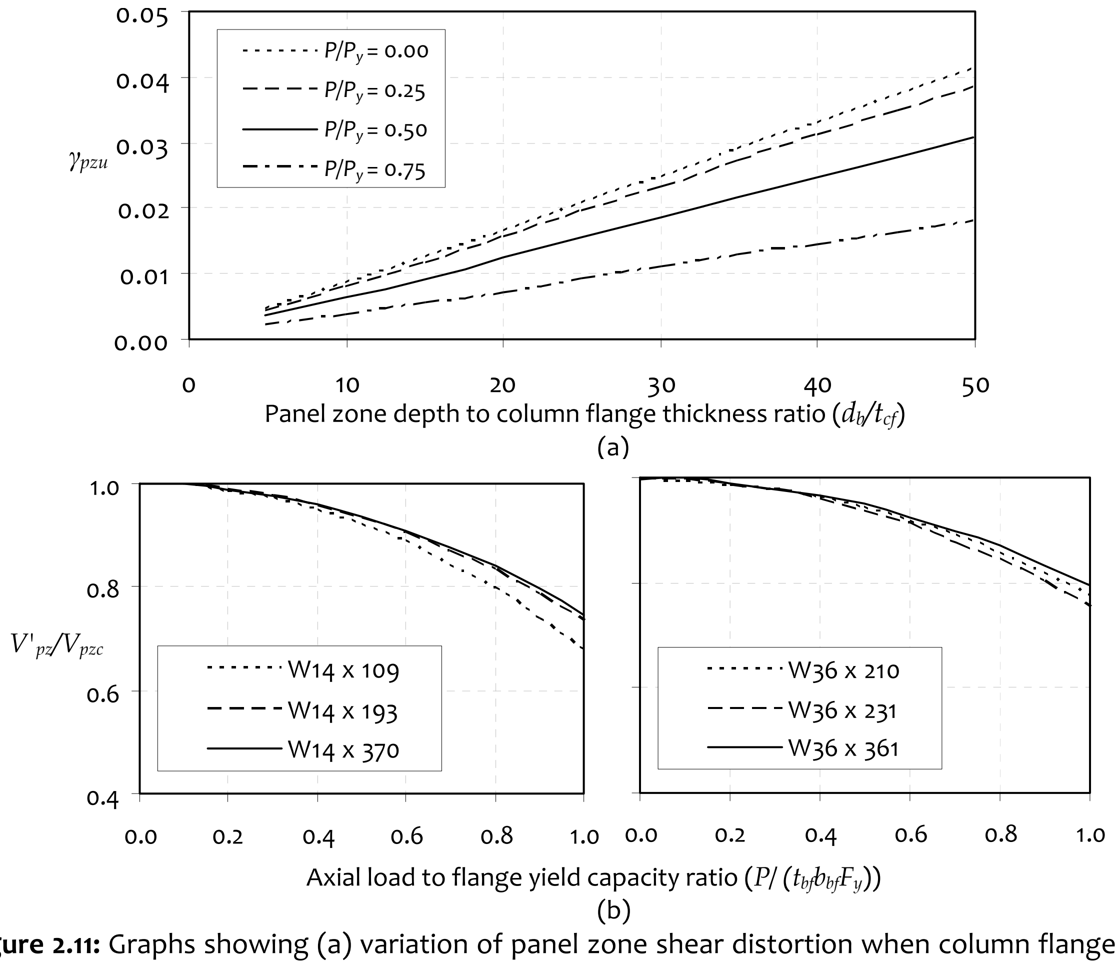


Figure 2.11: Graphs showing (a) variation of panel zone shear distortion when column flange kinks (γ_{pzu}) with respect to d_b/t_{cf} ratio for varying axial loads, and (b) variation of normalized panel zone capacity (panel zone shear capacity normalized by its shear capacity at zero axial load) for varying axial load ratio [Kim *et al.*, 2015].

2.4.2 Panel Zone Shear Demand

Similar to panel zone shear capacity, the accurate estimation of panel zone shear demand is also challenging. This is because, during nonlinear behaviour of structures under dynamic seismic shaking, the demand keeps on changing. The panel zone shear demand (V_{pzd}) is generally estimated by considering the equilibrium of the beam-column joint as shown in Figure 2.12, which represents the state of an interior beam-column joint when flexural plastic hinges are formed in both the beams. The overstrength moment capacity of beam (M_{pri}) is

$$M_{pri} = 1.1 R_{yi} Z_{pbi} F_{ybi}, \tag{2.7}$$

where, Z_{pbi} is the plastic section modulus of beam, F_{ybi} the minimum specified or characteristic yield stress of the beam, R_{yi} the ratio of expected material yield stress to the minimum specified yield stress. In Eq.(2.7) it is assumed that the strain hardening factor is 1.1. The moment at the column face (M_{cfi}) when plastic hinge forms in the beam is

$$M_{cfi} = M_{pri} + V_{bi} S_{hi}, \tag{2.8}$$

where, V_{bi} is the shear in the beam at plastic hinge location, S_{hi} the distance between column face and beam plastic hinge. V_{bi} is estimated as

$$V_{bi} = \frac{2M_{pri}}{L_i - d_{ci} - 2S_{hi}} + V_{gi}, \tag{2.9}$$

where, V_{gi} is the shear due to gravity load and d_{ci} the depth of the column. If V_c is the shear in the column, the panel zone shear demand can be estimated as (refer Figure 2.13)

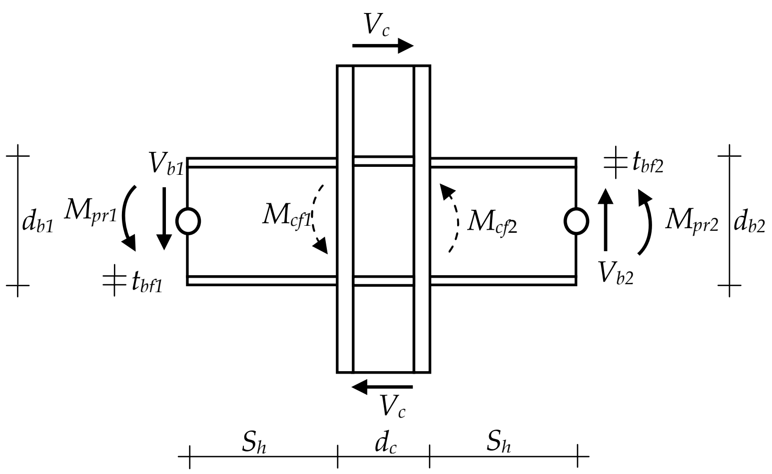


Figure 2.12: Free body diagram of interior beam-column joint of a beam-column sub-assemblage (refer Fig.3) under lateral load when flexural plastic hinge has formed in the beams.

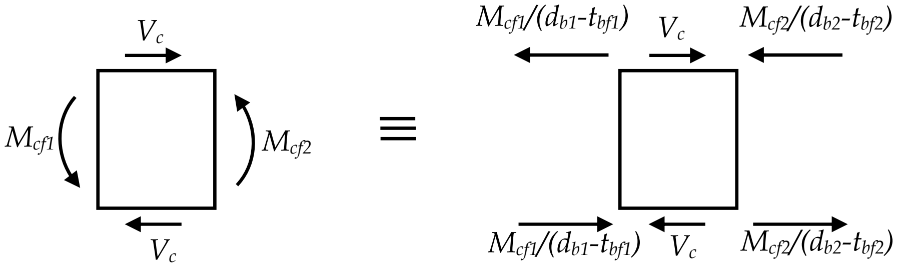


Figure 2.13: Free body diagram of horizontal forces acting at the panel zone used to estimate the panel zone shear demand.

$$V_{pzd} = \frac{M_{cf1}}{d_{b1} - t_{bf1}} + \frac{M_{cf2}}{d_{b2} - t_{bf2}} - V_c \text{, or} \tag{2.10}$$

$$V_{pzd} = \sum_{i=1}^{n} \left(\frac{M_{cfi}}{d_{bi} - t_{bfi}} \right) - V_c \text{ ,} \tag{2.11}$$

where, d_{bi} is the depth of the beam and t_{bfi} the thickness of beam flange. The shear in the column (V_c) can be calculated by considering the moment equilibrium of the beam column sub-assemblage, as shown in Figure 2.14, as

$$V_c = \frac{\sum_{i=1}^{n} \left(R_i \frac{L_i}{2} \right)}{\sum_{i=1}^{n} \left(\frac{h_i}{2} \right)} \text{ .} \tag{2.12}$$

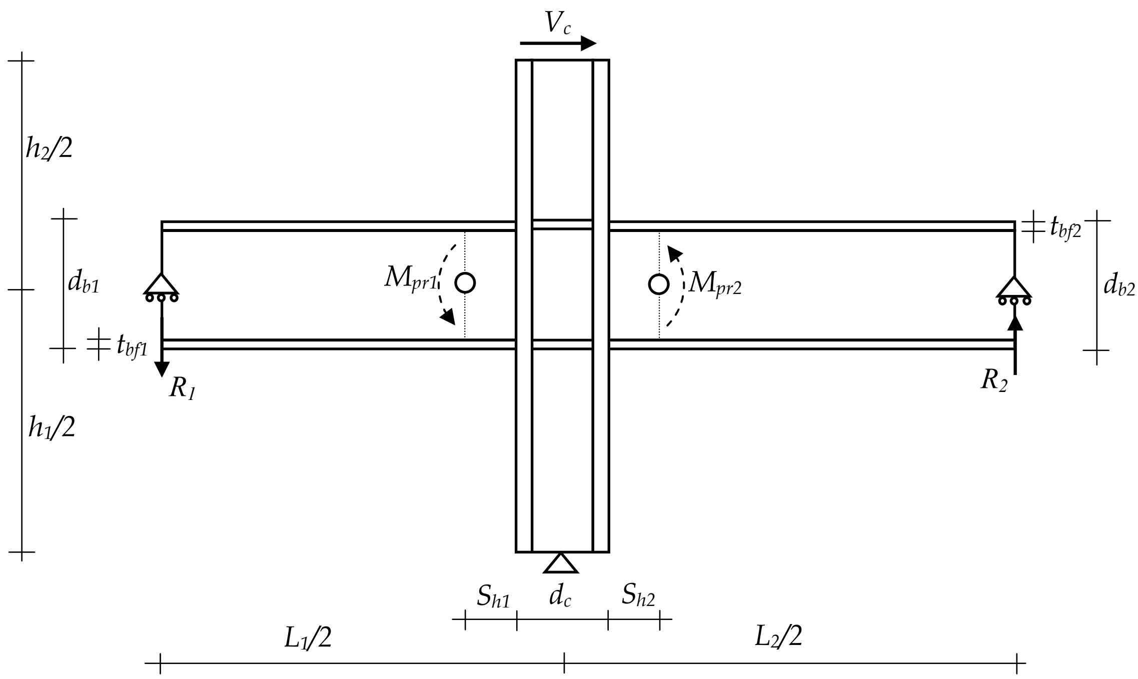


Figure 2.14: Interior beam-column sub-assemblage considered to estimate column shear when flexural plastic hinge has formed in both the beams.

Substituting Eq.(2.12) in Eq.(2.11), the column shear (V_c) can be expressed as

$$\therefore V_c = \sum_{i=1}^{n}\left(\frac{M_{pri}}{L_i - d_{ci} - 2L_{hi}} \times \frac{L_i}{h}\right), \tag{2.13}$$

where, h is the distance between centres of storey above and below the joint. Finally, substituting Eq.(2.13) in Eq.(2.11), the design panel zone shear (V_{pzd}) can be expressed as

$$V_{pzd} = \sum_{i=1}^{n}\left[\left(\frac{M_{cfi}}{d_{bi} - t_{bfi}}\right) - \left(\frac{M_{pri}}{L_i - d_{ci} - 2L_{hi}} \times \frac{L_i}{h}\right)\right]. \tag{2.14}$$

The panel zone demand estimate as per Eq.(2.14) is based on the assumption that inflection points in columns and beams lies at the centre of the span. However, during nonlinear behaviour, the inflection points do shift. Columns which deformed in double curvature during elastic analysis may shift to single curvature bending, owing to the influence of higher vibration modes and formation of plastic hinges during nonlinear analysis. Hence, the panel zone shear demand can vary from the estimate based on Eq.(2.14); panel zone shear demand can be even higher due to dynamic actions.

The presence of gravity loads were considered in the derivation of panel zone demand to estimate the expected moment at the column face (M_{cfi}). As the gravity load proportion increases with respect to the lateral load, the panel zone shear distortion decreases. This is because, when the gravity loads are high, plastic hinge forms in beams at only one side of a panel zone, thus effectively reducing the shear demand (Castro et al., 2008). Hence, Eq.(2.14) will lead to conservative estimates of panel zone shear demand for higher proportion of gravity load.

2.4.3 Panel Zone Design Philosophy

In literature, panel zones are classified as *strong, balanced,* and *weak* [Popov, 1987]. In general, panel zones are deemed: (a) *strong*, if they resist design earthquake shaking elastically; (b) *balanced,* if they yield along with flexural yielding of beams; and, (c) *weak*, if they yield before the flexural yielding of beams.

To begin with, panel zones were proportioned to remain elastic, *i.e.*, strong. Later, beam-column sub-assemblage experiments during 1970's demonstrated ductile and stable hysteretic behaviour of panel zones under cyclic loading [Krawinkler *et al.*, 1971; Fielding and Huang, 1971]. As can be observed from Figure 2.15, even at a large shear distortion of 0.06 radians (~23 times yield strain of panel zone with A572, grade 50 ksi (345 MPa) steel), there is no significant drop of strength and stiffness. The availability of large post-yield capacity, lead to the development of panel zone design approaches, where limited yielding of panel zones were also allowed, *i.e.*, balanced panel zone [Bertero et al., 1973; Krawinkler *et al.*, 1975; Krawinkler and Mohasseb, 1987]. Further, even w*eak* panel zone design approaches, which confines most of the inelastic action to the panel zones, were proposed too [Kawano, 1984]. However, experiments demonstrated that weak panel zones will lead to column flange kinking, subsequently leading to high strain demands in the connection welds [Popov *et al.*, 1985]. Hence, the idea of having weak panel zones, which can help in eliminating the use of doubler plates in joints, was not fully accepted by the research community. Experiments conducted on composite beam to column connections also demonstrated the stable hysteretic response of panel zones [Lee and Lu, 1989]. Further, as studies showed that strong panel zones will induce large rotation demands in connecting beams, balanced panel zone became the preferred choice [Ghobarah et al., 1992]. Even though, the intent was to have balanced panel zone, the panel zones proportioned turned out to be weak, since material strength uncertainty was not properly accounted in the pre-Northridge era. This problem was compounded by the fact that the demand and capacity equations used for balanced panel zone design can potentially lead to higher shear distortion of panel zone than intended, especially during higher shaking intensity.

The damages observed in the 1994 Northridge earthquake raised concerns over the reliability of weak and balanced panel zones. Statistical analysis done on the damaged Northridge connections concluded that the move towards weaker panel zones

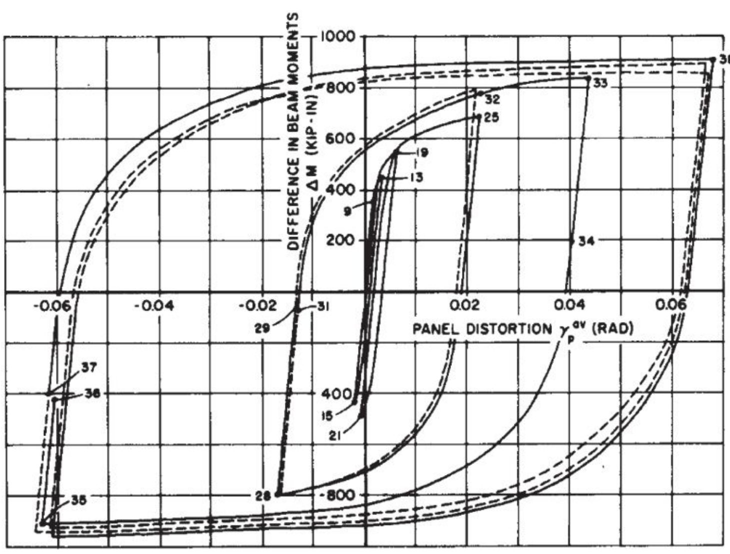


Figure 2.15: Shear distortion response of panel zone [Krawinkler, 1978].

contributed to the brittle connection failures observed [Roeder and Foutch, 1996]; this realization led to the discontinuation of weak panel zone in design. But, Northridge earthquake did not result in a complete u-turn back to use of strong panel zone, although increasingly stronger panel zones were being used. This is because post-Northridge studies presented a divided opinion on whether to use balanced or strong panel zones. The studies which support use of strong panel zone design approach are generally outnumbered by those in advocating use of balanced one. Even those studies which recommended use of stronger panel zones did not explicitly mentioned that elastic response of panel zone is required in design [Ricles *et al.*, 2002]. A considerable number of studies recommended balanced panel zone design instead [Da´vila-Arbona, 2006; Castro *et al.*, 2008]. However, it is questionable, whether under severe seismic shaking, the equations proposed in the above studies and the current code requirements would help in achieving balanced panel zone behaviour.

2.4.4 Panel Zone Shear Design

The behaviour of panel zone depends upon the combination of demand and capacity equations used to arrive at the panel zone design. Till 1980's, the strong panel zone design approach considered yield strength (Eq.(2.3)) as the capacity, and shear demand resulting from an unbalanced moment ΔM ($\Delta M=\sum M_p$), where M_p is the nominal plastic moment capacity of beam section) [SEOAC, 1980]. Later, for balanced panel zone design, the ultimate capacity (Eq.(2.5)) of the panel zone was considered rather than yield capacity, along with shear demand resulting from an unbalanced moment of $0.8\Delta M$ [Popov *et al.*, 1987]. Finally for weak panel zones, along with ultimate shear capacity on the capacity side, shear demand resulting from load combination of $\sum M_g$ + $\sum 1.85 M_e$ was considered. However, before the 1994 Northridge earthquake, very few studies reported the level of plastic distortion demands balanced and weak designs will undergo. Still, these approaches found their way to design codes [AISC, 1992]. Following Northridge earthquake, panel zone design provisions were amended. But even today, only limited number of studies report extensively on panel zone response under nonlinear dynamic actions [Gupta, 1999]. The following section describes the panel zone design guidelines available in the recent version of American, European, and Indian design codes.

2.4.4.1 American Design (AISC 360, 2016; AISC 341, 2016)

The current AISC *Provisions* [ANSI/AISC 341, 2016] intends to adopt balanced panel zone design for SMFs. It recommends the panel zone shear demand to be estimated as described previously in Eq.(2.11). Further, AISC *Specification* [AISC 360, 2016] recommends estimation of shear capacity (V_{pzc}) of panel zone as a function of the axial load in columns (Eqs.(2.15) and (2.16)).

$$V_{pzc} = 0.6F_y d_c t\left(1+\frac{3b_{cf}t_{cf}^2}{d_c t d_b}\right) \qquad \textit{If, } P_r < 0.75\ P_y\text{, or} \tag{2.15}$$

$$V_{pzc} = 0.6F_y d_c t\left(1+\frac{3b_{cf}t_{cf}^2}{d_c t d_b}\right)\left(1.9-\frac{1.2P_r}{P_y}\right) \qquad \textit{If, } P_r > 0.75\ P_y\text{,} \tag{2.16}$$

where t is the thickness of the panel zone, b_{cf} the breadth of the column flange, t_{cf} the thickness of column flange, d_b the depth of the beam, d_c the depth of the column, P_r the axial load in column from (LRFD) load combinations, and P_y the yield capacity of the column. The design panel zone shear capacity is taken as 0.9 (resistance factor in LRFD for web shear) times V_{pzc}. The term within the brackets in Eq.(2.16) accounts for the post-yield contribution of the column flanges towards the panel zone shear capacity. An additional requirement to limit the likelihood of shear buckling of the panel zone is given as,

$$t \geq \frac{(d_b - 2t_{bf}) + (d_c - 2t_{cf})}{90}. \tag{2.17}$$

If doubler plate and column web are not welded together to behave integrally, t in Eq.(2.18) needs to be replaced by the thickness of the doubler plate and the column web separately.

2.4.4.2 European Design (CEN, 2005)

The Eurocode, unlike AISC *Provisions*, does not provide closed form equation to evaluate panel zone shear demand under seismic actions. The design shear capacity of the panel zones (V_{pzc}) recommended in the Eurocode 3 [CEN, 2005] is given as,

$$V_{pzc} = \frac{0.9F_y A_v}{\gamma_{M0}\sqrt{3}} + \frac{b_{cf}t_{cf}^2}{d_b - t_{bf}}\ , \tag{2.18}$$

where A_v is the shear area of panel zone (~$0.95d_c t$), and γ_{M0} is the partial factor of safety for yield and buckling, which is taken equal to unity. The constant 0.9 represents the reduction in the shear capacity in the presence of axial loads [Castro *et al.*, 2008]. The first term in Eq.(2.18) denotes the shear yield capacity of panel zones while the second term denotes the post-yield contribution of column flanges towards the shear capacity. The second term in Eq.(2.19) can be used only if continuity plates are provided to stiffen the web of the column. The Eurocode restricts additional increase in shear capacity by the use doubler plate to the shear yield capacity of the column web without doubler plate. An additional requirement to limit the likelihood of shear buckling of panel zone is given as,

$$\frac{d_c}{t} \leq 69\xi \tag{2.19}$$

where ξ is a scaling factor (ξ = $(250/F_y)^{0.5}$) that accounts for the variation of yield stress from 250 MPa. If the doubler plate and column web are not welded together to behave integrally, t in Eq.(2.19) is to be replaced with thickness of doubler plate and column web separately.

2.4.4.3 Indian Design (IS 800, 2007)

IS 800 [IS 800, 2007] clause 12.11.2.3 states that '*the panel zone shall be checked for shear buckling in accordance with clause 8.4.2 at the design shear defined in clause 12.11.2.2*'. The Clause 12.11.2.2 defines the shear demand for *connections*, V_{con} as,

$$V_{con} = \frac{2 \times 1.2 \times Z_{pb} F_{yb}}{L - d_c - 2S_h} + V_g. \tag{2.20}$$

Designing panel zone by Eq.(2.20) will lead to gross underestimation of the shear demand. This apparently might have resulted from an unintentional but improper choice of words; this can be rectified by a slight change of wording in clause 12.11.2.3 to '*the panel zone shall be checked for shear <u>yielding</u> in accordance with clause 8.4 for the <u>loading condition</u> defined in clause 12.11.2.2*'. Thus, the panel zone shear demand should be estimated as in Eq.(2.11). And, the panel shear yield capacity, V_{pzc} defined as,

$$V_{pzc} = \frac{F_{yc} d_c t}{\sqrt{3}}. \tag{2.21}$$

Further, a minimum thickness of panel zone to prevent the likelihood of shear buckling is recommended in the Indian code, which is identical to that in the American code (Eq.2.17).

2.5 Column to Beam Strength Ratio (CBSR)

Seismic design codes implement Strong Column Weak Beam (SCWB) design by specifying a minimum Column to Beam Strength Ratio (*CBSR*) requirement. The primary intend of providing a minimum *CBSR* requirement is not to eliminate yielding of columns altogether, but to minimize the probability of *weak storey mechanism* during seismic shaking [AISC 341, 2016].

A handful of studies in the past have attempted to arrive at the *CBSR* required for elastic response of columns in steel SMFs during seismic shaking [Lee, 1996; Nakashima and Sawaizumi, 2000; Medina and Krawinkler, 2005; Choi and Park, 2012; Choi et al., 2013; Zaghi *et al.*, 2014; Wongpakdee and Leelataviwat, 2017]. Nakashima and Sawaizumi monitored the maximum moments in column (M_{Cmax}) during seismic excitations (based on nonlinear time history analysis) in a generic steel frame modelled such that plastic hinges occur only at the ends of beams and column base [Nakashima and Sawaizumi, 2000; Nakashima *et al.*, 2002a]. It was observed that the *CBSR* (*CBSR*= M_{Cmax} /M_{pb},, where M_{pb} is beam plastic moment capacity; a rigid plastic idealization was adopted for beam hinges) required for column elastic response increased steadily with increment of ground motion amplitude and reached about 1.5 for a ground motion amplitude of 0.5 m/s (equivalent to Japan's large design earthquake level [BCJ, 1997]), and 2.0 for ground motion amplitude of 1.0 m/s. This steady increase in minimum *CBSR* requirement is attributed primarily to the higher mode response that tends to increase with increasing ground motion amplitude.

From all the studies conducted in the past, the following observations can be generalized:

1. Minimum *CBSR* requirement to prevent yielding of columns increases with increasing levels of shaking for given column to beam stiffness ratio;
2. At a given *CBSR*, increasing the column to beam stiffness ratio increases the moment demand in the column; and
3. The code prescribed minimum *CBSR* requirement of 1 will lead to extensive yielding of columns at higher shaking intensities.

2.5.1 *CBSR* Requirement as per Different Seismic Design Codes

The *CBSR* requirement, as required by different seismic codes, is discussed in following section.

2.5.1.1 American Design Code (AISC 341, 2016)

The AISC *Seismic Provisions* [ANSI/AISC 341, 2016] requires the columns in steel SMFs to satisfy the ratio of *nominal* plastic flexural capacity of columns, considering axial-flexure interaction, to the *overstrength* flexural plastic capacity of beams at column centreline to be greater than unity, which is expressed as,

$$\frac{\sum M_{pc}}{\sum M_{bo}} > 1 \ , \tag{2.22}$$

where $\sum M_{bo}$ is sum of the *overstrength* flexural strength of beams at column centreline. Thus, $\sum M_{bo}$, in Eq.(2.22), is given by,

$$\sum M_{bo} = \sum (M_{pri} + M_v), \tag{2.23}$$

where M_{pri} is as defined previously in Eq.(2.7), and M_v is the moment due to shear amplification at the column centreline, given by,

$$M_v = V_b\left(S_h + \frac{d_c}{2}\right). \tag{2.24}$$

For uniaxial bending of columns, $\sum M_{pc}$, in Eq.(2.22), is the sum of the nominal flexural strength of columns considering axial-flexure interaction, given by,

$$\sum M_{pc} = \sum Z_{pc} F_{yc}\left(1 - \frac{P_r}{P_y}\right). \tag{2.25}$$

Further, apart from satisfying Eq.(2.22), columns must be also be checked for axial loads determined from overstrength seismic load combinations, specified in ASCE 7 [ASCE 7, 2016]. However, moment in column can be neglected while performing column design with amplified seismic load combinations, which implicitly allows for some flexural yielding in columns. This underscores the fact the purpose of a minimum *CBSR* is to eliminate the possibility of a WCSB collapse mechanism, but not to entirely avoid yielding in columns.

2.5.1.2 European Design Code (CEN, 2004)

The Eurocode 8 [CEN, 2004], rather than directly specifying a minimum *CBSR* requirement, recommends the columns to be designed considering the interaction of overstrength design actions in columns ($N_{Ed,col}$, $M_{Ed,col}$, and $V_{Ed,col}$ are overstrength axial, moment, and shear demands in column, respectively) which are,

$$N_{Ed,col} = N_{Ed,g} + 1.1\gamma_{ov}\Omega N_{Ed,E}\ , \tag{2.26}$$

$$M_{Ed,col} = M_{Ed,g} + 1.1\gamma_{ov}\Omega M_{Ed,E}\ \text{, and} \tag{2.27}$$

$$V_{Ed,col} = V_{Ed,g} + 1.1\gamma_{ov} \Omega V_{Ed,E}\,, \tag{2.28}$$

where, $N_{Ed,g}$, $M_{Ed,g}$, and $V_{Ed,g}$ are the axial force, bending moment, and shear force, respectively, in the column due to non-seismic actions included in the combination of actions for seismic design load combination, $N_{Ed,E}$, $M_{Ed,E}$, and $V_{Ed,E}$, the axial force, bending moment and shear force, respectively, in the column due design seismic action, γ_{ov} the material overstrength factor for beam, which is the ratio of *expected* to minimum *specified* yield stress of the material (similar to R_y in AISC 341 (2016)), and Ω, the beam overstrength factor ($\Omega = M_{pb}/M_{Ed}$, where M_{pb} is the plastic flexural capacity of beam, and M_{Ed} the design moment in beam). Thus, Ω is the product of partial factor of safety for beams (which is '1.0' as per Eurocode) and overstrength in beam while selecting the beam section. The constant 1.1 accounts for the effects of strain hardening of the material and strain rate of loading [Elghazouli, 2010]. It can be inferred from Eq.(2.27), that the right hand side of the equation estimates the overstrength flexural capacity of the beam similar to the AISC approach, while the left hand side represents the probable moment demand on column.

2.5.1.3 Indian Design Code (IS 800, 2007)

The Indian code [IS 800, 2007] requires columns to satisfy Eq.(2.29).

$$\frac{\sum M_{pc}}{\sum M_{pb}} > 1.2\,, \tag{2.29}$$

where, $\sum M_{pc}$ is sum of the plastic flexural capacities of columns at a joint (M_{pc} is given by $Z_{pc}F_{yc}$), and $\sum M_{pb}$ the sum of the plastic flexural capacities of beams at a joint (M_{pb} is given by $Z_{pb}F_{yb}$). The constant 1.2 is supposed to account for the effects of strain hardening and strain rate. Thus, the current provision recommended in the Indian code for minimum *CBSR* requirement: (a) does not consider the expected increase in material yield stress from the characteristic value and amplification of moment due to shear, and (b) appears to ignore the reduction in flexural capacity of columns in the presence of axial load. Further, the columns are to be checked for overstrength load combinations mentioned in IS 800 (2007), where only axial forces need to considered while moments can be neglected; this also can lead to flexural yielding in columns during strong earthquake shaking.

2.5.2 Limitations of *CBSR* Requirement Estimates

CBSR estimates adopted in seismic codes [AISC 341, 2016; CEN, 2005; IS 800, 2007] is based on linear static analysis with the assumption that the inflection point in columns lies at the midspans as shown in Figure 2.16. However, under seismic excitations, due to nonlinear behaviour, the columns can undergo bending in single curvature too, leading to much larger moment demands in the columns. Further the possibility for single curvature bending increases as column to beam stiffness ratio increases. Hence, satisfying minimum CBSR requirement equation (Eq. 2.22, 2.27, or 2.29) is not sufficient to prevent yielding of columns.

2.6 General Steel SMF Design Provisions

The previous sections presented an in-depth review regarding the panel zone design approach and *CBSR* requirement. This section presents, other important aspects pertaining to seismic design of steel SMF. Starting with material selection, cross-section requirements, member bracing requirements, stability criterion, and drift limits.

2.6.1 Material

Structural sections used for seismic applications must have adequate 1) toughness, 2) weldabilty, and 3) ductility. High strength steels generally have increased carbon content and use micro-alloys which increase strength, but bring down the toughness, weldabilty, and ductility. Hence, design codes regulate the use of high strength steels. Table 2.1 compares the material requirements as specified in American,

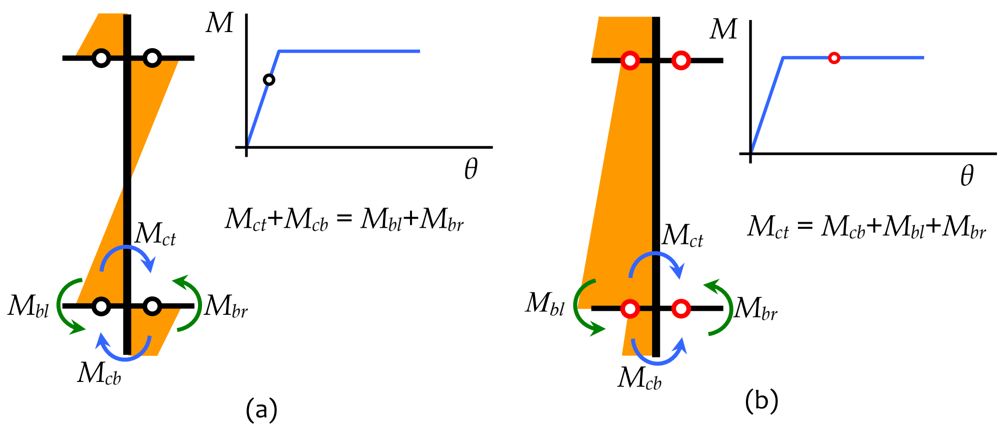


Figure 2.16: Column moment profile, (a) linear static behaviour and, (b) nonlinear behaviour as plastic hinges form and higher modes dominate the response.

Table 2.1: Mechanical properties of steels for seismic applications as per various design codes.

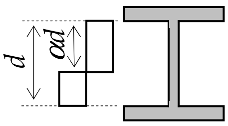
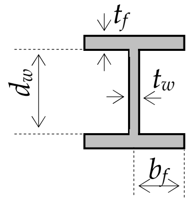
Property	AISC 341 [2016]	CEN [2005]	IS 800 [2007]#
Maximum Allowable Yield Strength for Beams	345 MPa*	450 MPa*	450 MPa*
Maximum Allowable Yield Strength for Columns	485 MPa*	450 MPa*	450 MPa*
CVN Toughness for Structural Steel	>27J @ 21^0C	>27J @ 21^0C	>27J @ 21^0C
Percentage Elongation for Structural Steel	>20%	>15%**	>20-23%##
$\frac{F_u}{F_y}$	>1.17	>1.1**	Not Specified
* Need to be adjusted corresponding to through thickness ** Data from CEN (2004) # Amended version of IS 800 (2007), given in NBC, Volume-1 (2016) ## Data from IS 2062 (2011)			

European, and Indian seismic codes. Most design codes also provide the R_y and R_t factors to aid designers. For heavy sections, additional material testing is required to account for the variation in through thickness properties that arises from differential cooling. Similar to steel sections, the weld filler material is also supposed to meet toughness and ductility related specifications as required by material standards [EN 1011-2, 2001; IS 814, 2004; AWS D1.8/D1.8M , 2016].

2.6.2 Section

Steel as a material having sufficient ductility does not necessarily mean that adequate section ductility is guaranteed. Premature local buckling curtails the section ductility. Local buckling causes high localized inelastic strain demands, which when repeated, leads to premature fracture due low cycle fatigue. Hence, for wide flange sections, the web and flange slenderness ratios are limited as mentioned in Table 2.2 to prevent premature local buckling.

Table 2.2: Limiting width-thickness ratios for hot-rolled sections allowed to be used for seismic applications as per various codes.

Ratio	**AISC 341 [2016]** (For highly ductile members)	**CEN [2005]** (Class 1, Plastic section)	**IS 800 [2007]** (Class 1, Plastic sections)
$\frac{b_f}{t_f}$	$0.32\sqrt{\frac{E}{R_y F_y}}$	$9\sqrt{\frac{235}{F_y}}$	$9.4\sqrt{\frac{250}{F_y}}$
$\frac{d_w}{t_w}$	$0.88\sqrt{\frac{E}{R_y F_y}}(2.68 - C_a), for\, C_a > 0.11$ $2.57\sqrt{\frac{E}{R_y F_y}}(1 - 1.04C_a), for\, C_a \le 0.1$	$\frac{396\sqrt{\frac{235}{F_y}}}{13\alpha - 1}, for\,\alpha \ge 0.$ $\frac{36\sqrt{\frac{235}{F_y}}}{\alpha}, for\,\alpha < 0.5$	$\frac{84\sqrt{\frac{250}{F_y}}}{1+r} \ge 42\sqrt{\frac{250}{F_y}}$
	$C_a = \frac{Axial\ Load\ \ from\ LRFD}{0.9 R_y F_y A_g}$		
			

2.6.3 Member

Good section ductility is translated to member level only if the member slenderness is controlled to preclude premature global buckling. For beams, lateral bracing needs to be provided to preclude lateral torsional buckling [Nakashima *et al.*, 2002b]. AISC 341 [ASCE 341, 2016] limits the maximum lateral brace spacing (L_b) for wide flange beams in steel SMFs to

$$\frac{L_b}{r_y} < 0.095\frac{E}{R_y F_y}, \tag{2.30}$$

where, r_y is the radius of gyration about minor axis. Both, the top and bottom flanges need to be braced laterally. European and Indian design codes require the beams to be sufficiently braced, but do not provide expressions to calculate L_b under seismic actions.

However, bracing should not be provided in the *protected zone*, *i.e.*, the region where beam flexural hinge is supposed to develop, since it can delay the hinge formation. For columns also, the slenderness ratio need to be regulated to preclude premature global buckling. An easy way to prevent the flexural torsional buckling and out of plane bending modes is to use stocky wide flange sections (*e.g.*, W14 sections as columns in SMFs) with smaller depths and considerable minor axis bending capacity compared to major axis capacity.

2.6.4 Structure

Ductility at global level is hampered by amplification of P-Δ effects at large drifts. Hence, seismic codes for steel SMFs attempts to secure the ductility of the system by limiting the drifts and/or by restricting the value of stability coefficient (Θ). ASCE 07 [ASCE 07, 2016] defines stability coefficient as

$$\Theta = \frac{P\Delta}{Vh} < \frac{0.5}{\beta C_d} < 0.25\,. \tag{2.31}$$

where, P is the axial load in the column, Δ the interstorey drift, V the shear demand in the column, h the storey height, β the ratio of shear demand to shear capacity, and C_d the displacement amplification coefficient (5.5 for SMFs as per ASCE 07 (2016)). Limiting drift or stability coefficient essentially results in a stiffer system thereby decreasing the P-Δ amplification. The design drift requirement as per different seismic codes are listed in Table 2.3.

Table 2.3: Design drift limits as per various design codes.

Drift Limit for SMF	ASCE 07 [2016]	CEN [2004]	IS 1893 [2016]
Drift ratio $\left(\frac{C_d \delta_e}{I\ h}\right)$	2%, 1.5% or 1%#	-	-
Elastic drift ratio $\left(\frac{\delta_e}{h}\right)$	-	0.5%, 0.75% or 1%*	0.4%

Depending on occupancy category
* Depending on infill material
h : Storey height
C_d : Displacement amplification factor [ASCE 07, 2016]
δ_e : Elastic interstorey drift from corresponding load combinations

2.7 Nonlinear Analysis

Nonlinear analysis has become a popular tool, especially to assess seismic behaviour of structures, especially wherein high levels of inelastic action are expected. However, even the most sophisticated nonlinear analysis tool cannot predict the exact response accurately enough due to uncertainties involved at multiple levels of modelling. Despite this, an exact response is not critical, since the ground motions anyway are going to induce much larger randomness [Krawinkler, 2006]. But, it is essential to use analytical tools, modelling assumptions, and component models which give results with sufficient accuracy. The following section briefly describes the two common nonlinear analysis methods currently used in practice.

2.7.1 Nonlinear Static Analysis (Pushover Analysis)

Nonlinear static analysis is a popular seismic performance assessment tool for preliminary analysis. Even though, nonlinear static procedures are inferior to nonlinear dynamic procedures in many respects, static procedures do have some advantages. First, nonlinear static procedures are computationally cheaper. Further, they use widely acceptable response spectrum rather than a suite of ground motions for seismic demand estimation. Additionally, the sequence of damage formation under a specific load pattern can be monitored. However, the main limitation arises from the fact that they use static procedures to capture dynamic behaviour.

A large number of nonlinear static procedures have been developed over the years. Despite this, the basic procedure has remained the same which involves: (1) selection of a load pattern, (2) generation of the pushover curve using the selected load pattern, (3) conversion of the pushover curve to capacity curve (*i.e.*, convert the MDOF structure response curve to equivalent SDOF response curve), (4) selection of a trial demand point on the bilinear approximation of the capacity curve, (5) estimation of the seismic demand using response spectrum and re-estimation of the demand point on the capacity curve, and finally, (6) verification of whether the trial demand point and the estimated demand point matches within the tolerance limit. If a match is obtained, the current demand point is designated as the *performance point*; else, iteration is continued till convergence is reached. These steps involved in nonlinear static analysis is illustrated in Figures 2.17.

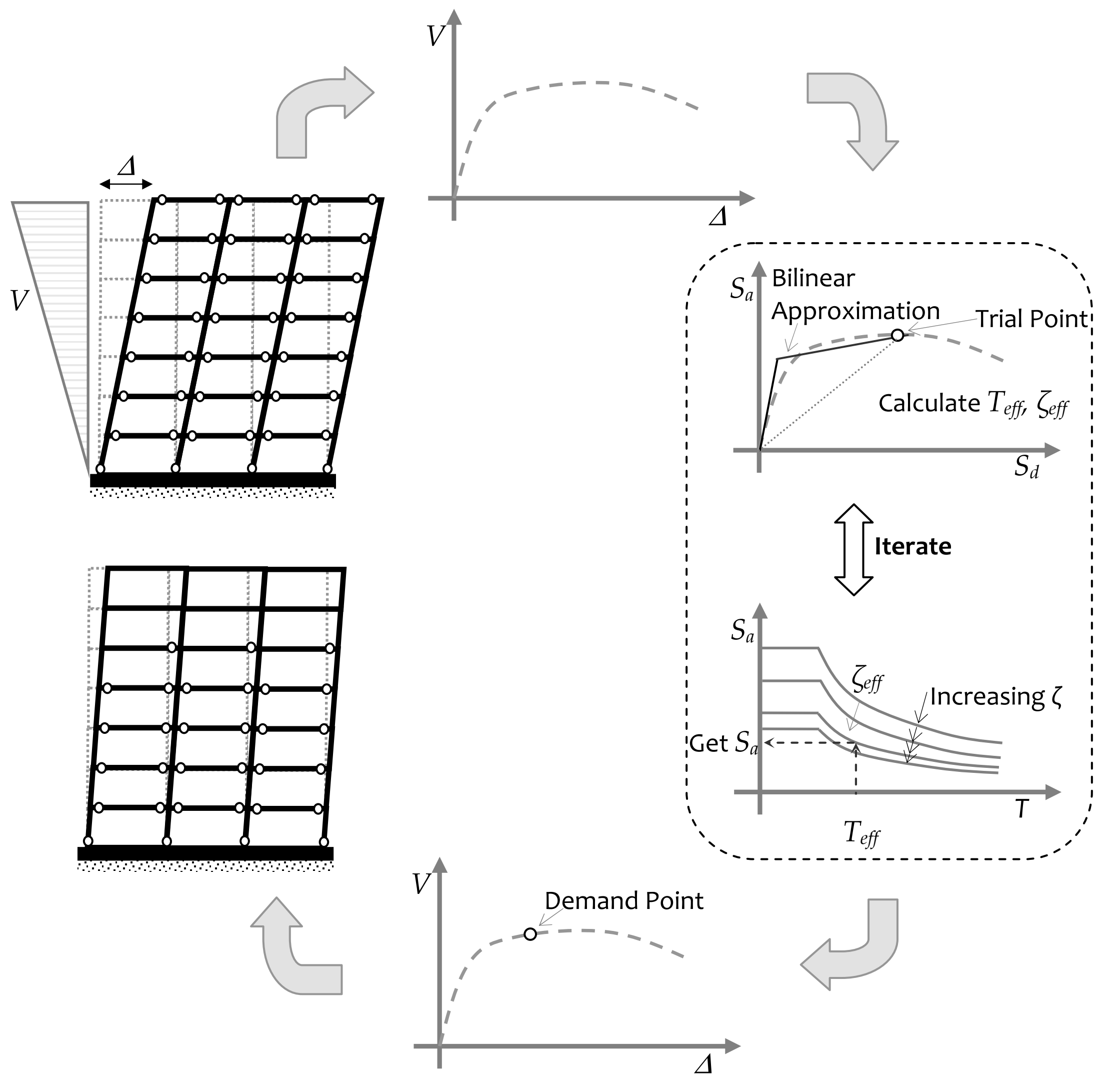
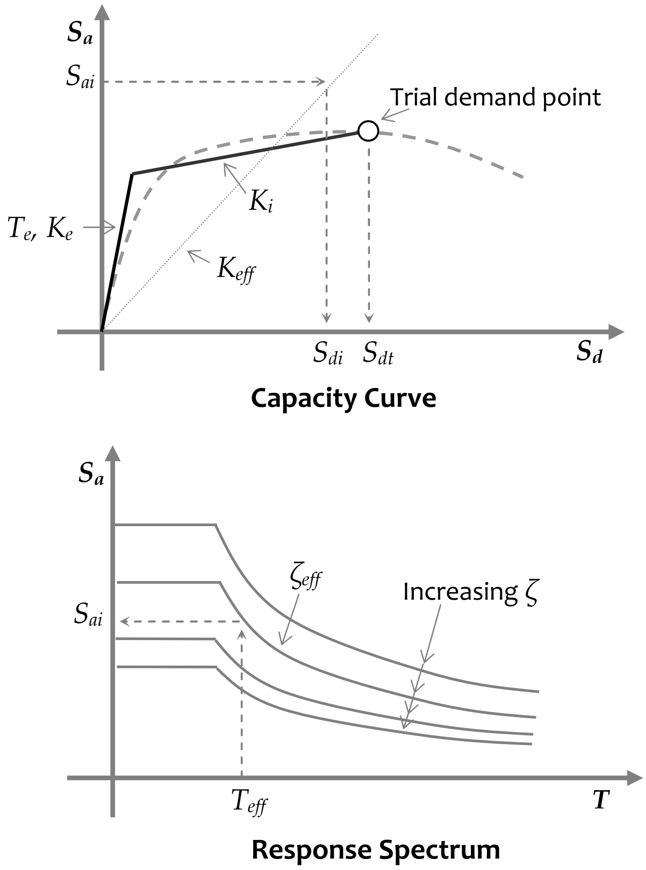


Figure 2.17: General steps in nonlinear static analysis.

