BEHAVIOUR ASSESSMENT OF MULTI-STOREYED RC FRAME BUILDINGS UNDER SEQUENTIAL EARTHQUAKE EVENTS

Submitted in partial fulfilment of the requirements
for the award of the degree of

DOCTOR OF PHILOSOPHY

in

CIVIL ENGINEERING

by

PRAVEEN OGGU

(Roll No. 715010)

Under the supervision of

Dr. K. GOPIKRISHNA

STRUCTURES DIVISION

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY WARANGAL

WARANGAL – 506 004, TELANGANA, INDIA

2021

NATIONAL INSTITUTE OF TECHNOLOGY
WARANGAL

CERTIFICATE

This is to certify that the thesis entitled "**BEHAVIOUR ASSESSMENT OF MULTI-STOREYED RC FRAME BUILDINGS UNDER SEQUENTIAL EARTHQUAKE EVENTS"** being submitted by **Mr. PRAVEEN OGGU** for the award of the degree of **DOCTOR OF PHILOSOPHY** to the Faculty of Engineering and Technology of **NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL** is a record of bonafide research work carried out by him under my supervision and it has not been submitted elsewhere for award of any degree.

Dr. K. GOPIKRISHNA

Thesis Supervisor
Assistant Professor
Department of Civil Engineering
National Institute of Technology
Warangal (T.S.) – INDIA

NATIONAL INSTITUTE OF TECHNOLOGY WARANGAL

DECLARATION

This is to certify that the work presented in the thesis entitled **"BEHAVIOUR ASSESSMENT OF MULTI-STOREYED RC FRAME BUILDINGS UNDER SEQUENTIAL EARTHQUAKE EVENTS"** is a bonafide work done by me under the supervision of **Dr. K. GOPIKRISHNA** and was not submitted elsewhere for the award of any degree.

I declare that this written submission represents my ideas in my own words and where others' ideas or words have been included, I have adequately cited and referenced the original sources. I also declare that I have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea / data / fact /source in my submission. I understand that any violation of the above will be a cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

____

(Signature)

(Name of the Student: **PRAVEEN OGGU**)

(Roll No. **715010**)

Date: _____________

ACKNOWLEDGEMENTS

With great pleasure and proud privilege, I express my wholehearted thankfulness to my research supervisor, Dr. K. Gopikrishna, Assistant Professor, Department of Civil Engineering, for his invaluable suggestions, insightful guidance, scholarly advice and comprehensive critical remarks in bringing out this research work, for which I am deeply indebted to, and without his moral support I could not have come this far.

I am grateful to Prof. P. Rathish Kumar, Head of the Department of Civil Engineering, and Chairman of Doctoral Scrutiny Committee; and also, members of Doctoral Scrutiny Committee: Prof. P. Bangaru Babu, Department of Mechanical Engineering; Dr. M.V.N. Sivakumar, Department of Civil Engineering; and Dr. T.P. Tezeswi, Department of Civil Engineering for their continuous encouragement, valuable suggestions and constructive criticism during the period of research work.

I am also thankful to Prof. G. Rajesh Kumar, Prof. C. B. Kameswara Rao, Prof. D. Ramaseshu, Prof. T.D. Gunneswara Rao, Sri. M. Sudhakar, Dr. S. Venkateswara Rao, Dr. D. Ravi Prasad, Dr. S. Anitha Priyadharshani, Dr. B Kavitha, and Dr. B Umesh, the faculty members of Structures Division, NITW for the moral support given during the period of research work.

I will be failing in my duty if I do not express my sincere gratitude to my parents who strived for my excellence, and because of whom I am what I am today.

Finally, I gratefully acknowledge everyone, who contributed either directly or indirectly to this work.

- Praveen Oggu

ABSTRACT

Earthquakes represent an inevitable natural hazard capable of creating a disaster. Several earthquakes that have been witnessed around the world (California, Japan, New Zealand, India, etc.) are an epitome of the disastrous nature this natural hazard. The Earthquake Disaster Risk Index (EDRI) report mentions that about 59% area of India in particular is vulnerable for moderate to major earthquakes. This was clearly evident from past earthquakes that occurred such as Manipur (2016), Nepal (2015), Sikkim (2011), Kashmir (2005), Bhuj (2001), Chamoli (1999), Jabalpur (1997) and Latur (1993) etc. in which even several engineered structures like the RC buildings, bridges etc. have experienced significant damages. Further, it has been reported that more than 90% of the casualties that occurred in past earthquakes in India are due to the collapse of numerous non-conforming commercial and residential structures (EDRI Report, 2019). This resulted in significant loss of life and property, which can be mitigated only by ensuring better code compliance of new constructions and rehabilitate/strengthen the existing structures to withstand the seismic hazard at the chosen location.

However, occurrences of earthquakes are in general random oriented and often repeated multiple number times even after a major earthquake, commonly referred as mainshocks and aftershocks. This phenomenon has been widely witnessed during major earthquake disasters that have occurred around different parts of the world. In several of these instances, sequence of seismic events succeeding the major earthquake were found to possess energy comparable or sometimes even higher than the major earthquake, characterized in terms of the earthquake magnitude scale. California (Mammoth Lakes, 1980; Coalinga, 1983; Whittier Narrows, 1987; Northridge 1994), Japan (Kobe, 1995; Niigata, 2004; Tohoku, 2011), New Zealand (Darfield, 2010; Christchurch, 2011), etc. are certain representative disasters reported in the literature, where sequential earthquakes were found to occur frequently possessing similar or higher magnitude of energy compared to the main earthquake event.

Therefore, the present study is mainly focused on the assessment of seismic behaviour in terms of various response parameters of three-dimensional (3D) RC building frames located in a moderate seismic zone under bi-directional sequential earthquakes. Given this scenario, reinforced concrete (RC) moment-resisting frames (MRFs) of medium-rise configuration with and without vertical irregularities, with and without unreinforced masonry (URM) infills located at Warangal city, Telangana state, India (characterised as Seismic Zone III, medium

soil profile) have been considered in this study. In this investigation, non-linear analysis approach has been adopted to assess the seismic performance of regular and vertically irregular models under bidirectional individual and sequential earthquake events. The non-linear response and structural damages have been analysed in terms of evaluation of several parameters such as storey displacements, permanent damage (residual displacements), local structural damage (plastic hinge formation), structural capacity evaluation (dynamic capacity curves with respect to average spectral acceleration), and collapse fragility estimation in terms of average spectral acceleration.

Further, the existing seismic design codes in most part of the world link the elastic and inelastic response of the structural MRFs in an elastic design by means of a constant value specified as R (termed as Response reduction factor/behaviour factor/Response modification factor etc.). This specification of R by most of the codes results in erroneous representation of seismic demand thereby leading to improper seismic design configuration. Therefore, accurate estimation of R is imminent in arriving at a safe and functional structural configuration throughout its life time. Hence, a methodology for estimation of modified R-factor for RC MRFs to achieve certain performance level, when subjected to single and sequential earthquake forces under Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) hazard levels. The target performance criteria were considered in accordance with seismic design philosophy specified in IS 1893. Therefore, the Life Safety (LS) and Collapse Prevention (CP) criteria has been considered in this study. The modified R values for the RC MRFs are calculated by defining Safety-Margin-Ratios (SMRs) in accordance with respective code demands at any specified location. Depending upon the choice of the stakeholders, performance level can be chosen, and the corresponding SMRs are evaluated, which results in computation of modified R-factor. This modified R-factor aids in estimation of adequate seismic demand necessary to obtain a safe and economical design configuration.

The proposed formulation for modified R-factor has been checked for an IS code designed multi-storeyed building configuration and found to provide promising results in meeting the target performance criteria. Further, this aids in integrating the performance-based design approach (PBD) in to conventional force-based design approach. Hence, this approach ensures evolution of safe and functional design configuration for the seismic forces prevalent at any chosen location.

Contents

List of Tables

List of Figures

List of notations and abbreviations

CP	Collapse Prevention
DBE	Design Basis Earthquake
E_c	Modulus of elasticity of concrete
EDP	Engineering Demand Parameter
E_m	Modulus of elasticity of masonry
FBD	Force-Based Design
GLD	Gravity-Load Design
h	Height of the wall
I	Importance factor
I_c	Moment of inertia of concrete member
IDA	Incremental Dynamic Analysis
IDR	Inter-storey Displacement Ratio
IM	Intensity Measure
IO	Immediate Occupancy
IS	Indian Standard
L_{ds}	Length of the diagonal strut
LS	Life Safety
MCE	Maximum Considered Earthquake
MDOF	Multiple-Degree-Of-Freedom
MIDR	Maximum Inter-storey Displacement Ratio
M_w	Moment magnitude
MRF	Moment Resisting Frame
NLA	Non-Linear Analysis
NLD	Non-Linear Dynamic
NLS	Non-Linear Static
OGS	Open Ground Storey
OMRF	Ordinary Moment Resisting Frame
PBD	Performance-Based Design
PGA	Peak Ground Acceleration
R	Response reduction factor
RC	Reinforced Concrete

R_R	Redundancy factor
R_S	Overstrength factor
R_μ	Ductility factor
R_ξ	Damping factor
SMR	Safety-Margin-Ratio
SMRF	Special Moment Resisting Frame
S_a	Spectral acceleration
$S_{a\ T}$	Spectral acceleration corresponding to a time period, T
$S_{a\ avg}$	Average Spectral acceleration
S_a/g	Average response acceleration coefficient (depends on T – Undamped Natural period of the structure)
SDOF	Single-Degree-Of-Freedom
t	Angle of the diagonal strut with the horizontal
T	Time period
UF	Utilisation Factor
W	Seismic weight of the building
W_{ds}	Equivalent width of the diagonal strut
Z	Zone factor
μ	Ductility ratio
θ	Thickness of the infill wall

CHAPTER 1

Introduction

1.1 General

Earthquakes are known to mankind as an inevitable natural hazard capable of creating a disaster and has a potential to even cripple the economy of a nation. The causalities caused, the vulnerabilities of the built infrastructure experienced and subsequent difficulties faced by mankind, witnessed during occurrences of several earthquakes around the world (California, Japan, New Zealand, India, etc.) are an epitome of the disastrous nature this natural hazard. The only way to alleviate the difficulties resulting from this hazard is mitigation of the built environment.