Of all the nonlinear static methods, Capacity Spectrum Method [ATC, 1996], FEMA 440 Coefficient Method [FEMA 440, 2005], and FEMA 440 Linearization Method are the most popular ones. Among the three, the FEMA 440 Linearization Method is found to be the most accurate one [Powell, 2006]. Key steps in FEMA 440 Linearization method are illustrated in Figure 2.18. However, nonlinear static methods in general are incapable of capturing local response of structures (maximum plastic rotation demands and interstorey drifts) with sufficient accuracy. This is because in structures with significant higher mode participation, usage of simple load patterns (*e.g.*, triangular, first-mode based, parabolic) for pushover analysis lead to erroneous results. Hence, static methods are, in general, reliable only for structures whose response is dominated by fundamental mode (usually regular low-rise building). Thus, nonlinear static procedures are generally recommended for preliminary assessment only [Powell, 2006].



(1) Select trial demand point on capacity curve and construct bilinear approximation curve. Find elastic stiffness (K_e), post-elastic stiffness (K_i), elastic time period (T_e), and ductility ratio (μ).

(2) Calculate effective period (T_{eff}), stiffness (K_{eff}) and damping (ζ_{eff}) using T_e, K_e, K_i, and μ from FEMA 440 linearization equations [FEMA 440, 2005].

(3) Given T_{eff} and ζ_{eff}, from response spectrum, get spectral acceleration demand.

(4) Given S_a, estimate the spectral displacement demand (S_{di}). Iterate till the spectral displacement corresponding initially selected trial demand point and estimated spectral displacement demand matches.

Figure 2.18: Steps involved in FEMA 440 Linearization method [FEMA 440, 2005]

2.7.2 Nonlinear Dynamic Analysis (Time History Analysis)

Nonlinear dynamic analysis or time history analysis is considered to be the most reliable analysis tool available to predict the response of structures subjected to earthquake ground motions. However, seismic performance assessment through nonlinear dynamic analysis has its own limitations, which mainly arise from: (1) selection of ground motions, and (2) nonlinear modelling of component response.

The ground motion selection needs to consider the location of the structure, since the expected ground motion at a location depends on the probable faulting mechanism associated with the region, the distance of the fault from the site, the local site effects, *etc*. Hence, this step introduces uncertainty to the assessment process. Once ground motions are selected, scaling of ground motions to evaluate the structural response at the desired intensity of shaking presents further challenge. Two popular scaling methods used are: (1) peak ground acceleration (PGA) scaling, and (2) spectral amplitude matching. Although, both the scaling methods preserve the frequency content of the ground motion while only the amplitude of ground motion is altered, the latter is recommended, since the response of a building towards seismic excitation at its fundamental period correlates reasonably well to the overall nonlinear behaviour of the building [Elnashai and Di Sarno, 2008].

Modelling assumptions used to represent the nonlinear behaviour also reduce the reliability of results from nonlinear dynamic analysis. For instance, the inelastic demands are heavily dependent on the component backbone curve and the hysteretic rule used in the analysis. Further, the non-consideration of P-Delta effects, joint flexibility, and lateral resistance offered by gravity frames, can significantly affect the nonlinear response of the building (see Figure 2.19 [Gupta and Krawinkler, 1999]). Also, a direct time history analysis at the design level earthquake usually does not provide information regarding the sequence of damage expected in the building. Thus, a variation of the nonlinear dynamic analysis, called the Incremental Dynamic Analysis (IDA), is developed wherein the ground motion intensity is gradually increased. This can now provide details regarding the sequence of damage and intensity of shaking required to cause dynamic instability in buildings [Vamvastikos and Cornell, 2002].

2.8 Numerical Model for Nonlinear Analysis

The nonlinear component models generally used can be classified into three types, which are: (1) concentrated (lumped) hinge models, (2) fibre hinge models, and (3) continuum models [ATC, 2017]. Even though, continuum model is the most sophisticated among the three, the simpler concentrated hinge model does have certain advantages over the others.

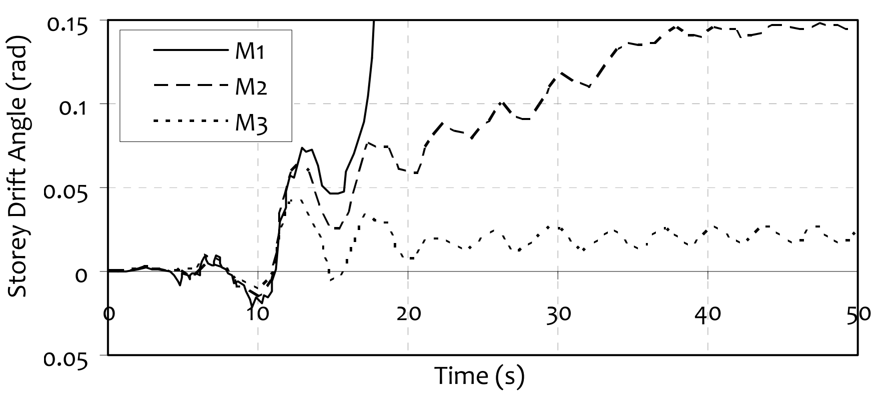


Figure 2.19: Response of a 20-storey MRF subjected to Tabas ground motion record for three analysis models: (a) M1 (centreline model), (b) M2 (panel zones also incorporated into the model), and (c) M3 (model also considers stabilization due to gravity frames) [Gupta and Krawinkler, 1999].

Continuum models are best suited to evaluate localized behaviour (*e.g.*, local buckling, fracture) which generally cannot be captured by simplified models. Further, continuum models do not enforce kinematic constraints (*e.g.*, Euler-Bernoulli hypothesis), which make them appropriate to study complex geometries (*e.g.*, reinforced connection). However, these models come with increased computational cost and the results are sensitive to the boundary conditions. Therefore, continuum models are seldom used to evaluate overall frame response. At the other extreme of continuum models are the concentrated hinge models, which are computationally efficient and are widely used to evaluate global response of buildings. The concentrated hinges are assigned to regions of *expected* inelasticity. The component models are often semi-empirically calibrated with experimental results, which enable them to capture the strength and stiffness degradation arising from local buckling with sufficient accuracy. In-between the continuum and concentrated models is the fibre hinge model. Similar to the continuum models, fibre models typically use uniaxial constitutive rules, but subjected to kinematic constraints. Like concentrated models, fibre hinge models are often assigned to the *expected* locations of inelastic action. Compared to concentrated models, these models are better suited to capture axial-flexural (*P-M*) interaction. However, fibre models typically do not capture local buckling and associated strength degradation and stiffness deterioration [ATC, 2017]. Despite this, fibre models are computationally efficient than continuum models, and hence, are widely used to evaluate global frame response.

Connections and splices in steel SMFs are generally designed as capacity protected elements, as a result of which, elastic response is expected from these components. Therefore, inelastic modelling of connections and splices are rarely done. However, if inelasticity is expected in connection, inelastic modelling of connections should be incorporated in the nonlinear building model. The following section describes beam, column, and panel zone nonlinear component models used for steel SMFs, with emphasis on concentrated and fibre models.

2.8.1 Beam

Concentrated hinge models are preferred for steel wide flange beams compared to fibre hinge models, since the former is better suited to capture the strength degradation and stiffness deterioration resulting from local buckling [PEER-ATC, 2010]. Hence, the discussion is limited to concentrated models, which can be specified either as Moment-Curvature (M-φ) or Moment-Rotation (M-θ) relation. M-φ hinge definitions require hinge length to be specified, which depend primarily on depth of the section, but also on beam span and section geometry. Hence, it is preferable to specify M-θ hinges, which are semi-empirically calibrated based on experimental results. Various models which estimate the M-θ relations of steel beams have been proposed [Ziemian et al., 1992; Gioncu and Petcu, 1997; Lignos and Krawinkler, 2010; ASCE 41, 2013; Hannamwale, 2014]. However, most of the models do not consider all the factors that lead to strength degradation and stiffness deterioration; two of the commonly used flexural hinge definitions are shown in Figure 2.20. The ASCE 41 hinge definition estimates pre-capping (θ_p) and ultimate (θ_u) rotations as a multiple of yield rotation (θ_y), which often results in overestimation of the rotation capacity and make the definition very sensitive to shear span to depth ratio. Further, ASCE 41 does not specify the post-capping negative stiffness, which is not experimentally calibrated. Hence, multivariate regression formulations were used to calibrate M-θ relations using over 300 experimental results [Lignos and Krawinkler, 2007]. The plastic rotation capacity (θ_p) is then given by,

$$\theta_p = 0.087\left(\frac{h}{t_w}\right)^{-0.365}\left(\frac{b_f}{2t_f}\right)^{-0.14}\left(\frac{L_s}{d}\right)^{0.34}\left(\frac{d}{0.0254\times 21}\right)^{-0.721}\left(\frac{F_{ye}}{0.145\times 50}\right)^{-0.23}, \quad (2.32)$$

where h/t_w is ratio of fillet to fillet web depth to web thickness, L_s/d the ratio of shear span to overall depth of the beam section, b_f/t_f the ratio of flange width to flange thickness, F_{ye} the *expected* yield stress, and d the depth of the section; in the above, linear dimensions are in metres while yield stress is in MPa. Further, the post-capping rotation (θ_{pc}) is computed as,

$$\theta_{pc} = 5.70\left(\frac{h}{t_w}\right)^{-0.565}\left(\frac{b_f}{2t_f}\right)^{-0.80}\left(\frac{d}{0.0254\times 21}\right)^{-0.28}\left(\frac{F_{ye}}{0.145\times 50}\right)^{-0.43}. \quad (2.33)$$

The effective yield strength, M_y is taken as $1.1M_p$, where M_p ($=Z_pF_{ye}$) is overstrength flexural plastic section capacity, the capping strength M_c as $1.1M_y$, and the residual strength, M_r as $0.4M_y$.

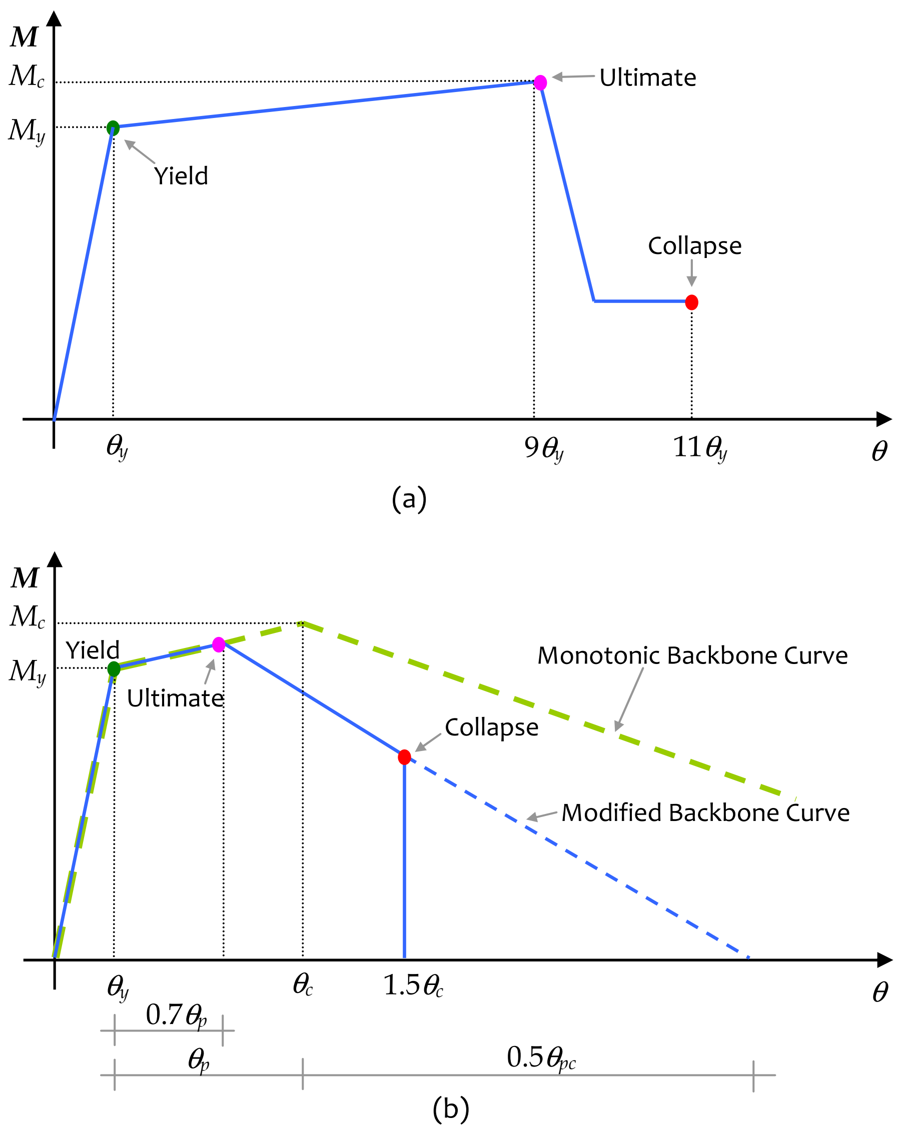


Figure 2.20: Moment-rotation backbone curve of steel wide flange sections as per (a) ASCE-41, 2013, and (b) Lignos and Krawinkler, 2010

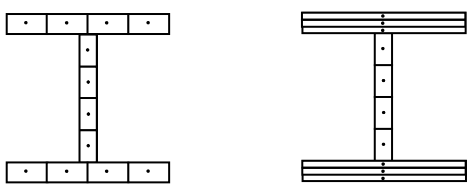
2.8.2 Column

Columns in steel SMFs can develop inelasticity under severe seismic excitation depending on the design CBSR used. Therefore, nonlinear modelling of column is required too. But unlike in beams, rotation capacity of columns is curtailed due to presence of axial loads. Further, the number of full-scale experimental tests done on columns is limited; hence semi-empirically calibrated concentrated hinge definitions available for columns are not reliable as compared to those available for beams. Also, limited number of tests done on stocky wide flange columns (W14 sections) have demonstrated that those sections have fairly stable hysteretic behaviour, as local and lateral buckling modes are dormant at drift ranges of practical interest [Newell and Uang, 2008]. Hence, fibre modelling is considered to be ideal for these stocky sections.

However, deep wide flange sections are prone to web local buckling, flexural torsional buckling, and out of plane twisting in the presence of axial loads; hence, fibre models which are unable to capture local buckling modes should be avoided [Newell and Uang, 2006; Elkady and Lignos, 2015]. Global analysis using fibre models can be made computationally efficient by optimizing the number, location, and orientation of fibres. The distribution of fibres as depicted in Figure 2.21(a) is shown to be capable of producing results with remarkable accuracy for columns [Kostic and Filippou, 2011]. For, uniaxial bending, a better discretization scheme is as shown in Figure 2.21(b).

2.8.3 Panel Zone

The shear behaviour of panel zones is best captured using models shown in Figure 2.22. The Scissor model is simple in its representation, but it cannot capture the kinematics of joint deformation accurately. Despite this, results obtained using the Scissor model was found to correlate well with experimental results in frames with equal bay and equal storey dimensions [Charney and Downs, 2004; Charney and Marshall, 2006]. In contrast, the Krawinkler model, though complex, do not have any such limitation. The Krawinkler model has 8 rigid elements which are interconnected at the four corners with hinges as shown in Figure 2.22(a), making an assembly that deforms as a parallelogram. Three among the four hinges are real and the final hinge is assigned the shear stiffness and strength of the panel zone; its use is widely accepted and recommended [ATC, 2017]. However, it is important to note that both models ignore axial and flexural deformations of the panel zone, as experiments on short links have demonstrated that shear deformations are dominant. However, axial deformations can be significant, especially in panel zones of exterior columns in tall buildings. In such cases, the column axial flexibility can be incorporated by changing the vertical rigid links in the panel zone model to links with finite axial stiffness.



(a) (b)

Figure 2.21: Fibre hinge discretization scheme for wide flange sections for (a) biaxial bending, and (b) uniaxial bending.

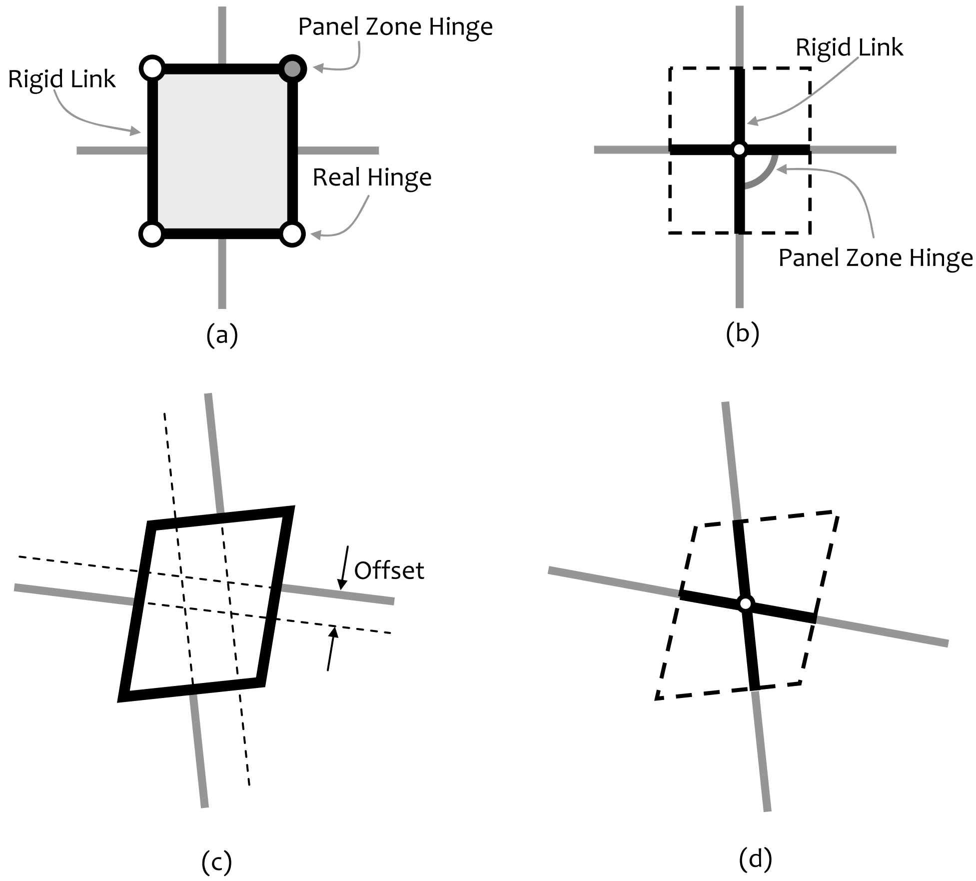


Figure 2.22: Panel zone models: geometry of (a) Krawinkler model, and of (b) Scissor model, and joint deformation kinematics in (c) Krawinkler model, and in (d) Scissor model.

2.9 Gap Areas

Structural design compliant to the provisions of seismic design codes are supposed to guarantee at least collapse prevention of the building during a severe seismic event. In that regard, the latest edition of Indian code [IS 800, 2007] for the design of steel SMFs came into existence in 2007, with few minor amendments following. However, till date, there are no extensive studies which report the seismic performance of steel SMFs designed compliant to the code. Most of the provisions of code are adopted from international codes [AISC 341, 2005; CEN, 2004]. Although, the intent of the design provisions of the design codes are similar, few critical differences persist between the provisions outlined in Indian seismic design code and other seismic design codes (more specifically, the American and the European ones). The key differences are: (a) in recognising the expected increase in material yield stress from the minimum specified, or characteristic, yield strength of the material, (b) in estimating the demand on and the capacity of panel zones, and (c) in recommending a minimum column to beam strength ratio (*CBSR*) for the design of columns in SMFs. Further, the load combinations and drift limits considered in the designs codes also vary. Hence, it is necessary to assess the implication of these critical differences on the seismic behaviour of steel SMF buildings.

Further, there is a difference in opinion among the research community on whether inelasticity should be allowed in panel zone during severe seismic actions. In addition, ambiguity still exists regarding the combination of panel zone shear demand and capacity to be considered to proportion a balanced or strong panel zone. While many studies are reported in literature of behaviour of panel zones based on component tests or analyses, extensive studies of actual building frames reporting the behaviour of code compliant panel zones are also limited.

Another important aspect concerned with seismic behaviour of moment frame is the minimum *CBSR* requirement. It is known that, the code requirement of minimum *CBSR* does not guarantee elastic response of columns; *CBSR* values greater than 1.5 is recommended to limit the probability of column yielding. In this context, use of higher grades of structural steel for columns in SMFs can help achieve capacity-protection of panel zones and columns with smaller column cross-sections. However, there is a lack of studies which investigate the effectiveness of using columns with higher grade of steel, *vis-à-vis* design and behaviour of panel zones. Finally, eliminating the need of using doubler plates in panel zones can have many advantages. However, studies which report the *CBSR* required to eliminate the use of doubler plate are also limited.

Thus, the gap areas identified can be summarized as:

1. Lack of studies which evaluate the seismic behaviour of steel SMF buildings designed as per Indian design codes [IS 800, 2007; IS 1893, 2016];
2. The strength hierarchy between the structural components (beam, column, and panel zone) to be followed in design of steel SMFs; to be specific, whether yielding of panel zones is to be allowed;
3. Ambiguity regarding the combination of panel zone shear demand and capacity to be considered to design balanced and strong panel zones;
4. Lack of studies which investigate the merits and demerits in using higher grade steel for columns; and
5. Investigations to find the *CBSR* required to eliminate the use of additional doubler plates in panel zones.

2.10 Objective and Scope

Based on the identified gap areas, the objective of the current study is to:

1. Identify improvements possible, if any, in provisions for design of steel SMF buildings included in the current Indian design code [IS 800, 2007];
2. Determine suitable combination of estimates of panel zone shear demand and capacity to achieve a balanced and/or strong panel zone, and identify the ideal panel zone design approach;
3. Investigate the merits and demerits in using higher grade steel for columns in steel SMF buildings; and
4. Evaluate the *CBSR* requirement to eliminate the use of doubler plate and recommend an appropriate *CBSR* for design of steel SMF.

The scope of the current study is limited to;

1. Use of steel regular buildings with perimeter SMFs having equal bays, fixed base condition, and founded on medium soil, to evaluate seismic behaviour of such buildings;
2. Bare frame building models are considered to evaluate the seismic behaviour; and
3. Connection response is assumed to be rigid and elastic.

…

Blank Page

Chapter 3

Seismic Performance Evaluation of Steel SMF Buildings Designed as per Indian Codes

3.0 Introduction

Seismic performance evaluation of two (a 3- and a 9-storey) steel Special Moment Frame (SMF) office buildings, designed to be compliant with the provisions of Indian codes [IS 800, 2007; IS 1893-Part 1, 2016] are presented in this Chapter. The two buildings have perimeter SMFs as the lateral load resisting system. Nonlinear static and dynamic analyses are employed to evaluate the seismic performance of the buildings. As discussed previously in Chapter 2, few *critical differences* persist between the design procedures outlined in Indian seismic design code and other seismic design codes (more specifically, the American and the European ones). The key differences are: (a) in recognising the expected increase in material yield stress from the minimum specified, or characteristic yield strength of the material, (b) in estimating the demand on and the capacity of panel zones, and (c) in recommending a minimum column to beam strength ratio (*CBSR*) for the design of columns in SMFs. Hence, the objective of the performance evaluation presented in this Chapter is to ascertain, whether the above mentioned *critical differences* can jeopardize the seismic performance of buildings with steel SMF (designed compliant to Indian codes) as lateral load resisting system.

3.1 Building Model

The geometric configuration of the office buildings considered for the current study is shown in Figure 3.1. These building have geometry similar to the 3- and 9-storey office buildings designed as part of SAC Steel Joint Venture Project; more details can be found in Appendix B of FEMA-355C [FEMA-355C, 2000e]. All the interior spans have simple framing designed to resist gravity loading. A typical SMF in the shorter direction (North South direction (refer Figure 3.1)) is considered for two dimensional nonlinear static and dynamic analyses. The total equivalent seismic force in a direction is shared equally by the two parallel SMFs in that direction. Further, these SMFs also resist gravity loads coming from half span of a bay in the perpendicular direction. Lateral load resisting system in both the buildings are considered to have fixed base.

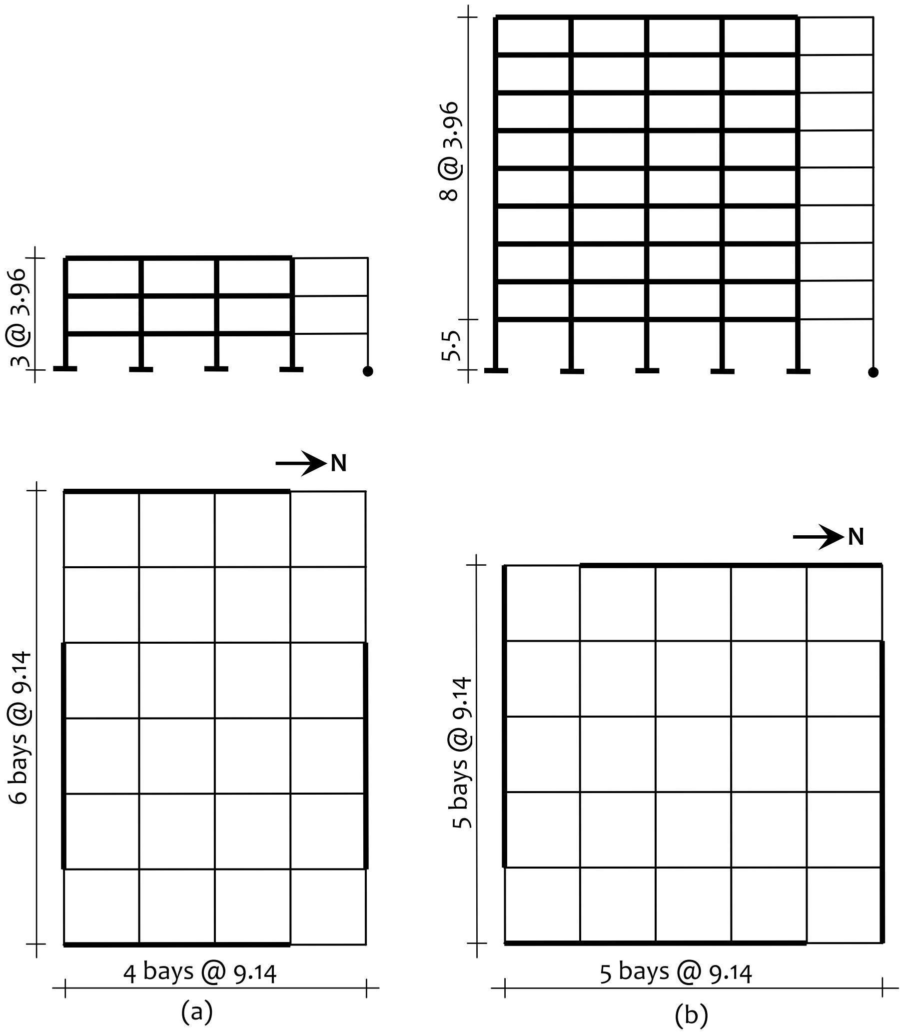


Figure 3.1: Plan and elevation of model buildings; (a) 3-storey, and (b) 9-storey (building dimensions in metres)

3.2 Seismic Design of SMFs

3.2.1 General Design Considerations

The SMF's in the shorter direction (North South) are designed to be compliant with the provisions of Indian codes [IS 800, 2007; IS 1893-Part 1, 2016], considering gravity loads and seismic effects. Dead and live load considered for a typical floor are 4.6 kN/m^2 and 2.4 kN/m^2 respectively. For the roof, dead and live load considered are 4.0 kN/m^2 and 1.0 kN/m^2. The study buildings are designed for seismic hazard level with Zone Factor (Z) [IS 1893-Part 1, 2016] equal to 0.4g (which is approximately equal to the highest level of shaking recognised by the current Indian code). A site with medium soil condition (Soil type-II) as per IS 1893-Part 1 (2016) is considered for seismic design. ASTM A36 steel with a minimum specified (or characteristic) yield stress and expected yield stress of 250 MPa and 375 MPa, respectively, is used for beams. ASTM A992 steel with a minimum yield stress and expected yield stress of 345 MPa and 379.5 MPa, respectively, is used for both the columns and the doubler plates in panel zones.

3.2.2 Design of Model Buildings

The load combinations as per IS 1893-Part 1 (2016) considered for the design of the study buildings are

1.5*DL* + 1.5*LL*
1.2*DL* + 1.2*LL* + 1.2*EQ*
1.5*DL* + 1.5*EQ*
0.9*DL* + 1.5*EQ*

where, *DL* is the dead load, *LL* is the live load, and *EQ* is the equivalent seismic force. It is the load combination 1.5*DL* + 1.5*EQ*, which governs the selection of beams for the study buildings. The minimum Column to Beam Strength Ratio (*CBSR*) governs the selection of columns. The *CBSR* requirement as specified by Indian code [IS 800, 2007] for SMFs is

$$CBSR_i = \frac{\sum Z_{pc} F_{yc}}{\sum Z_{pb} F_{yb}} > 1.2 \tag{3.1}$$

where, $CBSR_i$ is the *CBSR* definition according to Indian code [IS 800, 2007], Z_{pc} and Z_{pb} are the plastic section modulus of columns and beams, respectively, and, F_{yc} and F_{yb} are the minimum specified/characteristic yield stress of the columns and beams, respectively. The columns are also checked for the special/overstrength load combination specified in Indian code [IS 800, 2007]. Finally, the panel zones are designed for a shear demand, V_{pzd}

$$V_{pzd} = \sum_{i=1}^{2} \left[\left(\frac{M_{cfi}}{d_{bi} - t_{bfi}} \right) - \left(\frac{M_{pbi}}{L_i - d_{ci} - 2S_{hi}} \times \frac{L_i}{h} \right) \right] \tag{3.2}$$

where M_{cf} is the moment at the column face, M_{pb} is the plastic moment capacity of the beam section, L_i is the beam-to-beam centreline span, S_h is the distance of plastic hinge from column face, d_c and d_b are the depth of column and beam respectively, and t_{bf} is the thickness of beam flange. Satisfying the shear buckling capacity check for panel zones as per the Indian code [IS 800, 2007], the panel zones are designed to have the yield capacity as the strength capacity for the purpose of current design. The yield capacity of panel zone (V_{pzc}) is given as,

$$V_{pzc} = \frac{F_y(0.95d_c)t}{\sqrt{3}} \tag{3.3}$$

where, F_y is the minimum specified yield stress of the panel zone, d_c the depth of column section, and t the thickness of panel zone. It is assumed that the beam plastic hinge forms at a distance $0.5d_b$ from the column face. The details of the beams, columns and panel zones of the SMFs of the 3- and 9-storey study buildings are given in Table 3.1 and Table 3.2, respectively.

Table 3.1: Details of the SMF of the 3-storey study building designed as per Indian code considered for performance evaluation

3 Storey SMF	**Beam**	**Column**	***CBSR*i**	**Doubler Plate Thickness (mm)**	
Storey				**Interior**	**Exterior**
3	W24 × 76	W 14 × 233	1.50	12	0
2	W30 × 132	W 14 × 233	1.38	39	7
1	W30 × 132	W 14 × 233	1.38	39	7
Building Natural Period: **1.03 seconds** Design Base Shear: **1447 kN** *Note: CBSR is as per Eq.(3.1)*					

Table 3.2: Details of the SMF of the 9-storey study building designed as per Indian code considered for performance evaluation

9 Storey SMF	**Beam**	**Column**	***CBSR*i**	**Doubler Plate Thickness (mm)**	
Storey				**Interior**	**Exterior**
9	W24 × 76	W14 × 211	1.35	15	0
8	W24 × 76	W14 × 211	2.69	15	0
7	W30 × 124	W14 × 211	1.32	38	8
6	W30 × 124	W14 × 211	1.32	38	8
5	W30 × 124	W14 × 257	1.65	30	0
4	W30 × 148	W14 × 257	1.34	43	8
3	W30 × 148	W14 × 257	1.34	43	8
2	W30 × 148	W14 × 257	1.34	43	8
1	W36 × 150	W14 × 342	1.60	29	0
Building Natural Period: **2.68 seconds** Design Base Shear: **1878 kN** *Note: CBSR is as per Eq.(3.1)*					

3.2.3 Building Nonlinear Model

Two-dimensional numerical model used for nonlinear analyses of the 3-storey study building is shown in Figure 3.2; a similar model is developed for the 9-storey building too. To simulate the *P-Δ* (global) effects, leaning columns, made up of rigid plane frame element, are added to the numerical model. The gravity loads (1.0*DL*+0.2*LL*) from the interior frames are lumped at the nodes as shown in Figure 3.2. The effect of gravity frames and slabs are not considered in the model. Nonlinear analysis is carried out using commercial software Perform 3D [CSI, 2016]

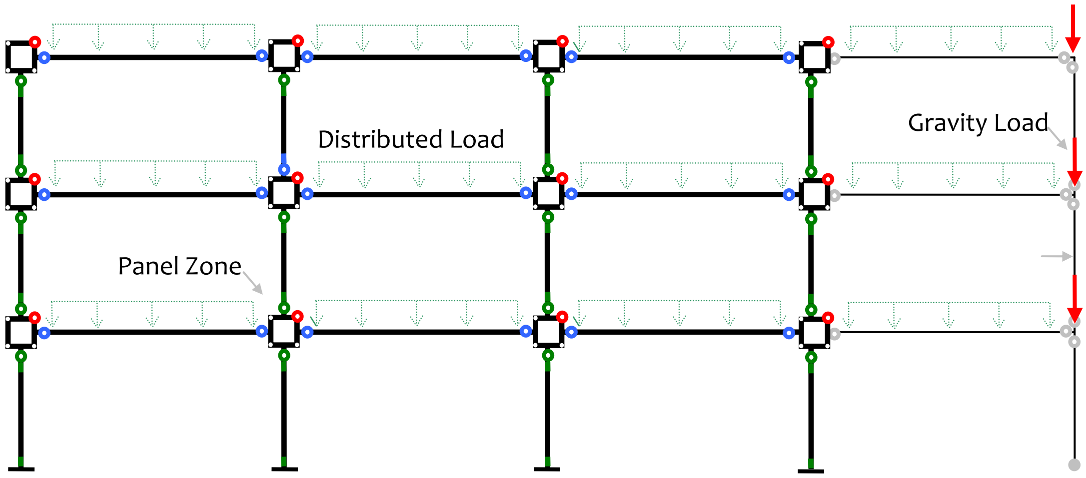


Figure 3.2: Two-dimensional numerical model of 3-storey frame

The beams are modelled with elastic frame elements with inelastic rotational springs lumped at both ends (at distance of $0.5d_b$ from the column faces). Monotonic hinge properties of the inelastic rotational springs used to represent the beam properties are adopted from PEER-ATC, Modelling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings [PEER-ATC, 2010; Lignos and Krawinkler, 2011]. The monotonic hinge properties are modified further to account for cyclic degradation as per the guidelines provided in PEER-ATC. Figure 3.3 shows the hinge definition of a typical beam section along with the damage limit states considered. The panel zones are modeled with four rigid elements connected with three pins and an inelastic rotational spring to capture the shear flexibility of the panel zone; the tri-linear inelastic response of the inelastic rotational spring of the panel zone is as shown in Figure 3.4 (Krawinkler, 1978). The limit states considered in study are also marked in the Figure 3.4. Finally, the columns are modelled with elastic column elements and inelastic fibre hinge elements distributed at both ends of a column. The fibre hinge length is taken as 1.4 times the depth of the section [Elkady and Lignos, 2015]. Bilinear stress strain relationship is considered for steel. Results of finite element simulations in the literature have concluded that the critical buckling strain for highly compact plastic sections can be conservatively assumed to around 25 times the yield strain (ε_y) [Hanamawale, 2014].

Based on the above work, the bilinear stress strain curve of steel used in the study adopted is depicted in Figure 3.5. Nonlinear analyses are carried out till any of the elements (beams, panel zones, or columns) reach their *collapse limit state*.

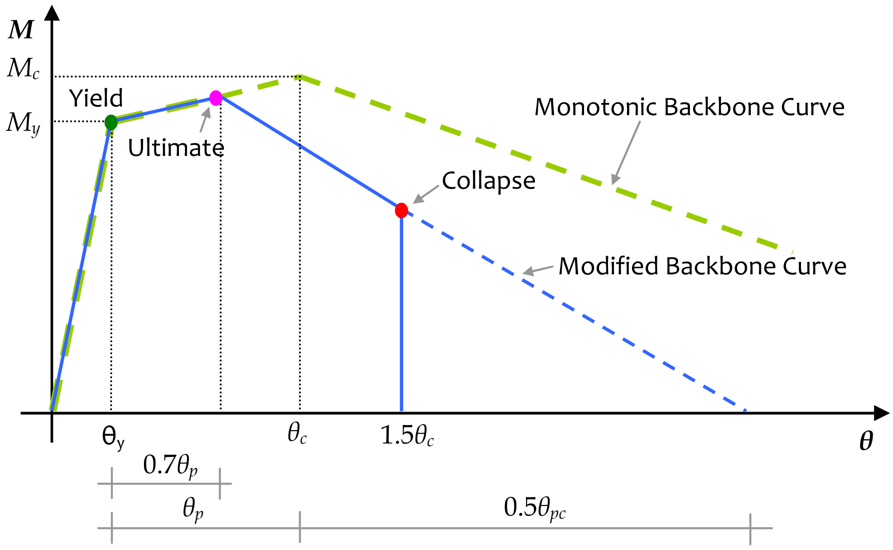


Figure 3.3: Backbone curve of beam plastic hinge [adapted from PEER-ATC, 2010]. Limit states considered in the study are marked too.

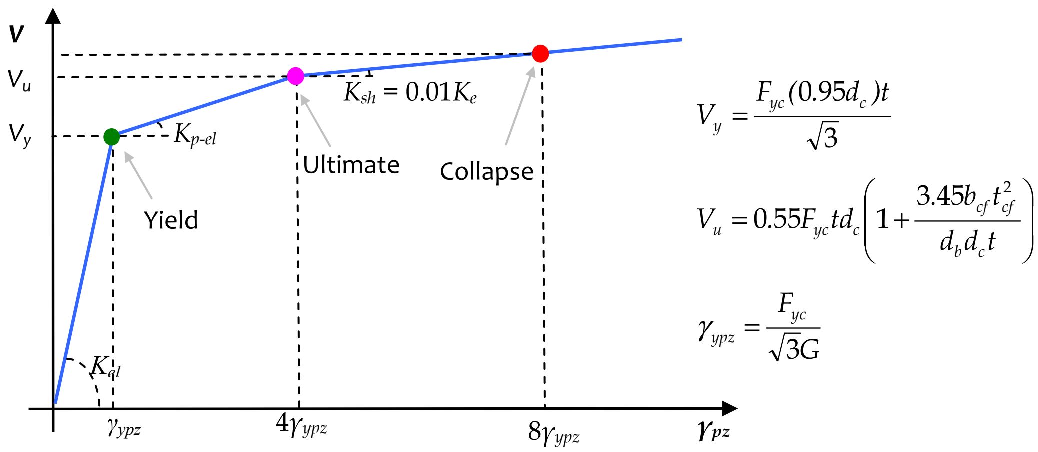


Figure 3.4: Shear-distortion response of panel zone [Krawinkler, 1978]. Limit states considered are marked too.

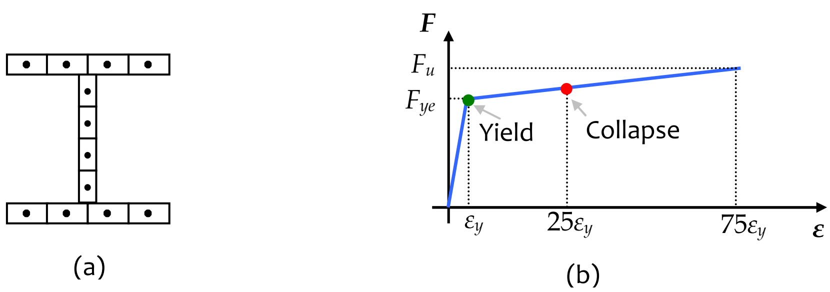


Figure 3.5: (a) Fibre hinge discretization of column sections, and (b) idealized stress-strain bilinear curve of steel for column fibres.

3.3 Nonlinear Analysis

Nonlinear static and dynamic analyses are employed to evaluate the seismic performance of the buildings. First mode based *Nonlinear Static Pushover Analysis* (*PoA*) was carried out till any of the deformation-controlled element reached its collapse limit state. *PoA* gave insight into the location of damage and the expected sequence of damage formation within the structural elements (beams, columns, and panel zones). Subsequently, response of the two study buildings is evaluated for thirty natural unidirectional earthquake ground motion accelerograms using *Nonlinear Dynamic Analysis*; the ground motions considered are listed in Table 3.3. These thirty ground motions are selected to have reasonable randomness in the basic ground motion characteristics, namely *PGA*, epicentral distance, significant duration of shaking, and frequency corresponding to peak Fourier amplitude. The statistical variation of the basic ground motion characteristics for the thirty accelerograms considered is given in Table 3.4, and the elastic acceleration response spectra are shown in Figure 3.6.

For dynamic analysis, every ground motion is scaled to match the spectral value of the 5% damped design spectrum at the fundamental natural period of each study building. IS 1893-Part 1 (2016) spectrum for medium or stiff soil (soil type II), scaled to the design hazard level (Z=0.4g), is considered as the design spectrum. Also, Rayleigh damping, with ratio of 4%, is considered at 0.2 and 0.9 times the fundamental period of the building models for analyses. Non-deteriorating multi-linear hysteretic model is adopted for the nonlinear hinge models. The usage of non-deteriorating model is possible, since the monotonic backbone curve of the beam hinges (which are supposed to be the main source of energy dissipation) are already modified to account for cyclic deterioration. Also, investigations in the past have demonstrated stable hysteretic behaviour of panel zones [Krawinkler, 1978]. Hence, a non-deteriorating model is suitable for panel zones too. Finally, since stocky W14 sections are used for columns, cyclic deterioration is expected to be limited at the interstorey drift ratios and axial force demands in the range of practical values during seismic shaking, again justifying use of a non-deteriorating model for columns [Newell and Uang, 2008]. Further, as in nonlinear static analyses, nonlinear dynamic analyses are continued until any of the deformation-controlled elements reached their *collapse* limit state. Finally, the nonlinear dynamic analysis carried out in the commercial software Perform 3D [CSI, 2016] uses the unconditionally stable Newmark's Constant Average Acceleration numerical scheme.

Table 3.3: Details of ground motions considered for nonlinear dynamic analysis in the current study

No	Event	Station	Year	M_w	PGA (g)	Epicentral Distance (km)
1	Kern County	Taft	1952	7.36	0.159	38.89
2	San Fernando	Palmdale Fire Station	1971	6.60	0.133	25.40
3		Lake Hughes			0.144	25.80
4	Tabas	Dayhook	1978	7.35	0.324	13.94
5	Imperial Valley	Plaster City	1979	6.50	0.042	31.70
6		Niland Fire Station			0.069	35.90
7		Delta			0.351	43.60
8		Coachella Canal#4			0.115	49.30
9	Park Field	Cholame 3W	1983	6.40	0.078	30.40
10		Gold Hill 3E			0.094	29.20
11		Fault Zone 3			0.139	36.40
12		Fault Zone 10			0.073	30.40
13	Superstition Hills	Imperial Wildlife	1987	6.30	0.207	24.70
14	Loma Prieta	Hollister-South Pine	1989	6.90	0.371	28.80
15		Red Wood City			0.273	47.90
16		Salinas-John and Work			0.091	32.60
17	Cape Mendocino	Eureka-Myrtle and West	1992	7.10	0.154	44.60
18		Fortuna-Blvd			0.116	23.60
19	Landers	Fire Station	1992	7.30	0.152	24.90
20		Palm Springs Airport			0.076	37.50
21		Desert Hot Spring			0.171	23.20
22	Northridge	Lake Hughes#1	1994	6.70	0.087	36.30
23		Downey-Co Maint Bldg			0.230	47.60
24		LA-116th Street School			0.133	41.90
25	Kobe	Nishi-Akashi	1995	6.90	0.483	7.08
26		Kakogawa			0.251	22.50
27		Morigawachi			0.214	24.80
28	Hector Mine	Hector	1999	7.13	0.265	11.66
29	Chi Chi	TCU 047	1999	7.62	0.298	35.00
30	Chamoli	Gopeshwar	1999	6.8	0.359	8.70

Table 3.4: Statistical variation of basic ground motion characteristics of the thirty ground motions considered in the current study

	Epicentral distance (km)	*PGA* (g)	*Significant duration* (s)	*Frequency corresponding to peak Fourier amplitude* (Hz)
Minimum	7.08	0.042	8.4	0.21
Maximum	49.30	0.483	50.33	2.64
Mean	30.48	0.188	19.72	1.28
CoV	36.9	58.8	51.8	52.9

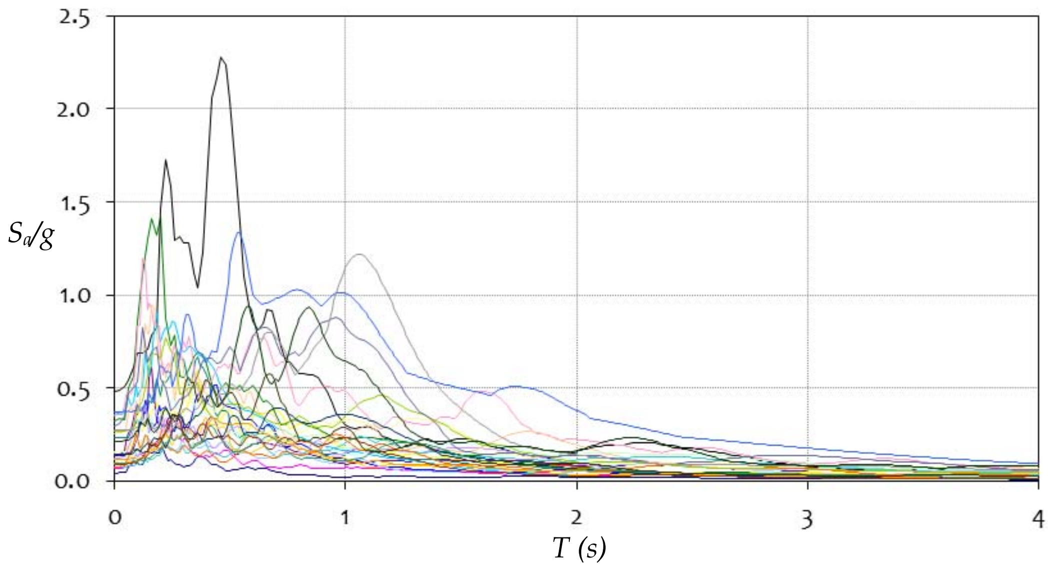


Figure 3.6: Elastic acceleration response spectra of the thirty ground motions considered in the study

3.3.1 Results of Nonlinear Analysis of Study Buildings

Results from the nonlinear static and dynamic analyses of the study buildings designed as per Indian code are presented in this section. Pushover curves have roof drift ratio and normalized base shear along the abscissa and ordinate, respectively. Roof drift ratio is the lateral displacement at the roof level normalized by the building height. And, normalized base shear is the base shear divided by the design seismic force; the maximum value of this quantity represents the overstrength of a building.

Statistical representation of data is required for meaningful interpretation of the results from nonlinear dynamic analysis. Median and counted 84th percentile values are used to represent seismic demands imposed on the three key deformation-controlled elements, namely beam plastic hinges, panel zones, and columns. To find the median and counted 84th percentile values, the seismic demand values corresponding to all thirty ground motions are sorted in order. Thereafter, average of 15th and 16th values is considered as the median value, while the average of 25th and 26th values is considered as the counted 84th percentile.

3.3.1.1 3-Storey Building

The first mode based normalized pushover curve of the 3-storey study building is shown in Figure 3.7; significant overstrength of about 3.9 is observed in the building. This overstrength is the combined result of (i) partial load factor, (ii) material partial safety factor (of 1.1), (iii) overstrength in beams due to increase in material yield stress from the minimum specified yield (characteristic) stress (by a factor of 1.5), (iv) redundancy, (v) strain hardening in beams (by about a factor of 1.1), and (vi) overstrength resulting from selection of higher member sizes during design. The yielding of the building initiated with yielding of column bases at lateral roof drift ratio of about 1.0%. Thereafter, panel zone yielding occurred at drift ratio of 1.2%. Finally, beam yielding is observed, but only at drift ratio of 1.9%. The delayed yielding of the beam is due to higher overstrength in beams compared to columns and panel zones. The overstrength in beams resulted from the non-consideration of the actual yield stress of the beam, while capacity-protected elements (panel zones and columns) were designed. It is found that the panel zones and columns are the main source of inelastic energy dissipation, which contradicts the seismic dissipation mechanism envisioned for SMFs, where beams are supposed to be the primary source of energy dissipation. This is further illustrated through the distribution of inelasticity observed in the building, as depicted in Figure 3.8. Figure 3.8(a) shows the distribution of inelasticity in the building at the *performance point* estimated by FEMA 440 linearization [FEMA, 2005] method; up to this point, only the panel zones and the column bases have yielded, while all the beams are still elastic. Similarly, the distribution of damage when a critical panel zone reached its collapse state is shown in Figure 3.8(b). It can be seen that only a single plastic hinge is formed in the beam at this stage. Further, the seismic demand is roof drift ratio of 1.6% estimated by FEMA 440 linearization method corresponding to the performance point at which the building is found to be safe against collapse, although the beams remained elastic while the panel zones and columns yielded. This concentration of inelasticity, primarily in panel zones and columns, clearly underscores the inadequacy of current Indian code provisions to guarantee acceptable seismic performance of steel SMFs.

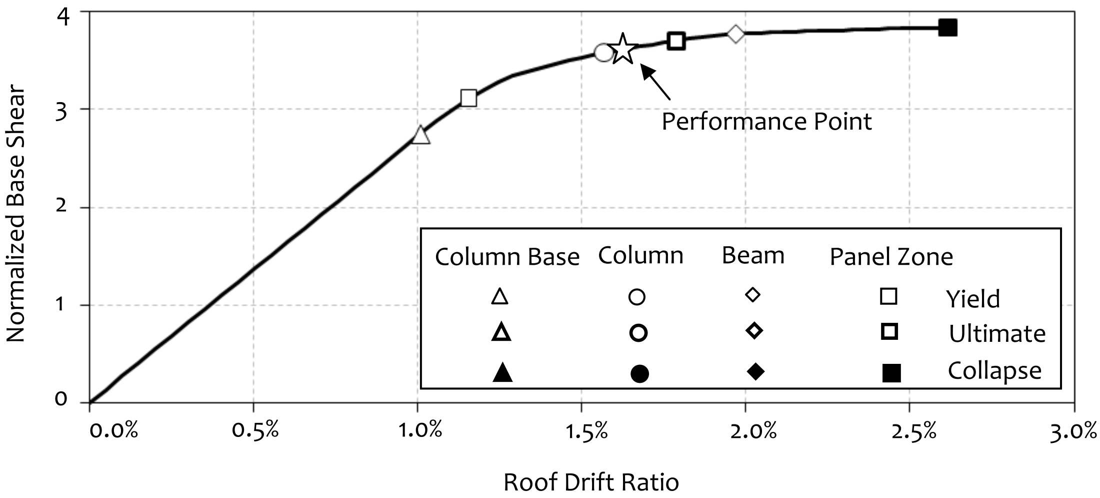


Figure 3.7: Pushover curve of the 3-storey study building designed as per Indian code; the onset of the key damage states is highlighted.

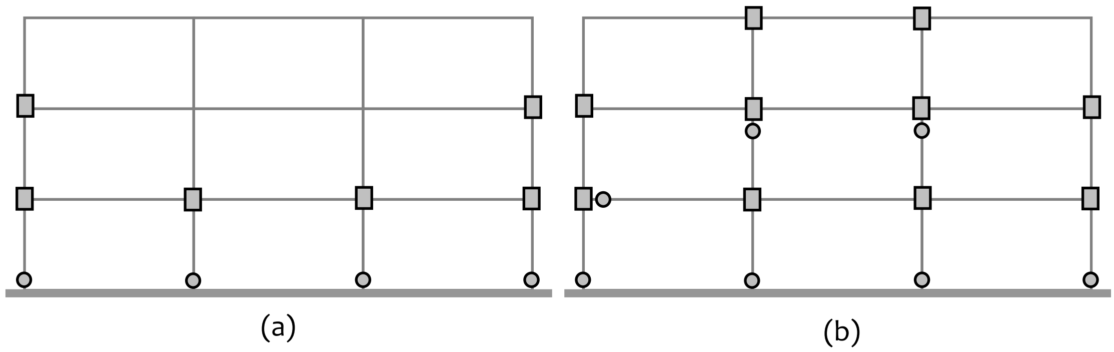


Figure 3.8: (a) Distribution of inelasticity at the performance point (roof drift ratio of 1.6%), and (b) distribution of inelasticity at collapse limit state (roof drift ratio of 2.6%), in the 3-storey SMF building designed as per Indian code. The damage distribution is based on first mode based nonlinear static pushover analysis; yielded components are shaded.

The results of nonlinear dynamic analyses of the 3-storey building, subjected to the thirty natural ground motion accelerograms, are presented in the following. The interstorey and total drift profiles of the 3-storey building are shown in Figure 3.9. It can be seen that the residual (Figure 3.9(b)) drift is relatively higher in the bottom storey due to formation of plastic hinges at the column bases..

The behaviour of interior panel zones and adjacent beams along the height of the 3-storey building is depicted in Figure 3.10. It can be seen from Figure 3.10(a) that the interior panel zones have yielded for more than half of the 30 ground motions, while the beams remained elastic (Figure 3.10(b)) even at Z=0.4g shaking, except for a single ground motion. Figure 3.10(c) shows the normal strain demand at the centre of column flange. As the building has fixed base, significant yielding can be observed in the column bases. A qualitative distribution of inelasticity in the building is depicted in

Figure 3.10(d). It can be seen that, almost all panel zones yielded along with column bases. It is interesting to note from Figure 3.10(c) and Figure 3.10(d) that, though the column bases yielded, there is no significant yielding in the columns, even with low *CBSR* (low *CBSR* resulted from the non-consideration of R_y factor and ignoring the reduction in moment capacity of columns in presence of axial loads, when designed according *CBSR* definition of IS 800 (2007)). This is because the panel zones sustained all the inelastic actions leaving the columns elastic. Thus, the results of nonlinear dynamic analyses confirms the observations drawn from nonlinear static analysis, *i.e.*, the provisions of the current Indian code for seismic design of steel SMFs can lead to panel zones becoming the primary source of inelastic energy dissipation, while beams may not participate in energy dissipation.

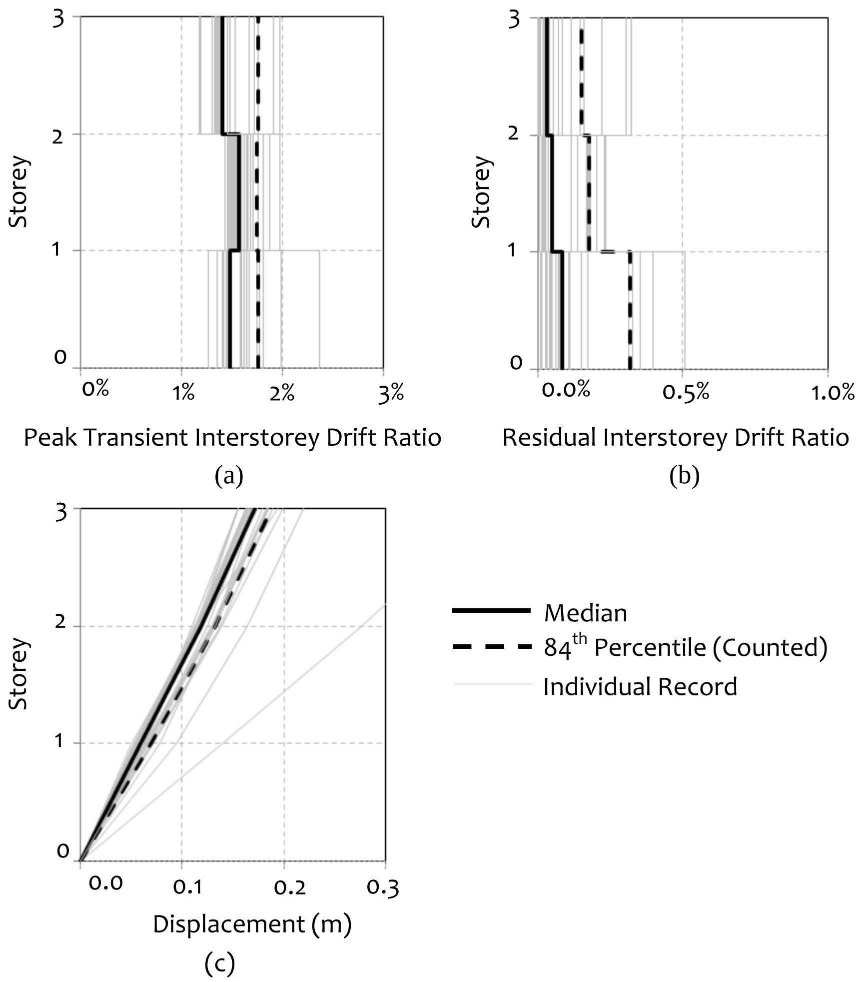


Figure 3.9: (a) Peak transient interstorey drift ratio, (b) residual interstorey drift ratio, and (c) displacement profile for the 3-storey building designed as per Indian code, as observed in the nonlinear dynamic analyses.

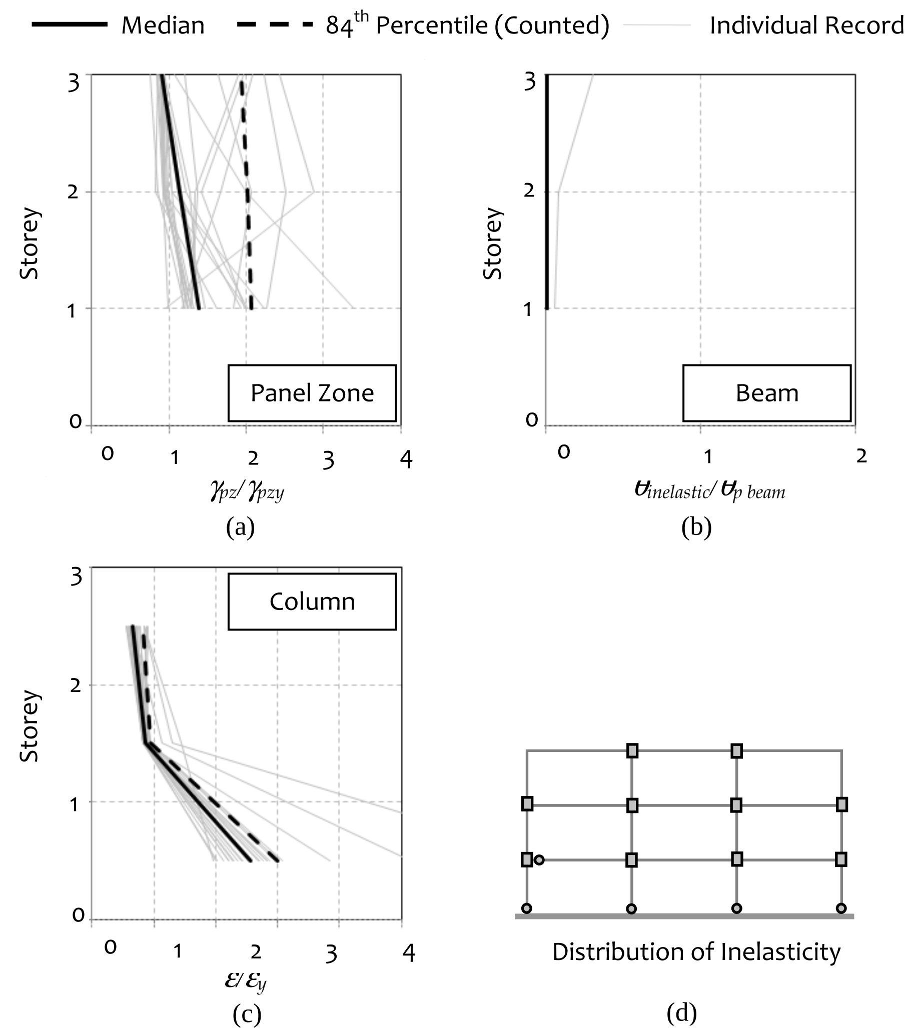


Figure 3.10: (a) Shear strain demand in interior panel zones, (b) plastic rotation demand in interior beams, and (c) normal strain demand in columns along height of the 3-storey building. The plastic rotation demands ($\theta_{inelastic}$) in beams are normalized with the modified pre capping plastic rotation capacity of the section (θ_p= $0.7\theta_{p\text{-}monotonic}$), while shear strain in panel zones and normal strain in columns are normalized with shear yield and normal yield strains (γ_{pzy} and ε_y) respectively, and (d) distribution of inelasticity under at least 15 ground motions; yielded components are shaded.

3.3.1.2 9-Storey Building

The first mode based normalized pushover curve of the 9-storey study building is shown in Figure 3.11 with overstrength of about 3.7 (which is slightly less than that of the 3-storey building). But unlike in the 3-storey building, inelasticity in the 9-storey building initiated with yielding of the panel zones at lateral roof drift ratio of about 0.9%. Thereafter, column base yielding followed at drift ratio of 1.1%. Finally, beam yielding is observed, but only at drift ratio of 1.4%. The delayed yielding of the beam is due to higher overstrength in beams compared to that in either the columns or the panel zones.

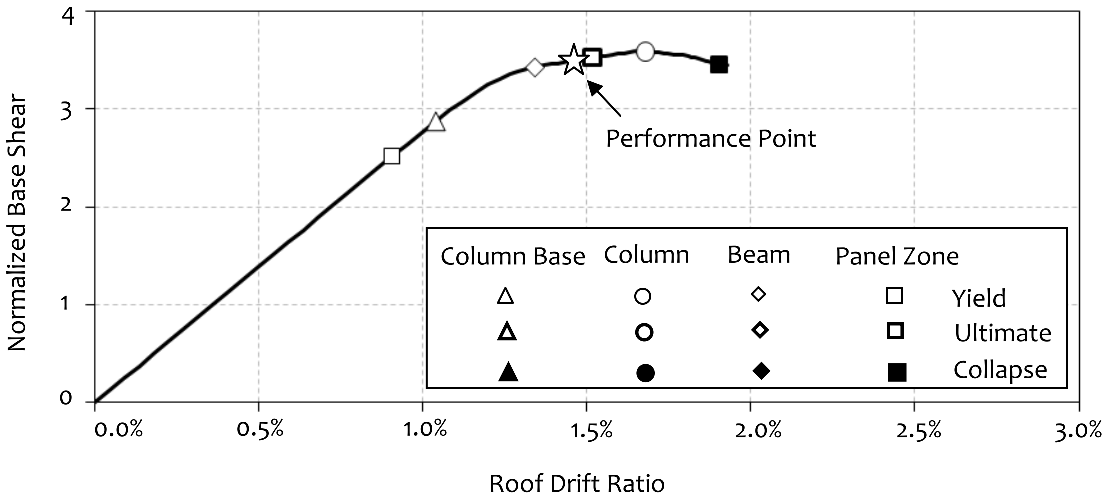


Figure 3.11: Pushover curve of the 9-storey study building designed as per Indian code; the onsets of the key damage states are highlighted.

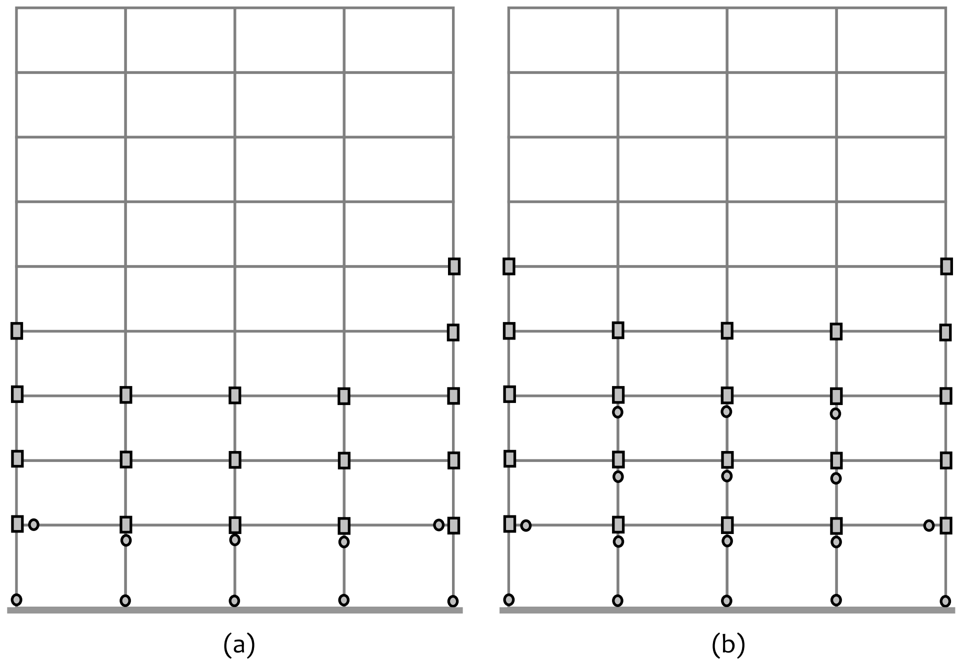


Figure 3.12: (a) Distribution of inelasticity at the performance point (roof drift ratio of 1.45%), and (b) distribution of inelasticity at collapse limit state (roof drift ratio of 1.9%), in the 9-storey SMF building designed as per Indian code. The damage distribution is based on first mode based nonlinear static pushover analysis; yielded components are shaded.

This distribution of inelasticity as observed in nonlinear static analysis is depicted in Figure 3.12. Figure 3.12(a) shows the distribution of inelasticity in the building at the performance point estimated by FEMA 440 linearization method, at which point, only the panel zones and the columns have yielded, while almost all the beams are still elastic. Similarly, the distribution of damage when a critical panel zone reached its collapse limit state is shown in Figure 3.12(b). It can be seen that only two plastic hinges are formed in the beams. Further, the seismic demand is drift ratio of 1.45% estimated by FEMA 440 linearization method corresponding to the performance point at which the building is found to be safe against collapse, although the beams remained nearly elastic while the panel zones and columns yielded. This concentration of inelasticity, primarily in panel zones, clearly highlights the inadequacy of current Indian code provisions to guarantee acceptable seismic performance of steel SMFs.

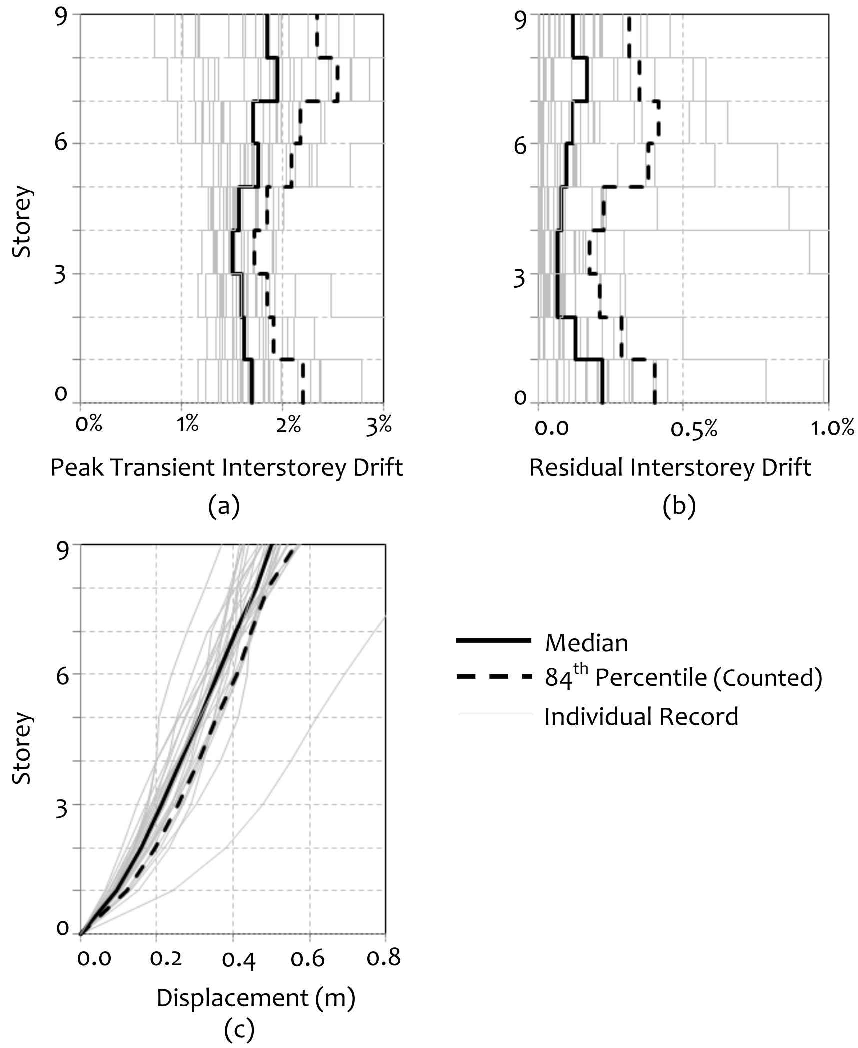


Figure 3.13: (a) Peak transient interstorey drift ratio, (b) Residual interstorey drift ratio, and (c) Displacement profile for the 9-storey building designed compliant with Indian code as observed in the nonlinear dynamic analysis.

The interstorey and total drift profiles of the 9-storey building, obtained from nonlinear dynamic analyses subjected to the thirty natural ground motion accelerograms, are shown in Figure 3.13. Again, as in the 3-storey building, the peak transient (Figure 3.13(a)) and residual (Figure 3.13(b)) drift profiles are relatively more in the bottom storey due to formation of plastic hinges at the column bases. Further, the abrupt amplification of the interstorey drift in the top storeys is due to the influence of higher vibration modes on seismic response of the building.

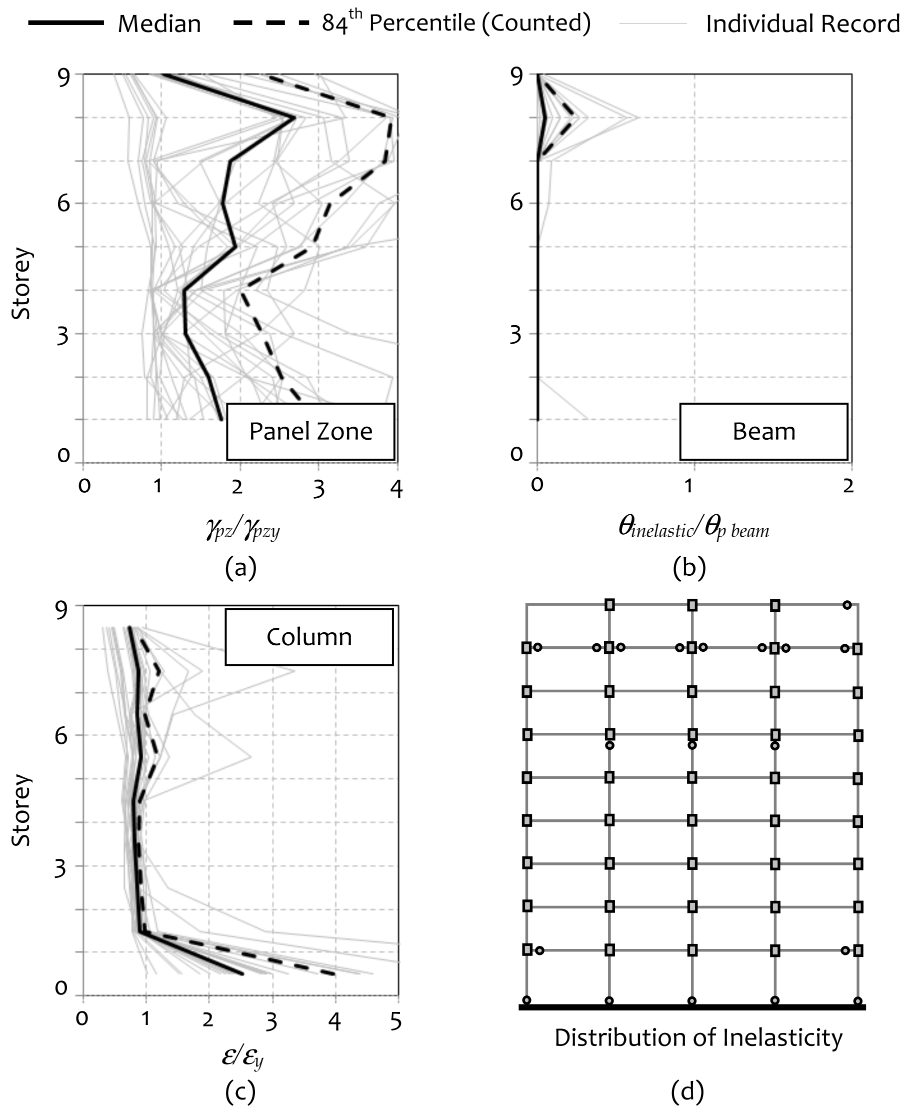


Figure 3.14: (a) Shear strain demand in interior panel zones, (b) plastic rotation demand in interior beams, and (c) normal strain demand in columns along height of the 9-storey building. The plastic rotation demands ($\theta_{inelastic}$) in beams are normalized with the modified pre capping plastic rotation capacity of the section ($\theta_p = 0.7\theta_{p\text{-}monotonic}$), while shear strain in panel zones and normal strain in columns are normalized with shear yield and normal yield strains (γ_{pzy} and ε_y) respectively, and (d) distribution of inelasticity under at least 15 ground motions; yielded components are shaded

The behaviour of interior panel zones and adjacent beams along the height of the 9-storey building is depicted in Figure 3.14. It can be seen from Figure 3.14 (a) that the interior panel zones have yielded significantly (γ_{pz} is about $2\gamma_{ypz}$). Beam response shown in Figure 3.14(b) indicates that the beams are elastic even at Z=0.4g shaking intensity, except in the eighth storey. The beams yielded in the eight storey due to combination of higher mode effect and relatively higher *CBSR* ratio provided in the eighth storey due to the specific choice of the column section. Figure 3.14(c) shows the normal strain demand at the centre of column flange. As the building has fixed base, significant yielding can be observed in the column bases. Limited yielding of columns is also observed in the top stories owing to higher mode response.

Additionally, it can be seen, from the qualitative distribution of inelasticity in the building shown in Figure 3.14(d) that, almost all panel zones yielded along with column bases. It is interesting to note from Figure 3.14(c) and Figure 3.14(d) that, though the column bases yield, there is no significant yielding in the columns, even with low *CBSR*. This is because the panel zones sustained all the inelastic actions leaving the columns elastic. Thus, the concentration of inelasticity primarily to the panel zones in both 3- and 9-storey buildings clearly highlights the inadequacy of current Indian code provisions for the design of steel SMFs as the panel zones become the primary source of inelastic energy dissipation while beams may not participate at all. The results also highlight another important issue - the distribution of inelasticity observed in the 9-storey building from nonlinear static and dynamic analysis are significantly different; nonlinear static analysis shows elastic behaviour in top storeys, which is misleading. This is because the first mode based nonlinear static analysis does not capture the effects of higher modes of vibration, which a dynamic analysis can.

3.4. Redesigned Buildings

The 3- and 9-storey study buildings designed as per the current Indian code demonstrated undesirable earthquake behaviour. The major drawbacks which resulted in the undesirable behaviour are, (i) neglecting material overstrength, R_y factor in the design of panel zones, and (ii) using an effectively low value of *CBSR* in the design of columns which again resulted from not considering R_y factor. Hence, redesigns of the same 3- and 9-storey study buildings are carried out where the above mentioned drawbacks are rectified. The geometry, loading, and design strategies are maintained to be same as in the previous study buildings, except for the fact that, R_y of 1.5 (corresponding to ASTM A36 grade steel) is used in the design of capacity-protected elements (*i.e.*, panel zones and columns). The details of the new design of the 3- and 9-storey buildings are given in Tables 3.5 and 3.6, respectively.

Table 3.5: Details of the redesigned SMF of the 3-storey study building

3 Storey SMF	Beam	Column	*CBSR*i (Interior Joint)	Doubler Plate Thickness (mm)	
Storey				**Interior Joint**	**Exterior Joint**
3	W24 × 76	W 14 × 342	2.30	10	0
2	W27 × 114	W 14 × 342	2.70	34	0
1	W27 × 114	W 14 × 342	2.70	34	0
Building Natural Period: **0.98 seconds** Design Base Shear: **1447 kN** *Note: CBSR is as per Eq.(3.1)*					

Table 3.6: Details of the redesigned SMF of the 9-storey study building

9 Storey SMF	Beam	Column	*CBSR*i (Interior Joint)	Doubler Plate Thickness (mm)	
Storey				**Interior Joint**	**Exterior Joint**
9	W21 × 83	W14 × 311	2.12	21	0
8	W21 × 83	W14 × 311	4.25	21	0
7	W24 × 117	W14 × 311	2.54	46	14
6	W24 × 117	W14 × 311	2.54	46	14
5	W24 × 117	W14 × 398	3.38	31	1
4	W30 × 132	W14 × 398	2.53	34	5
3	W30 × 132	W14 × 398	2.53	34	5
2	W30 × 132	W14 × 398	2.53	34	5
1	W30 × 148	W14 × 455	2.58	39	4
Building Natural Period: **2.61 seconds** Design Base Shear: **1878 kN** *Note: CBSR is as per Eq.(3.1)*					

3.4.1 Results of Nonlinear Analysis of Redesigned Buildings

Results from the nonlinear static and dynamic analyses of the redesigned study buildings are presented in this section.

3.4.1.1 3-Storey Building

The first mode based pushover curve of the redesigned 3-storey building is shown in Figure 3.15. It can be observed that, it is the beam yielding which controls the frame response. Even though the panel zones yield, they do not reach their ultimate limit state. Thus the redesigned buildings exhibit balanced panel zone behaviour where limited yielding of panel zone occurs. Further, the deformation capacity of the building is also significantly enhanced. A marginal increase in the overstrength capacity of the building is also observed owing to increase in strength of the capacity protected elements (panel zones and columns). Finally, at the collapse limit state (at a roof drift ratio of 5.0% which is not shown in figure), it is beam which reach its collapse state.

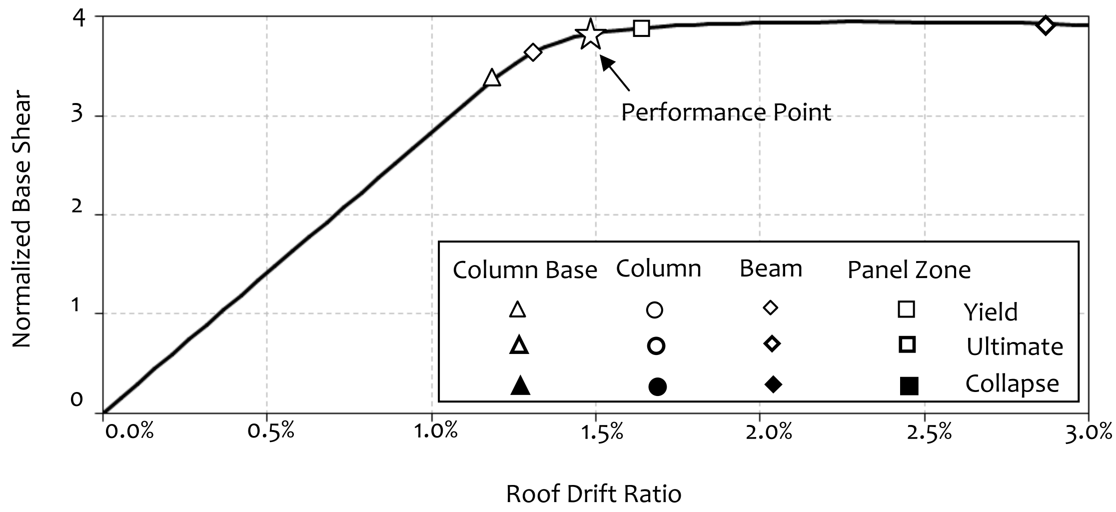


Figure 3.15: Pushover curve of the redesigned 3-storey study building; the onsets of the key damage states are highlighted.

The behaviour of interior panel zones and adjacent beams along the height of the redesigned 3-storey building, as observed from nonlinear dynamic analyses subjected to the thirty natural ground motion accelerograms, is depicted in Figure 3.16. It can be seen from Figure 3.16(a) that the panel zones respond elastically except for a single ground motion. In contrast, beam response shown in Figure 3.16(b) indicates that the beams are now the primary locations of inelasticity in the building. Figure 3.16(c) shows the normal strain demand at the centre of column flange. As the building has fixed base, significant yielding is still observed at the column bases. However, the columns do behave elastically during seismic shaking, thereby forcing the inelastic actions in the beams. Comparing Figure 3.10 and Figure 3.16, it can be clearly observed that the yielding of panel zone is significant in the former while panel zones do not yield in the latter. Moreover, the beams which did not participate in energy dissipation in the 3-storey study building designed compliant to current Indian code, now contribute to energy dissipation in the redesigned building.