Fig. 1.1 Structural damage caused by earthquakes (Sources: Saatcioglu, 2013. *Encyclopedia of Natural Hazards; Springer*; https://www.theguardian.com/cities/2015/apr/30/nepal-earthquake-disaster-building-collapse-resilience-kathmandu)

Fig. 1.2 Column hinging of the Imperial Valley Services Building during the 1979 Imperial Valley Earthquake (Source: Saatcioglu, 2013. *Encyclopedia of Natural Hazards; Springer*)

The Earthquake Disaster Risk Index (EDRI) report mentions about 59% area of Indian sub-continent is vulnerable for moderate to major earthquakes as depicted in Fig. 1.3. This was clearly evident from past earthquakes that occurred in India, in particular, in which several engineered structures (such as reinforced concrete buildings, bridges etc.) have experienced significant damages. In the last few decades, India has witnessed several devastating earthquakes with different levels of moment magnitude (M_w) like Bihar-Nepal border (1988) of M_w 6.4, Uttarkashi (1991) of M_w 6.6, Latur (1993) of M_w 6.3, Jabalpur (1997) of M_w 6, Chamoli (1999) of M_w 6.8, Bhuj (2001) of M_w 6.9, Sumatra (2005) of M_w 8.9, Kashmir (2005) of M_w 7.6, Himalayan (2011) of M_w 6.9, Nepal (2015) of M_w 7.8, Imphal (2016) of M_w 6.7, etc. Further, it has been reported that more than 90% of the casualties that occurred in past earthquakes in India are due to the collapse of numerous non-conforming commercial and residential structures (EDRI Report, 2019). This resulted in significant loss of life and property, which can be mitigated only by ensuring better code compliance of new constructions and rehabilitate/strengthen the existing structures to withstand the seismic hazard at the chosen location.

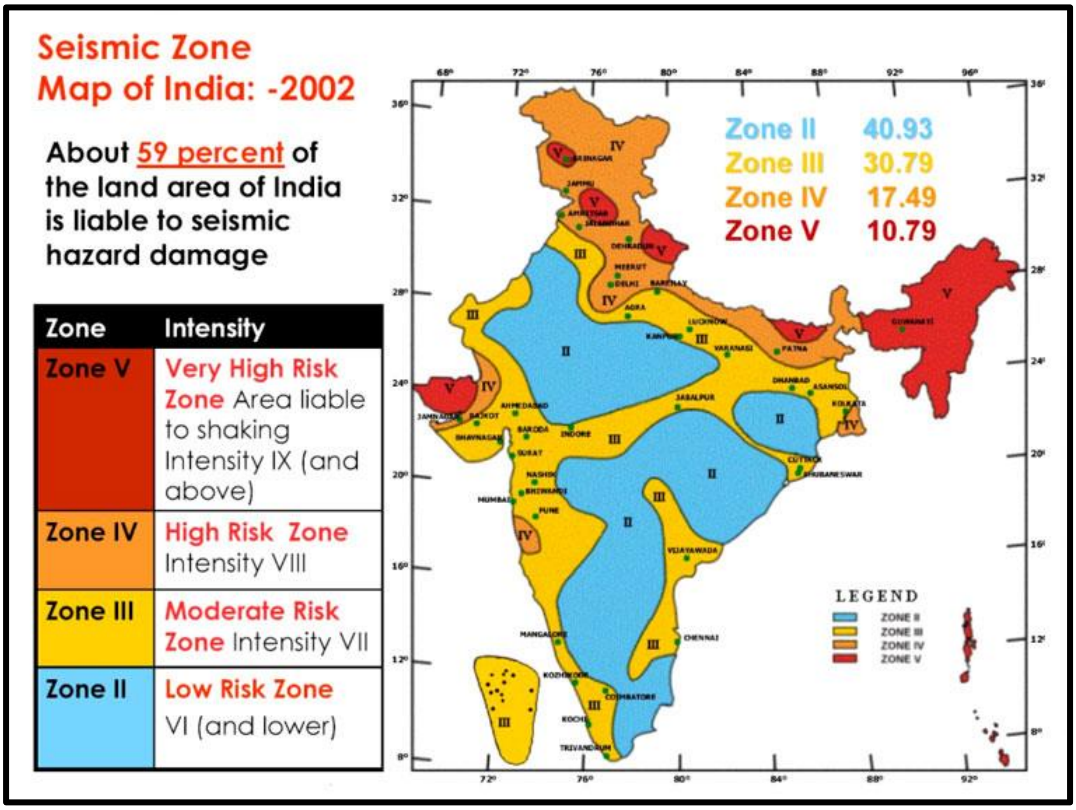


Fig. 1.3 Seismic zone map of India (Source: National Institute of Disaster Management, Ministry of Home Affairs, Govt. of India)

1.2 Sequential earthquake events

The occurrences of these earthquakes are in general, random oriented and often repeats itself multiple number times even after a major earthquake (commonly referred as mainshocks and aftershocks). This phenomenon has been witnessed during major earthquake disasters that have plagued different parts of the world. In several of these instances, sequence of seismic events succeeding the major earthquake were found to possess energy comparable or sometimes even higher than the major earthquake, characterized in terms of the earthquake magnitude scale. California (Mammoth Lakes, 1980; Coalinga, 1983; Whittier Narrows, 1987; Northridge 1994), Japan (Kobe, 1995; Niigata, 2004; Tohoku, 2011), New Zealand (Darfield, 2010; Christchurch, 2011), etc. are certain representative disasters reported in the literature, where multiple earthquakes were found to occur frequently possessing similar or higher magnitude of energy compared to the main earthquake event (Di Sarno, 2013). Sequential earthquake event is perceived as a low probability but high consequential event. Moreover, it has been observed that these repeated earthquake events often occur in a small interval of time, thereby impairs the repair/strengthening measures to be carried on any built infrastructure. This leads to accumulation of structural damages resulting in significant reduction in strength and stiffness characteristics of the structure often leading to collapse. In order to alleviate this effect on the structural behaviour, it is necessary to consider the effect of multiple earthquake forces during the structural modelling and analysis phase itself. This consideration of effect of multiple earthquake forces on evaluation of structural behaviour also aids in designing an appropriate retrofit/rehabilitation/strengthening measure for the existing structure ensuring its functionality throughout its serviceable lifetime.

Multi-storeyed RC buildings are considered to be a safe habitat in any urban environment. However, the lessons learnt from the major earthquakes that struck across the world and India in particular has cleared this myth by exposing the weaknesses of many residential and commercial RC buildings. In view of this, many Indian design codes for reinforced concrete (RC) buildings, including the earthquake resistant design codes for instance, have undergone several revisions to prevent global failure of the building structures. However, these existing seismic codes of practice for RC buildings consider only a scenario earthquake for analysis, often represented in terms of Design Basis Earthquake (DBE) or Maximum Considered Earthquake (MCE). These earthquake forces are often characterized in terms of response spectrum or chosen ground motion data for time history analysis at the specified location. This representation during modelling and analysis does not cater to the actual structural behaviour assessment due to non-consideration of the repeated nature of earthquake events.

In addition, the existing international standards and codes of practice for assessing the seismic performance of RC buildings specify a total inter-story drift limit, i.e., combined elastic and inelastic drifts as a performance measure. However, in the case of Indian standard code for earthquake resistant design of buildings for instance, a limit of 0.4% inter-story drift is recommended for elastic level design, ignoring the inelastic or plastic drifts (IS 1893, 2016). This being an essential criterion to assess RC buildings, there is an imminent need to evaluate the inelastic structural seismic capacity, utilising performance-based design methodology to ensure functionality of RC buildings for the targeted performance objective.