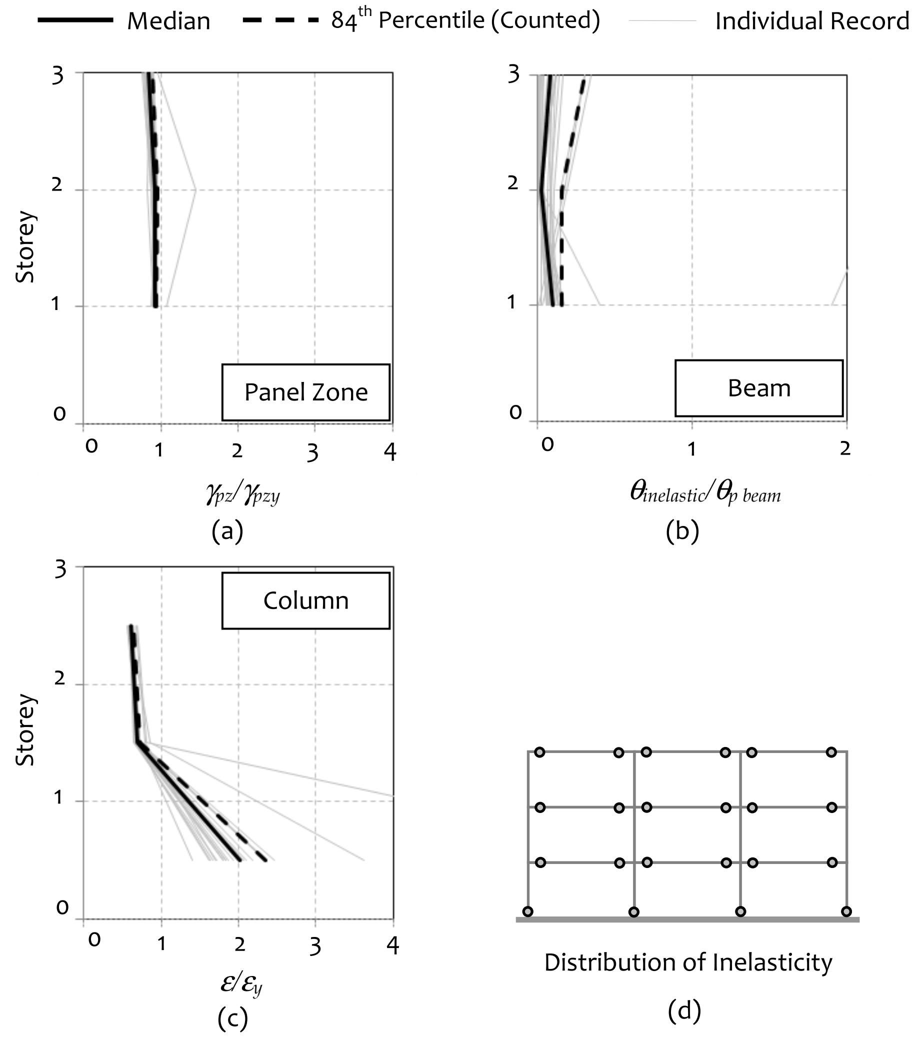


Figure 3.16: (a) Shear strain demand in interior panel zones, (b) plastic rotation demand in interior beams, and (c) normal strain demand in columns along height of the redesigned 3-storey building The plastic rotation demands ($\theta_{inelastic}$) in beams are normalized with the modified plastic rotation capacity of the section θ_p= $0.7\theta_{p\text{-}monotonic}$),), while shear strain in panel zones and normal strain in columns are normalized with shear yield and normal yield strains (γ_{pzy} and ε_y), respectively, and (d) distribution of inelasticity under at least 15 ground motions; yielded components are shaded

3.4.1.2 9-Storey Building

The first mode based pushover curve of the redesigned 9-storey building is shown in Figure 3.17. Similar to the redesigned 3-storey buildings, it is the beam yielding which controls the frame response in case of 9-storey buildings too. Limited panel zone yielding is also observed. Despite this, beams remain as the primary source of inelastic energy dissipation, which is in line with the dissipation mechanism envisioned for steel SMFs.

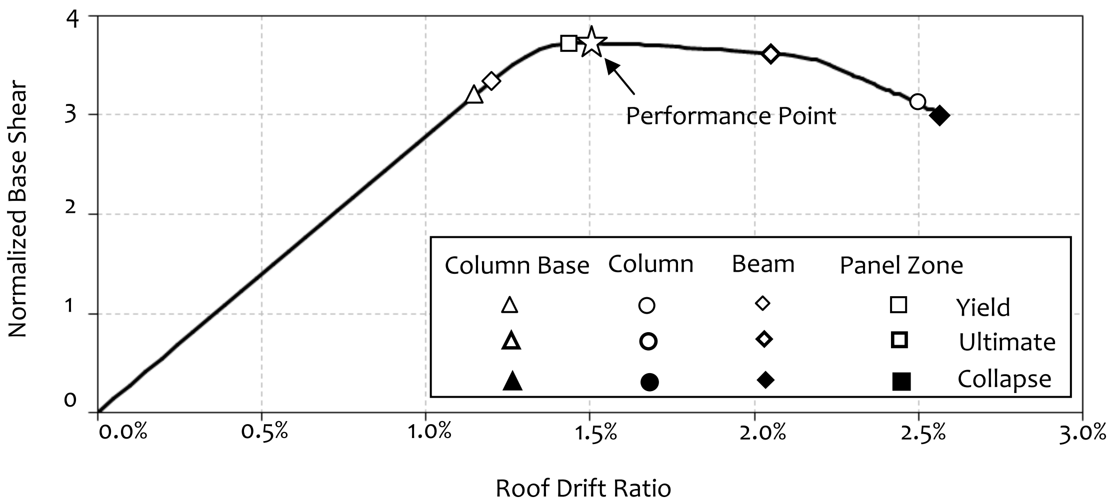


Figure 3.17: Pushover curve of the redesigned 9-storey study building; the onsets of the key damage states are highlighted.

The behaviour of interior panel zones and adjacent beams along the height of the 9-storey redesigned building is depicted in Figure 3.18. It can be seen from Figure 3.18(a) that the yielding of the panel zones are limited. Beam response shown in Figure 3.18(b) indicates significant yielding and energy dissipation in the beams. Also, significant yielding is observed at the column bases as the building has fixed base (Figure 3.18(c)). However, the columns do behave elastically during seismic shaking, thereby forcing the inelastic actions to the beams. Comparing Figure 3.14 and Figure 3.18, it can be clearly observed that the yielding of panel zone is significant in the former while limited yielding of panel zone is observed in the latter. Moreover, the beams which did not participate in energy dissipation in 9-storey study building designed compliant to current Indian code now contribute to energy dissipation in the redesigned building. Hence it can be concluded that the redesign leads to acceptable earthquake behaviour of steel SMFs wherein the beams become the major source of energy dissipation with minor yielding in the panel zones.

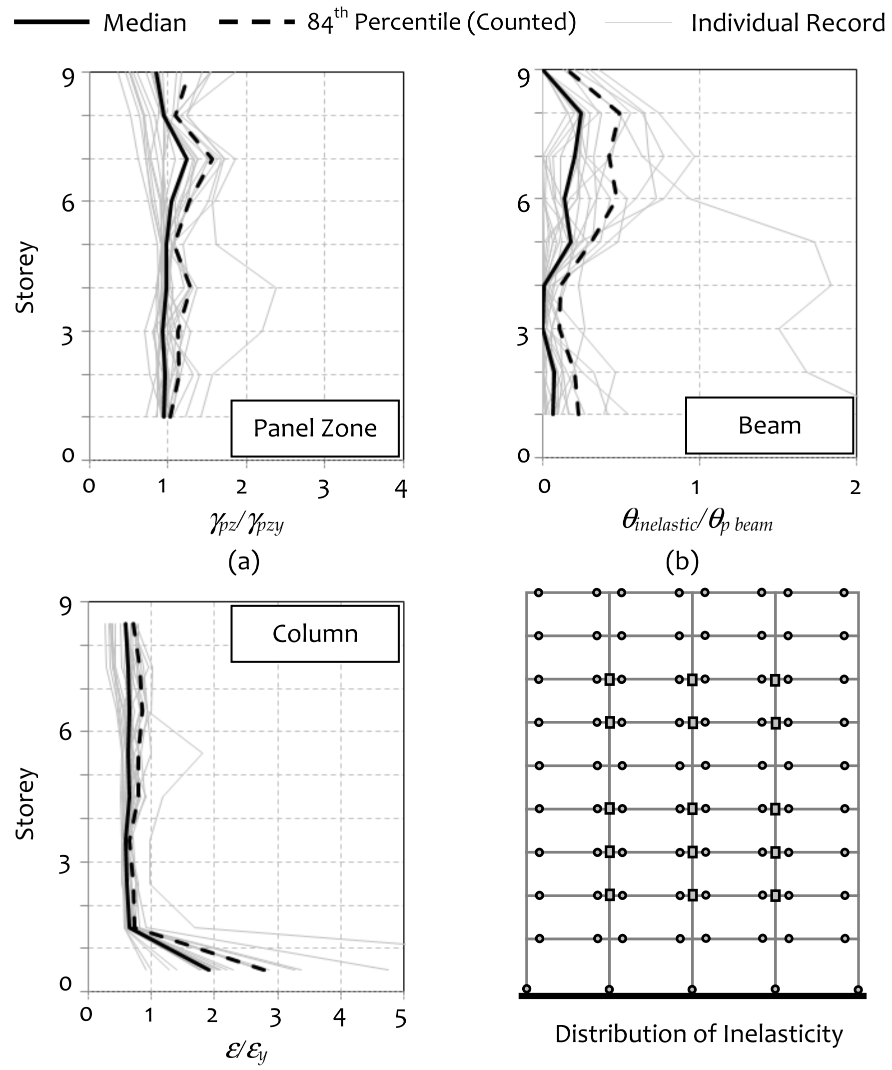


Figure 3.18: (a) Shear strain demand in interior panel zones, (b) plastic rotation demand in interior beams, and (c) normal strain demand in columns along height of the redesigned 9-storey building The plastic rotation demands ($\theta_{inelastic}$) in beams are normalized with the modified plastic rotation capacity of the section (θ_p= $0.7\theta_{p\text{-}monotonic}$), while shear strain in panel zones and normal strain in columns are normalized with shear yield and normal yield strains (γ_{pzy} and ε_y), respectively, and (d) distribution of inelasticity under at least 15 ground motions; yielded components are shaded

3.5. Summary of Maximum Inelastic Demands Imposed on Study Buildings

The inelastic demands imposed on the study buildings, as observed in nonlinear dynamic analyses, are summarized in Table 7. From the table it can be seen that, less than half the plastic rotation capacity of the beams are used. Typically the beams allowed to be used in SMFs, *i.e.* plastic sectionss have a pre-capping plastic rotation capacity (considering cyclic degradation) of 0.015-0.035 radian (which translates to total rotation (*i.e.*, yield rotation plus plastic rotation) of 0.025-0.050 radian). Hence, using plastic sections as required by design codes will guarantee adequate rotation capacity to resist an earthquake with shaking intensity of Z=0.4g. Further, proper capacity protection of capacity protected elements (panel zones and columns) as done in the redesigned buildings will limit the rotation demands imposed on those elements, well within their allowable deformation limits.

Table 7: Maximum inelastic demands as observed in the nonlinear dynamic analyses under design earthquake at Z=0.4g.

Maximum Inelastic Demands Observed under Design Earthquake		Beam Plastic Rotation (θ_p)	Panel Zone Yield Rotation (γ_{pzy})	Column Yield Strain (ε_y)	Column Base Yield Strain (ε_y)
3-Storey	Median	$0.00\theta_p^{\#}$ (0.0000 rad)	$1.35\gamma_{pzy}$ (0.0038 rad)	$0.85\varepsilon_y$ (0.0016)	$2.50\varepsilon_y$ (0.0047)
	84 Percentile	$0.00\theta_p^{\#}$ (0.0000 rad)	$2.10\gamma_{pzy}$ (0.0060 rad)	$0.90\varepsilon_y$ (0.0017)	$3.00\varepsilon_y$ (0.0057)
9-Storey	Median	$0.05\theta_p^{*}$ (0.0012 rad)	$2.70\gamma_{pzy}$ (0.0077 rad)	$0.95\varepsilon_y$ (0.0018)	$2.50\varepsilon_y$ (0.0047)
	84 Percentile	$0.25\theta_p^{*}$ (0.0061 rad)	$3.90\gamma_{pzy}$ (0.0111 rad)	$1.15\varepsilon_y$ (0.0022)	$4.00\varepsilon_y$ (0.0076)
3-Storey Redesigned	Median	$0.15\theta_p^{\#\#}$ (0.0036 rad)	$0.88\gamma_{pzy}$ (0.0025 rad)	$0.70\varepsilon_y$ (0.0013)	$2.00\varepsilon_y$ (0.0038)
	84 Percentile	$0.32\theta_p^{\#\#}$ (0.0074 rad)	$0.92\gamma_{pzy}$ (0.0026 rad)	$0.72\varepsilon_y$ (0.0014)	$2.30\varepsilon_y$ (0.0044)
9-Storey Redesigned	Median	$0.20\theta_p^{**}$ (0.0063 rad)	$1.20\gamma_{pzy}$ (0.0034 rad)	$0.72\varepsilon_y$ (0.0014)	$1.90\varepsilon_y$ (0.0036)
	84 Percentile	$0.45\theta_p^{**}$ (0.0142 rad)	$1.60\gamma_{pzy}$ (0.0046 rad)	$0.85\varepsilon_y$ (0.0016)	$2.70\varepsilon_y$ (0.0051)

θ_p : Plastic rotation capacity (considering cyclic degradation) of the section
$\theta_p^{\#}$: Plastic rotation capacity (considering cyclic degradation) of top storey beam of 3-srorey building
θ_p^{*} : Plastic rotation capacity (considering cyclic degradation) of eighth storey beam of 9-srorey building
$\theta_p^{\#\#}$: Plastic rotation capacity (considering cyclic degradation) of top storey beam of 3-srorey building
θ_p^{**} : Plastic rotation capacity (considering cyclic degradation) of eighth storey beam of 3-srorey building
γ_{pzy} : Yield distortion capacity of panel zone
ε_y : Yield strain of column
($\theta_p^{\#}$, θ_p^{*}, $\theta_p^{\#\#}$, θ_p^{**}, γ_{pzy}, and ε_y all can be calculated as per the hinge definitions provided in Section 2.8.1)

3.6 Conclusions from Performance Evaluation of Study Buildings

Performance assessment of 3- and 9-storey SMF buildings designed using the current Indian code indicates undesirable behaviour such as: (a) significant yielding of capacity-protected elements (*i.e.*, panel zones and columns), and (b) limited or no yielding of designated yielding elements (*i.e.*, beams). Two key factors that result in the undesirable behaviour of said buildings are: (a) underestimation of demand in capacity-protected elements, and (b) overestimation of capacity of capacity-protected elements. Therefore, mitigating this undesirable behaviour requires (a) estimating the demand in capacity-protected elements considering the expected material yield stress of the material rather than the characteristic yield stress, (b) providing closed form expressions to determine panel zone demand and capacity, (c) estimating the capacity of columns considering the possible reduction in their moment capacity due to presence of axial loads and recommending a consistent minimum *CBSR* ratio requirement. While addressing the last two issues is fairly straightforward, a systematic exercise is required, to assess the magnitude of uncertainty in yield strength of the steels being manufactured in the country, to address the first. A rigorous statistical analysis of actual data is required to ascertain meaningful values of the R_y factor, and then, incorporation of the same in the design code is essential to guarantee acceptable seismic performance of steel special moment frame buildings.

...

Chapter 4

Seismic Design Approaches for Panel Zone

4.0 Introduction

Force based seismic design of steel *Special Moment Frames* (SMFs), which is currently adopted by seismic design codes across the world, relies on *capacity design* procedures to confine the damage to pre-determined fuse elements. In SMFs, once the connections are designed to remain elastic during seismic activity, the two main tools which the seismic code provisions use to control the distribution of damage to different structural elements (*i.e.*, beams, columns and panel zones) are the panel zone design approach and the minimum column-to-beam strength ratio (*CBSR*) requirement. In Chapter 3, it was seen that the current provisions of IS 800 [IS 800, 2007) are unable to guarantee acceptable seismic performance of steel SMFs. Hence, in the current chapter, detailed investigations are carried out to understand seismic design and behaviour of panel zones in steel SMFs. To begin with, the limitations of estimating panel zone shear demand from beam-column sub-assemblage are explained. Further, three panel zone design approaches are investigated, which are intended to result in *weak*, *balanced*, and *strong* panel zone designs, respectively. The different panel zone design approaches are then adopted in design of study buildings to ascertain their seismic behaviour. Thereafter, the *pros* and *cons* of the three proposed panel zone design approaches are discussed.

4.1 Panel Zone Shear Demand

Panel zone shear demand (V_{pzd}) in steel perimeter SMFs is generally estimated using Eq.(4.1) whose derivation was discussed previously in Section 2.4.2.

$$V_{pzd} = \sum_{i=1}^{n}\left[\left(\frac{M_{cfi}}{d_{bi}-t_{bfi}}\right)-\left(\frac{M_{pri}}{L_i-d_{ci}-2L_{hi}}\times\frac{2L_i}{h_1+h_2}\right)\right] \tag{4.1}$$

The first term on the right hand side of Eq.(4.1) represents the shear demand resulting from the formation of plastic hinges in beams, and the second term represents the shear in the column. In Eq.(4.1), it is assumed that the points of inflection in columns and beams lie at the centre of the span of the respective members. However, during nonlinear behaviour, the inflection points in beams and in particular in columns do shift.

Columns which deform in double curvature during elastic analysis may even switch over to bending in single curvature, owing to the influence of higher vibration modes and formation of plastic hinges during nonlinear response. Even, under elastic response, the inflection points rarely lie at mid-span. Thus, the actual shear in column can be higher or lower than the column shear demand estimated based on the second term on the right hand side of Eq.(4.1). When the actual column shear is lesser than the estimated column shear as per Eq.(4.1), usage of Eq.(4.1) leads to unconservative estmate of panel zone shear demand (V_{pzd}). It is very much probable that, during nonlinear analysis, such a situation may arise as illustrated in Figure 4.1.

4.2 Panel Zone Shear Capacity

As discussed previously in Section 2.4.1, the actual panel zone shear capacity and shear distortion at the instant of column flange kinking (corresponding to ultimate capacity of panel zone) vary with the dimensions of panel zone and axial loads present. Despite this, the tri-linear response curve shown in Figure 4.2 represents the panel zone behaviour reasonably well in the practical range of interest. Hence, the tri-linear response as shown in Figure 4.2 is adopted, where the yield capacity (V_y) of panel zone is,

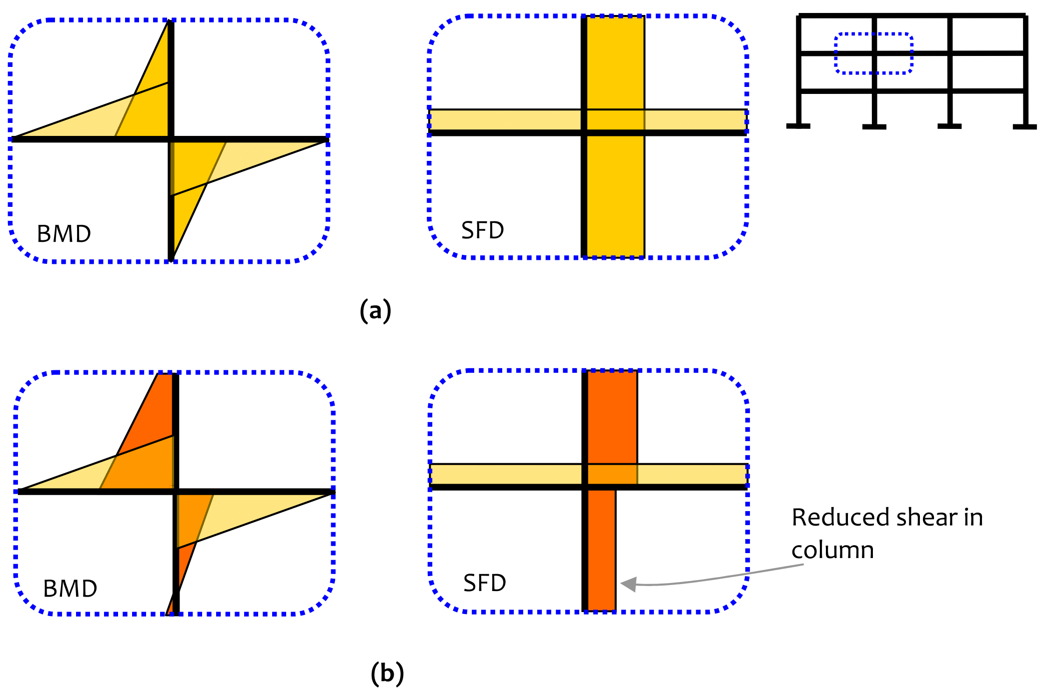


Figure 4.1: Bending moment and shear force profile near interior joint of moment frames. (a) An interior beam column sub-assemblage with inflection points at the mid-span of beams and columns. (b) A possible moment and shear profile in an interior location during seismic response.

$$V_y = \frac{F_y (0.95 d_c) t}{\sqrt{3}} \quad , \tag{4.2}$$

where, F_y is the yield stress, d_c the depth of column, and t the thickness of panel zone. The corresponding yield shear distortion (γ_{ypz}) is,

$$\gamma_{ypz} = \frac{F_y}{\sqrt{3G}} \ , \tag{4.3}$$

where, G is the shear modulus of elasticity. The ultimate shear capacity (V_u) of the panel zone is,

$$V_u = 0.55 f_y d_c t \left(1 + \frac{3.45 b_c t_{cf}^2}{d_b d_c t_{pz}}\right), \tag{4.4}$$

where, b_c is the breadth of the column, t_{cf} the thickness of column flange, and d_b the depth of the beam. The shear distortion corresponding to V_u is four times the yield strain, γ_{ypz}.. The post elastic stiffness, $K_{p\text{-}el}$ is given by

$$K_{p-el} = \frac{1.09 G b_c t_{cf}^2}{d_b} \ . \tag{4.5}$$

The final strain hardening stiffness, K_{sh}, of panel zone is reported to be around 0.01-0.02 times K_{el}; a value of $0.01K_{el}$ is considered for K_{sh} in the current study. Even though panel zones can sustain large shear distortion (as high as 0.08 radians), the panel zone distortion need to be limited to avoid column kinking and associated fracture in welds or heat affected zones [PEER-ATC, 2010]. Consequently, PEER-ATC recommends shear distortion to be limited to 0.02 radians ($\sim 8\gamma_{ypz}$) for panel zones. Hence a distortion corresponding to $8\gamma_{yp}$ is adopted in this study as the shear distortion at *collapse*.

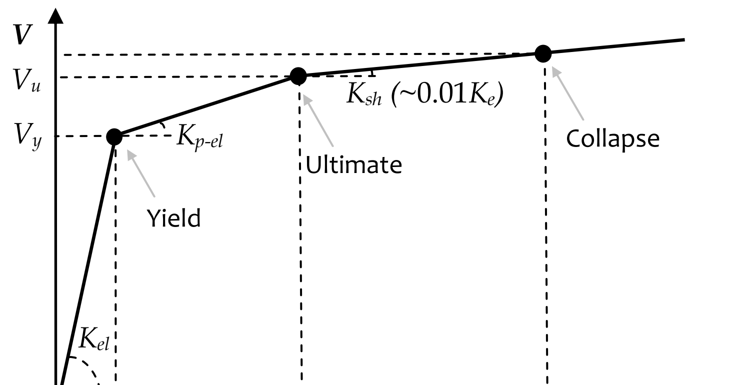


Figure 4.2: Shear-distortion response of panel zone adopted in the current study [Krawinkler, 1978]. Limit states considered are also marked in the figure.

4.3 Panel Zone Seismic Design Approaches

The shear force demand in panel zone varies throughout the nonlinear response of building during seismic activity. In particular, the estimate of panel zone shear demand using Eq.(4.1) can be unconservative. Hence, to design a panel zone which will remain elastic during seismic shaking, an upper bound estimate of the panel zone shear demand is required. Further, some connection configurations can force limited amount of inelasticity in panel zones, i.e., a balanced panel zone behaviour can be obtained. To design a balanced panel zone, a suitable combination of panel zone demand and capacity needs to be used. Further, there is a need to study the performance of panel zones designed by the current seismic provisions, which based on results of Chapter 3 indicates weak panel zone behaviour. Hence, three different panel zone design approaches are investigated in the following sections which are intended to result in weak, balanced, and strong panel zones.

The three panel zone design approaches hereafter are referred as PZ-I, PZ-II, and PZ-III. The PZ-I design approach is similar to the present AISC seismic provision [AISC 341, 2016]. Even though, the provision aims to achieve a balanced panel zone, as the post-yield strength is used to estimate the shear capacity of panel zone, PZ-I approach is bound to results in large panel zone shear distortions. Further, the panel zone shear demand is taken by reducing the shear in the column, which can be unconservative. PZ-II design approach is similar to the PZ-I approach except that the capacity of the panel zone is estimated without considering the post-yield strength of panel zone which comes from the contribution of column flanges. Hence PZ-II approach is intended to result in a balanced panel zone, which will yield less than the PZ-I approach. PZ-III approach is similar to PZ-II approach except that the panel zone demand is estimated without reducing the shear force coming from the column. Thus, the PZ-III approach is intended to result in a strong panel zone, where the panel zone is supposed to remain elastic during seismic activity. The shear demand and capacity estimates of the three panel zone design approaches are as listed in Table 4.1.

Table 4.1: Demand and capacity of the three panel zone design approaches considered in the current study.

Panel Zone Design Approach	Shear Demand	Shear Capacity
PZ-I	$\sum_{i=1}^{n}\left[\left(\frac{M_{cfi}}{d_{bi}-t_{bfi}}\right)-\left(\frac{M_{pri}}{L_i-d_{ci}-2L_{hi}}\times\frac{L_i}{h}\right)\right]$	$0.6F_y d_c t\left(1+\frac{3b_{cf}t_{cf}^2}{d_c t_{pz} d_b}\right)$
PZ-II	$\sum_{i=1}^{n}\left[\left(\frac{M_{cfi}}{d_{bi}-t_{bfi}}\right)-\left(\frac{M_{pri}}{L_i-d_{ci}-2L_{hi}}\times\frac{L_i}{h}\right)\right]$	$\frac{F_y(0.95d_c)t}{\sqrt{3}}$
PZ-III	$\sum_{i=1}^{n}\left[\frac{M_{cfi}}{d_{bi}-t_{bfi}}\right]$	$\frac{F_y(0.95d_c)t}{\sqrt{3}}$

M_{pr} : Probable strain hardened plastic moment capacity of the beam ($1.1R_yZ_{pb}F_{yb}$)
M_{cf} : Moment at the column face
d_b : Depth of the beam
d_c : Depth of the column
L : Centreline beam span
h : Distance between centres of storey above and below the joint
F_y : Yield Stress
t : Thickness of panel zone
t_{bf} : Thickness of beam flange
t_{cf} : Thickness of column flange

4.4 Methodology for Panel Zone Response Evaluation

The following methodology is adopted to evaluate the response of panel zones designed according to the three design approaches, namely, PZ-I, PZ-II, and PZ-III.

1. 3- and 9-storey study buildings are designed such that the minimum *CBSR* requirement according to all the codal provisions is satisfied with a margin, which is sufficient enough to limit the yielding in columns up to a shaking intensity corresponding to Z=0.6g. The study buildings are designed for three seismic hazard levels (Z=0.2g, 0.4g, and Z=0.6g).
2. The panel zones in each of the buildings are varied according to the three proposed panel zone design approaches, *i.e.*, PZ-I, PZ-II, and PZ-III. Thus a total of 18 study buildings are designed and evaluated in this section.
3. To begin with, first mode based nonlinear static pushover analysis (PoA) is performed on each study building to understand the sequence of damage initiation within its structural components.

4. Thereafter, nonlinear dynamic analyses are carried out of each study building using thirty natural unidirectional ground motion accelerograms to evaluate the panel zone response.

4.4.1 Study Buildings

The geometric configuration of the 3- and 9-storey study buildings considered for panel zone response evaluation is same as that of the study buildings described previously in Chapter 3. The details of the study buildings (beams and column sections, and doubler plate thickness) designed for panel zone response evaluation is given in Table 4.2 and 4.3. Study buildings are designed for three seismic hazard levels, Z=0.2g, 0.4g, and Z=0.6g according to the Indian Standards [IS 800, 2007; IS 1893 Part-1, 2016]. The *CBSR* is ensured be compliant with the seismic codal provisions [IS 800, 2007; AISC 341, 2016; EN 1993-1-8, 2004] for the design of steel SMFs, with a margin sufficient enough to limit yielding in columns up to a shaking intensity of Z=0.6g. Finally the design of panel zone is varied according to the three panel zone design approaches described in Section 4.3. The nonlinear building model is created as described previously in Section 3.2.3.

Table 4.2: Details of the SMFs of the 3-storey study building with three different panel zone designs

3-Storey	Storey	Beam	Column	Doubler Plate Thickness (mm)					
				PZ-I		PZ-II		PZ-III	
				Int	Ext	Int	Ext	Int	Ext
Z=0.2g	3	W 21 × 57	W 14 × 211	4	0	16	0	23	0
	2	W 24 × 76	W 14 × 211	18	0	30	4	40	9
	1	W 24 × 76	W 14 × 211	18	0	30	4	40	9
	Building Natural Period (s)			*1.391*		*1.377*		*1.368*	
	Design Base Shear: ***723.2*** *kN*								
Z=0.4g	3	W 24 76	W 14 × 342	0	0	10	0	19	0
	2	W 27 × 114	W 14 × 342	12	0	34	0	50	6
	1	W 27 × 114	W 14 × 342	12	0	34	0	50	6
	Building Natural Period (s)			*0.993*		*0.982*		*0.975*	
	Design Base Shear: ***1446.4*** *kN*								
Z=0.6g	3	W 24 × 84	W 14 × 455	0	0	0	0	9	0
	2	W 33 × 147	W 14 × 455	1	0	29	0	51	1
	1	W 33 × 147	W 14 × 455	1	0	29	0	51	1
	Building Natural Period (s)			*0.815*		*0.804*		*0.799*	
	Design Base Shear: ***2169.6*** *kN*								

Table 4.3: Details of the SMFs of the 9-storey study building with three different panel zone designs

9-Storey	Storey	Beam	Column	Doubler Plate Thickness (mm)					
				PZ-I		PZ-II		PZ-III	
				Int	Ext	Int	Ext	Int	Ext
Z=0.2g	9	W 21 × 57	W 14 × 257	0	0	8	0	15	0
	8	W 21 × 57	W 14 × 257	0	0	9	0	15	0
	7	W 27 × 84	W 14 × 257	12	0	26	0	38	5
	6	W 27 × 84	W 14 × 257	12	0	26	0	38	5
	5	W 27 × 84	W 14 × 283	6	0	23	0	34	2
	4	W 24 × 104	W 14 × 283	22	0	41	5	55	12
	3	W 24 × 104	W 14 × 283	22	0	41	5	55	12
	2	W 24 × 104	W 14 × 283	22	0	41	5	55	12
	1	W 30 × 99	W 14 × 311	10	0	28	0	40	3
	Building Natural Period (s)			*3.332*		*3.282*		*3.256*	
	Design Base Shear: ***939.5*** *kN*								
Z=0.4g	9	W 21 × 83	W 14 × 311	0	0	21	0	30	0
	8	W 21 × 83	W 14 × 311	0	0	21	0	30	0
	7	W 24 × 117	W 14 × 311	24	0	46	6	61	14
	6	W 24 × 117	W 14 × 311	24	0	46	6	61	14
	5	W 24 × 117	W 14 × 398	2	0	31	0	45	1
	4	W 30 × 132	W 14 × 398	9	0	34	0	54	5
	3	W 30 × 132	W 14 × 398	9	0	34	0	54	5
	2	W 30 × 132	W 14 × 398	9	0	34	0	54	5
	1	W 30 × 148	W 14 × 455	9	0	39	0	57	4
	Building Natural Period (s)			2.659		2.608		2.586	
	Design Base Shear: ***1879.0*** *kN*								
Z=0.6g	9	W 24 × 84	W 14 × 426	0	0	4	0	13	0
	8	W 24 × 84	W 14 × 426	0	0	4	0	13	0
	7	W 33 × 130	W 14 × 426	1	0	26	0	47	0
	6	W 33 × 130	W 14 × 426	1	0	26	0	47	0
	5	W 33 × 130	W 14 × 550	0	0	8	0	27	0
	4	W 36 × 160	W 14 × 550	0	0	23	0	48	0
	3	W 36 × 160	W 14 × 550	0	0	23	0	48	0
	2	W 36 × 160	W 14 × 550	0	0	23	0	48	0
	1	W 36 × 182	W 14 × 605	0	0	30	0	55	0
	Building Natural Period (s)			2.076		2.045		2.025	
	Design Base Shear: ***2818.5*** *kN*								

4.5 Results of Nonlinear Analysis

Nonlinear static and dynamic analyses are employed to evaluate the seismic response of the panel zones in the study buildings described in Section 4.4.1. First mode based nonlinear static pushover analysis (PoA) was carried out till any of the deformation-controlled element reached its collapse limit state. The nonlinear dynamic time history analysis was carried out for the thirty unidirectional ground motion accelerograms mentioned in Section 3.3. The scaling of the ground motion accelerograms and Rayleigh damping considered for the study buildings are the same as described previously in Section 3.3.

4.5.1 Nonlinear Static Analysis

Figures 4.3, 4.4, and 4.5 shows the pushover curve of the 3-storey study buildings designed for Z=0.2g, 0.4g, and 0.6g level of shaking, respectively. Performance point obtained from the FEMA 440 linearization method is also marked in the Figures. Demand estimation by FEMA 440 linearization method shows that, the study buildings

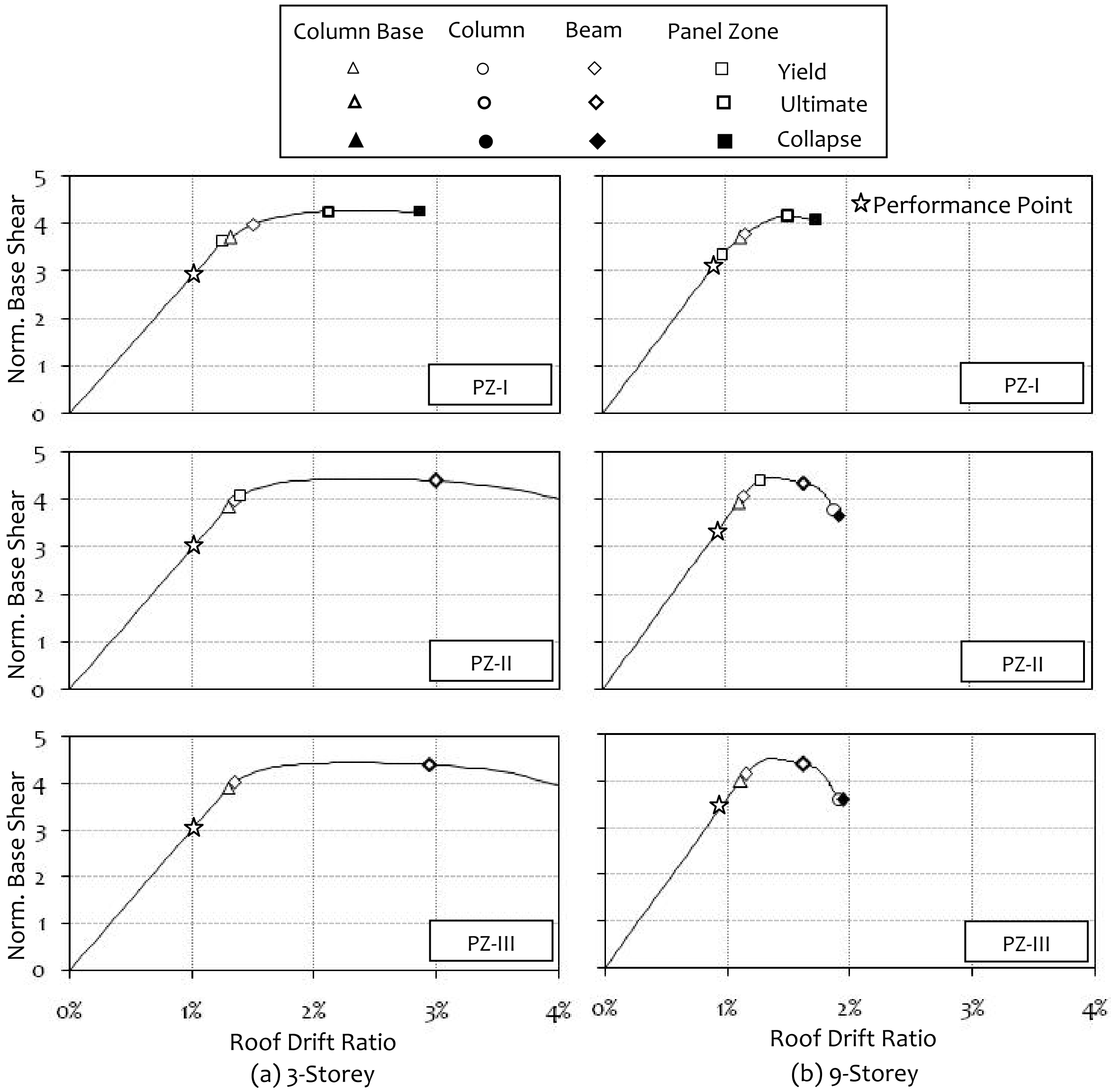


Figure 4.3: Pushover curves of (a) 3-storey and (b) 9-storey study buildings designed for a seismic hazard level of Z=0.2g; the onset of the key damage states are highlighted.

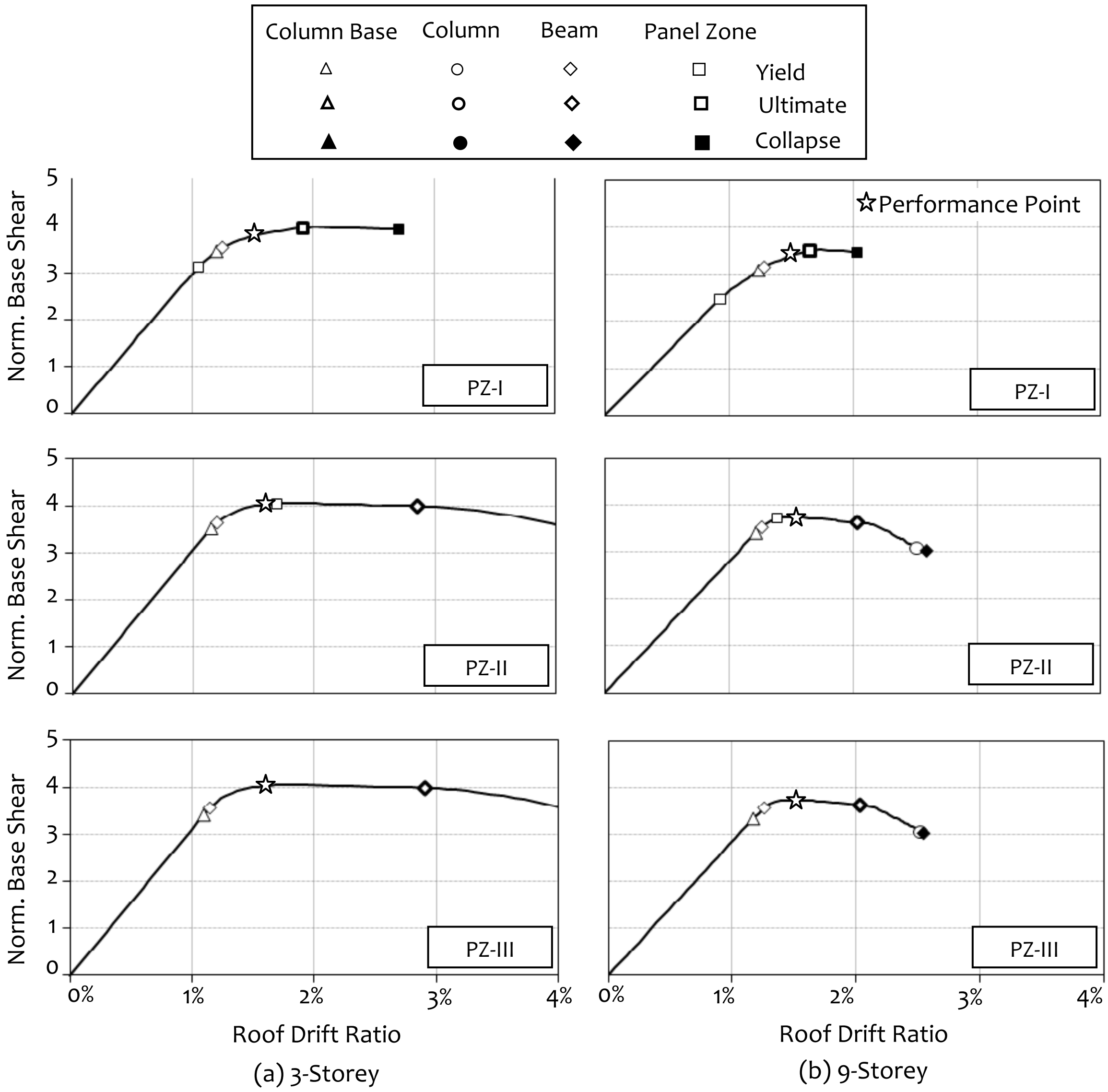


Figure 4.4: Pushover curves of (a) 3-storey and (b) 9-storey study buildings designed for a seismic hazard level of Z=0.4g; the onset of the key damage states are highlighted.

designed for a shaking intensity corresponding to Z=0.2g remain nearly elastic. This can be attributed to the significant overstrength (~4) developed by the buildings. This overstrength is the combined result of (i) partial load factor (of 1.5), (ii) material partial safety factor (of 1.1), (iii) overstrength in beams due to increase in material yield stress from the minimum specified (characteristic) yield stress (by a factor of 1.5), (iv) redundancy, (v) strain hardening in beams (by a factor of 1.1), and (vi) overstrength resulting from selection of higher member sizes during design. The overstrength developed marginally decreases (~3.8) for study buildings designed for higher shaking intensity (*i.e.*, Z=0.4g and 0.6g), since the proportion of seismic demand to gravity demand increases. Significant inelastic action is observed in the buildings designed for higher seismic hazard levels, especially buildings designed for Z=0.6g.

Even though, the level of inelasticity observed varies for buildings designed for different hazard levels, the sequence of damage initiation among the structural components essentially remain same for a given panel zone design approach adopted. Hence, the results of study buildings designed for seismic hazard level Z=0.4g, which is shown in Figure 4.4, is the primary focus of discussion in the following section.

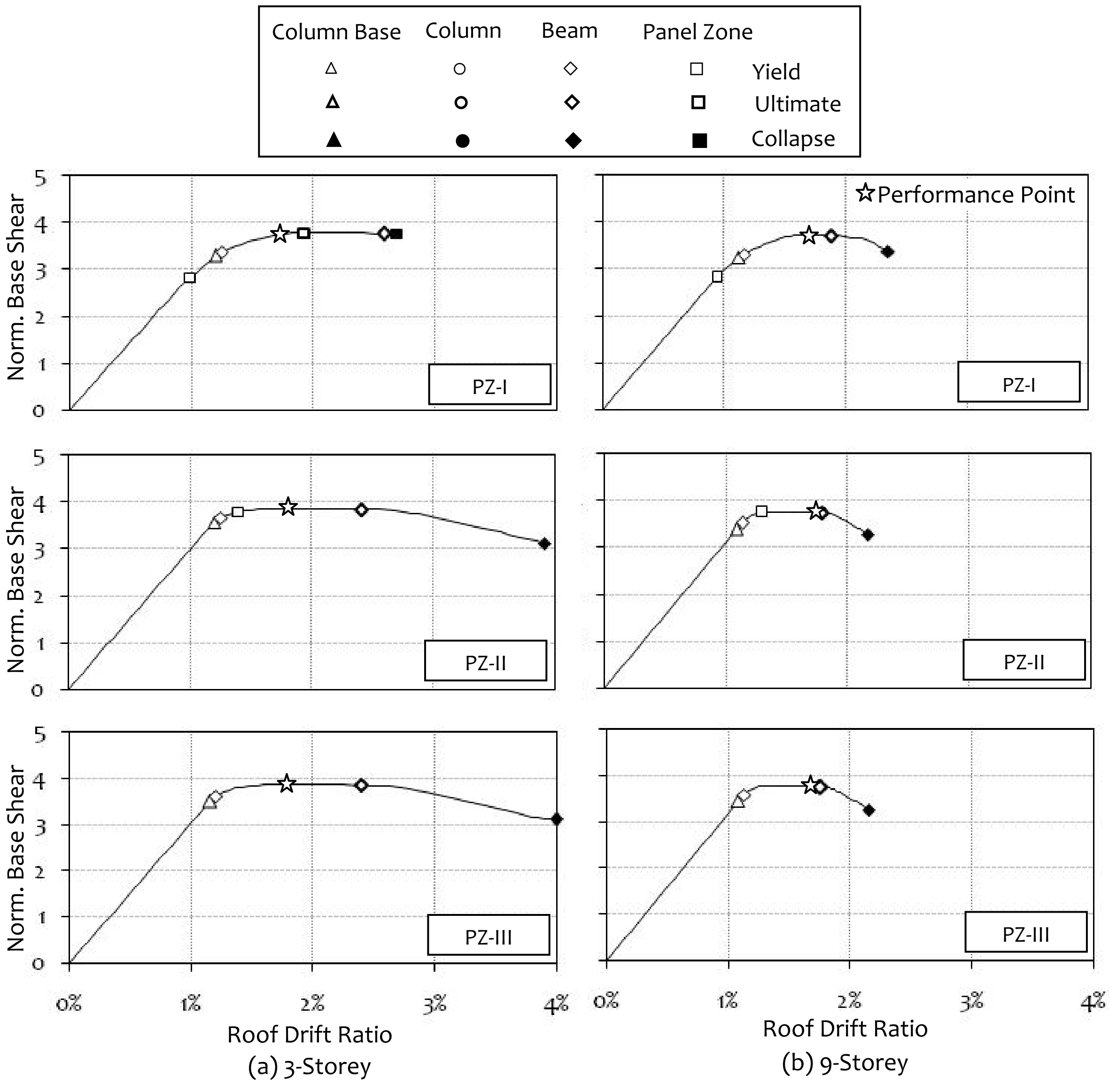


Figure 4.5: Pushover curves of (a) 3-storey and (b) 9-storey study buildings designed for a seismic hazard level of Z=0.6g; the onset of the key damage states are highlighted.

4.5.1.1 PZ-I Design Approach

For the 3- and 9-storey study buildings designed according PZ-I approach, the yielding of the buildings starts with the onset of panel zone yielding at drift ratio around 1.0%. Thereafter, the column bases and beams begin to yield. Finally, the deformability of the buildings is limited by the panel zone shear deformations reaching the collapse limit state. Thus, it can be inferred that, panel zones are the primary source of energy dissipation rather than the beams if PZ-I design is adopted. Such behaviour is not preferred under seismic activity, since the post-yield response of the panel zone is associated with plastic hinging of column flanges. This results in kinking of columns at the beam flange level, which further induces large strain demand on the welds leading to brittle fracture of connection. Hence it can be concluded that the PZ-I approach will lead to weak panel zones, which should be avoided in seismic design of steel SMFs.

4.5.1.2 PZ-II Design Approach

The PZ-II design approach results in balanced panel zone behaviour. The 3- and 9- storey study buildings behave elastically till a roof drift ratio of 1.0%. The inelastic response of the structure initiates with the onset of column base and beam yielding at roof drift ratio beyond 1.0%, after which, panel zone yielding initiates. Unlike in the study buildings designed as per PZ-I approach, the study buildings designed as per PZ-II approach have considerable inelastic action in the beams. Further the inelasticity in panel zones is limited, as it can be observed from pushover curves that none of the panel zones reach their ultimate limit state. The drop in the pushover curve of the 9-storey frame after a drift of 2.0% is due to global P-Δ effects coupled with the exhaustion of plastic rotation capacity of the beam sections. Finally, as the panel zone shear deformations are limited, an increase in deformability capacity is observed compared to the buildings designed as per PZ-I approach. Thus, the behaviour of the buildings with PZ-II design approach results in acceptable seismic performance provided the beam-to-column connection can accommodate the additional strain demand generated from mild panel zone yielding.

4.5.1.3 PZ-III Design Approach

The behaviour of study buildings with PZ-III design approach is similar to study buildings with PZ-II design approach except that panel zones now do not yield at all. Thus, PZ-III panel zone design approach leads to strong panel zone behaviour. The behaviour of study buildings with PZ-III panel design approach shows acceptable seismic performance, where beams are primary source of energy dissipation, with yielding of panel zones eliminated altogether.

4.5.2 Nonlinear Dynamic Analysis

The nonlinear dynamic time history analysis is carried out for the thirty unidirectional ground motions which were scaled at their natural period to match the corresponding design spectrum [IS 1893, 2016].

4.5.2.1 3-Storey Building (Panel Zone Response)

The behaviour of interior panel zones and adjacent beams along the height of the 3-storey study buildings designed for Z=0.2g, 0.4g, and 0.6g is depicted in Figures 4.6, 4.7, and 4.8, respectively.

a) PZ-I Design Approach

The inelasticity in the panel zones is limited in buildings designed for a seismic hazard level of Z=0.2g (~1 γ_{ypz}). However, in buildings designed for shaking intensity of Z=0.4g and 0.6g, significant inelasticity can be observed in the panel zones, while inelastic action in beams are limited. The limited inelasticity observed at Z=0.2g level of shaking is due to the fact that, intensity of shaking is not severe enough to push the SMF into significant inelastic action, which is evident from the elastic response of the beams and near elastic response of panel zones. The elastic response observed at Z=0.2g confirms the observation made previously based on the results of nonlinear static analysis (FEMA 440 Linearization). Hence the use of, PZ-I panel zone design approach can be justified for regular SMFs up to 3 storeys for a seismic hazard level of Z=0.2g. But, PZ-I approach surely need to be avoided for SMFs designed for higher hazard levels. For the buildings designed for higher hazard levels, inelasticity observed in beams is very limited (median response indicates that, less than 20% of plastic rotation capacity of the critical beams are utilized even for the building designed for Z=0.6g) and it is the panel zones which contribute primarily towards seismic energy dissipation (~3γ_{ypz} and ~4 γ_{ypz} for building designed for Z=0.4g and 0.6g respectively). This mode of energy dissipation contradicts the dissipation mechanism envisioned for SMFs, where beams are supposed to be the primary source of energy dissipation.

b) PZ-II Design Approach

The inelasticity observed in panel zone is very limited in buildings designed using PZ-II design approach. The panel zones exhibit almost an elastic response in SMFs designed for Z=0.2g and 0.4g levels of shaking, where yielding is observed for just two of the thirty ground motions. Even in buildings designed for Z=0.6g level of shaking, the panel zones do not yield beyond 1.5 times the yield strain (γ_{ypz}). Further, the responses of the beams indicate considerable inelastic action in beam, which did not occur in the PZ-I design. Also, all the beams do yield when designed as per PZ-II approach, while only limited number of beams underwent yielding when designed as per PZ-I approach. Hence PZ-II design approach results in energy dissipation mechanism which is in line with the dissipation mechanism desired for SMFs. Therefore, PZ-II panel zone design approach is suitable for regular SMFs up to 3 storeys. However, a more stringent provision than PZ-II approach need to be used for connection configurations which cannot tolerate even minor panel zone yielding and the associated additional strain demands on the connections. Another observation that can be made by comparing the response of buildings designed as per PZ-I and PZ-II approach is that, the total energy dissipation is shared between panel zones and beams depending upon their relative strengths. As the panel zone thickness increases moving from PZ-I to PZ-II design, the inelastic participation of beams increases, while that of panel zones reduces. Similar to PZ-I approach, near elastic response of frames is observed at Z=0.2g level of shaking.

c) PZ-III Design Approach

PZ-III panel zone design approach results in elastic response of panel zones in SMFs designed for all levels of shaking considered in the study. Since the panel zones do not yield, the inelastic action is moved entirely from panel zones to beams, which is evident from the enhanced inelasticity observed in beam response in PZ-III approach compared to PZ-II and PZ-I approaches. Thus, the behaviour of the SMFs as observed in PZ-III approach results in acceptable seismic energy dissipation mechanism, however at the price of having increased doubler plate thickness compared to PZ-II approach. Further as PZ-III approach results in increased seismic demand in beams, it will be necessary to ensure sufficient plastic rotation capacity is available in the beam sections used in such design.

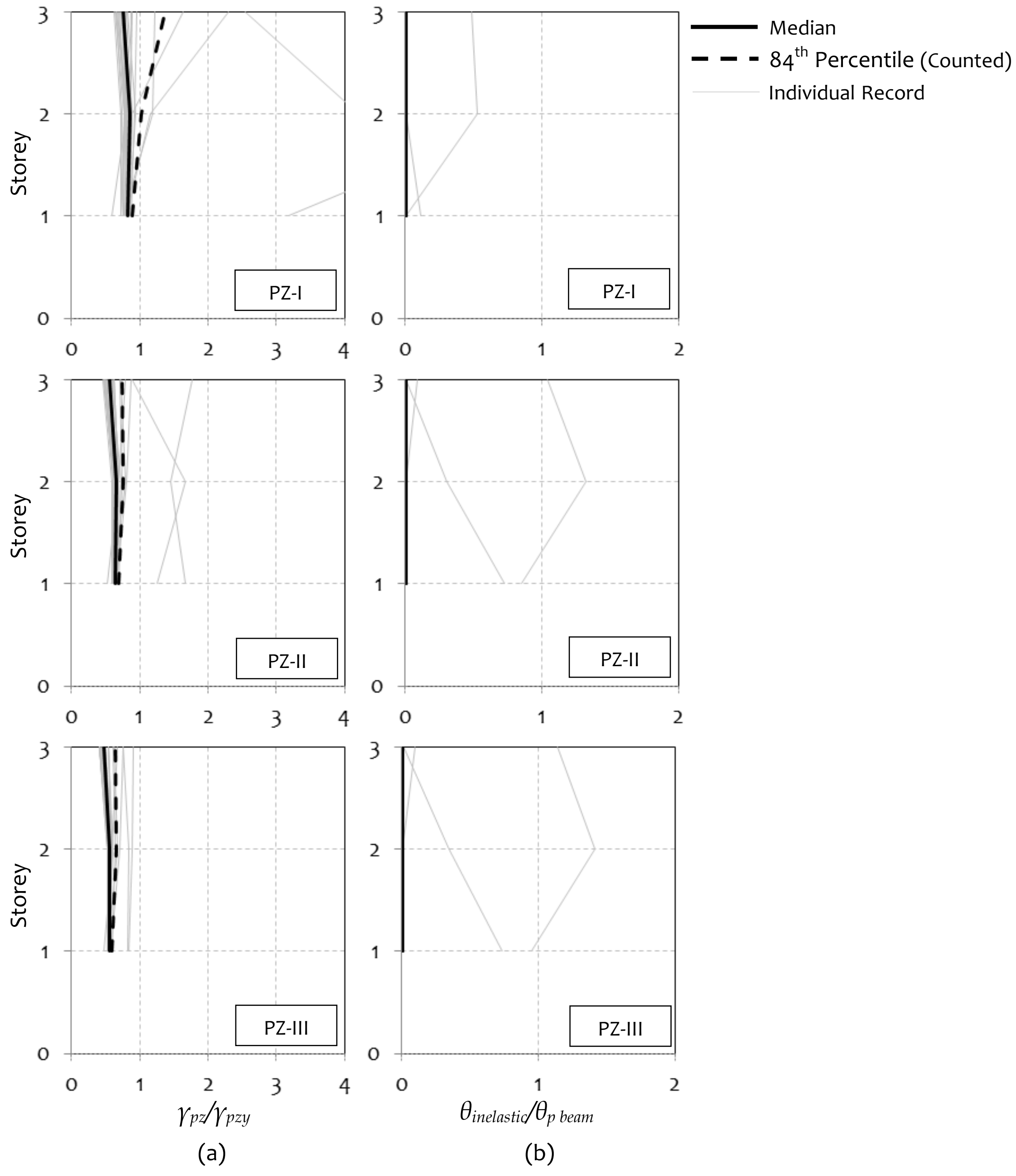


Figure 4.6: (a) Panel zone shear deformation normalized with respect to yield, and (b) inelastic rotation demand in beam normalized with respect to the modified plastic rotation capacity of the beam for the 3-storey building designed for a seismic hazard level of Z=0.2g.

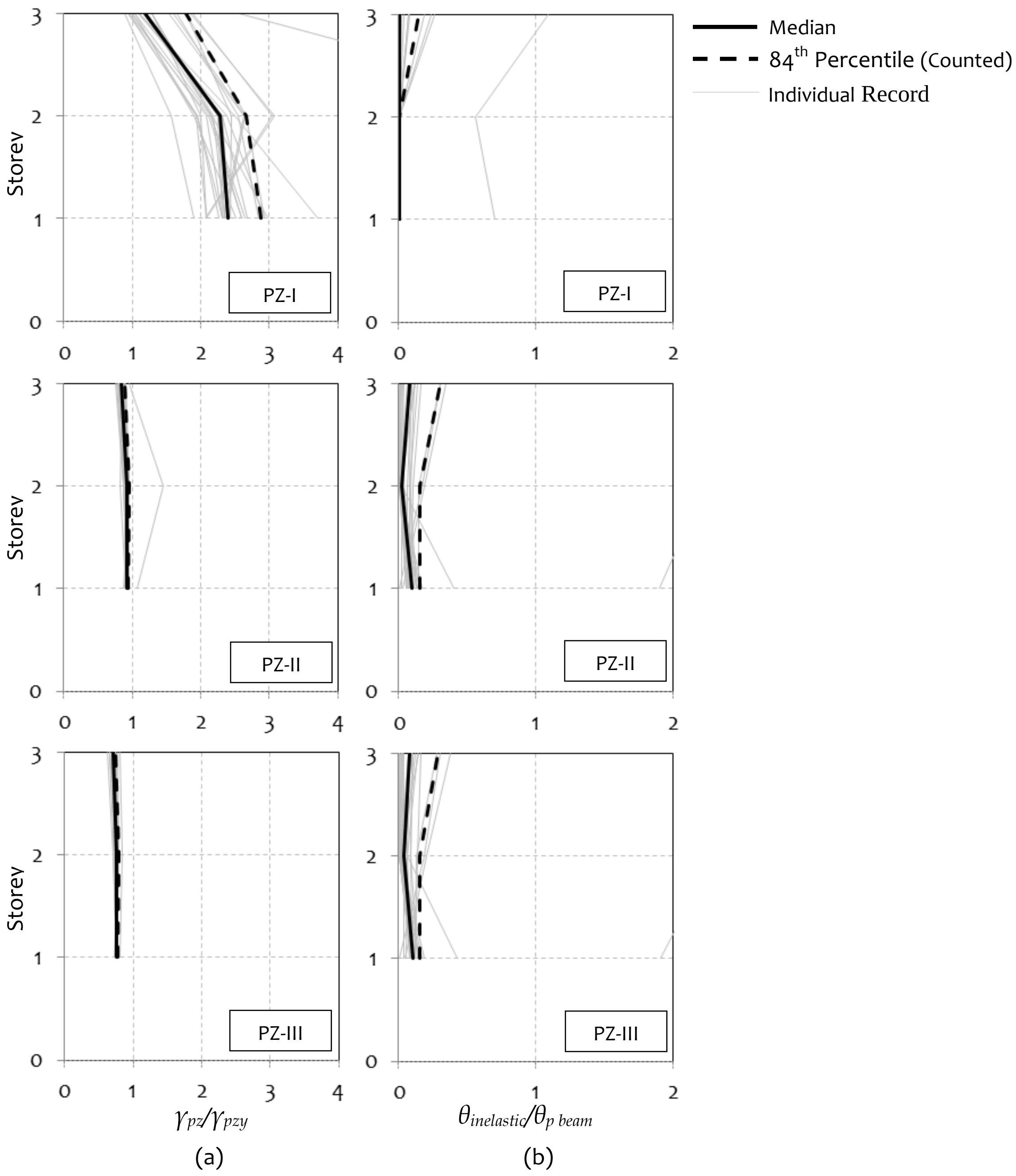


Figure 4.7: (a) Panel zone shear deformation normalized with respect to yield, and (b) inelastic rotation demand in beam normalized with respect to the modified plastic rotation capacity of the beam for the 3-storey building designed for a seismic hazard level of Z=0.4g.

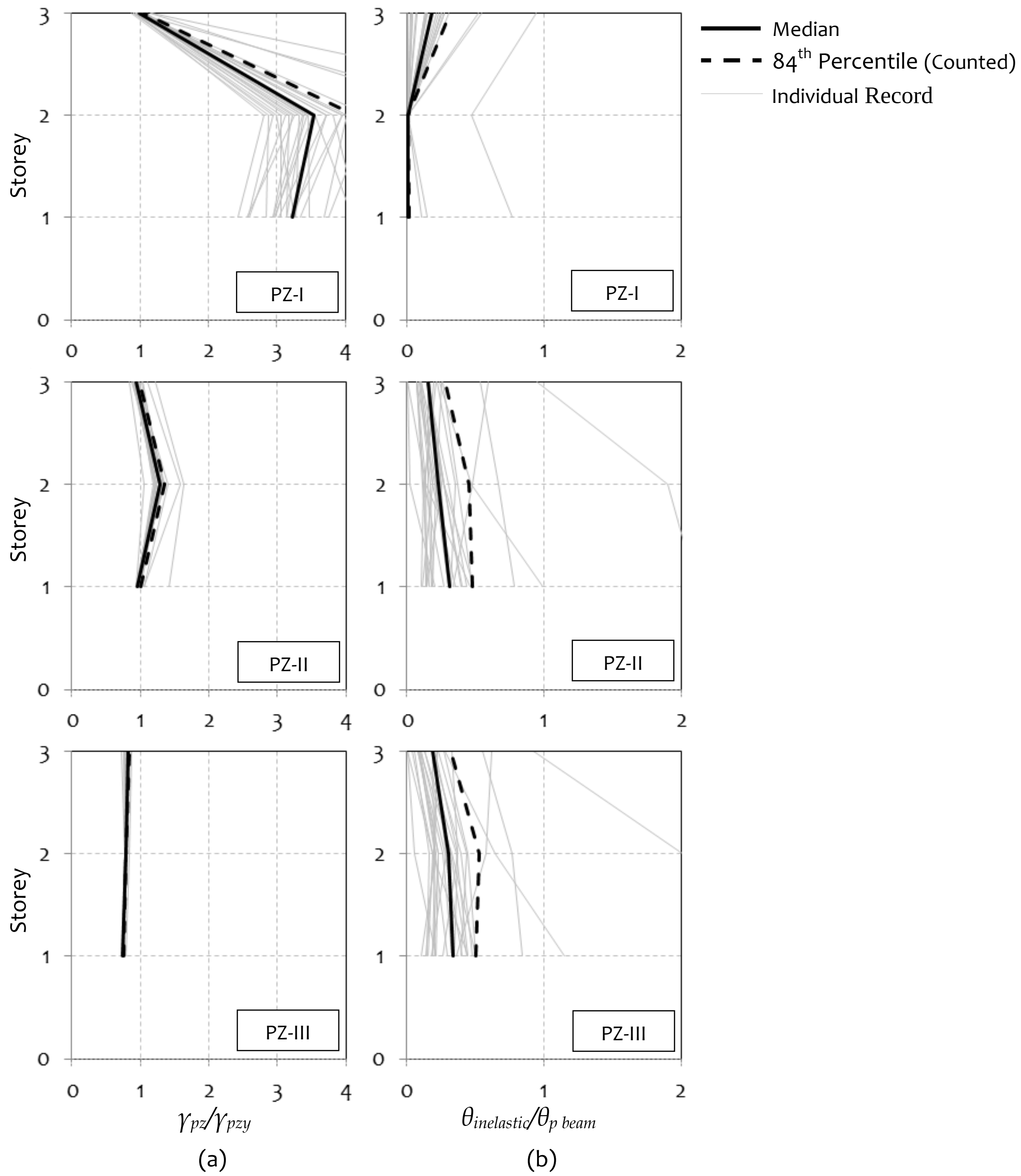


Figure 4.8: (a) Panel zone shear deformation normalized with respect to yield, and (b) inelastic rotation demand in beam normalized with respect to the modified plastic rotation capacity of the beam for the 3-storey building designed for a seismic hazard level of Z=0.6g.

4.5.2.2 9-Storey Building (Panel Zone Response)

The behaviour of interior panel zones and adjacent beams along the height of the 9-storey study buildings designed for Z=0.2g, 0.4g, and 0.6g is depicted in Figures 4.9, 4.10, and 4.11, respectively.

a) PZ-I Design Approach

The panel zones undergo considerable inelastic action (~2γ_{ypz}) with PZ-I design approach even in the SMF designed for a low seismic hazard level of Z=0.2g. Median panel zone shear deformations of 3 and 4 times the yield are observed in the SMFs designed for Z=0.4g and Z=0.6g levels of shaking. Further, the participation of the beams in inelastic action is minimal even in the SMFs designed for Z=0.6g shaking intensity, with many beams not at all participating in energy dissipation. Thus, it can be concluded that PZ-I design approach will lead to weak panel zone behaviour, which is undesirable. Hence, PZ-I approach should be avoided, especially for connection configurations where panel zone yielding and subsequent kinking of column flange will eventually lead to brittle fracture of the connection. The amplification of the beam and panel zone demands observed in the top storeys of 9-storey SMFs is due to the influence of higher vibration modes on seismic response.

b) PZ-II Design Approach

The shear distortion of panel zone is limited, even in SMF designed for Z=0.6g shaking intensity, where the shear distortion is within twice the yield strain. Further, considerable yielding of beam flexural hinges are also observed. Thus the PZ-II panel zone design approach results in balanced panel zone behaviour for regular steel SMFs upto 9 storeys.

c) PZ-III Design Approach

Panel zones exhibit elastic response in SMFs designed using PZ-III for all levels of shaking. However, there is no significant increase in the beam inelastic demand compared to what was observed in PZ-II design approach. Thus PZ-III design approach results in strong panel zone behaviour, which assures acceptable seismic behaviour, provided the beam sections used have sufficient plastic rotation capacity. Most of the standard beam sections, which are plastic, do have the required plastic rotation capacity.

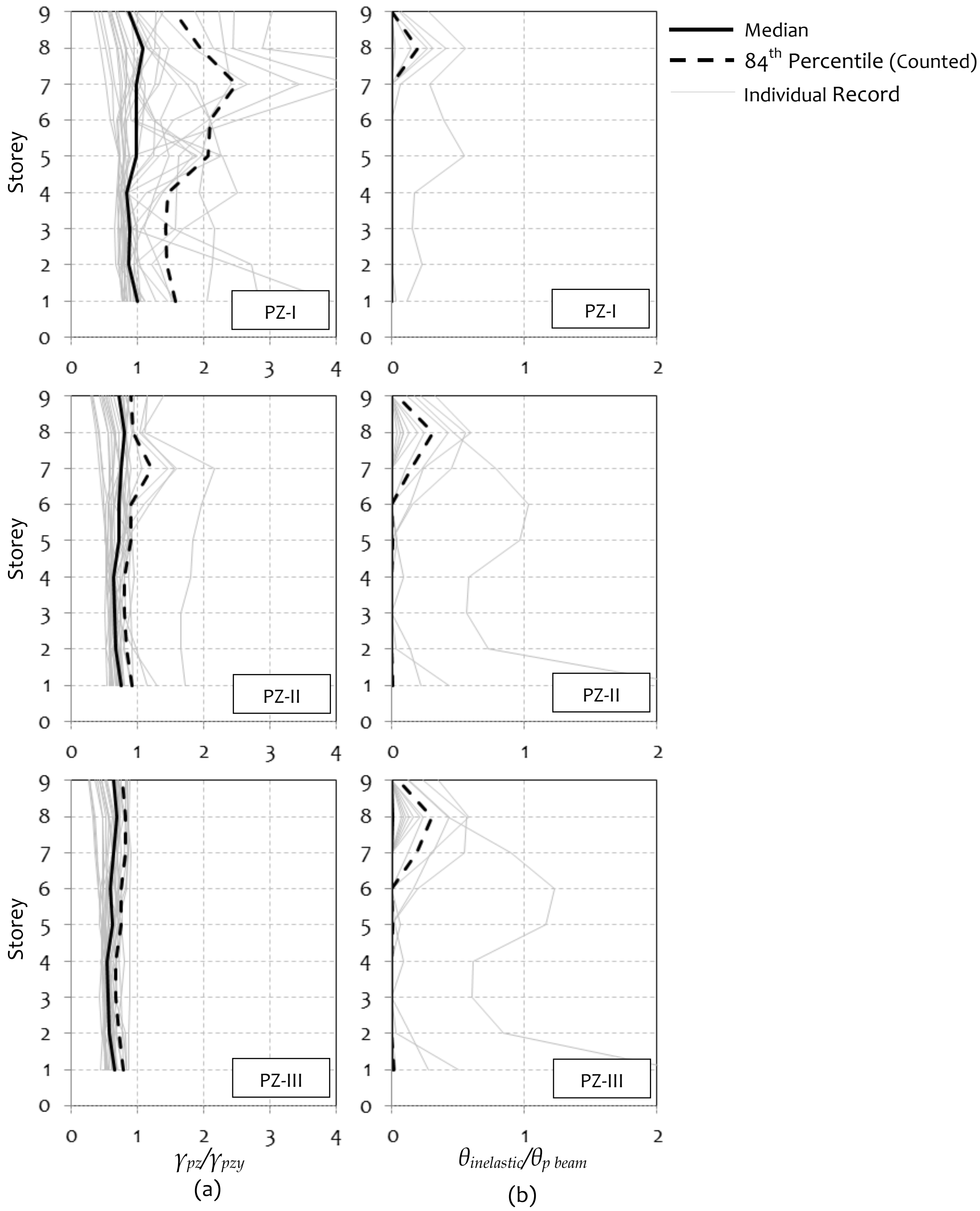


Figure 4.9: (a) Panel zone shear deformation normalized with respect to yield, and (b) inelastic rotation demand in beam normalized with respect to the modified plastic rotation capacity of the beam for the 9-storey building designed for a seismic hazard level of Z=0.2g.

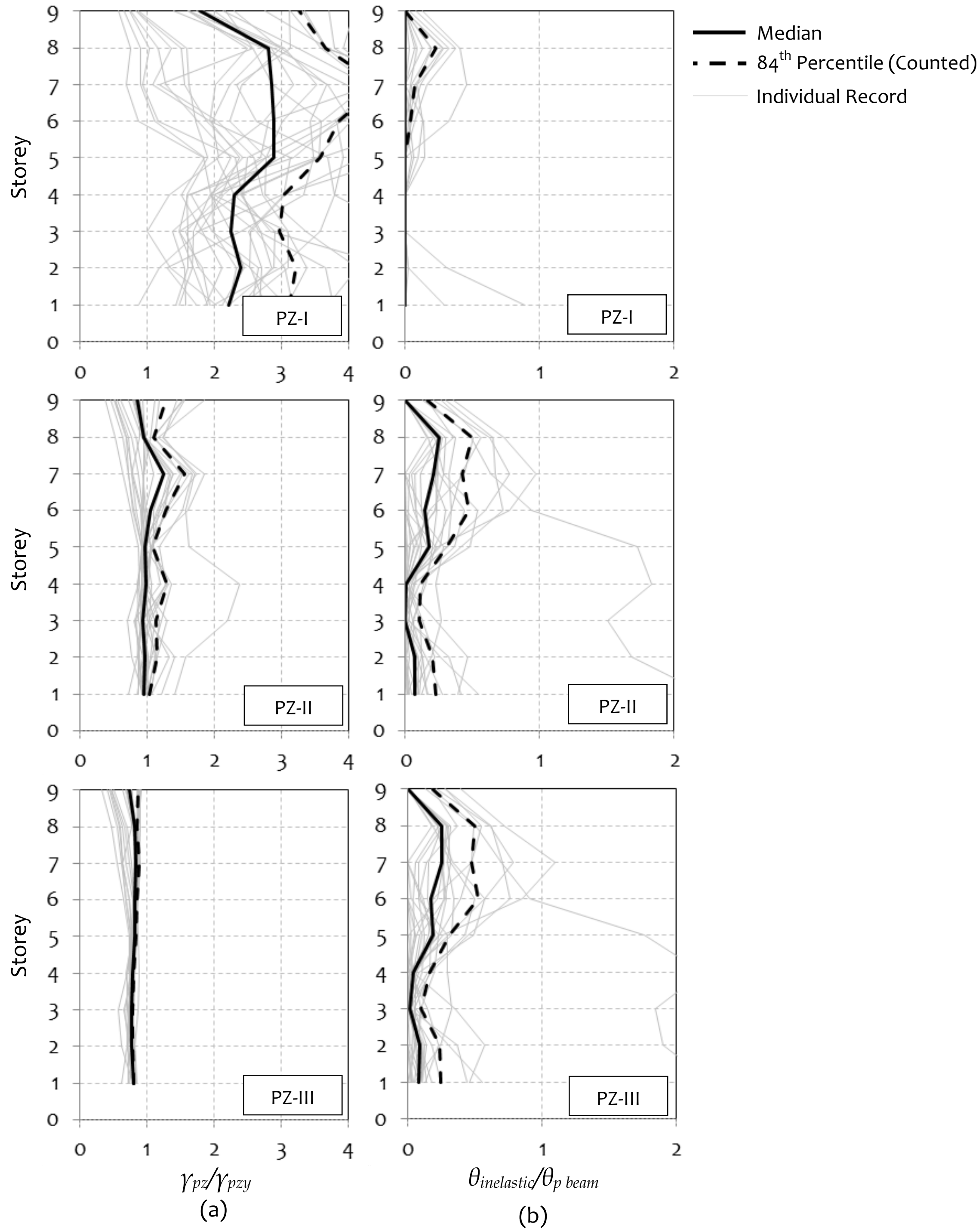


Figure 4.10: (a) Panel zone shear deformation normalized with respect to yield, and (b) inelastic rotation demand in beam normalized with respect to the modified plastic rotation capacity of the beam for the 9-storey building designed for a seismic hazard level of Z=0.4g.

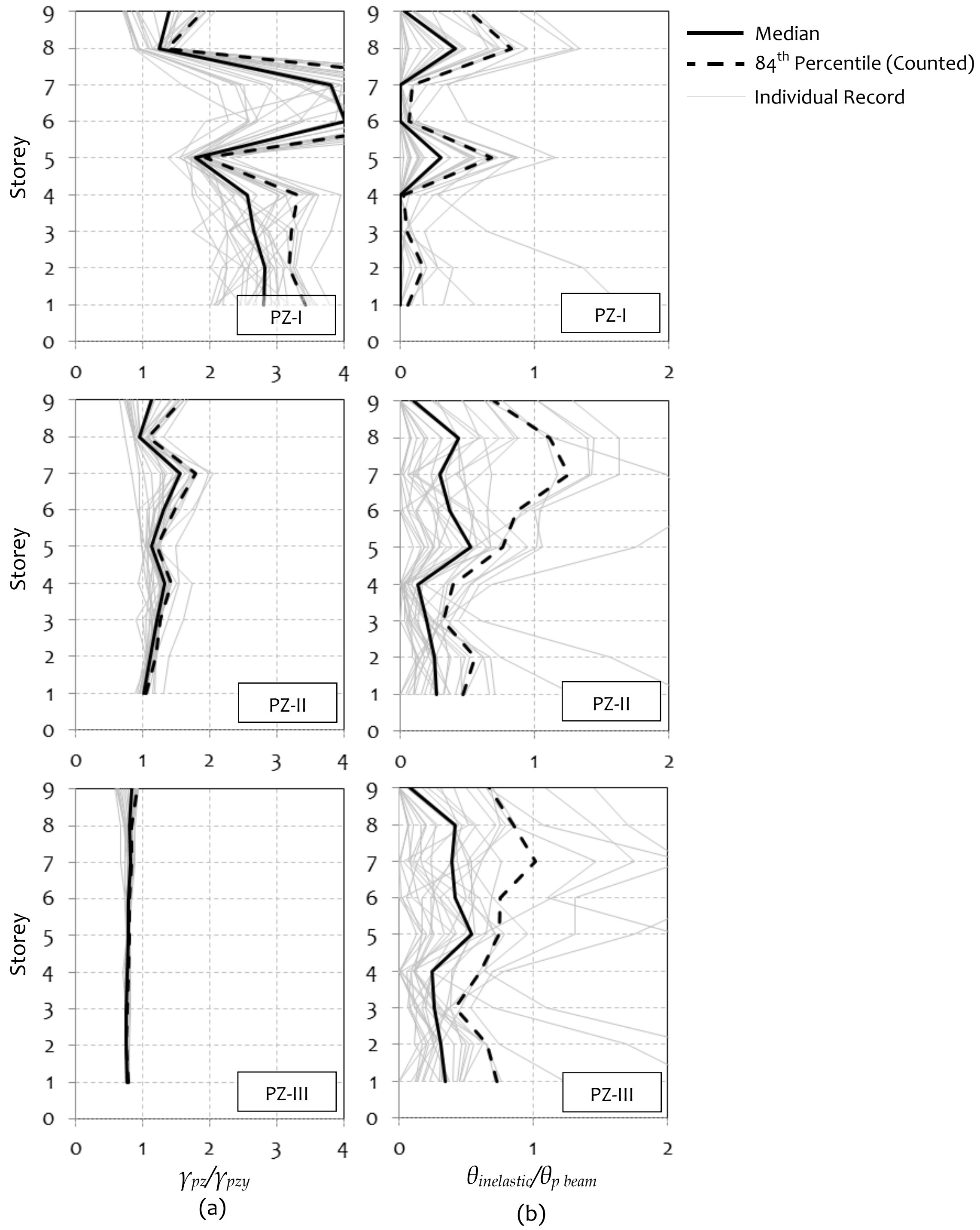


Figure 4.11: (a) Panel zone shear deformation normalized with respect to yield, and (b) inelastic rotation demand in beam normalized with respect to the modified plastic rotation capacity of the beam for the 9-storey building designed for a seismic hazard level of Z=0.6g.

4.4.5.3 Summary of Panel Zone Response at Design Level Earthquake

Figure 4.12 summarizes the interior panel zone response observed in the nonlinear dynamic analyses of all the study buildings considered in the study, where the maximum panel zone shear strain observed in each study building under each ground motion is normalized with panel zone yield strain. It can be inferred that the inelasticity of the panel zone increases in buildings designed for increasing level of shaking. This is because displacement demands are higher in the buildings designed for higher levels of shaking. Further distortion demands are consistently higher in 9-storey buildings compared to 3-storey buildings due to increased localization of inelasticity, *i.e.*, in the 3-storey frames, the energy dissipation is well distributed between all elements, while in the 9-storey frames, few elements participate much more in energy dissipation compared to others.

4.4.5.4 Incremental Dynamic Analysis

The 9-storey study building designed for seismic hazard level of Z=0.4g is subjected to ground motions scaled to design response spectrum corresponding to Z=0.5g and 0.6g, to evaluate the behaviour of panel zones at shaking intensities larger than the design intensity. The behaviour of interior panel zones and adjacent beams along the height of the 9-storey study buildings is shown in Figures 4.13 and 4.14, respectively. It is observed that the shear strain demand in panel zones of study building with PZ-I approach increases significantly (from a median demand of ~3γ_{ypz} at Z=0.4g to ~6γ_{ypz} at Z=0.6g) when subjected ground motions greater than the design intensity. Further, in PZ-I approach, the increase in beam ductility demand is marginal as shaking intensity increases. These observations underscore the fact that, PZ-I approach results in weak panel zone behaviour. Study buildings with PZ-II design approach shows only marginal increase in panel zone shear strain demand (from a median strain demand of 1.2γ_{ypz} at Z=0.4g to 1.5γ_{ypz} at Z=0.6g) but with increase in beam ductility demand which increases from 20% utilization of plastic rotation capacity at Z=0.4 to 60% utilization at Z=0.6g.. Thus, PZ-II approach results in a balanced panel zone behaviour, where panel zones do yield and contribute to energy dissipation, but beams are the primary source of energy dissipation. Finally, panel zones remain elastic when designed as per PZ-III approach, even at shaking intensities greater than the design level. Hence, PZ-III approach results in strong panel zone, where inelasticity is concentrated in beams alone without any yielding of panel zones. A summary of panel zone response in the study buildings designed for Z=0.4g is presented in Figure 4.15.

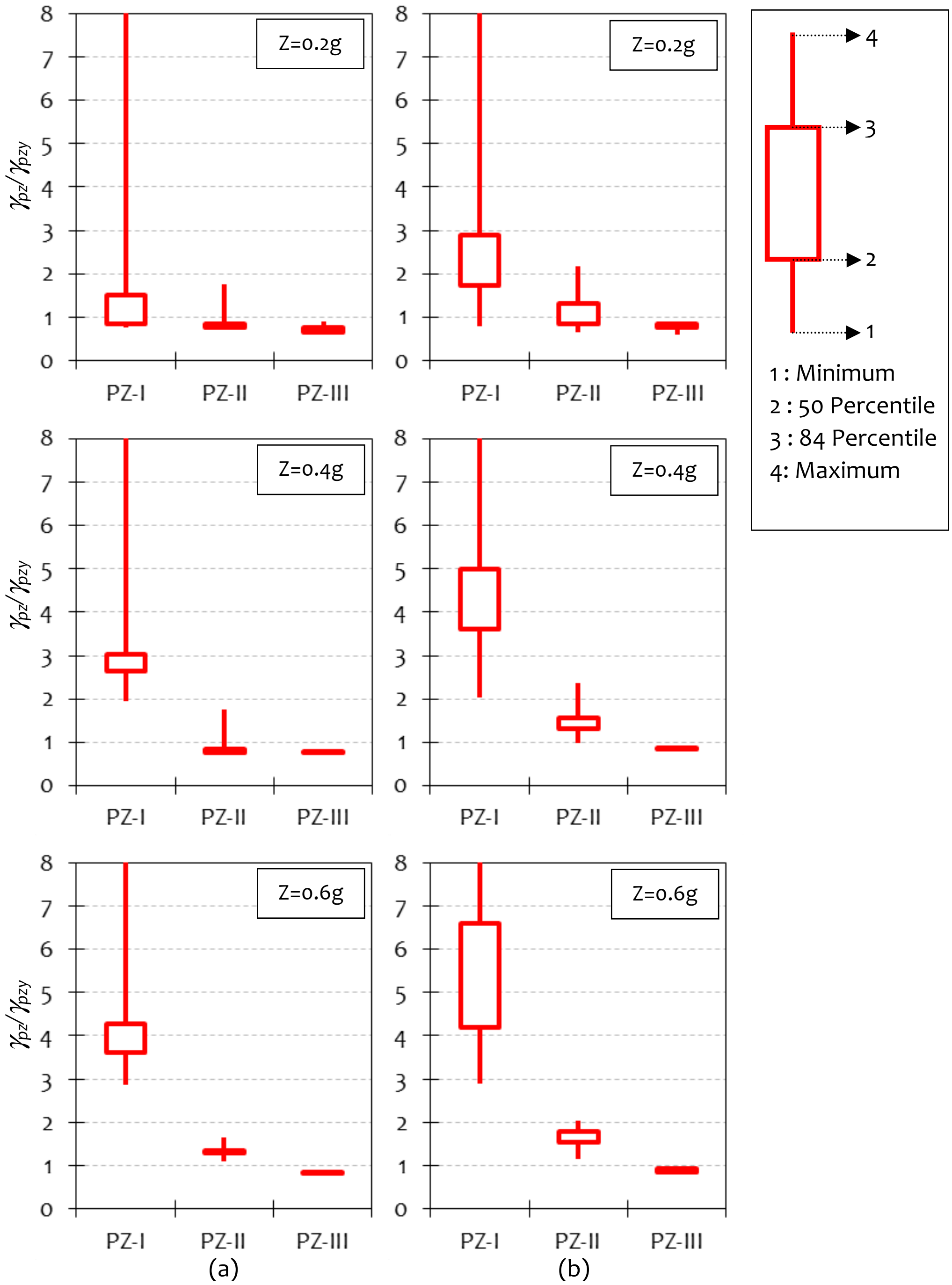
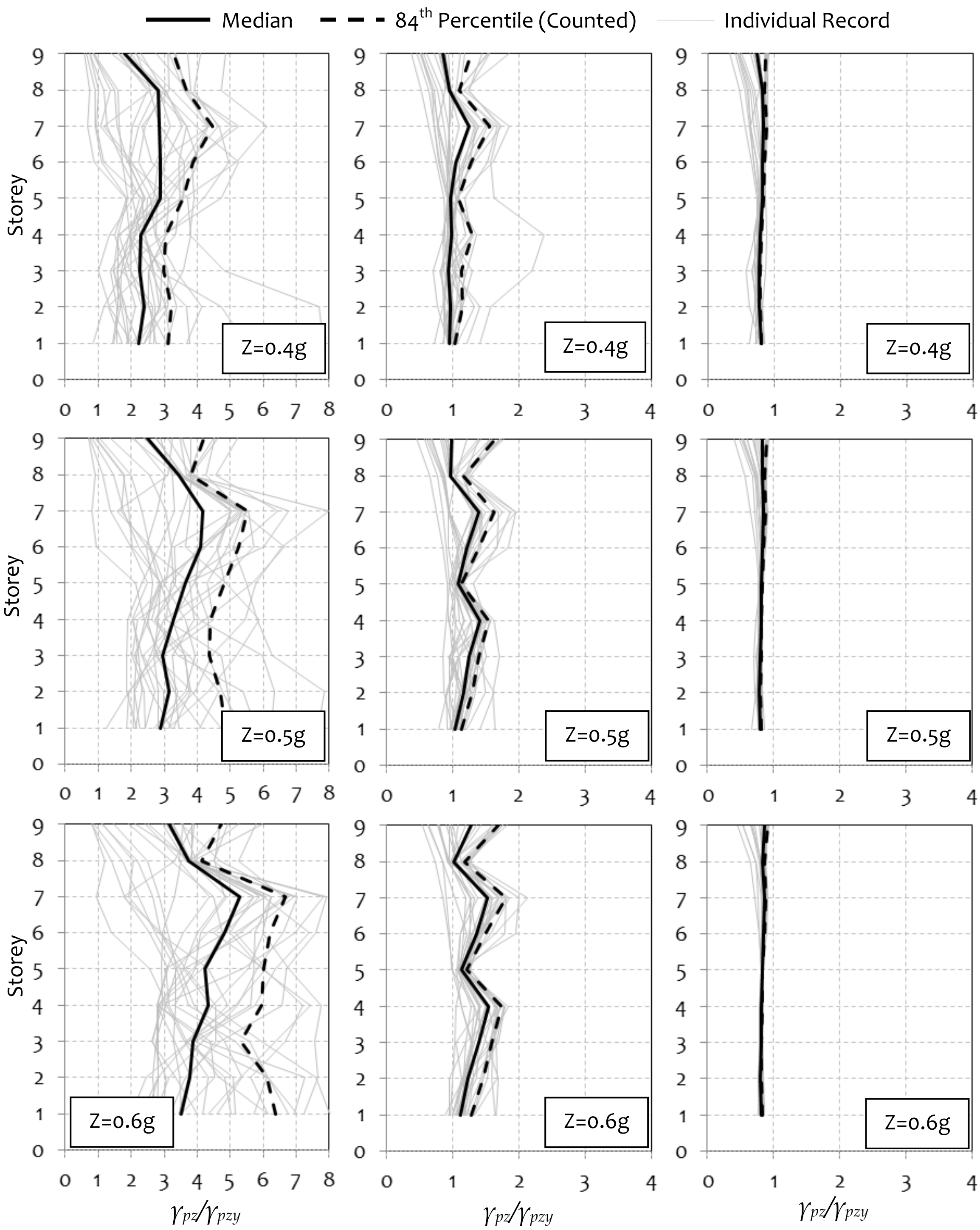


Figure 4.12: Range of maximum panel zone deformations from time history analysis at the MCE level. (a) 3-storey, and (b) 9-storey.