1.3 Response reduction factor (R)

The existing seismic design codes in most part of the world still adopt a force-based design approach. The non-linear response of the RC buildings represented by moment-resisting frames (MRFs) is accounted using an implicit representation of constant scale factor referred to as response reduction factor in a linear elastic design. This factor is also referred to as response modification factor, behaviour factor, response reduction factor, etc. in other international codes of practice, and symbolically represented as ‘R’ (EC 8, 2004; ASCE 7, 2010; IS 1893, 2016). The concept of response reduction factor was originally proposed to split the seismic-resistant design process into the quantification of the actual seismic demand (assuming that the structure remains elastic during the expected level of excitation) and prediction of the reserved capacity of a structural system (ATC 19, 1995). Various codes of practice existing around the world classify RC buildings in to different categories with appropriate R value. ASCE 7 classifies RC frame buildings as Ordinary (OMRF), Intermediate (IMRF), and Special Moment Resisting Frames (SMRF) with appropriate reduction factors 3, 5, and 8 respectively (ASCE 7, 2010). European and Mexican codes account for ductility requirements only and ignore the reserve strength. Besides, certain international codes (EC 8, 2004; ECP 203, 2007; ECP 201, 2012) do not differentiate between steel and RC frames in assigning ‘R’. However, the NEHRP regulations of the USA provide high ‘R’ compared to India, Mexico, Japan, and European seismic provisions (ATC 19, 1995; FEMA 273, 1997).

In view of aforementioned observations in literature, the majority of existing seismic codes have undergone numerous revisions in arriving at a realistic estimation of seismic demand. According to seismic provisions specified by IS 1893 (2016), moment-resisting frames of RC buildings are specified as Ordinary Moment Resisting Frames (OMRFs) and Special Moment Resisting Frames (SMRFs) with appropriate response reduction factors given as 3 and 5 respectively. However, these constant values fail to represent the influence of the changes in structural configurations with in an RC MRF, (viz., building height, number of bays present, bay width, irregularities arising out of mass and stiffness, etc.). These changes significantly alter the dynamic characteristics of the building structure (Chaulagain *et al.,* 2014).

Analytically, 'R' can be computed for any structural configuration using non-linear static analysis (NLS) and non-linear dynamic analysis (NLD) approaches. Several investigations presented in the literature for the estimation of the response reduction factor has adopted only non-linear static analysis (NLS) approach owing to its ease of implementation. Moreover, these investigations are focussed on consideration of fundamental mode of vibration for estimation of seismic response. This consideration of fundamental mode alone does not address the influence of irregularities present in the building configurations, which necessitates the multi-modal participation in dynamic response evaluation. Hence, it is imperative to consider the influence of irregularities in the estimation of 'R' for a particular structural configuration (Miranda *et al.,* 1994; Mwafy *et al.,* 2002; Arslan *et al.,* 2007; Ceylan *et al.,* 2010). In view of these observations, there is a necessity to focus on development of a modified R-factor which considers the changed dynamic characteristics of a structure, to arrive at a safe, economical and functional structural configuration even under repeated earthquake events.

1.4 Need for sequential forces in seismic analysis – Case Studies

Salient points from certain representative earthquake case studies collected from literature are presented to understand the influence of repeated nature of earthquakes on the built infrastructure.

1.4.1 2011 Christchurch earthquake, New Zealand

- The Christchurch earthquake sequence was initiated by the 7.0 magnitude Canterbury earthquake on September 4, 2010.

- Large magnitude earthquakes occurred later on the most devastating earthquake nucleated underneath Christchurch on February 22, 2011.
- More than 361 aftershocks occurred in the first week following the 6.3-magnitude earthquake.
- 185 people from more than 20 countries died in the earthquake.
- The latter earthquake caused more damage to structures and lifeline systems than the former although it was of a smaller magnitude, with an estimation of about 10,000 houses requiring demolition and over 1,00,000 houses damaged.

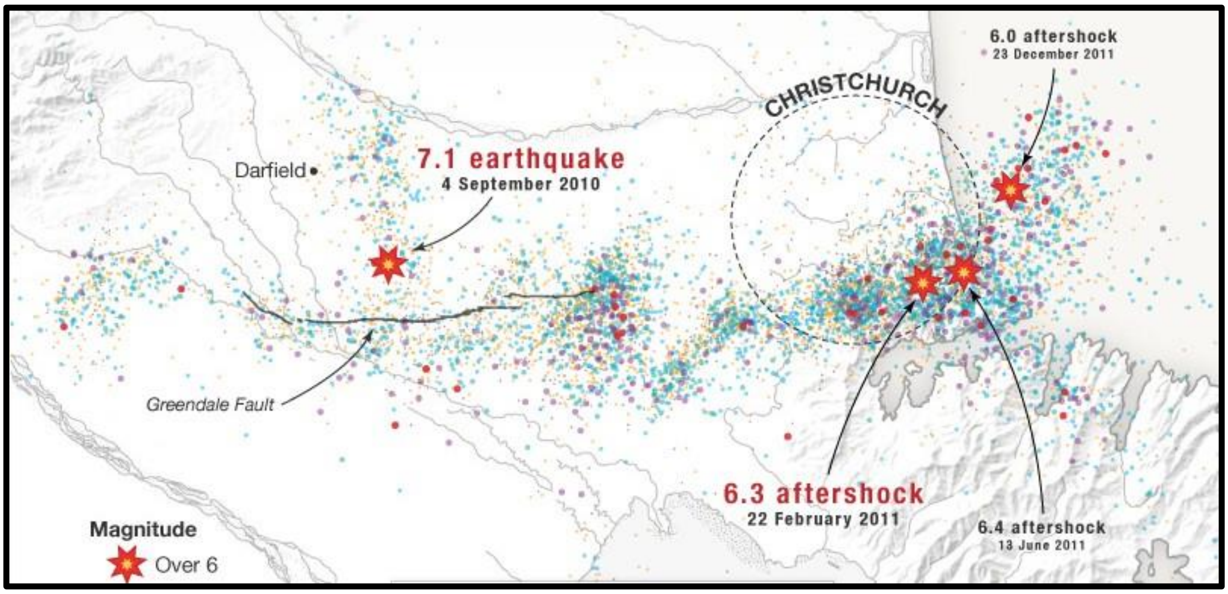


Fig. 1.4 Aftershocks of various magnitudes in the surrounding regions

(Source: https://mch.govt.nz/perspectives/earthquakes/)

Some of the real examples where structures failed due to aftershocks around the world reported in various news articles are listed below:

New Zealand hit by aftershocks after severe earthquake (dt. 14 November 2016)
https://www.bbc.com/news/world-asia-37970775

- Strong aftershocks have rocked New Zealand following a 7.8-magnitude earthquake that killed two people.
- The South Island has seen hundreds of tremors, including a 6.3-magnitude quake, after the initial one struck after midnight on Monday.

- Residents near the epicentre begin cleaning up after the earthquake and several aftershocks caused severe damage.

'Thousands of homes need to go' (dt. Jun 15 2011)

http://www.stuff.co.nz/business/rebuilding-christchurch/5139229/Thousands-of-homes-need-to-go

- A new fault, south of the Port Hills fault, is now believed responsible for yesterday's major aftershocks in Christchurch.
- GNS Science seismologists said the newly-confirmed fault had already generated a number of quakes since the deadly February 22 event.
- Dr Bill Fry said the dominant energy in yesterday's magnitude 5.7 and 6.3 aftershocks had been horizontal, compared with the vertical action in February's 6.3 quake.
- This meant they were felt differently.
- A total of 29 aftershocks have rocked the city since yesterday's two powerful earthquakes.
- Speaking at the same press conference, Sutton announced, “75 previously undamaged buildings in the red zone would need to be demolished in the wake of yesterday's quakes.”

Landmarks suffer further damage (dt. Jun 15 2011)

https://www.stuff.co.nz/the-press/5144934/Landmarks-suffer-further-damage

- Monday's aftershocks shattered many Christchurch landmarks already damaged in the February earthquake.
- The collapsed Christ Church Cathedral rose window cannot be fixed, the Cathedral of the Blessed Sacrament dome is more perilous than before, the Arts Centre has suffered major damage and the timeball from the destroyed Lyttelton Timeball Station was thrown 15 metres downhill.
- The cathedral has sustained more significant damage.
- There had been "significant additional damage" in the red zone, with 147 buildings suffering more damage. Some of the buildings had been damaged in previous quakes, while others had not.