(a) PZ-I (b) PZ-II (c) PZ-III

Figure 4.13: Shear strain demand in interior panel zones of 9-storey study buildings as observed in nonlinear dynamic analysis. The shear strain demand (γ_{pz}) is normalized with panel zone yield strain (γ_{pzy}).

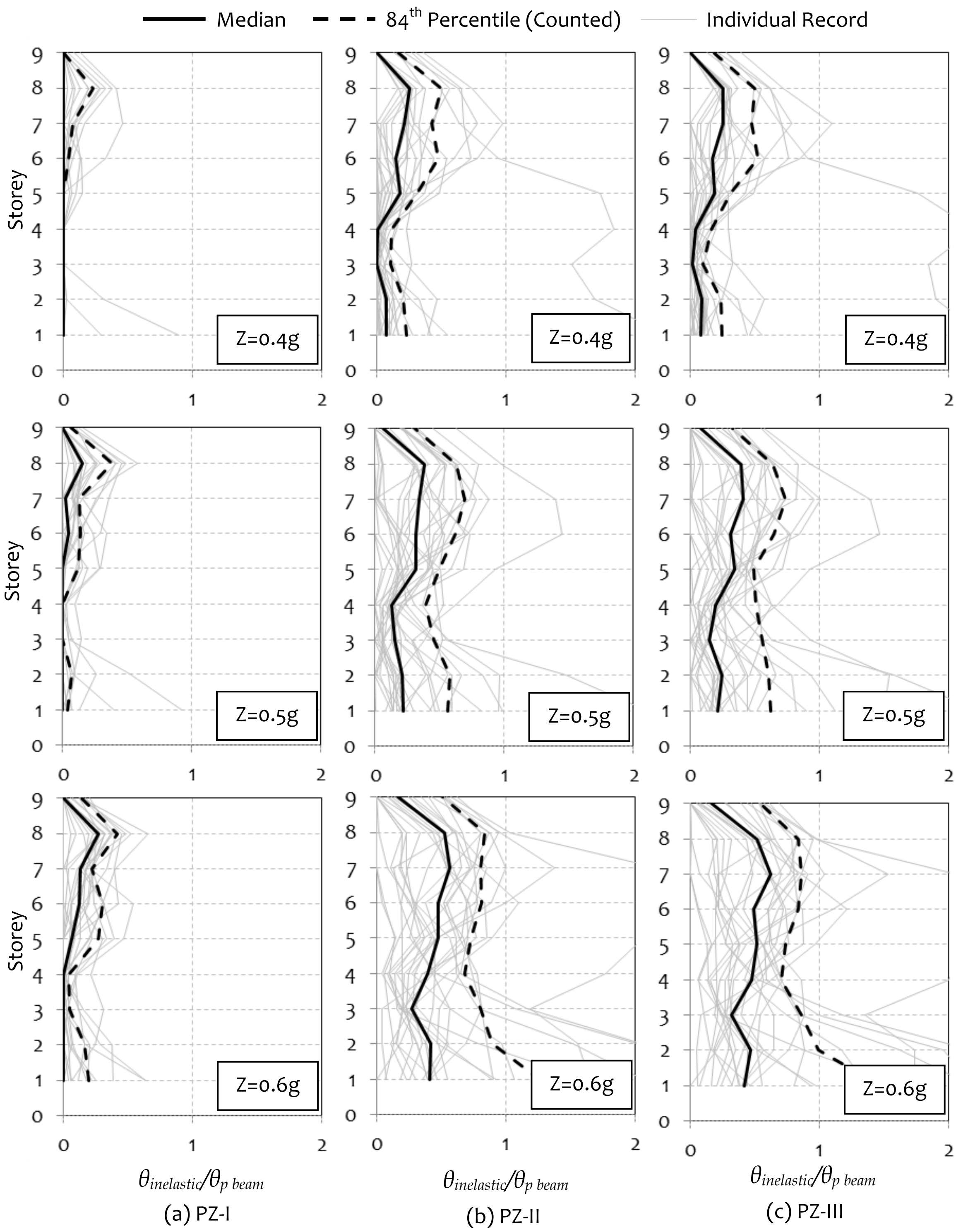


Figure 4.14: Inelastic rotational demand in interior beams of 3-storey study buildings as observed in nonlinear analysis. The inelastic rotational demand ($\theta_{inelastic}$) is normalized with modified plastic rotation capacity of the section ($\theta_{p\ beam} = 0.7\theta_{p\text{-}monotonic}$).

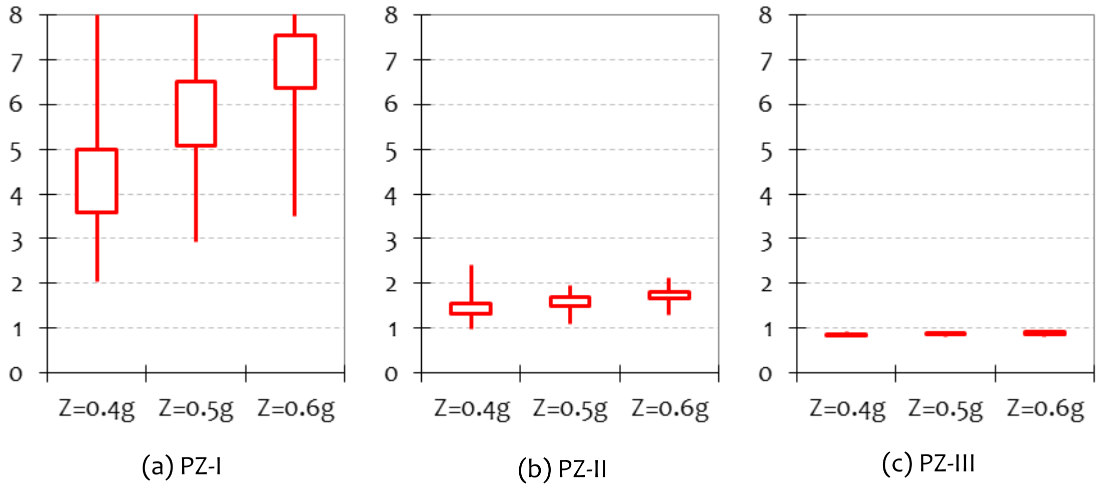


Figure 4.15: Range of maximum panel zone deformations of the 9-storey study building designed for Z=0.4g level of shaking.

4.4.5.5 Global Response of Study Buildings with Different Panel Zones

The results of nonlinear dynamic analyses under design level ground motions, indicate that the global response (interstorey drift) of steel SMFs is insensitive to the variation of panel zone design approach (*i.e.*, PZ-I, PZ-II, and PZ-III) adopted, especially for lower levels of shaking (up to Z=0.4g). This is because the dynamic characteristics (natural period) of the study buildings do not vary significantly. However, the variation in response becomes more and more apparent in buildings designed for higher levels of shaking. For instance, Figure 4.16 shows the response of 9-storey study building designed for Z=0.4g under the ground motion (scaled to Z=0.4g) recorded at Downey Company Maint Bldg station during the 1994 Northridge earthquake. It can be seen that the roof displacement, roof acceleration, and roof acceleration response spectrum (5% damped) have almost an exact match. However, Figure 4.14, which depicts the response of the 9-storey study building designed for Z=0.6g under the same ground motion, but scaled to Z=0.6g design response spectrum, indicates that the response becomes sensitive to the panel zone design approach. Hence, it is only when significant nonlinear action occurs in the building that the panel zone design approach begins to influence the global behaviour. Despite this, the three panel zone design methods considered in the study (*i.e.*, PZ-I, PZ-II, and PZ-III) do not lead to significant variation in the global frame response in general. This observation is reinforced from the results of Figures 4.18 to 4.25, where the maximum transient and residual drifts are seen to be similar for all the panel zone design approaches till Z=0.4g. However, the results begin to show slight variation for study buildings designed for Z=0.6g.

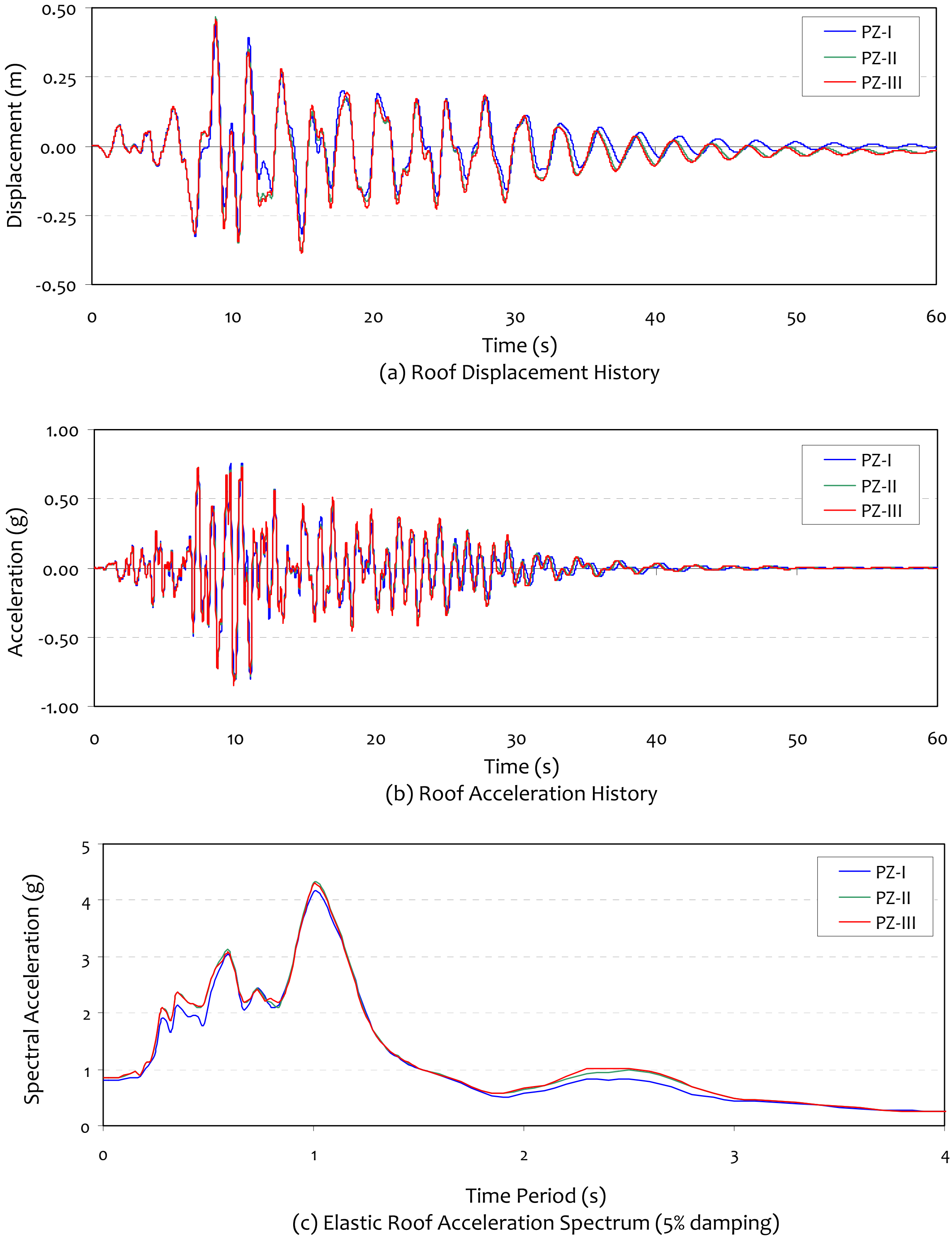
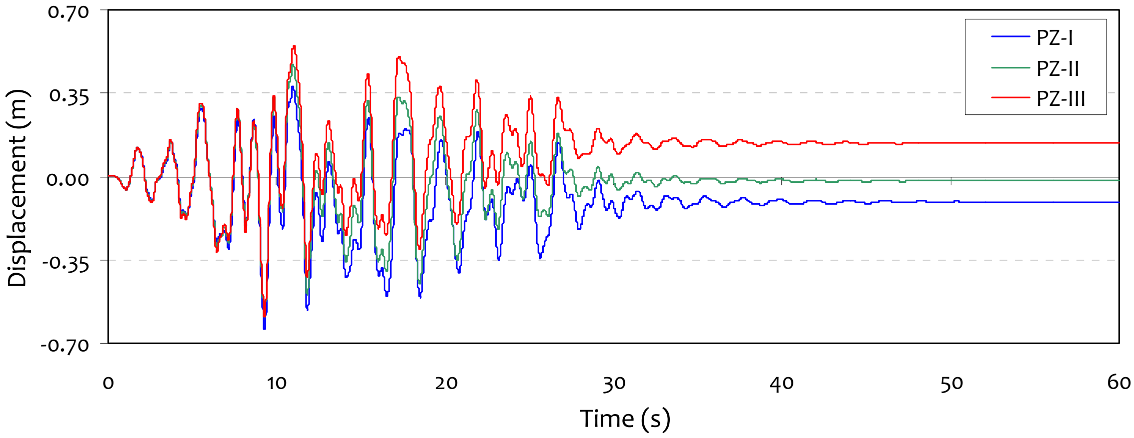
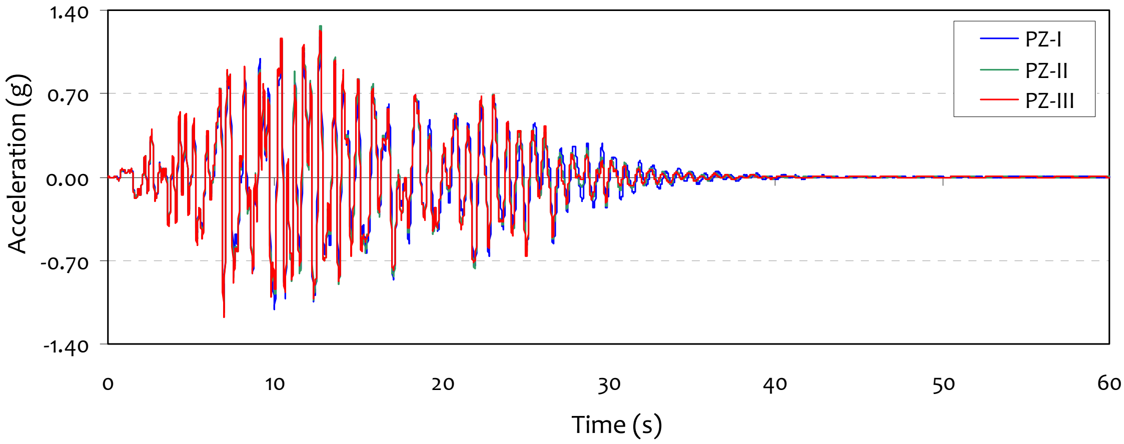


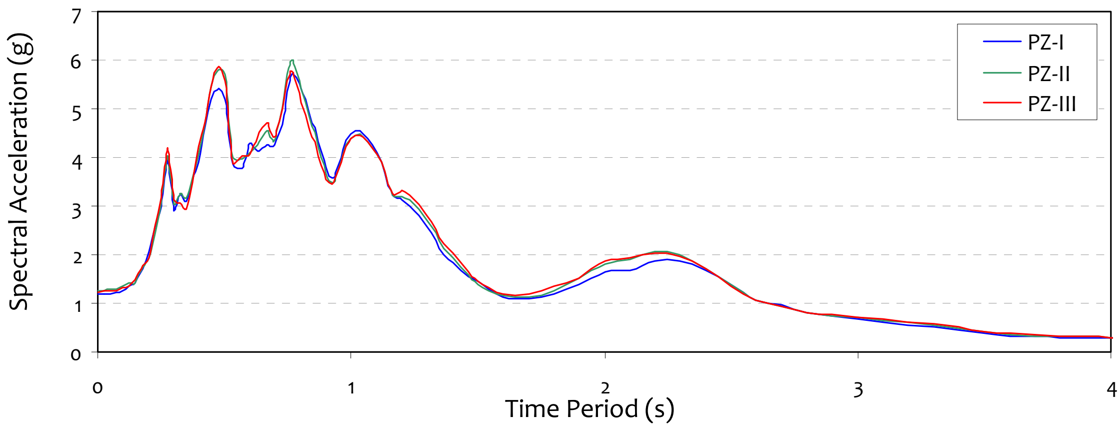
Figure 4.16: Response of 9-storey study building designed for Z=0.4g under the ground motion recorded at Downey Company Maint Bldg station during the 1994 Northridge earthquake, scaled to the design level.



(a) Roof Displacement History



(b) Roof Acceleration History



(c) Elastic Roof Acceleration Spectrum (5% damping)

Figure 4.17: Response of 9-storey study building designed for Z=0.6g under the ground motion recorded at Downey Company Maint Bldg station during the 1994 Northridge earthquake, scaled to the design level.

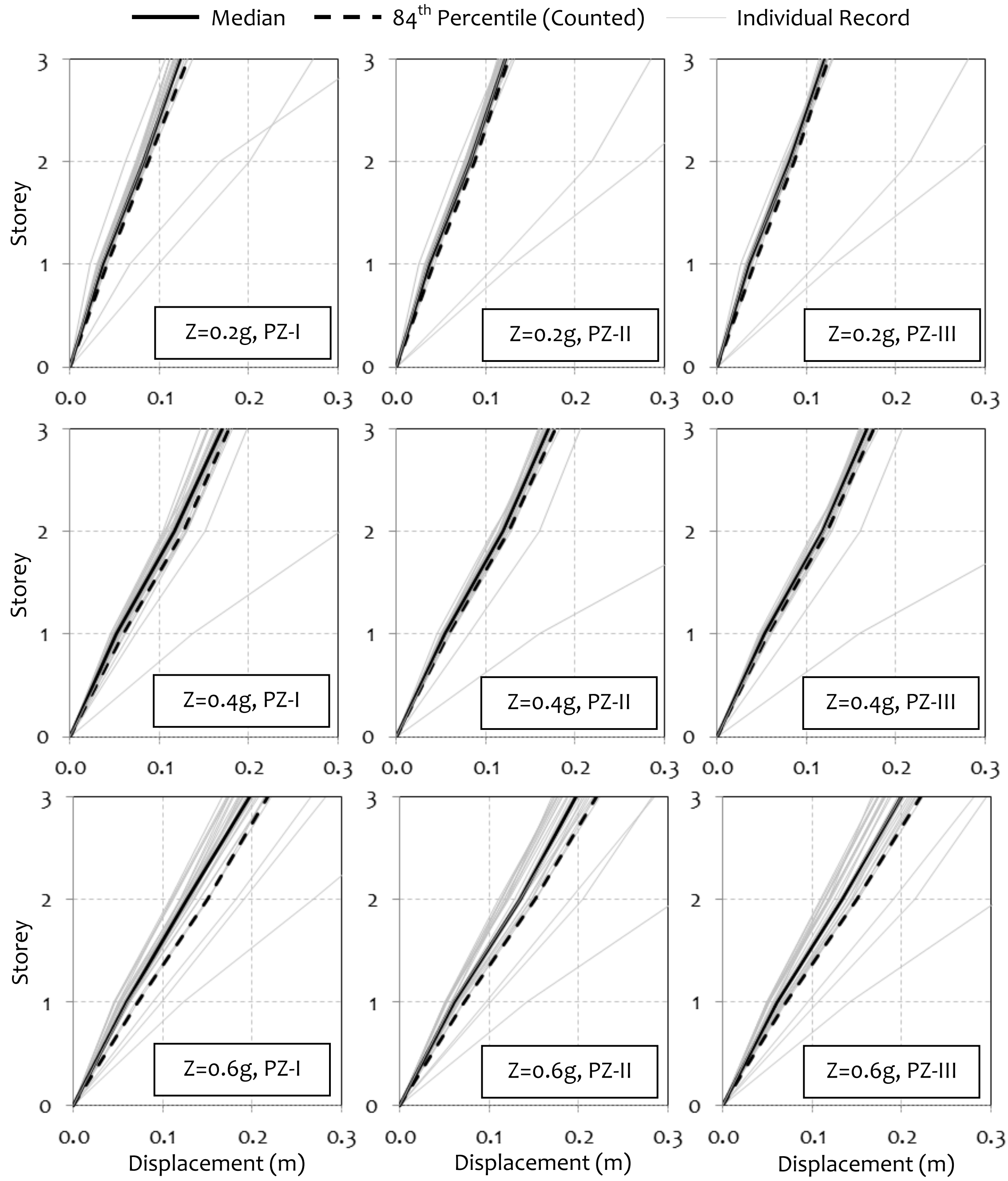


Figure 4.18: Peak transient displacement profile of the 3-storey study buildings with different panel zone design approaches and seismic hazard levels.

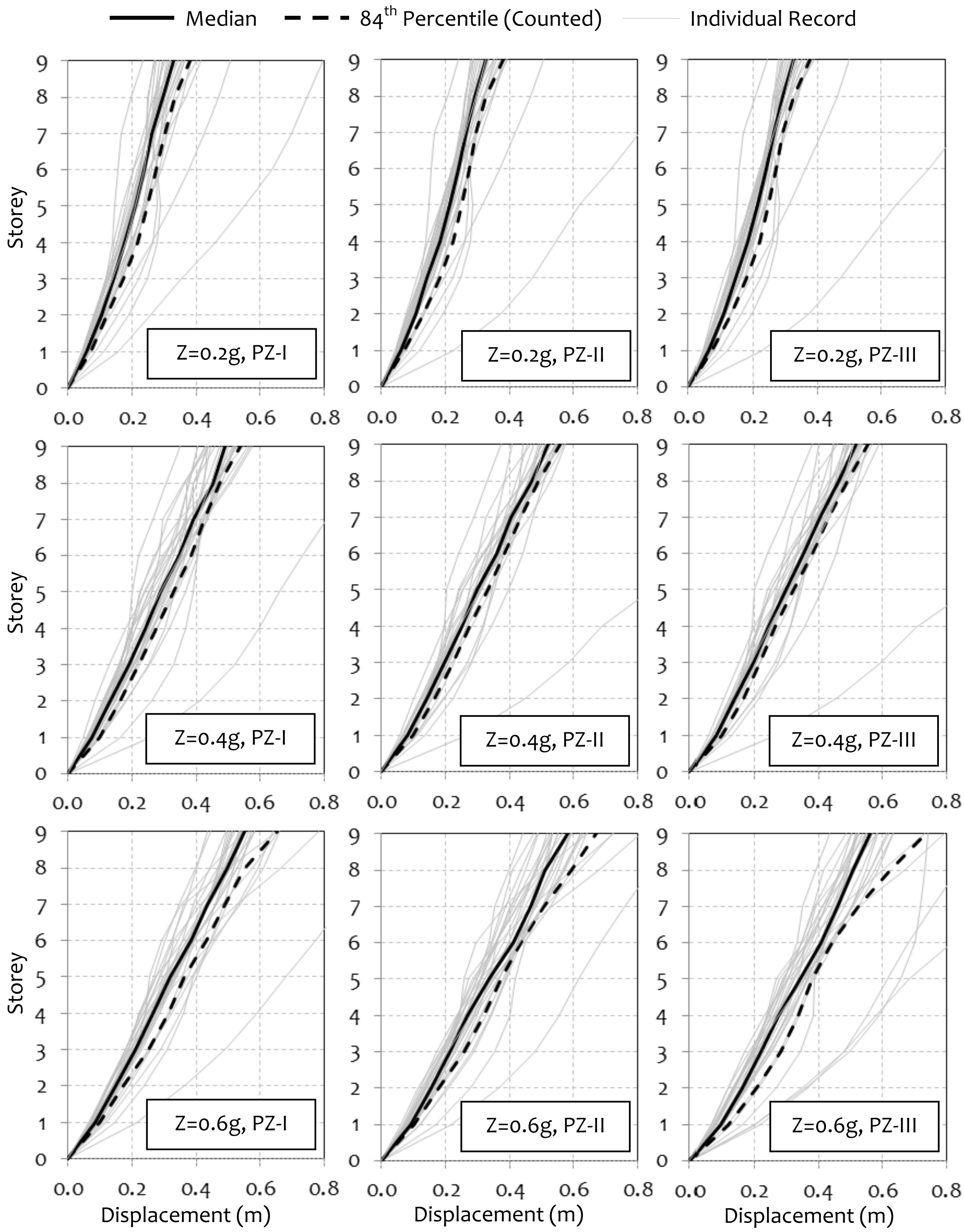


Figure 4.19: Peak transient displacement profile of the 9-storey study buildings with different panel zone design approaches and seismic hazard levels.

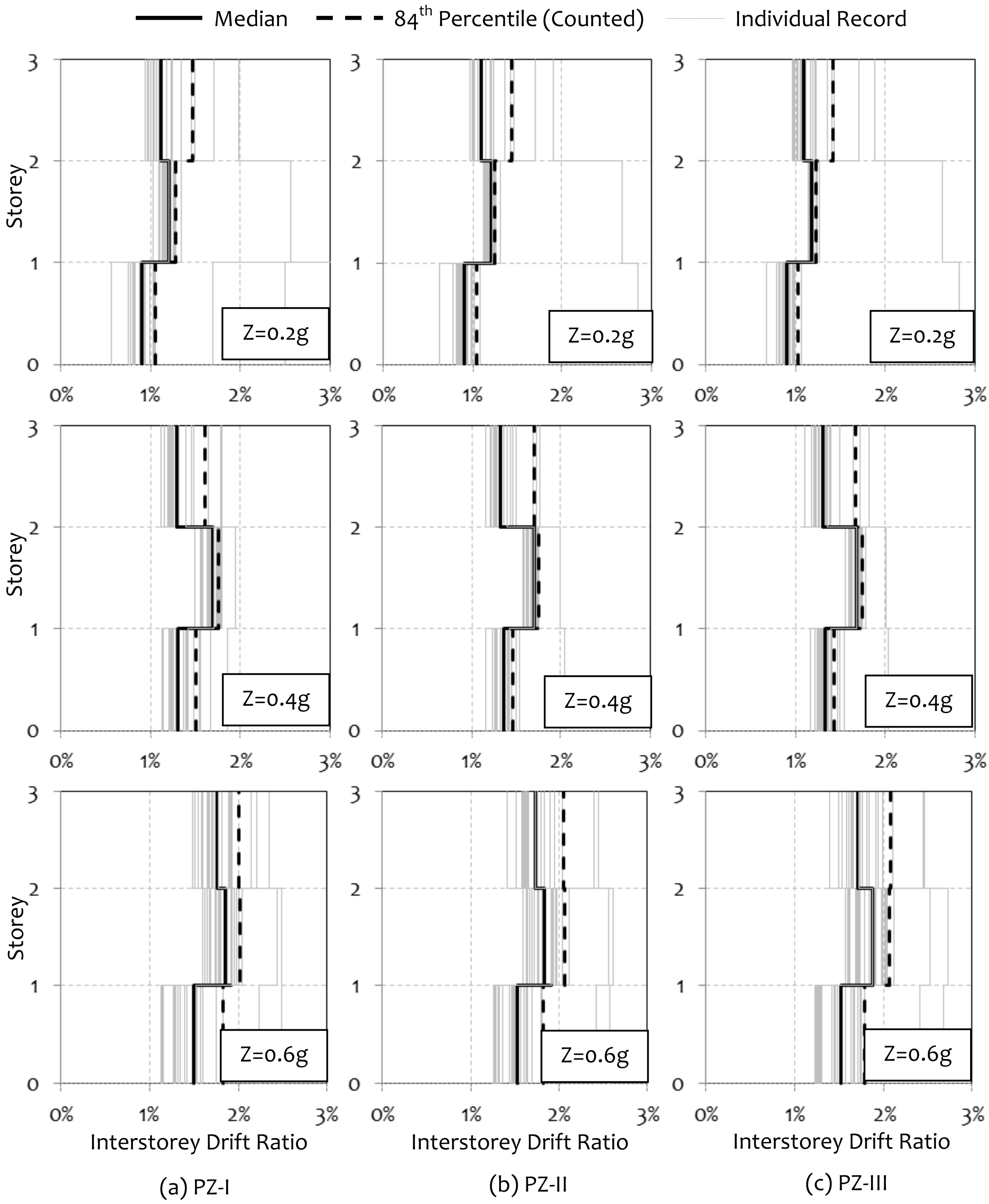


Figure 4.20: Peak transient interstorey drift profile of the 3-storey study buildings with different panel zone design approaches and seismic hazard levels.

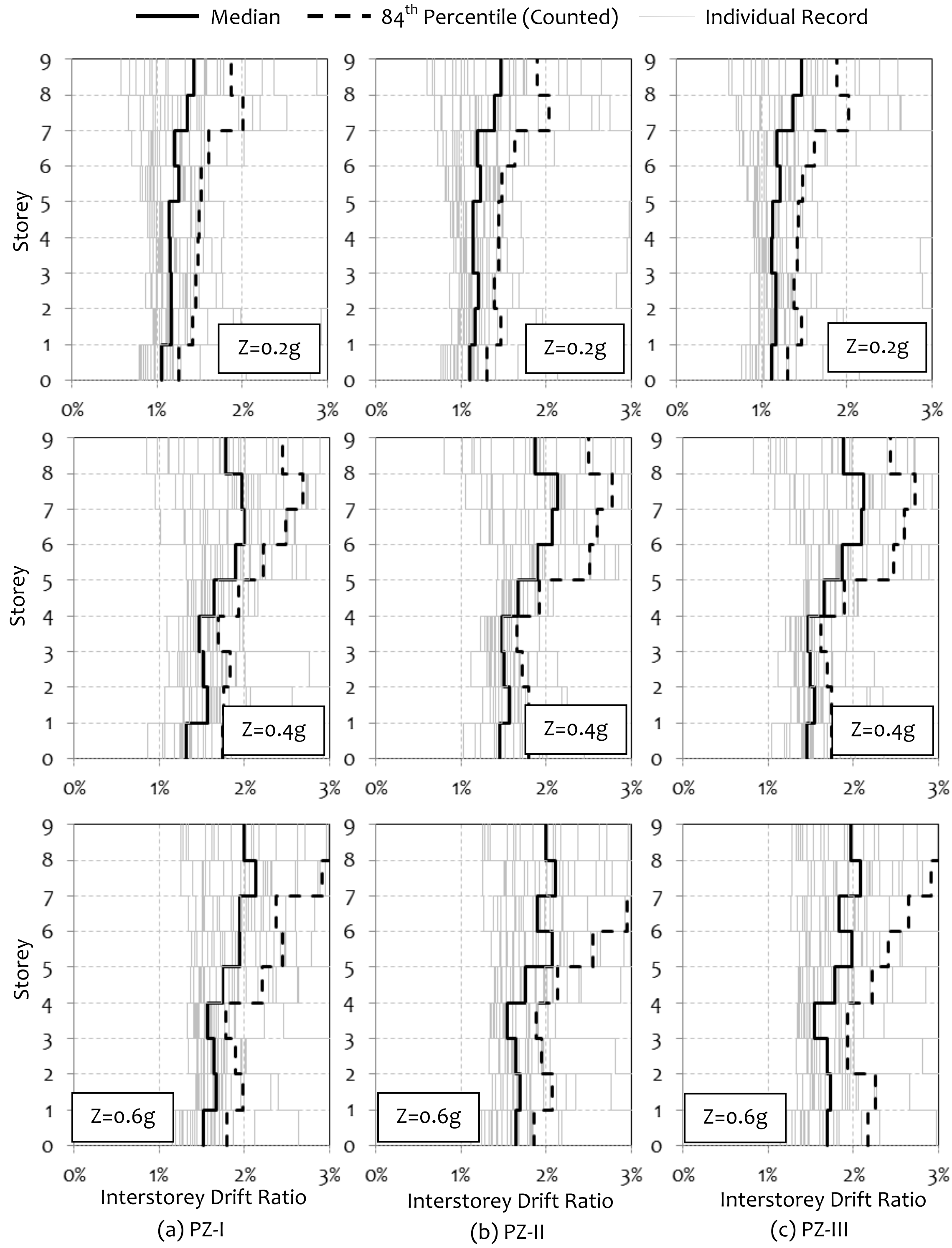


Figure 4.21: Peak transient interstorey drift profile of the 9-storey study buildings with different panel zone design approaches and seismic hazard levels.

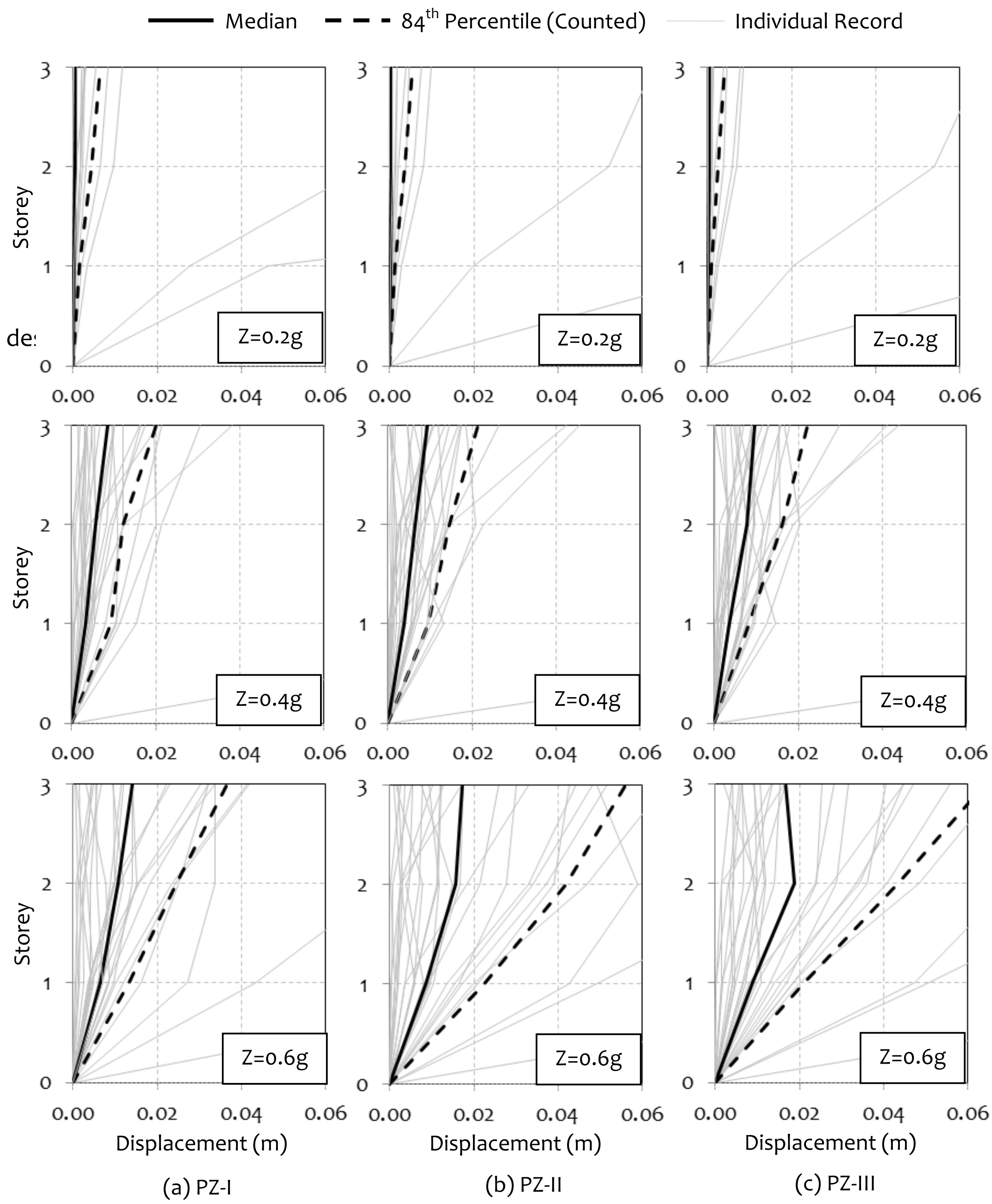


Figure 4.22: Residual displacement profile of the 3-storey study buildings with different panel zone design approaches and seismic hazard levels.

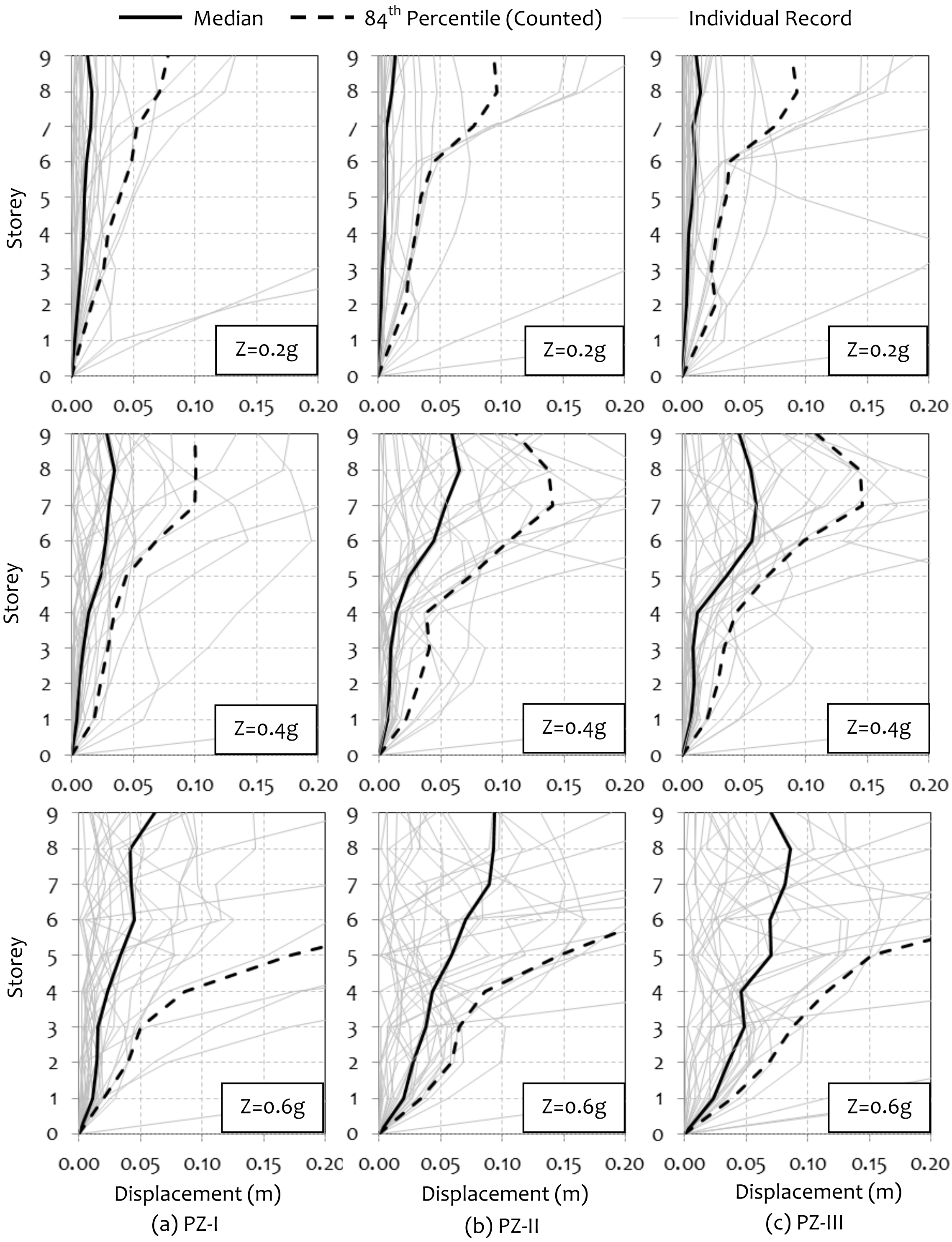
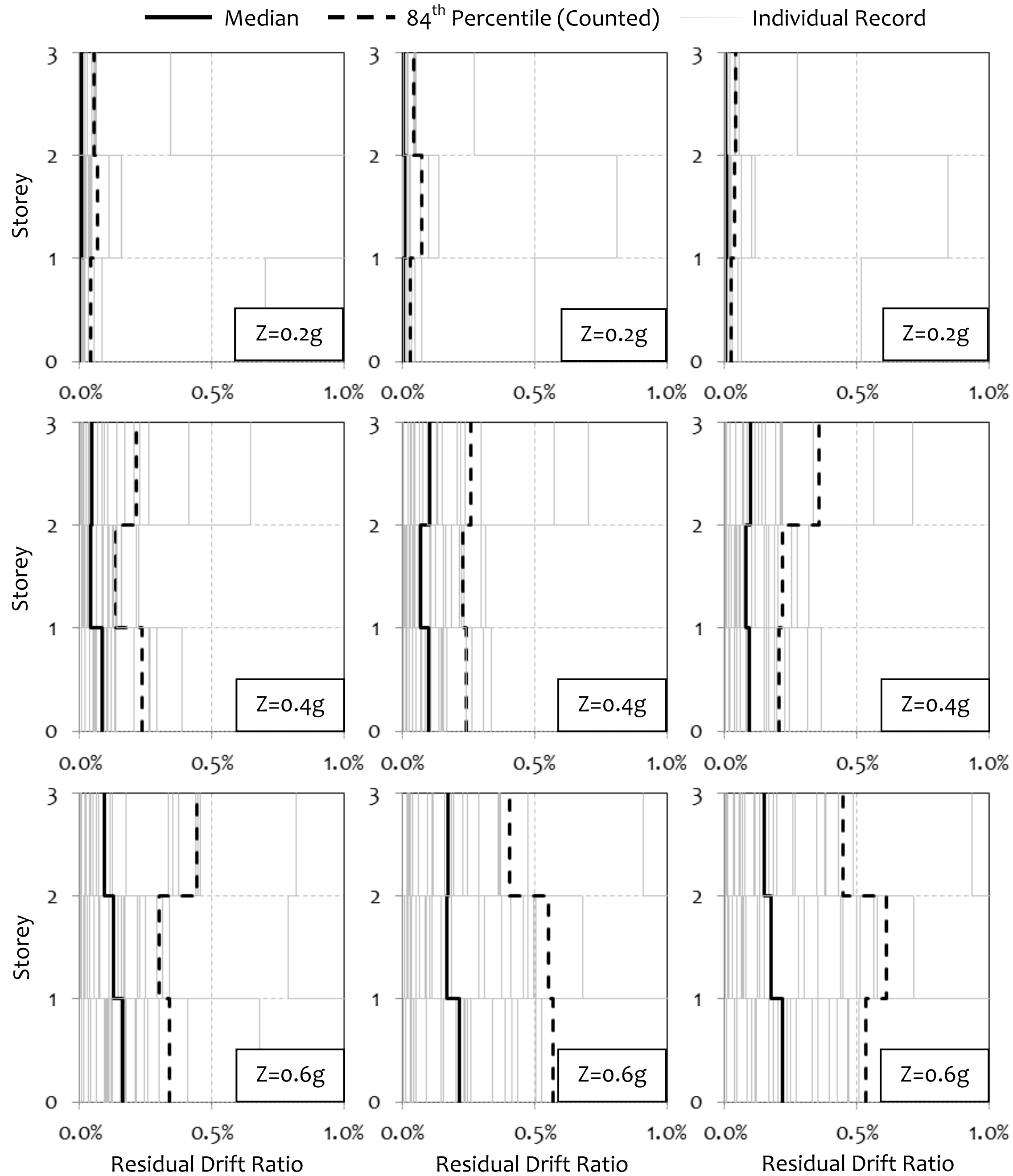


Figure 4.23: Residual displacement profile of the 9-storey study buildings with different panel zone design approaches and seismic hazard levels.