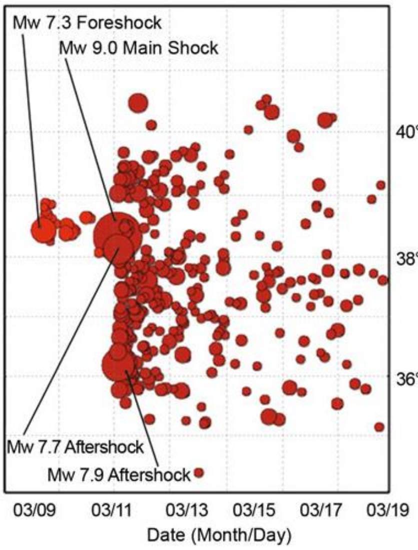
More significant quakes hit Canterbury (dt. 14 June 2011)

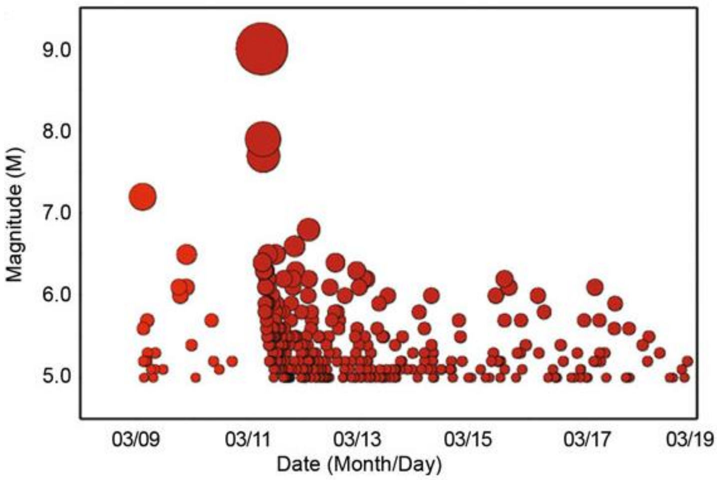
https://www.rnz.co.nz/news/canterbury-earthquake/77629/more-significant-quakes-hit-canterbury

- Powerful earthquakes have again rocked Canterbury on Monday, causing injury and damage.
- Some buildings already damaged in the February quake and due for demolition collapsed on Monday.

1.4.2 2011 Tohoku earthquake, Japan

- The Tohoku earthquake sequence, in Japan, generated more than 1,000 aftershocks of magnitudes 4+.
- After the M9.1 earthquake on March 11, 2011 Japan had experienced
    - 900 aftershocks in total
    - 60 aftershocks being over magnitude 6.0
    - three over magnitude 7.0
- The epicentre of this aftershock was close to the Japanese shore; consequently, it caused heavy damage and collapse of buildings in cities located near the shore.
- It is worth noting that these buildings were slightly damaged after the mainshock on March 11.





(a) (b)

Fig. 1.5 (a) Distribution of foreshocks, main shock, and aftershocks (b) Occurrences of aftershocks for several days after mainshock (Source: Liu and Zhou, 2012)

1.4.3 2015 Gorkha earthquake, Nepal

Quake-hit building collapses, triggers scare in south Delhi:

"Six months back, after the Nepal earthquake, a huge crack had developed in the building. We called a neighbourhood builder who said it was fine and won't cause any serious problem. After the tremors of October 26 quake, the crack got bigger and it got scary. All of us moved to other places nearby. This morning the whole building just tilted and almost fell on the adjoining structures," Manish Pawar, one of the occupants of the damaged building said.

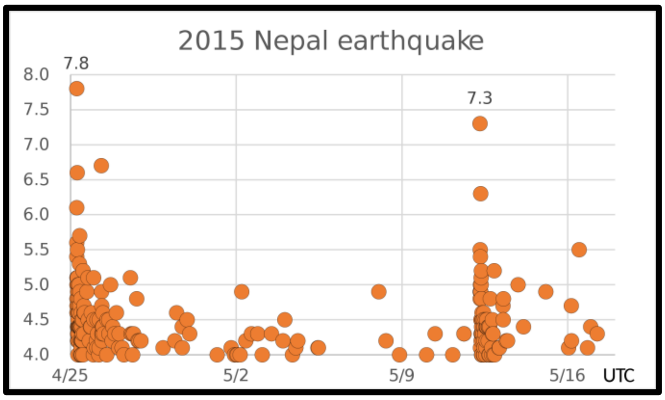


Fig. 1.6 Damaged building in Delhi, India due to aftershocks of Gorkha earthquake, Nepal
Source: https://www.hindustantimes.com/delhi/quake-hit-building-collapses-triggers-scare-in-south-delhi/story-lvCDJWwLsLheQDWFI3ZdyO.html

It can be observed that the repeated nature of earthquakes has significantly contributed in impairing the repair/rehabilitation/strengthening measures leading to accumulation of damages resulting in loss of functionality or collapse of structures. Hence, there is a necessity to consider these repetitive nature of earthquake forces to accurately predict the seismic behaviour.

Nepal: 3 injured, several houses destroyed in aftershock of 2015 earthquake (dt. May 19, 2021)

https://www.aninews.in/news/world/asia/nepal-3-injured-several-houses-destroyed-in-aftershock-of-2015-earthquake20210519100712/

- A 5.8 magnitude aftershock that struck Lamjung in Nepal in the early hours of Wednesday left three people injured, and damaged at least seven houses, officials confirmed.

- "We can confirm now that it's an aftershock of 2015's earthquake which had epicenter at Gorkha. Aftershocks as such sometimes continues for long time," Dr Lokbijay Adhikari, Senior Seismologist at National Earthquake Monitoring and Research Center told ANI over the phone.

Nepal earthquake: such huge aftershocks are rare (dt. May 15, 2015)

https://theconversation.com/nepal-earthquake-such-huge-aftershocks-are-rare-41833

- The 7.3 magnitude earthquake that hit Nepal on May 12, just weeks after the devastating 7.8 magnitude event, should be classed as an aftershock rather than a second earthquake.
- Although there are relatively few examples of such big aftershocks in history, the tragic events in Nepal demonstrate that we must always be prepared for them.

1.4.4 2020 Petrinja earthquake, Croatia

Croatia earthquake: Strong aftershocks hit after quake kills seven (dt. 30 December 2020)

https://www.bbc.com/news/world-europe-55485106

- A series of powerful aftershocks have rocked central Croatia following a magnitude 6.4 earthquake on Tuesday.
- The new 4.8 and 4.7 magnitude tremors struck at around 06:15 local time (05:15 GMT) on Wednesday, causing further damage to buildings.
- "This morning we were hit by the third, if not the fourth earthquake," the mayor of Petrinja, Darinko Dumbovic, told state television early on Wednesday.
- "Everything that has not yet fallen is falling," he added.

Fig. 1.7 Half the town Petrinja had been destroyed

(Source: https://www.bbc.com/news/world-europe-55485106)

1.4.5 2020 Cyprus earthquake, Cyprus

5.7 magnitude earthquake, aftershocks caused estimated $1 million damage to Cyprus High School (Apr 6, 2020)

https://www.deseret.com/utah/2020/4/6/21210076/utah-earthquake-magnitude-5-7-damage-cyprus-high-school

- The Magna area was the epicenter of the 5.7 magnitude earthquake that rocked the Salt Lake Valley the morning of March 18.
- The damage was caused by a recent 5.7 magnitude earthquake its aftershocks centered near the town.

1.4.6 2021 Crete earthquake, Greece

Greece: 'Surprise' aftershock adds damage to quake-hit area (dt. 5 March 2021)

https://abcnews.go.com/International/wireStory/greece-surprise-aftershock-adds-damage-quake-hit-area-76257800

- A powerful aftershock with magnitude 5.9 has caused additional damage in central Greece, a day after an earthquake damaged hundreds of homes in the same area.
- Authorities said some empty houses, which had been abandoned by their inhabitants after suffering damage Wednesday, collapsed but no injuries were reported.
- It caused no serious injuries to public, but scores of homes were damaged in several villages.

1.5 Research significance

Most of the existing seismic codes around the world, consider earthquakes as a single ground motion often represented in the form of response spectrum at that particular location. These representations of seismic forces do not address the effects of repeated nature of earthquakes (represented in terms of seismic sequences) on built infrastructure (RC buildings in particular). Hence, research on understanding the dynamic response of structures under seismic sequences have received attention since 2010. Further, capability of these structural systems to meet the intended seismic performance objectives need to be evaluated in order to design a safe and resilient structural system.

In general, seismic analysis is being caried out on two-dimensional (2D) structural models considering seismic force along one particular direction at once. This consideration will not be able to capture the out-of-plane interaction of the structural components with the RC frame. Hence, analysis on three-dimensional (3D) RC structural models subjected to simultaneous bi-directional force need to be carried out to capture the actual behaviour of structural components, albeit the computational cost. Also, evaluation of dynamic response of IS code-designed buildings in particular for sequential earthquake events have not been reported in literature yet in spite of witnessed vulnerabilities to the built infrastructure and causalities in the past earthquakes that occurred in India.

Therefore, to address the aforementioned issues, an attempt has been made to investigate the non-linear response and adequacy of code-mentioned ‘R’ factor under single/sequential earthquake forces for various Indian code-designed RC buildings located in moderate seismic zone.

1.6 Scope of the study

In order to address the issues discussed above in section 1.5, the seismic behaviour assessment of 3D RC MRFs of regular and vertically irregular configurations (with and without interaction of infills with MRF) under simultaneous bidirectional representation of sequential earthquake events are carried out. This involves:

- ❖ Modelling and analysis of hypothetical RC building frames conforming to the seismic provisions of IS code located in moderate seismic zone with appropriate site conditions (Warangal City, Telangana State).
- ❖ Consideration of interaction of infill with RC MRF by modelling of infill wall as a diagonal strut, in accordance with IS 1893 (Part 1): 2016 regulations.
- ❖ Development of spectrum compatible accelerograms from real ground motion records in accordance with the response spectrum corresponding to the codal provisions at the considered site location. This facilitates in the development of ground motion records for necessary single as well as sequential earthquake.
- ❖ Assessment of seismic behaviour of structural configurations considered using non-linear analysis under simultaneous single and sequential bi-directional earthquake forces in terms of structural response parameters.
- ❖ Checking the adequacy of code-specified R-factor in appropriate representation of inelastic seismic capacities of the RC building frames under single and sequential earthquakes.
- ❖ To propose a methodology for development of modified R-factor for RC frame buildings under single and sequential earthquake forces considering different hazard levels and performance levels to arrive at safe and functional structural design configurations.

1.7 Objectives of the study

In order to investigate the seismic behaviour of 3D RC buildings under repeated earthquake events, characterised as sequential forces and to provide an appropriate R-factor to represent more accurate seismic demand, the following objectives were framed for this study.

1). Behaviour assessment of 3D RC frame buildings under single earthquake event

2). Behaviour assessment of 3D RC frame buildings considering sequential earthquake event succeeding the primary or first earthquake event

3). Adequacy of code-specified Response reduction factor (R) in computing design lateral forces for RC buildings.
4). Propose an analytical formulation for development of modified R-factor under single and sequential earthquake events, utilizing inelastic capacity of the RC buildings under different hazard levels for targeted performance criteria.

1.8 Thesis organisation

The thesis is organised into eight chapters and corresponding details regarding contents of the chapters is also specified as mentioned below.

Chapter 1: Deals with brief introduction to importance of seismic analysis of built infrastructure along with the scope and objectives of this study.

Chapter 2: Literature review pertinent to the area of investigation in terms available journal articles are presented and thorough review of these articles helped to arrive at the aforementioned objectives for the study as discussed in section 1.2 for the study.

Chapter 3: Deals with various modelling and analysis procedures for seismic behaviour assessment of RC buildings.

Chapter 4: Deals with evaluation of seismic behaviour of three-dimensional RC building frames (with vertical setbacks and with and without infill wall contribution) under bi-directional single earthquake events are presented. Here, the seismic behaviour assessment is represented in terms of several response parameters obtained from NLD (non-linear dynamic analysis) of the structures. Further, the influence of masonry infill wall interaction with surrounding frame on the behaviour of the building structures designed as per the seismic provisions of Indian code is also discussed.

Chapter 5: The seismic behaviour assessment of three-dimensional RC building frames (with vertical setbacks and with and without infill wall contribution) conformed to the Indian standard codes of practice under bi-directional sequential earthquake events is presented. Here the seismic behaviour is evaluated in terms of several response parameters arrived from NLD analysis of the structures. In addition, the vulnerability assessment of RC building frames under single and sequential earthquake forces is also discussed.

Chapter 6: Presents adequacy of code-specified Response reduction factor (R) in accurate estimation of design lateral forces for RC building frames. The analytical estimation of R-factor based on inelastic capacity of frame is also discussed.

Chapter 7: Proposed analytical formulation for development of modified R-factor under single/sequential earthquake events has been discussed. Further application of this proposed modified R-factor is presented with a case study.

Chapter 8: The conclusions drawn from overall observations on seismic behaviour assessment of RC frames under single/sequential earthquakes have been presented. Further the significant contributions from this study, and limitations for this study, along with scope for further research is also discussed.

CHAPTER 2

Literature Review

A detailed literature review has been carried out considering various aspects of assessment of structural behaviour of RC buildings, and categorised into different groups as presented below:

2.1 Setback buildings

Wong and Tso (1994) investigated seismic response of structures with setback irregularity using elastic response spectrum analysis, and it was observed that the structures with setbacks had mass participation from higher modes of vibration, which further lead to different seismic load distributions other than the static code-specified procedure.

Kappos and Scott (1998) performed a comparative study between static and dynamic methods of analysis for assessing the seismic response of concrete frames with setbacks. On comparison of results, it was concluded that dynamic analysis generated results different from that of static analysis.

Karavasilis et. al. (2008) studied the inelastic seismic response of about 120 two-dimensional European code conforming steel moment resisting frames with setbacks. All the frames were subjected to a set of 30 earthquake ground motions scaled to different intensities. It was concluded that extent of plastic deformations and structural geometrical configurations played a significant role in distributing the deformation demands along the height of the structure. The maximum deformation demands had seemed to concentrate in the tower for tower-like structures and in the regions surrounding the setbacks present in the buildings.

Athanassiadou (2008) investigated the seismic performance of 10-storey two-dimensional plane RC frames having various vertically irregular configurations. All those frames were designed as per the guidelines of Eurocode 8 (2004), and then analysed by both non-linear static analysis and non-linear dynamic analysis for selected time histories. It was concluded that the level of ductility has a negligible impact on the construction cost. In addition, traditional pushover analysis seems to underestimate the responses of the upper floors of the irregular frame. According to their study, the irregular buildings when designed in accordance with EC8 also perform equally well as regular buildings when subjected to earthquake loads.

Soni and Mistry (2006) reviewed research on the seismic behaviour of vertically irregular structures and their findings in existing building codes and literature, and summarized the information on the seismic response of irregular building frames. The building code provides criteria for classifying vertical irregular structures and recommends dynamic analysis to estimate the lateral design forces. It was also observed that most of the studies agree with the increased drift requirements of the tower portion of the irregular structures, and the increased seismic requirements of buildings with discontinuous mass, stiffness and strength distributions.

Sarkar et al. (2010) proposed a new method to quantify irregularity in building frames by accounting for dynamic properties such as mass and stiffness. The authors had developed a new parameter called as regularity index to assess the amount of irregularity of building frames with vertical setbacks. A new modified version of code-specified empirical formula to estimate the fundamental time periods of the irregular building frames was also proposed and validated on 78 different building frames with different levels of setback irregularities.

Varadharajan et al. (2013) conducted an extensive parametric study on two-dimensional RC moment resisting frames with setbacks. A parameter called irregularity index was proposed based on the dynamic characteristics of the frame in order to quantify the setback irregularity. Later, the effect of setback on inelastic deformation demands was also investigated by modelling building frames with different configurations of setbacks, which were designed as per the guidelines of European standard code of practice. These frames were then subjected to a set of 13 ground motions and analysed by time history analysis. The results indicated a strong influence of the parameters like number of storeys and geometrical irregularity on inelastic seismic demands of the frames.

Bhosale et al. (2017) studied the adequacy of fundamental mode properties of buildings in quantifying the vertical irregularity. An attempt was made to check the correlation between existing vertical irregularity indicators and their associated seismic risks in terms of fragility curve, annual probability of collapse, and drift hazard curve. The results indicated that there is no correlation between existing vertical irregularity indicators based on fundamental mode properties and seismic risks. It was also evident that the seismic risks of building frames with open-ground story and floating columns were higher than that of a similar regular building.

Bhosale et al. (2018) developed a new parameter called 'seismic vulnerability indicator' (SVI) based on the inter-story drift ratio to estimate the expected seismic risk of any vertically irregular building. The proposed SVI was found to be computationally simple and also correlates well with the potential seismic risk for different categories of vertically irregular RC frame buildings. This study can also be further extended to include other local damage parameters.

2.2 Sequential earthquakes

Hatzigeorgiou and Beskos (2009) investigated the effect of multiple earthquakes on the nonlinear behaviour of RC structures by evaluating the inelastic displacement ratios of SDOF systems on the basis of empirical expressions obtained after extensive parametric studies. Also, quantified the seismic sequence effect directly on the inelastic displacement ratio. It was concluded that multiple earthquakes require increased displacement demands in comparison with single seismic events (design earthquake), and the traditional seismic design procedure, which is essentially based on the isolated design earthquake, should be reconsidered.

Hatzigeorgiou and Liolios (2010) extensively studied the inelastic response of RC planar, regular and vertically irregular frames subjected to sequential ground motions. A significant accumulation of damage was observed due to lack of time to rehabilitate the frame to sustain the repeated earthquakes. It was also concluded that the ductility demands of the sequential ground motions can be accurately estimated using appropriate combinations of the corresponding demands of single ground motions.

Loulelis et al. (2012) carried out an investigation on the seismic behaviour of two-dimensional moment resisting steel frames (MRFs) subjected to repeated ground motions. For that, thirty-six MRFs designed for gravity and lateral loads as per the guidelines of European codes were subjected to 5 real and 60 artificial seismic sequences separately. It was reported that damage for repeated earthquakes is higher than that for single earthquakes, of the order 72% and 27% for local and global damage values. Also, the maximum top horizontal displacement was found to be increased by 100% or more, which is very important observation to be taken into account for the seismic design of structures. Overall, it was concluded that the displacement demands and ductility demands had significantly increased in case of repeated earthquakes when compared to that for single earthquakes.

Di Sarno (2013) studied the effect of multiple earthquakes by examining the inelastic constant ductility spectra and force reduction factor spectra. The results gave indications of the levels of lack of conservatism in the safety of conventionally-designed structures when subjected to multiple earthquakes. It was also confirmed that multiple earthquakes deserve extensive and urgent studies.