(a) PZ-I (b) PZ-II (c) PZ-III

Figure 4.24: Residual interstorey drift profile of the 3-storey study buildings with different panel zone design approaches and seismic hazard levels.

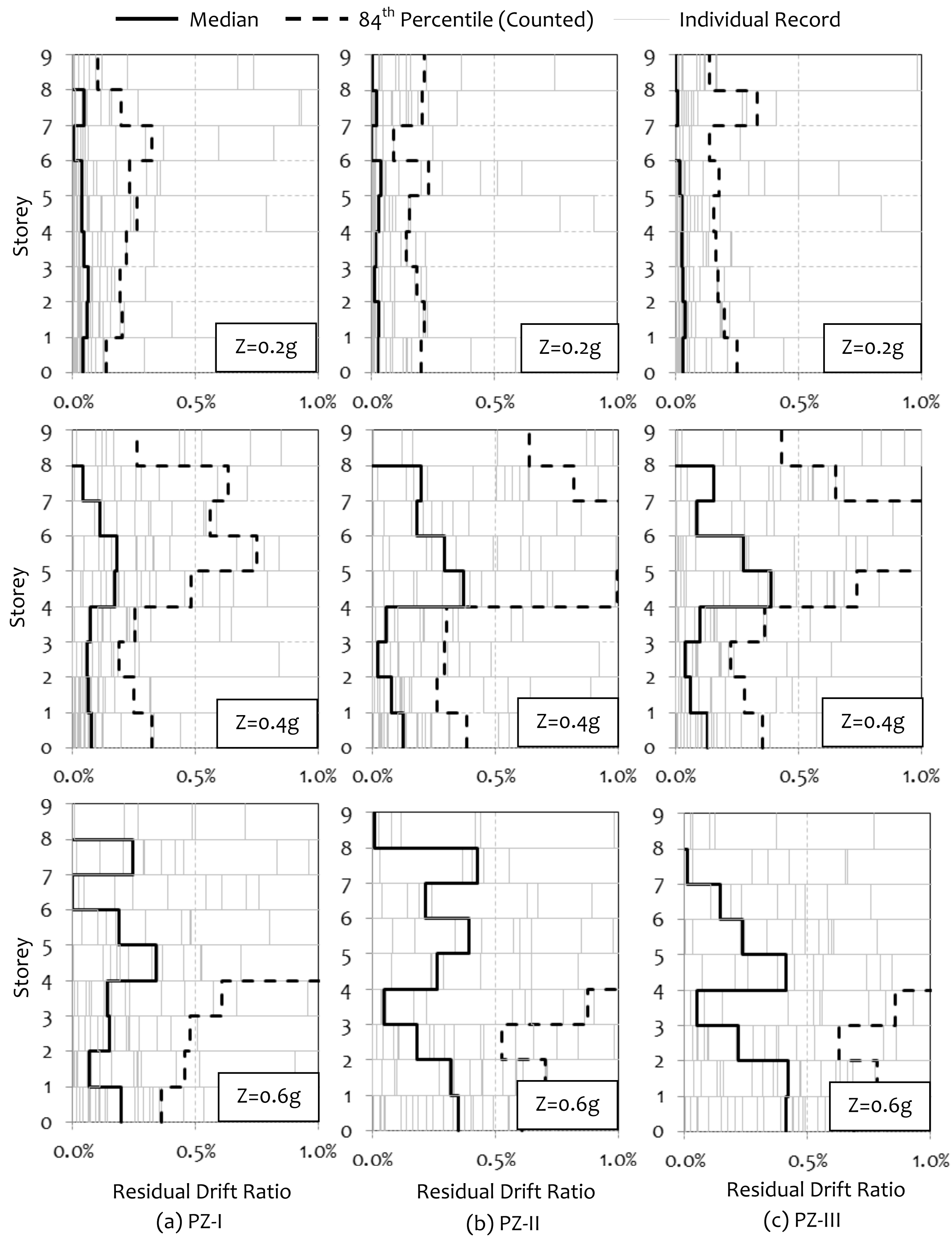


Figure 4.25: Residual interstorey drift profile of the 9-storey study buildings with different panel zone design approaches and seismic hazard levels.

4.6 Conclusions from Panel Zone Design Approach

The seismic behaviour of three panel zone design approaches (namely PZ-I, PZ-II, and PZ-III) is investigated, primarily at design level ground motions. The salient conclusions drawn from study are:

1. The panel zone shear demand of PZ-III design approach can safely be considered to be the upper bound estimate of shear demand imposed on panel zones in regular SMRF buildings upto 9-storeys and designed up to Z=0.6g.
2. PZ-I design approach, which is similar to the current AISC 341 (2016) design, can lead to weak panel zone behavior, especially for SMRF's designed for higher shaking intensity (e.g., Z=0.4g and above). Hence PZ-I approach need to be avoided in design practice, especially for connection configurations where yielding of panel zone can eventually lead to brittle fracture of connections.
3. PZ-II approach results in balanced panel zone behavior. Hence, PZ-II approach is suitable for SMRF's. However, as PZ-II approach exhibits balanced panel zone behaviour, it is certain that yielding of panel zone is bound to occur, especially for buildings designed for higher seismic hazard levels.
4. PZ-III approach results in strong panel zone behavior, which again results in desired seismic behavior, but at the price of having increased doubler plate thickness
5. The panel zone design approach adopted does not significantly affect the global seismic behaviour of the building, at least in terms of inter-storey and residual drift ratios, especially for steel SMFs designed for lower intensity shaking (Z=0.2g and 0.4g). However, for steel SMFs designed for higher intensities (Z=0.6g), the panel zone design approach adopted begins to influence the global behaviour owing to significant inelastic action.

...

Chapter 5

Column to Beam Strength Ratio Requirement

5.0 Introduction

An important design parameter that affects seismic behaviour of special moment frame (SMF) buildings is the minimum column-to-beam strength ratio (*CBSR*) requirement. It is by specifying a minimum *CBSR* requirement, seismic design codes aim to control the distribution of inelasticity between beams and columns in SMFs, once the panel zones are designed appropriately. This apportioning of damage is primarily intended to minimize the probability of *weak storey* mechanism during seismic activity, and not to entirely eliminate column yielding. Use of low *CBSR* value (of about 1) in design requires excessive doubler plate thickness (sometimes more than the thickness of column web), to achieve balanced panel zone behaviour. And, this also leads to extensive yielding of columns during severe seismic shaking. Further, the addition of doubler plates significantly increases the fabrication cost, and creates heat affected zones which can lead to unanticipated brittle modes of damage. Hence, in the current chapter, to begin with, a study is carried out to find the minimum *CBSR* requirement to eliminate the use of doubler plate in steel SMFs. Emphasis is given to arrive at the appropriate *CBSR* requirement for PZ-II design approach (Section 4.3), as this approach results in the most desirable frame behaviour, as discussed previously in Chapter 4.

Also, it is expected that the *CBSR* requirement to eliminate the use of doubler plates will be significantly large than the code requirement. An easy way to satisfy this high *CBSR* requirement would be to use higher grades of steel for columns. However, usage of higher grade steel will result in flexible buildings, leading to larger displacement demands. Hence, an investigation on the effectiveness of using higher grade steel for columns is also presented in this Chapter.

5.1 *CBSR* Required to Limit Column Yielding

Various definitions are used in the literature to estimate *CBSR*. Hence, it is important to use the appropriate minimum requirement of *CBSR* for each definition. American seismic provisions [AISC 341, 2016] define *CBSR* as the ratio of *nominal* plastic moment capacity of columns, considering axial-flexure interaction, to the *overstrength* plastic moment capacity of beam projected at column centreline, which is expressed as,

$$CBSR = \frac{\sum Z_{pc} F_{yc}\left(1 - \frac{P_r}{P_y}\right)}{\sum 1.1 R_y Z_{pb} F_{yb} + M_v}, \tag{5.1}$$

where Z_{pc} and Z_{pb} are the plastic section modulus of column and beam, respectively, F_{yc} and F_{yb} are minimum specified (characteristic) yield strength of column and beam, respectively, P_r the axial load in columns considering overstrength load combinations, P_y the tensile yield capacity of the column section, M_v the additional moment due to shear amplification, and R_y the material yield overstrength factor. It is reported in literature that a CBSR *greater than 1.5* as per Eq.(5.1) is required to *limit yielding* in columns for the highest design earthquake level recognised by the Japanese code [Nakashima and Sawaizumi, 2000]. However, a value greater the 1.0 as per Eq.(5.1) is found to be sufficient to *avoid weak storey* mechanism [AISC 341, 2016].

At the same time, the current Indian design provision [IS 800, 2007] define *CBSR* as the ratio of nominal plastic moment capacity of column to that of beam, which is expressed as,

$$CBSR = \frac{\sum Z_{pc} F_{yc}}{\sum Z_{pb} F_{yb}}. \tag{5.2}$$

CBSR value, computed as in Eq.(5.2), of greater than 2.6 and 3.9 are required to avoid weak storey mechanism and limit yielding of columns, respectively; this assumes axial load ratio (P_r/P_y) of 0.3, R_y fatcor of 1.5, and shear amplification of 10% of beam overstrength moment capacity. However, the current Indian provision requires *CBSR* calculated as per Eq. (5.2) to be only greater than 1.2. Hence, a thorough investigation of the minimum *CBSR* requirement is required to propose the required value of *CBSR* to be maintained in design, along with an appropriate improved definition of *CBSR*. As a reasoable definition of CBSR, and for the purpose of discussion, if not mentioned specifically hereinafter, *CBSR* is proposed to be computed as:

$$CBSR = \frac{\sum Z_{pc} F_{yc}}{\sum 1.1 R_y Z_{pb} F_{yb}}. \tag{5.3}$$

5.2 *CBSR* Required to Eliminate use of Doubler Plate

The current section presents ballpark *CBSR* values required in steel SMFs to eliminate the use of doubler plate which is used to strengthen the column web against shear. For the purpose of discussion, study buildings with geometric configuration similar to the ones described previously in Section 3.1 are considered. Again, *CBSR* requirement to eliminate the use of doubler plate for the three panel zone design approaches described previously in Section 4.2 (PZ-I, PZ-II and PZ-III) are only discussed, with emphasis on PZ-II approach.

Arriving at the exact *CBSR* required to eliminate the use of doubler plate in a steel SMF is a very specific problem, which depends upon the panel zone design approach adopted, the properties of beams and columns, building geometry (*i.e.*, bay span and storey height), and the connection configuration which dictates the location of plastic hinge in beams. Hence, the following discussion attempts to arrive at a *CBSR* value to be maintained to eliminate use of doubler plate for a particular case. Despite this, the resulting *CBSR* requirement will give a fair idea regarding the *CBSR* to be targeted during preliminary design, to eliminate the use of doubler plate altogether or to avoid the use of excessively thick doubler plates which will be difficult to weld.

Table 5.1 presents the *CBSR* required to eliminate the use of doubler plate for an interior panel zone, if PZ-II panel zone design approach is adopted in the study buildings presented previously in Section 3.1. Further, ASTM A36 (F_y=250 MPa, R_y=1.5) and ASTM A992 Grade 50 (F_y=345 MPa, R_y=1.1) steel are used for the beams and columns, respectively. A dash (as "-") in a cell in Table 5.1 indicates that the given combination of beam and column requires the use of doubler plate for interior panel zones. It can be observed that as the beam depth increases, the *CBSR* required decreases to eliminate the use of doubler plate. Such a trend is expected, since panel zone shear demand is inversely proportional to beam depth. Again, as the plastic section modulus of the section increases, the number of available column sections decreases which do not require additional doubler plate, owing to the increase in panel zone shear demand. In the above discussion, ASTM A36 steel (F_y=250 MPa, R_y=1.5) is considered for beams. But even if ASTM A992 steel (F_y=345 MPa, R_y=1.1) is considered for the beams in place of ASTM A36 steel, the resulting *CBSR* requirement does not change drastically as both steel grades have similar expected yield stress (*i.e.*, 250×1.5=375 MPa and 345×1.1=380MPa).

Table 5.1: *CBSR* required to eliminate the use of doubler plate, if PZ-II design approach is adopted for different combinations of beam and ASTM W14 column sections.

Beam	*Column*					
	W14×398	W14×426	W14×455	W14×500	W14×550	W14×605
W24×146	-	-	-	-	-	-
W24×131	-	-	-	-	-	-
W24×117	-	-	-	-	-	3.38
W24×104	-	-	-	-	3.41	3.82
W24×103	-	-	-	-	3.52	3.94
W24×94	-	-	-	3.46	3.89	4.35
W24×84	-	3.24	3.49	3.92	4.41	4.93
W24×76	3.35	3.63	3.91	4.39	4.93	5.52
W27×178	-	-	-	-	-	-
W27×161	-	-	-	-	-	-
W27×146	-	-	-	-	-	-
W27×129	-	-	-	-	-	2.79
W27×114	-	-	-	-	2.88	3.22
W27×102	-	-	-	2.88	3.24	3.62
W27×94	-	-	2.82	3.16	3.55	3.97
W27×84	-	2.98	3.21	3.60	4.04	4.52
W30×173	-	-	-	-	-	-
W30×148	-	-	-	-	-	-
W30×132	-	-	-	-	-	-
W30×124	-	-	-	-	2.00	2.71
W30×116	-	-	-	-	2.61	2.92
W30×108	-	-	-	2.54	2.85	3.19
W30×99	-	-	2.51	2.81	3.16	3.54
W30×90	-	2.57	2.77	3.10	3.49	3.90

Along the same lines, the range of minimum *CBSR* required to be maintained for wide flange steel W14 and W27 column sections for various beam depths, corresponding to the three panel zone design approaches are listed in Table 5.2. It can be seen that, *CBSR* requirement is least for PZ-I (weak panel zone) design, and the maximum for PZ-III (strong panel zone) design. Designs following PZ-III approach, which does not require doubler plate, will be uneconomical owing to the large *CBSR* requirement. Further, as PZ-III approach results in higher inelastic demands in beams, it is preferable adopt PZ-II approach (balanced panel zone) where the *CBSR* requirement to eliminate the use of doubler plate is also reasonable. Additionally, the *CBSR* values required to eliminate the use of doubler plate in PZ-II approach also coincides well with *CBSR* to be maintained to reduce the probability of column yielding at severe shaking intensities as reported in literature [Medina and Krawinkler, 2006]. Finally, the range of minimum *CBSR* required to be maintained to eliminate the use of doubler design following PZ-II approach for typical ASTM W14, W24 or W27 sections is listed in Table 5.3 for ASTM W24, W27, W30, W33 or W36 beam sections. The results indicate that, on average, a *CBSR* of 2.5 will be sufficient enough to eliminate the use of doubler plate altogether or to avoid the use of excessively thick doubler plates in steel SMFs. Hence, a value of 2.5 is recommended for preliminary proportioning of steel SMFs when PZ-II approach is used for panel zone design.

Table 5.2: Range of minimum *CBSR* required to eliminate the use of doubler plate for different combinations of beam and ASTM W14 & W27 column sections, for the three panel zone design approaches.

Beam	*W14 Column*			*W27 Column*		
	PZ-I	PZ-II	PZ-III	PZ-I	PZ-II	PZ-III
W21	1.7 - 2.8	3.2 - 4.3	3.9 - 4.6	3.0 - 3.7	4.5 - 5.0	5.2 - 5.9
W24	1.5 - 2.4	2.7 - 3.3	3.4 - 4.2	2.6 - 3.4	3.7 - 4.4	4.6 - 5.1
W27	1.4 - 2.1	2.4 - 3.0	3.3 - 3.7	2.2 - 2.9	3.2 - 3.5	4.1 - 4.2
W30	1.3 - 1.8	2.2 - 2.6	2.7 - 3.2	2.1 - 2.5	2.8 - 3.2	4
W33	1.1 - 1.5	2.0 - 2.1	2.7	1.8 - 2.1	2.5 - 2.8	-
W36	1.0 - 1.3	1.6 - 1.9	-	1.7 -1.9	2.2 - 2.4	-
W40	0.9 - 1.2	1.5 - 1.8	-	1.6 - 1.8	2.0 - 2.2	-

Table 5.3: Range of minimum *CBSR* required to eliminate the use of doubler plate, if PZ-II design approach is adopted in the study building for different combinations of beam sections for ASTM W14, W24 and W27 column sections

Beam	*Column*		
	W14	W24	W27
W21	3.2 - 4.3	4.1 - 4.5	4.5 - 5.0
W24	2.7 - 3.3	3.5 -4.2	3.7 - 4.4
W27	2.4 - 3.0	2.9 - 3.2	3.2 - 3.5
W30	2.2 - 2.6	2.6 -2.8	2.8 - 3.2
W33	2.0 - 2.1	2.2 - 2.3	2.5 - 2.8
W36	1.6 - 1.9	2.0 - 2.1	2.2 - 2.4
W40	1.5 - 1.8	1.8 - 2.0	2.0 - 2.2

5.3 Higher Grade Steel for Columns in SMFs

A minimum *CBSR* of 2.5 is required for initial proportioning of members in SMFs and to eliminate the use of doubler plate altogether or to restrict the doubler plate thickness to reasonable values for PZ-II design approach. Further, this value also comes close to the median *CBSR* recommended in literature to avoid column yielding at severe shaking intensities [Medina and Krawinkler, 2006; Zaghi *et.al.*, 2015]. However, a minimum *CBSR* requirement of 2.5 seems high. A pragmatic way to satisfy this seemingly high *CBSR* requirement is to use higher grade of steel for columns. Hence, in the current section, effectiveness of using higher grade steel for columns is investigated with the objective to find the suitable grade of column to be used in steel SMFs.

5.3.1 Methodology

The following methodology is adopted to study the effectiveness of using higher grade steel for columns in steel SMFs:

1. The 3- and 9-storey study buildings are redesigned as per Indian codes [IS 800, 2007; IS 1893 Part-1, 2016], with the actual material yield stress for beams and PZ-III panel zone design approach (refer Section 4.3) are considered in the design. PZ-III approach is considered instead of PZ-II approach, as this will make the columns more vulnerable towards yielding;
2. To enable meaningful comparison of results of study buildings with different grades of steel for columns, the *CBSR* is kept nearly constant by changing the column sections appropriately; the beam sections are kept unchanged;
3. Seismic response of the study buildings are evaluated by nonlinear dynamic analyses (similar analyses as described in Section 3.2).

The scope of the above investigation is limited to three grades of steel which are commonly manufactured, *i.e.*, steels with yield stress of 250, 345, and 450 MPa, hereafter referred as 250, 345, and 450 grade steels, respectively. In these analyses, the material uncertainty factor (R_y) is not considered for columns to obtain upper bound results.

5.3.2 Study Buildings

The geometric configuration of the 3- and 9-storey office buildings considered is same as presented in Section 3.1. To begin with, study buildings with 345 grade columns and 250 grade beams are designed for three levels of seismic shaking (Z=0.2g, 0.4g, and

0.6g). Thereafter, the columns of the benchmark buildings (*i.e.*, study buildings with grade 345 grade columns), are replaced with 250 and 450 grade steel columns. While changing the grade of steel of columns, it is ensured that the *CBSR* is kept nearly constant. Hence, study buildings with 250 grade steel for columns will be stiffer than the study buildings with 450 grade columns. Thus, the dataset of results of the study buildings will also allow evaluating the effect of varying *column to beam stiffness ratio* on the seismic behaviour, while *CBSR* is kept unchanged. It is also ensured that all the study buildings do meet the drift requirement specified in the Indian code [IS 1893 Part-1, 2016]. The details of the 3- and 9-storey buildings are listed in Tables 5.4 to 5.6, and Table 5.7 to 5.9, respectively. A total of 18 study buildings are assessed to determine the suitable grade of steel for columns in SMFs.

Table 5.4: Details of beams, columns, and doubler plates of 3-storey study buildings designed for Z = 0.2g

Z=0.2g	**Beam**	**Column**	***CBSR***	**Doubler Plate Thickness (mm)**	
3-Storey				**Interior**	**Exterior**
Column grade: 250 MPa					
3	W 21 × 57	W 14 × 283	1.27	29	0
2	W 24 × 76	W 14 × 283	1.64	52	11
1	W 24 × 76	W 14 × 283	1.64	52	11
Building Natural Period: 1.270 s					
Column grade: 345 MPa					
3	W 21 × 57	W 14 × 211	1.26	23	0
2	W 24 × 76	W 14 × 211	1.63	40	9
1	W 24 × 76	W 14 × 211	1.63	40	9
Building Natural Period: 1.368 s					
Column grade: 450 MPa					
3	W 21 × 57	W 14 × 159	1.21	19	1
2	W 24 × 76	W 14 × 159	1.57	33	8
1	W 24 × 76	W 14 × 159	1.57	33	8
Building Natural Period: 1.474 s					

Table 5.5: Details of beams, columns, and doubler plates of 3-storey study buildings designed for Z = 0.4g

Z=0.4g	Beam	Column	*CBSR*	Doubler Plate Thickness (mm)	
3-Storey				Interior	Exterior
Column grade: 250 MPa					
3	W 24 × 76	W 14 × 455	1.42	23	0
2	W 27 × 114	W 14 × 455	1.65	62	6
1	W 27 × 114	W 14 × 455	1.65	62	6
Building Natural Period: 0.896 s					
Column grade: 345 MPa					
3	W 24 × 76	W 14 × 342	1.41	19	0
2	W 27 × 114	W 14 × 342	1.64	50	6
1	W 27 × 114	W 14 × 342	1.64	50	6
Building Natural Period: 0.975 s					
Column grade: 450 MPa					
3	W 24 × 76	W 14 × 257	1.33	18	0
2	W 27 × 114	W 14 × 257	1.55	43	7
1	W 27 × 114	W 14 × 257	1.55	43	7
Building Natural Period: 1.062 s					

Table 5.6: Details of beams, columns, and doubler plates of 3-storey study buildings designed for Z = 0.6g

Z=0.6g	Beam	Column	*CBSR*	Doubler Plate Thickness (mm)	
3-Storey				Interior	Exterior
Column grade: 250 MPa					
3	W 24 × 84	W 14 × 605	1.79	9	0
2	W 33 × 147	W 14 × 605	1.56	62	0
1	W 33 × 147	W 14 × 605	1.56	62	0
Building Natural Period: 0.736 s					
Column grade: 345 MPa					
3	W 24 × 84	W 14 × 455	1.75	9	0
2	W 33 × 147	W 14 × 455	1.52	51	1
1	W 33 × 147	W 14 × 455	1.52	51	1
Building Natural Period: 0.799 s					
Column grade: 450 MPa					
3	W 24 × 84	W 14 × 370	1.79	7	0
2	W 33 × 147	W 14 × 370	1.56	41	0
1	W 33 × 147	W 14 × 370	1.56	41	0
Building Natural Period: 0.851 s					

Table 5.7: Details of beams, columns, and doubler plates of 9-storey study buildings designed for Z = 0.2g

Z=0.2g	Beam	Column	*CBSR*	Doubler Plate Thickness (mm)	
9-Storey				**Interior**	**Exterior**
Column grade: 250 MPa					
9	W 21 × 57	W 14 × 342	1.58	19	0
8	W 21 × 57	W 14 × 342	3.16	19	0
7	W 27 × 84	W 14 × 342	1.67	49	6
6	W 27 × 84	W 14 × 342	1.67	49	6
5	W 27 × 84	W 14 × 370	1.83	44	2
4	W 24 × 104	W 14 × 370	1.54	71	16
3	W 24 × 104	W 14 × 370	1.54	71	16
2	W 24 × 104	W 14 × 370	1.54	71	16
1	W 30 × 99	W 14 × 398	1.56	53	5
Building Natural Period: 3.081 s					
Column grade: 345 MPa					
9	W 21 × 57	W 14 × 257	1.58	15	0
8	W 21 × 57	W 14 × 257	3.16	15	0
7	W 27 × 84	W 14 × 257	1.67	38	5
6	W 27 × 84	W 14 × 257	1.67	38	5
5	W 27 × 84	W 14 × 283	1.86	34	2
4	W 24 × 104	W 14 × 283	1.57	55	12
3	W 24 × 104	W 14 × 283	1.57	55	12
2	W 24 × 104	W 14 × 283	1.57	55	12
1	W 30 × 99	W 14 × 311	1.62	40	3
Building Natural Period: 3.256 s					
Column grade: 450 MPa					
9	W 21 × 57	W 14 × 193	1.50	14	0
8	W 21 × 57	W 14 × 193	3.00	14	0
7	W 27 × 84	W 14 × 193	1.59	33	6
6	W 27 × 84	W 14 × 193	1.59	33	6
5	W 27 × 84	W 14 × 233	1.95	27	1
4	W 24 × 104	W 14 × 233	1.65	43	9
3	W 24 × 104	W 14 × 233	1.65	43	9
2	W 24 × 104	W 14 × 233	1.65	43	9
1	W 30 × 99	W 14 × 233	1.52	34	5
Building Natural Period: 3.457 s					

Table 5.8: Details of beams, columns, and doubler plates of 9-storey study buildings designed for Z = 0.4g

Z=0.4g	Beam	Column	*CBSR*	Doubler Plate Thickness (mm)	
9-Storey				Interior	Exterior
Column grade: 250 MPa					
9	W 21 × 83	W 14 × 426	1.34	35	0
8	W 21 × 83	W 14 × 426	2.69	35	0
7	W 24 × 117	W 14 × 426	1.61	74	14
6	W 24 × 117	W 14 × 426	1.61	74	14
5	W 24 × 117	W 14 × 550	2.19	53	0
4	W 30 × 132	W 14 × 550	1.64	63	2
3	W 30 × 132	W 14 × 550	1.64	63	2
2	W 30 × 132	W 14 × 550	1.64	63	2
1	W 30 × 148	W 14 × 605	1.60	69	3
Building Natural Period: 2.424 s					
Column grade: 345 MPa					
9	W 21 × 83	W 14 × 311	1.29	30	0
8	W 21 × 83	W 14 × 311	2.57	30	0
7	W 24 × 117	W 14 × 311	1.54	61	14
6	W 24 × 117	W 14 × 311	1.54	61	14
5	W 24 × 117	W 14 × 398	2.05	45	1
4	W 30 × 132	W 14 × 398	1.53	54	5
3	W 30 × 132	W 14 × 398	1.53	54	5
2	W 30 × 132	W 14 × 398	1.53	54	5
1	W 30 × 148	W 14 × 455	1.57	57	4
Building Natural Period: 2.586 s					
Column grade: 450 MPa					
9	W 21 × 83	W 14 × 257	1.36	23	0
8	W 21 × 83	W 14 × 257	2.71	23	0
7	W 24 × 117	W 14 × 257	1.62	47	10
6	W 24 × 117	W 14 × 257	1.62	47	10
5	W 24 × 117	W 14 × 311	2.01	38	2
4	W 30 × 132	W 14 × 311	1.51	45	5
3	W 30 × 132	W 14 × 311	1.51	45	5
2	W 30 × 132	W 14 × 311	1.51	45	5
1	W 30 × 148	W 14 × 370	1.61	46	3
Building Natural Period: 2.733 s					

Table 5.9: Details of beams, columns, and doubler plates of 9-storey study buildings designed for Z = 0.6g

Z=0.6g	Beam	Column	*CBSR*	Doubler Plate Thickness (mm)	
9-Storey				Interior	Exterior
Column grade: 250 MPa					
9	W 24 × 84	W 14 × 550	1.60	17	0
8	W 24 × 84	W 14 × 550	3.19	17	0
7	W 33 × 130	W 14 × 550	1.53	60	1
6	W 33 × 130	W 14 × 550	1.53	60	1
5	W 33 × 130	W 14 × 730	2.15	31	0
4	W 36 × 160	W 14 × 730	1.61	57	0
3	W 36 × 160	W 14 × 730	1.61	57	0
2	W 36 × 160	W 14 × 730	1.61	57	0
1	W 36 × 182	W 14 × 730	1.40	77	1
Building Natural Period: 1.914 s					
Column grade: 345 MPa					
9	W 24 × 84	W 14 × 426	1.62	13	0
8	W 24 × 84	W 14 × 426	3.24	13	0
7	W 33 × 130	W 14 × 426	1.56	47	0
6	W 33 × 130	W 14 × 426	1.56	47	0
5	W 33 × 130	W 14 × 550	2.11	27	0
4	W 36 × 160	W 14 × 550	1.58	48	0
3	W 36 × 160	W 14 × 550	1.58	48	0
2	W 36 × 160	W 14 × 550	1.58	48	0
1	W 36 × 182	W 14 × 605	1.54	55	0
Building Natural Period: 2.025 s					
Column grade: 450 MPa					
9	W 24 × 84	W 14 × 342	1.64	11	0
8	W 24 × 84	W 14 × 342	3.27	11	0
7	W 33 × 130	W 14 × 342	1.57	38	0
6	W 33 × 130	W 14 × 342	1.57	38	0
5	W 33 × 130	W 14 × 455	2.19	20	0
4	W 36 × 160	W 14 × 455	1.64	37	0
3	W 36 × 160	W 14 × 455	1.64	37	0
2	W 36 × 160	W 14 × 455	1.64	37	0
1	W 36 × 182	W 14 × 500	1.60	43	0
Building Natural Period: 2.138 s					

5.3.3 Results of Nonlinear Analyses

The column material strain ductility demand, inter-storey drift profile, and beam plastic rotation demand, as obtained from the results of nonlinear dynamic analyses of the study buildings, are presented and discussed in this section. As earlier, median and counted 84th percentile values are used to represent seismic demands imposed.

5.3.3.1 3-Storey Buildings

The column material strain ductility demands, from the results of nonlinear dynamic analyses, are depicted in Figure 5.1. It can be inferred that, as the grade of steel of columns increases, the column ductility demand decreases. However, the strain demand in the columns increases with increasing grade of steel. This is because, higher grade steel has higher yield strain. The comparatively higher strain demand in columns of buildings with higher grade steel in columns is expected, since those buildings are relatively flexible compared to the buildings with lower grade of steel in columns. It is interesting to note that the columns do remain elastic for almost all ground motions, even in the buildings designed for Z=0.6g level of shaking. Hence, a *CBSR* around 1.5 is sufficient to eliminate the possibility of yielding of columns under design earthquake for regular 3-storey steel SMFs. But, the column bases do yield significantly, especially for buildings designed for higher shaking intensity with lower grades of steel in columns.

Figure 5.2 shows the interstorey drift profile of the buildings as observed in nonlinear dynamic analyses. As expected, the interstorey drift is higher for the flexible buildings, *i.e.*, the buildings with higher grade of steel in columns. Weak storey mechanisms do not occur, since the *CBSR* is well above 1. Finally, the inelastic rotation demands in beams are shown in Figure 5.3. There is no significant variation in the rotation demands in the beams. However, the inelastic rotation demands are slightly higher in the beams of flexible buildings, owing to the relatively higher interstorey drifts demand that they are subjected to during seismic shaking. Further the beam inelastic rotational demands do increase in buildings designed for higher level of seismic shaking, as expected. Hence, from the nonlinear dynamic analyses results, it can be concluded that a *CBSR* of 1.5 is sufficient enough to eliminate yielding of columns under design earthquakes upto a shaking intensity corresponding to Z=0.6g for 3-storey regular SMFs. However, column bases do yield at *CBSR*s of about 1.5. It seems preferable to use higher grade of steel for columns, which allows limiting the thickness of panel zone and excessive yielding at the column base. However, use of 450 grade steel for columns will result in increased rotation demands since the buildings would be relatively very flexible; the same is not advisable.

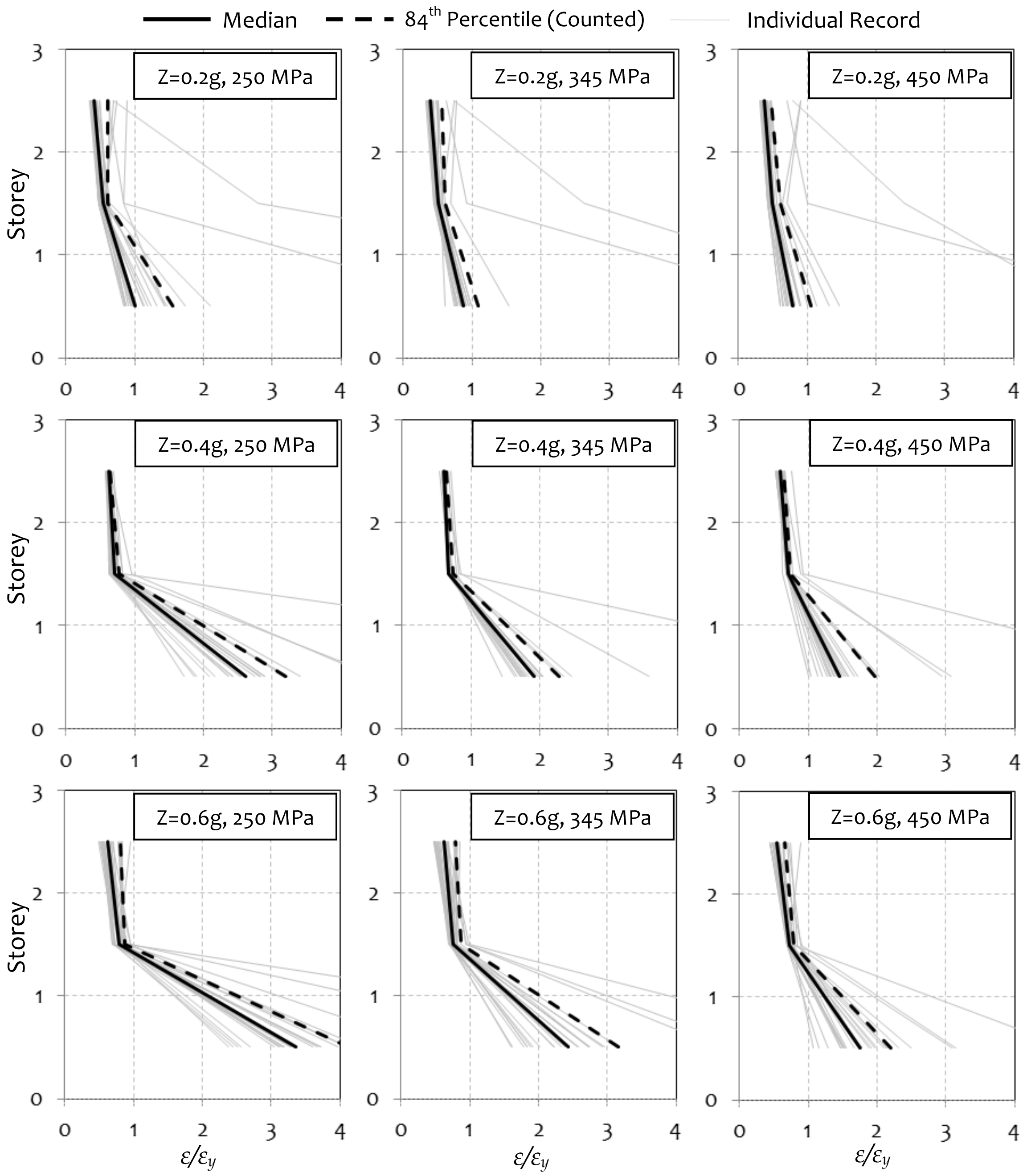


Figure 5.1: Column material strain ductility demand in 3-storey study buildings with different grades of steel in columns.

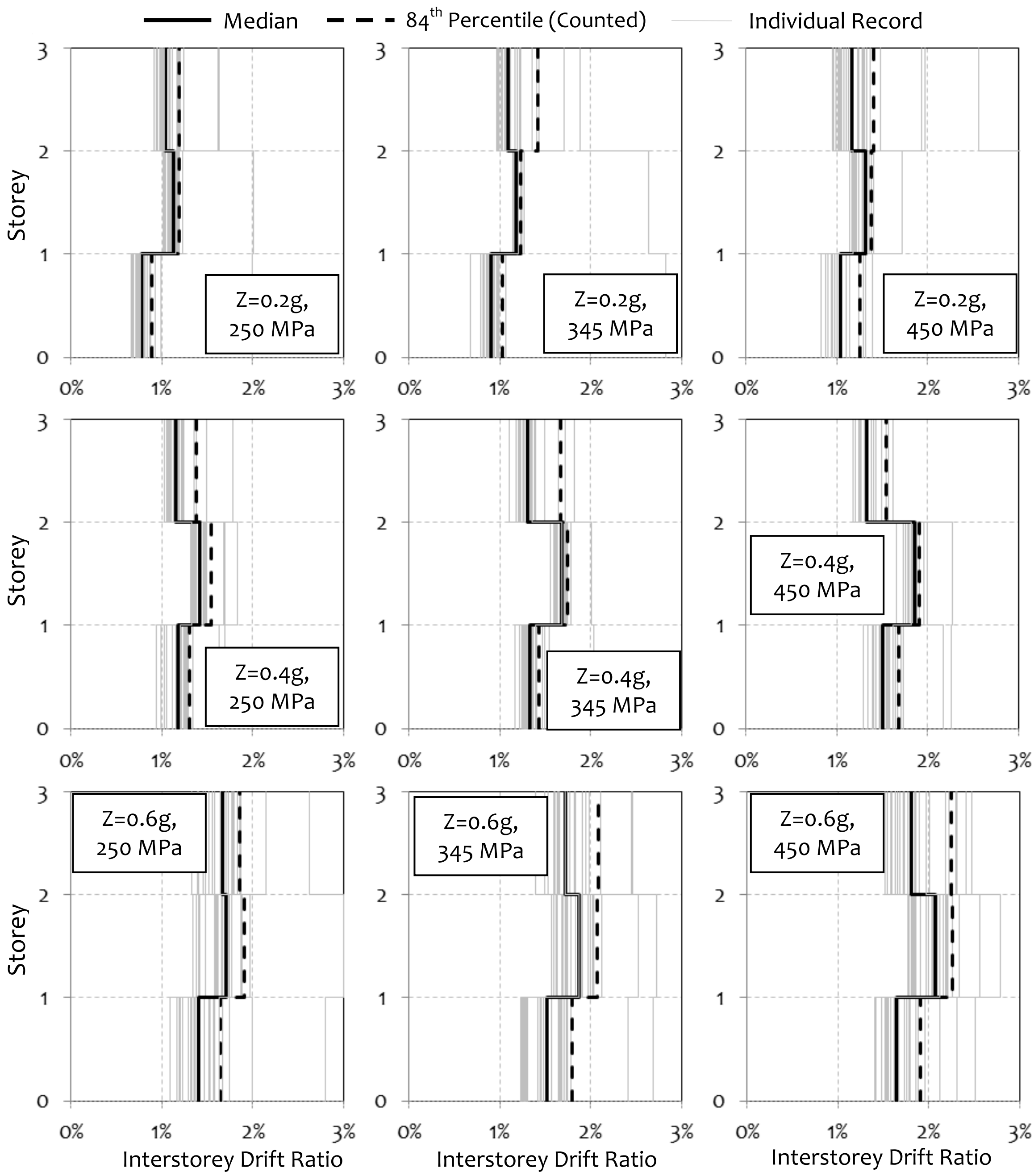


Figure 5.2: Interstorey drift demand in 3-storey study buildings with different grades of steel in columns.