Hatzivassiliou and Hatzigeorgiou (2015) studied the effects of seismic sequences on three-dimensional reinforced concrete buildings, and observed that the seismic demands are increased due to multiple earthquakes. Another significant observation was that using artificial sequences lead to unreliable results due to the uncertainty in various characteristics of the mainshock and aftershock, such as magnitude, intensity, frequency content, and duration.

Hosseinpour and Abdelnaby (2017) investigated the effect of different aspects of multiple earthquakes on the nonlinear behaviour of RC structures by deriving fragility curves and damage probability under different earthquake intensities. It was concluded that it is necessary to consider the effect of the damage from an event and its impacts on the nonlinear behaviour of structures under subsequent events.

Oyguc et al. (2018) performed a detailed reconnaissance studies on the Tohoku earthquake. It was reported that most of structures in the earthquake-affected region had collapsed due to occurrences of multiple earthquake excitations. The strength and stiffness degradations were found to be the main reason for the damages occurred. Hence, the results indicated that effects of multiple earthquakes are significant, and irregularity effects increased the dispersed damage under these excitation sequences.

Amiri and Rajabi (2018) investigated the effect and potential of consecutive earthquakes on the response and behaviour of six moment resisting concrete frames (3, 5, 7, 10, 12 and 15 storeys) were designed and analysed for two different records with seismic sequences from both real and artificial cases. From the results, it was observed that the consecutive earthquakes have increased the accumulated damages and response of structures to almost 2 times. Therefore, it is definitely necessary to consider this effect in the design procedure of structures. It was also reported that the use of artificial seismic sequences as design earthquake can lead to non-conservative estimation of behaviour and damage of structures, and hence the usage of real seismic sequences is recommended.

Manafpour et al. (2019) investigated the response of a RC Single-Degree-Of-Freedom system subjected to a number of sequential earthquakes comprising both near- and far-field records. From the results, it was confirmed that the direction and amount of residual drift under the first earthquake affects the behaviour significantly. It was also reported that the set of near-field records resulted in a more critical structural behaviour.

2.3 Response reduction factor

Hatzigeorgiou (2010) evaluated the behaviour factors for nonlinear structures subjected to multiple near-fault earthquakes. The influence of period of vibration, post-yield stiffness ratio and viscous damping ratio is also taken into account. Proposed a new procedure for the ductility demands control of single-degree-of freedom systems under repeated near-fault earthquakes. Examining the influence of seismic sequences, it is concluded that frequent/smaller near-fault earthquakes necessitate equal or smaller behaviour factors in comparison with the 'design earthquake'.

Mondal et al. (2013) assessed R-factors for RC SMRFs of different heights (2-, 4-, 7-, and 12-storey frames) designed as per Indian standards. The frames were assumed to be located in higher seismic zone (zone IV). In this study, a deterministic framework was used, and non-linear static analysis was performed on all the models. From the results it was observed that the design of the frames using code-specified R was inadequate to ensure life safety performance limit. This was understood from the obtained R-factors which were lower than the code-specified value of 5. The structural behaviour is not validated by any nonlinear time-history analysis.

Chaulagain et al. (2014) estimated the actual 'R' value for RC buildings designed and constructed following the code requirements located in Kathmandu valley. The seismic performance of the buildings considered was evaluated using non-linear static analysis on the structural models. The 'R' values for RC buildings of different geometrical configurations obtained from analysis were found to be lesser than those recommended in the IS 1893 (2002). The structural behaviour was not checked by any other non-linear time history analysis.

Al-Ahmar and Al-Samara (2015) investigated two case studies which included 25 numerical analyses, and obtained the R factor of SMRFs through non-linear static analysis. The influence of the number and span of bays and the number of storeys on R factor was investigated. It was reported that the R factor significantly changes based on the number of storeys, where it decreases as the number of storeys increases. In contrast, compared with the number of storeys, the number and length of the bays had a negligible effect on R. Hence, it was concluded that using a fixed R value may lead to unsafe or uneconomical building design.

Zhang et al. (2017) investigated the strength reduction factor of single-degree-of-freedom (SDOF) system subjected to the mainshock–aftershock sequence-type ground motions, considering the effects of displacement ductility and cumulative damage on estimation of the reduction factor. It was found that the aftershock ground motion has significant influence on strength reduction factors, and the damage-based strength reduction factor is about 0.6–0.9 times of the ductility-based strength reduction factor.

Abou-Elfath and Elhout (2018) evaluated the R values of RC MRFs designed according to Egyptian code specifications providing sufficient ductility. In this study, both non-linear static analysis and non-linear dynamic time history analysis were performed to compute the actual R values of nine RC MRFs with different geometric configurations. The value of R-factor decreases as the height of the frame increases. It was also reported that the changes in the number of bays and the spans of the bays have negligible effect on R-factor.

Vona and Mastroberti (2018) evaluated the seismic capacity of the main existing RC MRF building types. In order to accurately evaluate the seismic force demands, the actual values of R factor computed using force-based seismic design procedure have been proposed and compared with the Italian seismic code-recommended R values. Due to the variations in building structural configurations (such as distribution and effectiveness of infill interaction, number of storeys) and mechanical properties (such as concrete strength) which substantially effect the seismic response of buildings, the code-recommended R values seemed unable to represent the real variability of the R factors. In most cases, the code-recommended R values, underestimated the actual R values.

Badal and Sinha (2020) investigated archetypical buildings of different performance groups (2- to 12-storey frames) designed as per Indian standards for performance limit corresponding to Collapse Prevention at MCE hazard level. It was reported that the effect of the number of bays and bay-width of an RC frame building on R is not significant, and do not influence their seismic performance, and the low- and mid-rise buildings located in seismic zone-V were not able to meet the expected seismic performance.

2.4 URM infill walls

Kaushik et al. (2007) investigated the stress-strain characteristics of clay brick masonry under uniaxial compression, and proposed a simple analytical model for obtaining the stress-strain curves for masonry that can be used in the analysis and design procedures.

Haldar et al. (2012) studied the seismic performance of Indian code-designed RC frame buildings with and without URM infills, and also performed fragility analysis. It was suggested that URM infills result in a significant increase in the seismic vulnerability of RC frames and their effect needs to be properly incorporated in design codes.

Uva et al. (2012) presented a case study on a critical comparison of bare frame and infilled frame, and deduced some observations about the modelling of the infill. It was reported that while choosing an equivalent strut model to represent the effect of infill panels, it is crucial to adopt multi-strut systems. Also, the realistic assessment of the masonry mechanical parameters to be introduced in the strut models is important.

Haldar et al. (2013) reported a study on various failure modes of buildings identified from a stock of available earthquake damage survey reports, experimental studies, analytical models and design codes. The study also presented a review of available models for estimating the strength of infills and frame members in various failure modes. In addition, guidelines for simulation of seismic behaviour of infilled frames were developed based on the governing failure modes of infills

Burton and Deierlein (2014) presented improved methods of analysis and guidelines to perform simulations of seismic collapse of non-ductile concrete frames with URM infills. The post-peak behaviour of the masonry infill and the infill-frame interaction was modelled with the help of the proposed inelastic dual-strut model. The infill-frame interaction was found to be critical in predicting the collapse capacity of the considered non-ductile frames with URM infills. It was also reported that the strength of the infill strut had significantly greater effect on the collapse performance of the structures than that of the deformation parameters of the infill strut.

Cavaleri and Trapani (2014) discussed a criterion for modelling the structural behaviour of infills based on a macro-modelling approach, that is to say on the substitution of infills with diagonal pin jointed struts. The validation of Pivot modelling approach was carried out comparing experimental results and computer simulations of the experimental tests.

Morfidis and Kostinakis (2017) used 54 three-dimensional reinforced concrete buildings with different heights, structural systems, and masonry infill distribution to perform a comprehensive assessment. The considered buildings were analysed by non-linear time history analysis for about 80 bi-directional earthquake sequences. In order to consider the influence of the angle of incidence on the response of the structure, two horizontal acceleration maps of each movement of the ground are applied along the horizontal orthogonal axis that forms 12 different angles with the axis of the structure. From the results it was evident that compared to the bare structure, the seismic sequential forces had a greater impact on the structural damage of the infill building.

Oinam et al. (2017) investigated three RC frames with different configurations of masonry infills. Non-linear cyclic pushover analysis was carried out to study the effect of masonry infills and hysteretic behaviour on the response of RC frames. It was observed that yielding in all the frames started at a drift of 0.75%, but load degradation has started to occur at 2.75% drift in case of bare frames and at 3.5% drift in case of infill frames and open ground storey frames. Hence, it was reported that overall performance of fully infilled frame is far better than that of the bare frame and open ground frame.

Pokhrel et al. (2018) studied the effect of variation on infill masonry walls in the seismic performance of low-rise soft-storey RC buildings. Non-linear pushover analysis for a bare frame and the masonry infill extent as 25%, 50%, 75%, and 100% for a representative four-storeyed soft-story RC building, and extracted results in the form of various parameters such as base shear, performance point, inter-story drift, story level deflection, and fundamental natural period of the building. It was concluded that infill walls significantly affect the seismic performance of RC buildings, and hence deserve substantial care in design process.

2.5 Seismic analysis of RC buildings

Ghersi and Rossi (2001) studied the influence of bidirectional seismic excitation on seismic responses of stiffness eccentric one-storey building systems using elastic and inelastic analysis. The responses from the inelastic seismic analysis when compared with the responses of elastic seismic analysis had showed that the consideration of effects of bidirectional seismic excitation had resulted in variation in seismic response. On the other hand, elastic analysis using unidirectional seismic excitation was found to overestimate the seismic response.

Vamvatsikos and Cornell (2002) proposed a novel method of analysis called Incremental Dynamic Analysis (IDA), to thoroughly estimate the structural performance under seismic loads. New terminology and suitable algorithms were also presented to investigate both the behaviour of SDOF and MDOF systems. In this analysis method, a structure is subjected to multiple ground motions, with each one scaled to multiple levels in order to get the complete picture/behaviour of structures starting from its elastic response to yielding to inelastic response, and until reaching collapse state.

Inel and Ozmen (2006) investigated to identify any differences between pushover analyses on structures modelled using user-defined hinges, and those modelled using default hinges available in finite element software packages (i.e., SAP2000 based on the ATC-40 or FEMA-356 documents). Four- and seven-story buildings representing low- and medium-rise buildings were considered for this study. It was observed that the user-defined hinges were better than the default hinges in accurately representing the non-linear behaviour of pre-code buildings. However, for evaluating modern code compliant buildings, when default hinges modelled cautiously give accurate behaviour of the buildings.

Barbat et al. (2008) analysed the seismic risk of the buildings of Barcelona, Spain using Capacity spectrum method. The procedure to develop fragility curves for those buildings from the capacity curves obtained from non-linear static analysis was explained. It was concluded that in spite of being a low-to-moderate seismic region, the buildings present in Barcelona were highly vulnerable to earthquakes.

Choudhury and Kaushik (2015) reviewed various methods used in the past to quantify all the main components required for vulnerability and risk assessment, compared them using a step-by-step method, and discussed the advantages and disadvantages of those methods. Firstly, the seismic risk of the location was assessed where there is demand curves were available. It was reported that there is deficiency in maps providing site-specific hazard data and ground motion data at various regions in India. Further, the buildings were grouped categorically to evaluate representative capacity of the building category. Finally, fragility curves were developed for the corresponding buildings for all damage states considering various uncertainties associated with the analysis procedures. This ultimately leads to the development of location-wise vulnerabilities, which can further help estimate the earthquake risk due to specific disasters in a given area and a given reference period. Hence, it was stated that these data are essential for developing seismic risk maps of the area.

Dhir et al. (2018) performed vulnerability studies on gravity load–designed (GLD) reinforced concrete (RC) buildings. The relative seismic vulnerability of a GLD building subjected to seismic hazards in a practical load and resistance factor format corresponding to various seismic zones of India was evaluated. The relative vulnerability of the GLD building when compared to a building designed for seismic loads was found to be on much higher side. This vulnerability increased from lower to higher seismic zone. It also reported that the GLD buildings existing in higher seismic zones of India (IV and V) should be immediately uninhabited to avoid devastating catastrophe.

2.6 Summary of literature review

Following observations are made from the detailed literature review:

- ❖ Sequential earthquakes events require increased lateral displacement demands in comparison with that of single seismic events i.e., design earthquake (Hatzigeorgiou and Beskos, 2009; Hatzivassiliou and Hatzigeorgiou, 2015; Hosseinpour and Abdelnaby, 2017).
- ❖ Sequential earthquake events have shown increased damages in case of irregular building configurations (Oyguc *et al.*, 2018).
- ❖ Sequential earthquakes are a reality and needs immediate attention in order to design a safe, functional and seismic resilient infrastructure systems (Di Sarno, 2013).
- ❖ Real ground motion records need to be considered based on the site location for generating sequential earthquake forces to be used for seismic analysis, instead of

generating artificial earthquake ground motions (Hatzivassiliou and Hatzigeorgiou, 2015).

- The infill-frame interaction is critical for prediction of the collapse capacity of RC building frames in case of residential structures (Haldar *et al.*, 2012; Burton and Deierlein, 2014).
- Response reduction factor (R-factor) is an essential parameter to analyse the inelastic capacity of structural systems. Most of the existing seismic codes, including IS 1893 in particular recommend a constant value for a particular structural type, irrespective of its configuration (Hatzigeorgiou, 2010; Al-Ahmar and Al-Samara, 2015; Abou-Elfath and Elhout, 2018; Badal and Sinha, 2020).
- Code-recommended values of R-factor appears inappropriate in capturing the actual behaviour of multi-storeyed RC frame buildings, which significantly affect the seismic response of a structure thereby estimation of its structural behaviour (Mondal *et al.*, 2013; Chaulagain *et al.*, 2014; Vona and Mastroberti, 2018).

CHAPTER 3

Modelling and analysis

3.1 Introduction

In this chapter, the necessary background required for modelling and analysis of RC buildings is presented. The relevant concepts and definitions like seismic design philosophy, non-linearity in reinforced concrete framed buildings, different structural configurations, unreinforced masonry infill walls modelling, seismic analysis methods, performance assessment parameters, and fragility analysis are discussed.

3.1.1 Seismic design philosophy

Engineers do not try to design earthquake-proof buildings which will not be damaged even when subjected to a strong earthquake (which are rare), because such buildings would be too strong and also too expensive to build. Instead, the practical goal is to make buildings earthquake-resistant, which are resistant to the effects of ground shaking; though those buildings can get severely damaged, they would not collapse during a strong earthquake. Thereby, ensuring the safety of people and property in earthquake-resistant buildings, and thus a disaster is avoided. It is a major goal and philosophy of seismic design codes around the world. The seismic design philosophy may be summarized as follows:

a) Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however, building parts that do not carry load may sustain repairable damage.
b) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake; and
c) Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse.

The consequences of damage must be taken into account in the design philosophy. For example, critical buildings, such as hospitals and fire stations, play an important role in post-earthquake operations and must remain operational immediately after an earthquake. These structures must withstand very little damage and must be designed to provide a higher level of earthquake protection.

The design requirements for lateral loads are fundamentally different from those for vertical loads (dead and alive). Designing for seismic loads deals with events with a lower probability of occurrence. Therefore, it can be very uneconomical to design earthquake-resistant structures for higher performance levels. The seismic load can reach only part of the weight of the structure (~30-40%), acting horizontally. If the plastic design concepts used for primary loads are used for seismic loads, extremely heavy and expensive structures will emerge. Therefore, the necessary seismic design uses the concepts of controlled damage and collapse prevention. Indeed, buildings are generally designed to withstand only part of the elastic force (~15-20%) of an earthquake. This is illustrated in Fig. 3.1, where the elastic and inelastic reactions are described, and the concept of equal energy is used to reduce the design force from V_e to V_d (representing the elastic force and the design force respectively). Therefore, damage is inevitable in the seismic response and design. The type, location and extent of damage are the goals of the earthquake engineering and design process.

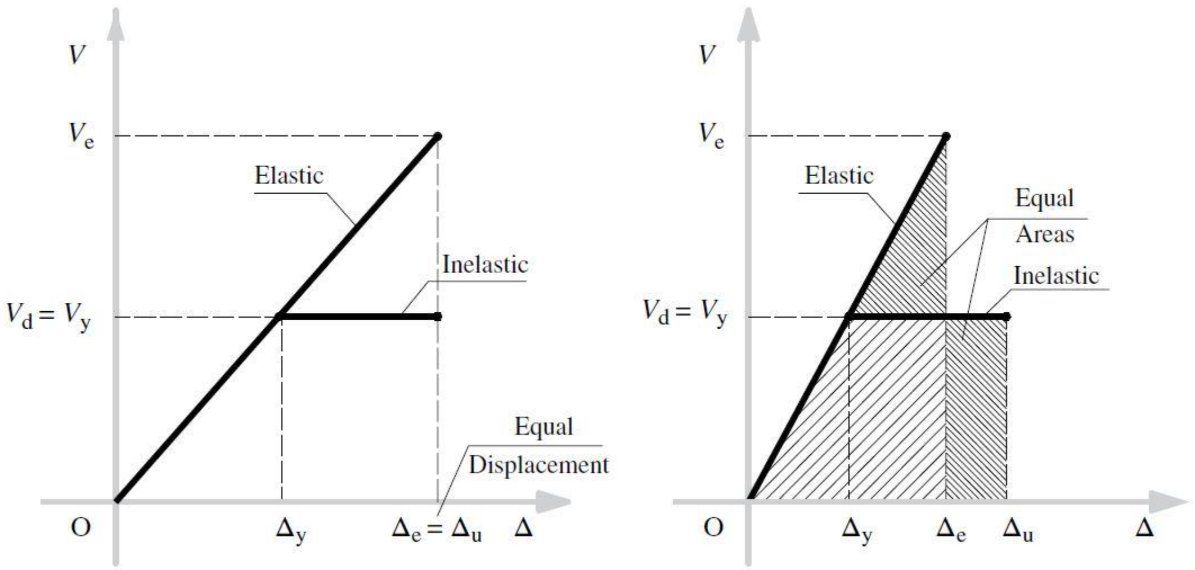


Fig. 3.1 Force – displacement relationships for inelastic single degree of freedom systems (Source: Fundamentals of Earthquake Engineering by Amr S. Elnashai and Luigi Di Sarno)

Δe = elastic displacement; Δy = yield displacement; Δu = ultimate displacement; V_d = design base shear; V_e = elastic base shear; V_y = yield base shear and V_u = ultimate base shear

3.2 Modelling

3.2.1 *Nonlinear behaviour definition*

Reinforced Concrete is a widely used construction material in many parts the world. In most construction works, RCC is preferred over other building materials (i.e., masonry, structural steel, timber, etc.) for its strength, mouldability, economy, and ready availability. From the perspective of structural analysis and design, RCC is a very complex composite material. It is about combining concrete and steel with completely different mechanical properties to develop a composite material that behaves like an elastoplastic material and responds differently to tensile and compressive stresses. Furthermore, due to cracking of the concrete, even the cross-section and structural characteristics depend on the nature and magnitude of the applied load. These complexities generally manifest themselves when the structure is subjected to dynamic loads such as earthquakes, wind, storms, and waves. Although material non-linearity or time-varying characteristics are important, they are rarely considered in the analysis and design of RCC structures.