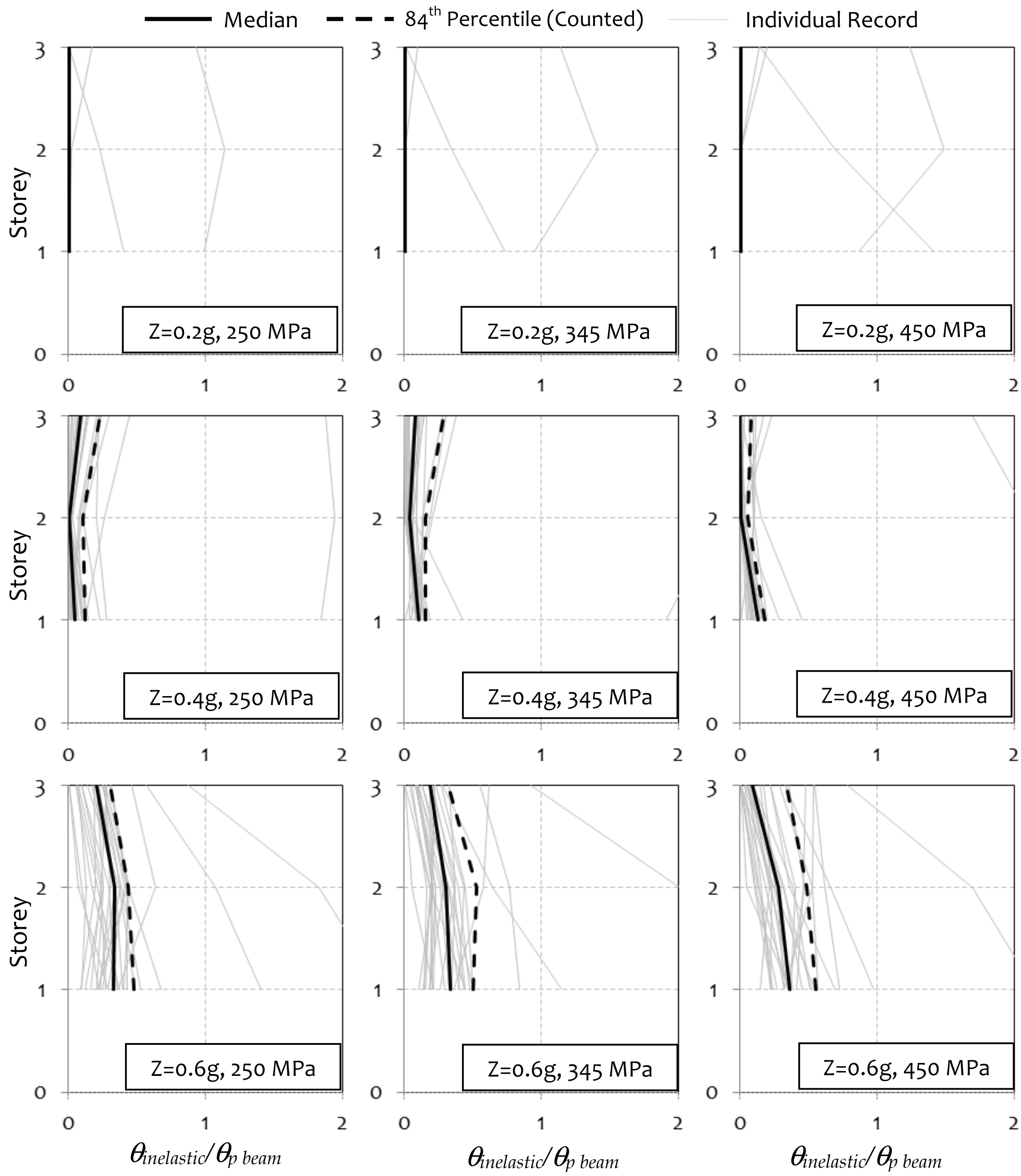


Figure 5.3: Plastic rotation demands ($\theta_{inelastic}$) in beams in 3-storey study buildings with different grades of steel in columns. The plastic rotations are normalized with the modified plastic rotation capacity of the section (θ_p= 0.7$\theta_{p\text{-}monotonic}$).

5.3.3.2 9-Storey Buildings

The responses of 9-storey study buildings are similar to the responses of 3-storey study buildings in many aspects. For instance, the column material strain ductility demand in columns decreases with increasing grade of steel of columns, but the overall strain demands increases as shown in Figure 5.4. However, unlike in the 3-storey study buildings, columns in the 9-storey study buildings do yield for considerable number of ground motions (~5-10), especially for buildings designed for Z=0.6g and 0.4g levels of shaking with 250 grade steel in columns (Figure 5.4). Minor yielding (<2ε_y) of columns is also observed in study buildings with 345 grade columns for few ground motions (~5). Further, the interstorey drift demands are higher in study buildings made up of higher grade steel columns, since they are flexible (Figure 5.5). Finally, the response of the beams shown in Figure 5.6 indicates that the overall inelastic demand in beams increases marginally with the use of higher grade of steel in columns, which can be attributed to higher interstorey drifts in those buildings. Thus, the use of higher grade steel in columns, is generally helpful in bringing down the ductility demand in the columns, especially column bases, however at the cost of having higher strain demands in columns, higher interstorey drifts, and increased beam inelastic rotation demands.

From the investigations to find the suitable material grade (*i.e.*, 250, 345, or 450 grade of steel) for columns in steel SMFs, the following conclusions can be drawn:

1. 250 grade steel for columns is least preferred as its usage requires larger column section sizes to satisfy *CBSR* requirement combined with thicker doubler plates to strengthen panel zones. Furthermore, the probability of column yielding is significantly higher in case of 250 grade steel columns owing to lower yield strain. Again, columns being stiffer in case of 250 grade compared to 345 and 450 grade steels result in increased flexural mode behaviour leading to enhanced single curvature bending, which eventually increases the probability of column yielding.
2. 345 grade steel for columns is the preferred choice as it helps in minimizing the probability of column yielding (only 5 ground motions resulted in minor column yielding) even in study buildings designed for Z=0.6 shaking, with reasonable column section dimensions.
3. 450 grade steel seems to be a good choice as the columns do not yield even in the study buildings designed for Z=0.6g shaking. However, usage of 450 grade steel in columns results in relatively flexible buildings leading to larger inelastic demands in beams. Further, the connections also need to be designed to accommodate this increased beam rotation demands. Again, higher grade steels typically have lower toughness and relatively poor weldabilty characteristics compared to lower grade steels. Finally, as very few experimental studies are available in literature using 450 grade steel, it is prudent to avoid its use in SMFs for the time being.

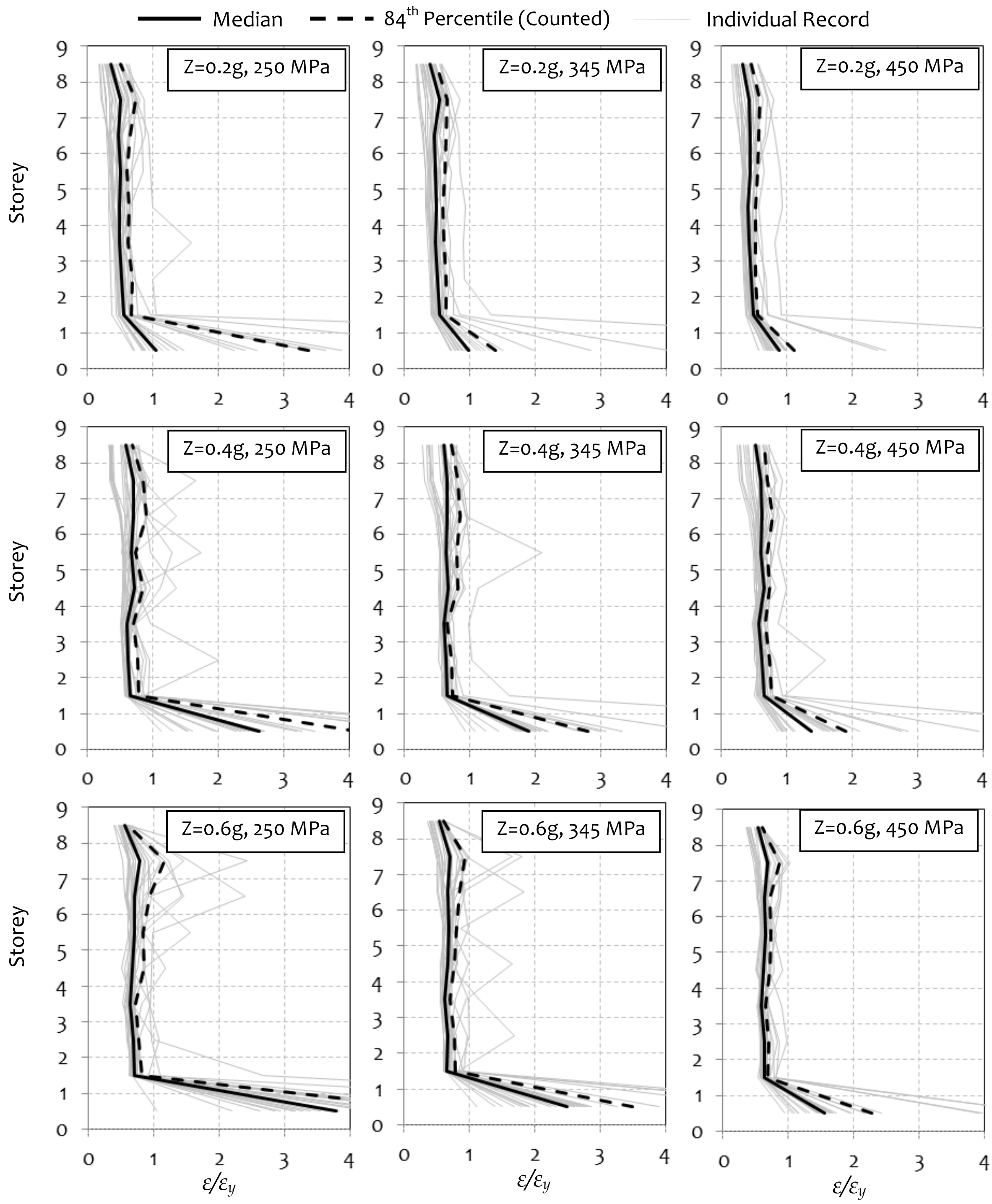


Figure 5.4: Column material strain ductility demand in 9-storey study buildings with different grades of steel in columns.

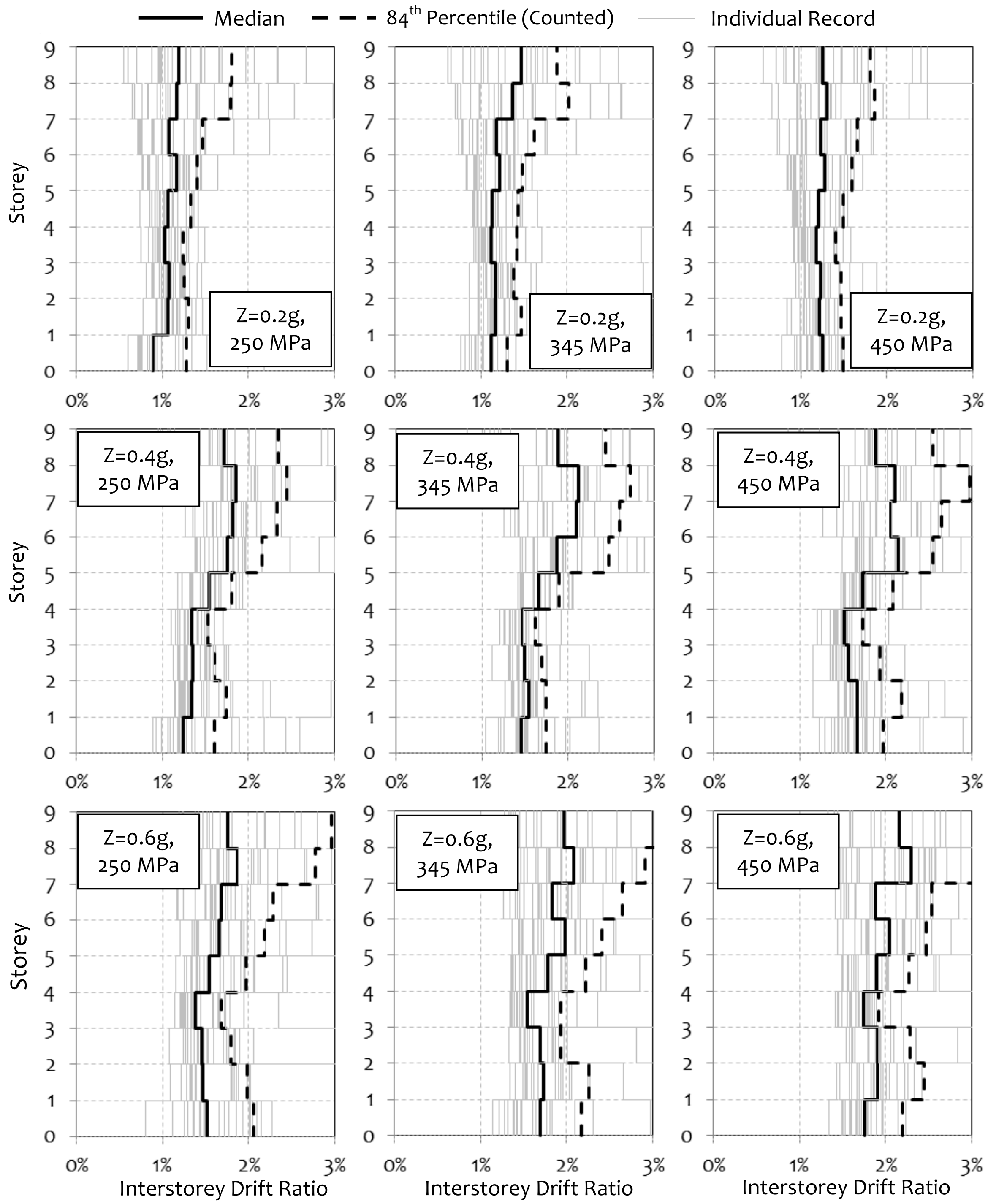


Figure 5.5: Interstorey drift demands in 9-storey study buildings with different grades of steel in columns.

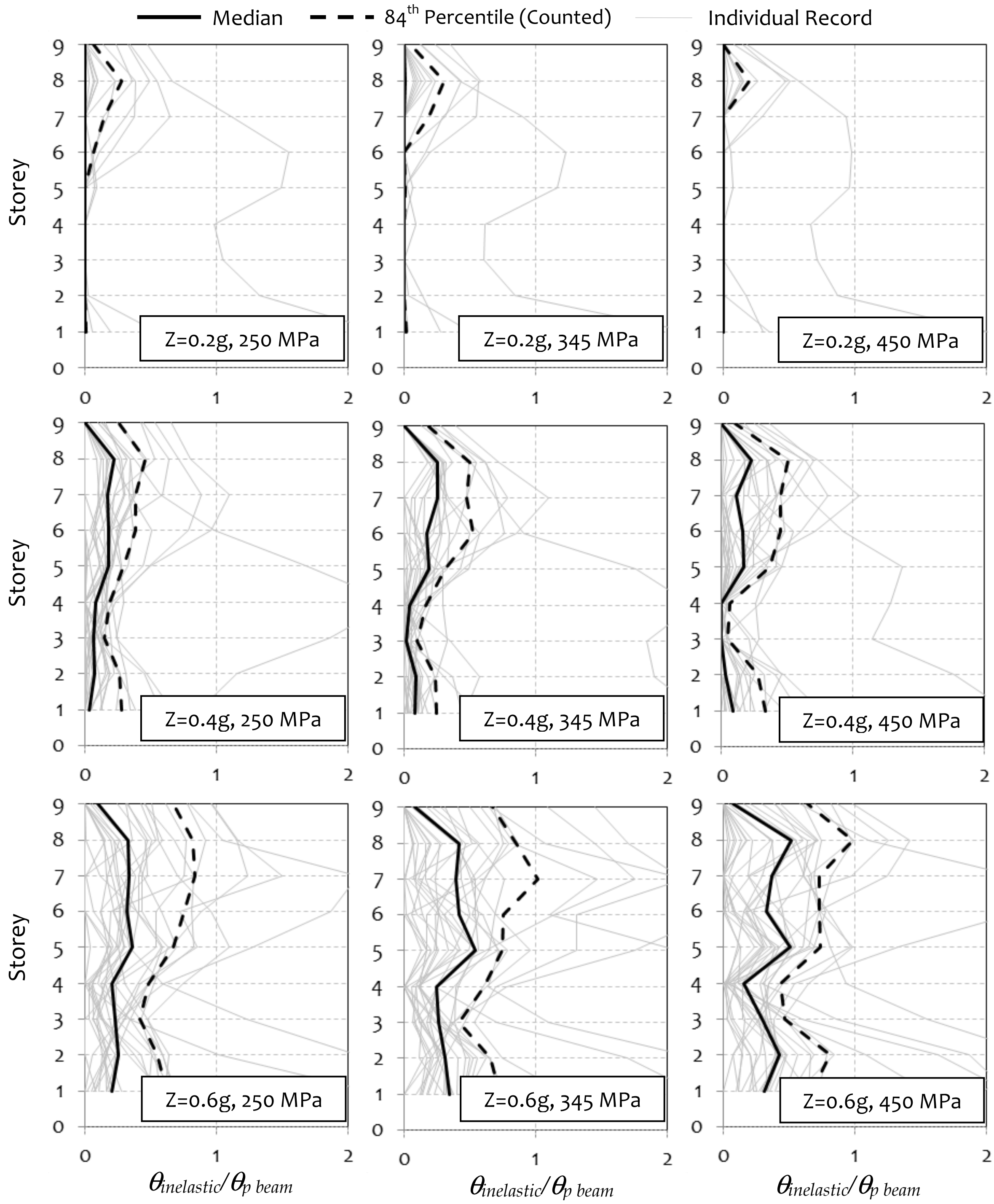


Figure 5.6: Plastic rotation demands ($\theta_{inelastic}$) in beams in 9-storey study buildings with different grades of steel in. The plastic rotations are normalized with the modified plastic rotation capacity of the section ($\theta_p = 0.7\theta_{p\text{-}monotonic}$).

5.4 Conclusions

In the current chapter, *CBSR* required to eliminate the use of doubler plate in interior panel zones of steel SMFs is explored. Thereafter, the effectiveness of using higher grade of steel for columns in steel SMFs is investigated. The salient conclusions drawn are:

1. At *CBSR* greater than 1.5, the probability of column yielding in steel SMFs is low for shaking intensities up to Z=0.6g.
2. 345 grade steel for column is the preferred choice, as it helps in minimizing the probability of column yielding even in buildings designed for Z=0.6 shaking (provided *CBSR* is above 1.5), with reasonable column section dimensions.
3. Use of 250 and 450 grade steels for columns in steel SMFs is not recommended. The use of former will require unduly large sections to meet the *CBSR* and panel zone design requirements. The latter, on the other hand, will lead to relatively flexible systems, which impose larger demand on beams, columns, and connections. Furthermore, as of now, issues pertaining to weldabilty and toughness of 450 grade steels are not fully resolved.
4. A *CBSR* value of 2.5 or greater is recommend for steel SMFs for preliminary proportioning to avoid the use of doubler plates or to reduce the thickness to reasonable values (for PZ-II design approach). Furthermore, at these *CBSRs*, the probability of column yielding is very low for shaking intensities up to Z=0.6g.

...

Chapter 6

Summary and Conclusions

6.0 Introduction

The performance of steel special moment frame (SMF) buildings designed using Indian design codes is studied. It is observed that the current provisions are inadequate to guarantee desirable seismic behaviour of steel SMF buildings. The conventional expectation of resisting strong earthquake ground motions through ductile flexural plastic actions at the beam ends in SMFs is not realised; excessive yielding of panel zones and column bases are observed instead. Hence, further investigation is carried out to understand the implications of three different panel zone design approaches on seismic behaviour of steel SMF buildings. The three design approaches are intended to achieve strong, balanced, and weak panel zones in steel SMFs. Thereafter, the study investigates the effectiveness of using higher grade of steel for columns in SMFs. Finally, the column-to-beam strength ratio (*CBSR*) required to avoid the use of doubler plate in panel zones and to limit the yielding of columns during strong earthquake shaking is evaluated.

6.1 Summary

The following is the summary of the work done as a part of this study:

1. The study initially compares the provisions for design of capacity-protected elements recommended by a few seismic design codes (more specifically, American, European, and Indian codes) to benchmark the provisions related to design of panel zones and columns in steel SMF buildings. Few critical differences are identified;
2. To assess the implication of the identified critical differences, two buildings, a 3-storey and a 9-storey with steel SMFs as the lateral load resisting system are designed compliant to the Indian codes; thereafter, the performance of the two buildings is evaluated using nonlinear static and dynamic analyses. The performance assessment of the designed buildings indicates undesirable behaviour based on which, the shortcomings of the design provisions are identified;
3. To improve the seismic behaviour, the two buildings are redesigned considering the identified shortcomings. The redesigned buildings are seen to demonstrate

acceptable earthquake behaviour. However, minor yielding of panel zone was observed;

4. To deepen the understanding on panel zone behaviour, three panel zone design approaches, namely PZ-I, PZ-II, and PZ-III (refer Section 4.3) are studied. These three approaches are intended to result in weak, balanced, and strong panel zones. Thereafter, the three panel zone design approaches are employed in the design of panel zones in study buildings. The 3- and 9-storey study buildings are designed for three shaking intensities (Z=0.2g, 0.4g, and 0.6g) to understand panel zone behaviour at different seismic hazard levels. Panel zone response is evaluated by nonlinear static and dynamic analysis. The behaviour of the panel zones is found to be as intended;
5. Further, the effectiveness in using higher grade steel for columns is investigated by adopting them in the study buildings. Here also, the study buildings are designed for three levels of shaking intensity (Z=0.2g, 0.4g, and 0.6g); and
6. Finally, the minimum column-to-beam strength ratio (*CBSR*) required to avoid the use of column doubler plate and to limit the yielding of columns during strong earthquake shaking is evaluated for the three panel zone design approaches investigated in the study.

6.2 Conclusions

The following are the salient conclusions drawn from this study:

1. Seismic design of steel SMF buildings following the provisions of current Indian codes will result in unsatisfactory seismic behaviour. This is because the current provisions [IS 800, 2007] fail (a) in recognising the expected increase in material yield stress over and above the minimum specified, or characteristic, yield strength of the material, (b) in properly estimating the demand on panel zones, and (c) in recommending an adequate minimum *CBSR* for the design of columns in steel SMFs;
2. Panel Zone Design Approach-1, (PZ-I) [refer Section 4.3] results in a relatively weak panel zone. Hence proportioning of panel zone by PZ-I approach which is similar to the current American specification [ANSI/AISC 360, 2016] can lead to high shear distortion (~4 γ_y) in panel zones, especially for buildings designed for higher shaking intensities. Therefore, use of PZ-I design approach should be avoided;
3. Panel Zone Design Approach-2, (PZ-II) [refer Section 4.3] results in balanced panel zone, where inelastic energy dissipation is shared primarily by the beams and to a

limited extent by the panel zones and columns. Hence, PZ-II approach is recommended for use in design of panel zone in steel SMFs;

4. Panel Zone Design Approach-3, (PZ-III) [refer Section 4.3] results in a strong panel zone, which again results in desired seismic behaviour, but at the cost of having increased doubler plate thickness and higher inelastic demands in beams. Hence, it is preferable to select PZ-II design approach over PZ-III design approach for design of panel zones;
5. Columns with yield stress of 345 MPa are the preferred choice of steel grade for steel SMF buildings (provided *CBSR* as given in Eq.(5.1) is above 1.5). Further, a *CBSR* greater than 1.5, is found be sufficient to limit the probability of column yielding in steel SMFs up to a shaking intensity of Z=0.6g; and
6. A *CBSR* value of 2.5, computed as per Eq.(5.1), or greater is recommend for steel SMFs for preliminary proportioning of members to avoid the use of or to reduce the thickness of doubler plates to reasonable values. Furthermore, at these *CBSRs*, the probability of column yielding is low for shaking intensities up to Z=0.6g.

6.3 Limitations of the Current Study and Scope for Future Work

The current study has the following limitations:

1. A simple hysteretic model, which does not *explicitly* capture stiffness degradation and strength deterioration, is considered for nonlinear analyses. Use of more advanced models may cause deviation of results, particularly at collapse limit states;
2. Bare frame building models are used to evaluate seismic behaviour of buildings. Hence, the influence infills, especially unreinforced masonry walls on seismic response of such buildings are not accounted. Further, beam nonlinear hinge definition used does not consider the influence of floor diaphragm on the moment-rotation response; and
3. Two dimensional frames are considered for nonlinear dynamic analyses. Hence, the bi-directional effects of ground shaking, which may alter the seismic response was not accounted.

Based on the limitations of the current study, the following are identified as the scope for possible future work:

1. As panel zone shear demand and capacity varies depending on the connection configuration adopted (*e.g.*, in case of reinforced connections, an increase in the effective depth of the beam at column face will help in reducing panel zone shear

demand), a similar study may be carried out for different commonly used connections to arrive at the appropriate panel zone design approaches; and

2. A similar study using three-dimensional nonlinear model of buildings will help to evaluate the bi-directional effects of earthquake ground shaking.
3. A comparative study on seismic performance and cost of steel moment frame buildings designed as per *Force Based Design* and *Performance Based Plastic Design* methods may be carried out.

…

References

1. American Institute of Steel Construction (AISC), (2016), *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Application* (ANSI/AISC 358-16), Chicago, Illinois, USA

2. American Institute of Steel Construction (AISC), (1992), *Seismic Provision for Structural Steel Buildings* (ANSI/AISC 341-92), Chicago, Illinois, USA

3. American Institute of Steel Construction (AISC), (2016), *Seismic Provision for Structural Steel Buildings* (ANSI/AISC 341-16), Chicago, Illinois, USA

4. American Institute of Steel Construction (AISC), (2016), *Specification for Structural Steel Buildings* (ANSI/AISC 360-16), Chicago, Illinois, USA

5. American Society of Civil Engineers (ASCE), (2016), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures: Provisions* (ASCE/SEI 07-16), Virginia, USA

6. American Society of Civil Engineers (ASCE), (2013), *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE/SEI 41-13), Virginia, USA

7. American Welding Society (2016), *Structural Welding Code-Seismic Supplement* (ANSI/AWS D1.8/D1.8M: 2016), American Welding Society, Florida, USA

8. Applied Technology Council (ATC), (2017), *Guidelines for Nonlinear Structural Analysis for Design of Buildings-Part IIa-Steel Moment Frames* (Report NIST GCR 17-917-46v2), National Institute of Standards and Technology, California, USA

9. Applied Technology Council (ATC), (1996), *Seismic Evaluation and Retrofit of Concrete Buildings* (Report No.ATC-40), Seismic Safety Commission, California, USA

10. Architectural Institute of Japan (AIJ), (1995), *Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake*, Architectural Institute of Japan, Tokyo, Japan

11. BCJ (1997), *Seismic Provisions for Design of Building Structures*, The Building Center of Japan, Tokyo, Japan

12. Bertero, V.V., Krawinkler, H. and Popov, E.P., (1973), *Further Studies on Seismic Behaviour of Steel Beam-column Subassemblages* (Report No. EERC 73-27), Earthquake Engineering Research Centre, College of Engineering, University of California at Berkeley, California, USA

13. Bertero, V.V., Anderson, J.C. and Krawinkler, H., (1994), *Performance of Steel Building Structures During the Northridge Earthquake* (Report No. UCB/EERC-94-09), Earthquake Engineering Research Centre, University of California at Berkeley, California, USA

14. Bertero, V.V., (1997), Performance-based seismic engineering: A critical review of proposed guidelines, *In Proceedings of the International Workshop on Seismic Design Methodologies for the Next Generation of Code,* pp. 1-32

15. Brandonisio, G., De Luca, A. and Mele, E., (2011), Shear instability of panel zone in beam-to-column connections, *Journal of Constructional Steel Research*, 67(5), pp.891-903

16. Bruneau, M., Uang, C.M. and Sabelli, S.R., (2011), *Ductile Design of Steel Structures*, McGraw Hill Professional, New York, USA

17. Castro, J.M., Davila-Arbona, F.J. and Elghazouli, A.Y., (2008), Seismic design approaches for panel zones in steel moment frames, *Journal of Earthquake Engineering*, 12(S1), pp.34-51

18. CEN (2004), *Eurocode 8: Design Provisions for Earthquake Resistance of Structures, Part 1: General rules, Seismic Actions and Rules for buildings* (EN 1998–1), European Committee for Standardization, Brussels, Belgium

19. CEN (2005), *Eurocode 3: Design of Steel Structures - Part 1.8: Design of Joints* (EN 1993–1–8), European Committee for Standardization, Brussels, Belgium

20. Challa, V.R.M., (1992), *Nonlinear Seismic Behaviour of Steel Planar Moment-Resisting Frames*, Doctoral Thesis, Caltech, USA

21. Charney, F.A. and Downs, W.M., (2004), Modeling procedures for panel zone deformations in moment resisting frames, Connections in steel structures V: behaviour strength and design, *Bouwen met Staal*, 5(1), pp.121-130

22. Charney, F.A. and Marshall, J., (2006), A comparison of the Krawinkler and scissors models for including beam-column joint deformations in the analysis of moment-resisting steel frames, *Engineering Journal,* 43(1), pp.31-48

23. Choi, S.W. and Park, H.S., (2012), Multi-objective seismic design method for ensuring beam-hinging mechanism in steel frames, *Journal of Constructional Steel Research*, 74(3), pp.17-25

24. Choi, S.W., Kim, Y., Lee, J., Hong, K. and Park, H.S., (2013), Minimum column-to-beam strength ratios for beam–hinge mechanisms based on multi-objective seismic design, *Journal of Constructional Steel Research*, 88(6), pp.53-62

25. Computers and Structure Inc. (CSI), (2016), *Nonlinear Analysis and Performance Assessment for 3D Structures,* PERFORM-3D, California, USA

26. Da´vila-Arbona, F. J., (2006), *Panel Zone Behaviour in Steel Moment-Resisting Frames*, MSc Thesis, European School for Advanced Studies in Reduction of Seismic Risk, Rose School, Pavia, Italy

27. Earthquake Engineering Research Institute (EERI), (1995), *Northridge Earthquake Reconnaissance*, Earthquake Spectra, Supplement C to Volume 11, Oakland, California, USA

28. Elghazouli, A.Y., (2010), Assessment of European seismic design procedures for steel framed structures, *Bulletin of Earthquake Engineering*, 8(1), pp.65-89

29. Elkady, A. and Lignos, D.G., (2015), Analytical investigation of the cyclic behaviour and plastic hinge formation in deep wide-flange steel beam-columns, *Bulletin of Earthquake Engineering*, 13(4), pp.1097-1118

30. Elnashai, A.S. and Di Sarno, L., (2008), *Fundamentals of Earthquake Engineering*, Wiley, New York, USA

31. EN 1011-2, (2001), *Welding: Recommendations for Welding of Metallic Materials. Arc Welding of Ferritic Steels* (EN 1011-2), European Committee for Standardization, Brussels, Belgium

32. Engelhardt, M.D. and Sabol, T.A., (1997), Seismic-resistant steel moment connections: developments since the 1994 Northridge earthquake, *Progress in Structural Engineering and Materials*, 1(1), pp.68-77

33. FEMA-350, (2000), *Recommended Seismic Design Criteria for New Steel Moment- Frame Buildings* (Report. FEMA-350), Federal Emergency Management Agency, Washington DC, USA

34. FEMA-351, (2000), *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* (Report. FEMA-351), Federal Emergency Management Agency, Washington DC, USA

35. FEMA-352, (2000), *Recommended Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (Report. FEMA-352), Federal Emergency Management Agency, Washington DC, USA

36. FEMA-353, (2000), *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications* (Report. FEMA-353), Federal Emergency Management Agency, Washington DC, USA

37. FEMA-355C, (2000), *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking* (Report. FEMA-355C), Federal Emergency Management Agency, Washington DC, USA

38. FEMA-355D, (2000), *State-of-the-Art Report on Connection Performance* (Report. FEMA-355D), Federal Emergency Management Agency, Washington DC, USA

39. FEMA-355E, (2000), *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes* (Report. FEMA-355E), Federal Emergency Management Agency, Washington DC, USA

40. FEMA-440, (2005), *Improvement of Nonlinear Static Seismic Analysis Procedures* (Report. FEMA 440), Federal Emergency Management Agency, Washington DC, USA

41. Fielding, D.J. and Huang, J.S., (1971), Shear in steel beam-to-column connections, *Welding Journal*, 50(7), pp.313-326

42. Ghobarah, A., Korol, R.M. and Osman, A., (1992), Cyclic behaviour of extended end-plate joints, *Journal of Structural Engineering*, 118(5), pp.1333-1353

43. Gioncu, V. and Petcu, D., (1997), Available rotation capacity of wide-flange beams and beam-columns Part 1. Theoretical approaches, *Journal of Constructional Steel Research*, 43(1-3), pp.161-217

44. Gioncu, V., and Mazzolani, F., (2002), *Ductility of Seismic-Resistant Steel Structures*, CRC Press, London, UK

45. Goswami, R., (2007), *Seismic Design of Welded Connections in Steel Moment Resisting Frame Buildings with Square Box Columns,* Doctoral Thesis, Indian Institute of Technology Kanpur, Kanpur, India

46. Gupta, A. and Krawinkler, H., (1999), *Seismic Demands for the Performance Evaluation of Steel Moment Resisting Frame Structures*, Doctoral Thesis, Stanford University, California, USA

47. Hamburger, R.O., and Malley, J.O., (2009), *Seismic Design of Steel Special Moment Frames* (Report No. NIST GCR, 09-917-3), National Institute of Standards and Technology, California, USA

48. Hannamwale, M.T., (2014), *Quantification of Curvature Ductility of Steel Beam Section*, M.Tech Project, Indian Institute of Technology Madras, Chennai, India

49. Indian Standards (IS), (2004), *Covered Electrodes for Manual Metal Arc Welding of Carbon and Carbon Manganese Steel-Specification* (IS 814, 2004), Bureau of Indian Standards, New Delhi, India

50. Indian Standards (IS), (2016), *Criteria for Earthquake Resistant Design of Structures - Part 1 General Provisions and Buildings* (IS 1893 (Part 1), 2016), Bureau of Indian Standards, New Delhi, India

51. Indian Standards (IS), (2007), *General Construction in Steel-Code of Practice* (IS 800, 2007), Bureau of Indian Standards, New Delhi, India

52. Indian Standards (IS), (2011), *Hot Rolled Medium and High Tensile Structural Steel-Specification* (IS 2062, 2011), Bureau of Indian Standards, New Delhi, India

53. Kasar, A.A., Bharti, S.D., Shrimali, M.K. and Goswami, R., (2017), Mechanics based force-deformation curve of steel beam to column moment joints, *Steel and Composite Structures*, 25(1), pp.19-34

54. Kawano, A., (1984), Inelastic behaviour of low-rise steel frame based on a weak beam-to-column connection philosophy to earthquake motion. *In Proceedings of the Eighth World Conference on Earthquake Engineering,* pp. 519-526

55. Kaufmann, E.J., Xue, M., Lu, L.W. and Fisher, J.W., (1996) Achieving ductile behaviour of moment connections, *Modern Steel Construction*, 36(1), pp.30-39

56. Kim, D.W., Blaney, C. and Uang, C.M., (2015), Panel zone deformation capacity as affected by weld fracture at column kinking location, *Engineering Journal*, 53(1), pp.27-46

57. Kim, K.D. and Engelhardt, M.D., (1995), *Development of Analytical Models for Earthquake Analysis of Steel Moment Frames* (Report No. PMFSEL 95-2), Phil M. Ferguson Structural Engineering Laboratory, University of Texas at Austin, USA

58. Kim, K.D. and Engelhardt, M.D., (2002), Monotonic and cyclic loading models for panel zones in steel moment frames, *Journal of Constructional Steel Research*, 58(5-8), pp.605-635

59. Kostic, S.M. and Filippou, F.C., (2011), Section discretization of fiber beam-column elements for cyclic inelastic response, *Journal of Structural Engineering*, 138(5), pp.592-601

60. Krawinkler, H. and Mohasseb, S., (1987), Effects of panel zone deformations on seismic response, *Journal of Constructional Steel Research*, 8(10), pp.233-250

61. Krawinkler, H., (2006), Importance of good nonlinear analysis, *The Structural Design of Tall and Special Buildings*, 15(5), pp.515-531

62. Krawinkler, H., Bertero, V.V. and Popov, E.P., (1971), *Inelastic Behaviour of Steel Beam-to-Column Subassemblages* (Report No. UBC/EERC-71/7), Earthquake Engineering Research Centre, University of California at Berkeley, California, USA

63. Krawinkler, H., Popov, E.P. and Bertero, V.V., (1975), Shear behaviour of steel frame joints, *Journal of the Structural Division*, ASCE, 101(11), pp.2317-2336

64. Krawinkler, H., (1978), Shear in beam-column joints in seismic design of steel frames, *Engineering Journal*, 15(3), pp.82-91

65. Lee, S.J. and Lu, L.W., (1989), Cyclic tests of full-scale composite joint subassemblages, *Journal of Structural Engineering*, 115(8), pp.1977-1998

66. Lee, H.S., (1996), Revised rule for concept of strong-column weak-girder design, *Journal of Structural Engineering*, 122(4), pp.359-364

67. Lignos, D.G. and Krawinkler, H., (2007), A database in support of modeling of component deterioration for collapse prediction of steel frame structures, *In Proceedings of the Structural Engineering Research Frontiers Congress*, pp. 1-12

68. Lignos, D.G. and Krawinkler, H., (2010), Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading, *Journal of Structural Engineering*, 137(11), pp.1291-1302

69. Mahin, S.A., (1998), Lessons from damage to steel buildings during the Northridge earthquake, *Engineering Structures*, 20(4-6), pp.261-270

70. Malley, J.O., (1998), SAC Steel Project: Summary of Phase 1 testing investigation results, *Engineering Structures*, 20(4-6), pp.300-309

71. Medina, R.A. and Krawinkler, H., (2005), Strength demand issues relevant for the seismic design of moment-resisting frames, *Earthquake Spectra*, 21(2), pp.415-439

72. Miller, D.K., (1998), Lessons learned from the Northridge earthquake, *Engineering Structures,* 20(4-6), pp.249-260

73. Murty, C.V.R., Goswami, R., Vijayanarayanan, A.R. and Mehta, V.V,, (2012), *Some Concepts in Earthquake Behaviour of Buildings*, Gujarat State Disaster Management Authority, Gandhinagar, India

74. Nakashima, M., Inoue, K. and Tada, M., (1998), Classification of damage to steel buildings observed in the 1995 Hyogoken-Nanbu earthquake, *Engineering Structures*, 20(4-6), pp.271-281

75. Nakashima, M., Roeder, C.W. and Maruoka, Y., (2000), Steel moment frames for earthquakes in United States and Japan, *Journal of Structural Engineering*, 126(8), pp.861-868

76. Nakashima, M. and Sawaizumi, S., (2000), Column-to-beam strength ratio required for ensuring beam-collapse mechanisms in earthquake responses of steel moment frames, *In Proceedings of the 12th World Conference on Earthquake Engineering*, Paper No. 1109

77. Nakashima, M., Ogawa, K. and Inoue, K., (2002a), Generic frame model for simulation of earthquake responses of steel moment frames, *Earthquake Engineering & Structural Dynamics*, 31(3), pp.671-692

78. Nakashima, M., Kanao, I. and Liu, D., (2002b), Lateral instability and lateral bracing of steel beams subjected to cyclic loading, *Journal of Structural Engineering*, 128(10), pp.1308-1316

79. Newell, J.D. and Uang, C.M., (2008), Cyclic behaviour of steel wide-flange columns subjected to large drift, *Journal of Structural Engineering*, 134(8), pp.1334-1342

80. Park, R. and Paulay, T., (1975), *Reinforced Concrete Structures*, John Wiley & Sons, London, UK

81. PEER-ATC, (2010), *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings* (PEER-ATC 72-1), Pacific Earthquake Engineering Centre and Applied Technology Council, California, USA

82. Popov, E.P., (1987), Panel zone flexibility in seismic moment joints, *Journal of Constructional Steel Research*, 8(5), pp.91-118

83. Popov, E.P., Amin, N.R., Louie, J.J. and Stephen, R.M., (1985), Cyclic behaviour of large beam-column assemblies, *Earthquake Spectra*, 1(2), pp.203-238

84. Powell, G.H., (2006), Static pushover methods – explanation, comparison and implementation, Modeling for Structural Analysis, *In Proceedings of 8th U.S. National Conference on Earthquake Engineering*, pp.4353-4363

85. Ricles, J.M., Mao, C., Lu, L.W. and Fisher, J.W., (2002), Inelastic cyclic testing of welded unreinforced moment connections, *Journal of Structural Engineering*, 128(4), pp.429-440

86. Roeder, C.W., Carpenter, J.E. and Taniguchi, H., (1989), Predicted ductility demands for steel moment resisting frames, *Earthquake Spectra*, 5(2), pp.409-427

87. Roeder, C.W. and Foutch, D.A., (1996), Experimental results for seismic resistant steel moment frame connections, *Journal of Structural Engineering*, 122(6), pp.581-588

88. Schneider, S.P., Roeder, C.W. and Carpenter, J.E., (1993), Seismic behaviour of moment-resisting steel frames: experimental study, *Journal of Structural Engineering*, 119(6), pp.1885-1902

89. Tremblay, R., Filiatrault, A., Timler, P. and Bruneau, M., (1995), Performance of steel structures during the 1994 Northridge earthquake, *Canadian Journal of Civil Engineering*, *22*(2), pp.338-360

90. Tremblay, R., Filiatrault, A., Bruneau, M., Nakashima, M., Prion, H.G. and DeVall, R., (1996), Seismic design of steel buildings: lessons from the 1995 Hyogo-ken Nanbu earthquake, *Canadian Journal of Civil Engineering*, 23(3), pp.727-756

91. Tsai, K.C. and Popov, E.P., (1988), *Steel Beam-Column Joints in Seismic Moment Resisting Frames* (Report No. UCB/EERC-89-19), Earthquake Engineering Research Centre, University of California at Berkeley, California, USA

92. Vamvatsikos, D. and Cornell, C.A., (2002), Incremental dynamic analysis, *Earthquake Engineering & Structural Dynamics*, 31(3), pp.491-514

93. SAC, (1995), *Surveys and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994* (Report SAC-95-06), SAC Joint Venture, California, USA

94. SEAOC, (1980), *Recommended Lateral Force Requirements and Commentary*, Structural Engineers Association of California, Sacramento, California, USA

95. SEAOC Vision 2000 Committee, (1995), *Performance-Based Seismic Engineering*, Structural Engineers Association of California, Sacramento, California, USA

96. Wheeler and Gray, (2019), *Ventura County Medical Centre*, Retrieved from http://www.wheelerandgray.com/project-vcmc-office-building/ on 22/11/2019

97. Wongpakdee, N. and Leelataviwat, S., (2017), Influence of column strength and stiffness on the inelastic behavior of strong-column-weak-beam frames, *Journal of Structural Engineering*, 143(9), pp.1-12

98. Xue, M., Kaufmann, E.J., Lu, L.W. and Fisher, J.W., (1996), Achieving ductile behaviour of moment connections, Part II, *Modern Steel Construction*, 36(6), pp.38-42

99. Zaghi, A.E., Soroushian, S., Itani, A., Maragakis, E.M., Pekcan, G. and Mehrraoufi, M., (2015), Impact of column-to-beam strength ratio on the seismic response of steel MRFs, *Bulletin of Earthquake Engineering*, 13(2), pp.635-652

100. Ziemian, R.D., McGuire, W. and Deierlein, G.G., (1992), Inelastic limit states design Part I: Planar frame studies, *Journal of Structural Engineering*, 118(9), pp.2532-2549

...

Jones Joju
Department of Civil Engineering
Indian Institute of Technology Madras

December 2019