In earthquake engineering the inelastic earthquake response of buildings is of great importance, as most buildings, when subjected to strong earthquakes, are expected to deform past the limit point for elastic behaviour. When the stress (or strain) level in structural components buildings exceeds a certain value, the building enters into a non-linear state. In general, the non-linearity in a building occurs due to non-linearity in the material, the structural components, or the combination of both. The non-linearity is due to changing geometric configuration of structure creating additional effects and large deformations. Non-linearity due to changing geometric configuration is called as geometric non-linearity, and the non-linearity due to material is called material is called material non-linearity.

3.2.1.1 *Material non-linearity*

In real structures, materials undergo plastic deformations under loads, and the constitutive relationship of the material, is no longer linear. This plastic (or non-linear) behaviour of materials has to be considered while modelling and appropriate non-linear material models have to be used to represent accurate behaviour of the materials.

The stress-strain curve of concrete in compression is the basis for analysing any reinforced concrete section. The characteristic curve and the design stress-strain curve specified in most design codes (IS 456: 2000, BS 8110) do not really reflect the actual stress-strain behaviour in the post-peak region, because for the convenience of the calculation, it assumes that the stress in this region is constant (strain between 0.002 and 0.0035). In reality, as the experimental tests show, the post-peak behaviour is characterized by descending branches, which are attributed to softening and microcracks in the concrete. Different building codes also provide guidelines on the yield strain and ultimate strain of the both concrete and steel. BS 8110 recommends 0.0035 as the ultimate concrete strain, while ACI 318 recommends 0.003. Also, the models based on these codes do not consider the improvement of strength and ductility due to confinement.

In various literature, many empirical stress–strain relations are proposed for the confined concrete, based on results obtained from experimental investigations, accounting for additional strength and ductility from providing confinement. Some important stress-strain models are *Kent-Park* Stress-Strain Model (1971), *Mander et al.* Stress-Strain Model (1988), *Scott et al.* Stress-Strain Model (1982), *Yong et al.* Stress-Strain Model (1989), *Bjerkeli et al.* Stress-Strain Model (1990), *Li et al.* Stress-Strain Model (2000).

One of the first models was the Kent and Park (1971) model, which suggested not to use additional strength from restraints, but to consider increasing ductility (as the number of confinement bars increases). The model proposed by Mander, Priestley and Park (1988) is most widely accepted and used in many research works. Similarly, steel is generally considered elastoplastic, but in fact, it has the ability to show additional strength after yielding. The strain hardening (post-yield) part of the stress-strain curve of steel can be used very intelligently to create many new configurations of steel with higher ductility and high strength.

Hence, in this study, *Mander et al.* model and *Park et al.* model was used in characterizing the constitutive stress-strain behaviour of concrete and steel rebars as shown in Fig. 3.2 (Mander *et al.*, 1988; CSi, 2016). Both confined and unconfined stress models defined in Mander et al. were used to model confined concrete and cover concrete (unconfined concrete) respectively. The materials used for modelling were M25 grade concrete (characteristic compressive strength of 25 MPa) and Fe415 grade reinforcing steel (yield strength of 415 MPa). Elastic material properties of concrete are taken as per Indian Standard IS 456 (2000). The modulus of elasticity (E_c) of concrete is taken as: $E_c = 5000\sqrt{f_{ck}}$ MPa, where f_{ck} is characteristic compressive strength of concrete cube in MPa at 28-day (25 MPa in this case). Moreover, as per the recommendations of IS 1893 (2016), moments of inertia of beams and columns were reduced to 35% and 70% for beams and columns respectively while performing non-linear structural analysis.

The stiffness degradation caused by the onset of concrete cracking and steel yielding can be modelled by existing hysteretic relationships such as: Takayanagi and Schnobrich (1979), Clough and Johnston (1966); Saiidi and Sozen (1979), Takeda *et al.* (1970), Park *et al.* (1987), Ibarra *et al.* (2005). In our study, the Takeda hysteresis model has been adopted to incorporate the degradation under cyclic loading which is available in the library of SAP2000 software, as depicted in Fig. 3.3.

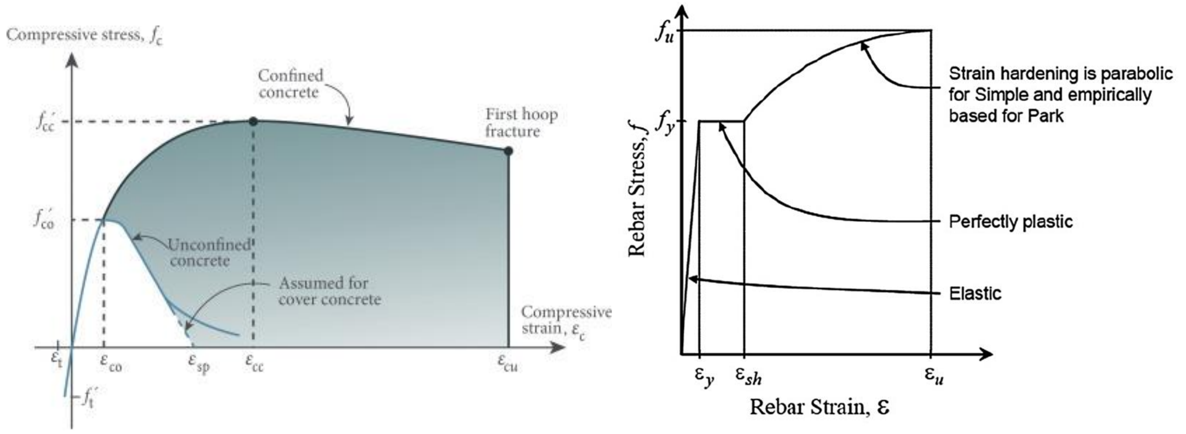


Fig. 3.2 (a) Mander *et al.* stress-strain model (b) Park *et al.* stress-strain model

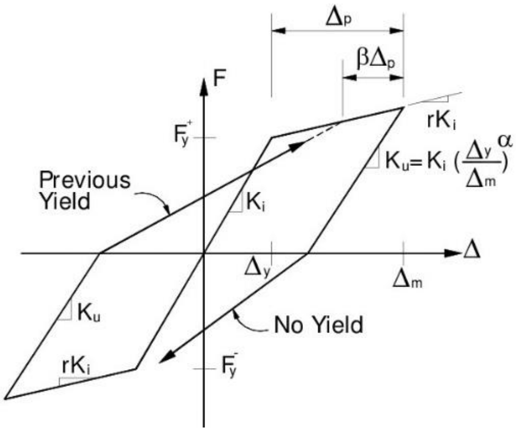


Fig. 3.3 Takeda cyclic degradation model (Takeda *et al.*, 1970)

3.2.1.2 *Geometric non-linearity*

When a structure experiences large deformation, its changing geometric configuration can cause the structure to respond non-linearly, which is also referred as P-Δ effect. It leads to abrupt changes in base shear, overturning moment and/or the axial force distribution at the base of RC structure when it is subjected to large lateral displacements. The structure is subjected to additional overturning moments resulting from seismic forces 'P' acting through a lateral displacement 'Δ'. This is due to horizontal seismic inertial forces.

In most cases, this increases the effective fundamental period of the structure also. This effect reduces a structure's initial elastic stiffness slightly, and will therefore have small influence if a building is subjected to an earthquake small enough that it remains in the elastic range. However, the P−Δ effect can make a substantial difference on the post-yield response of a structure. P−Δ effects can be considered partially or completely directly in SAP2000 by carrying out direct integration time history analysis. This analysis takes large amount of computational time.

3.2.1.3 Plastic hinge mechanism

When performing a non-linear analysis, the model must consider the non-linear behaviour of the structural elements. The flexural hinge is generally defined by a moment-rotation curve calculated based on the cross-section and reinforcement details at the possible hinge location. To calculate the properties of the hinge, it is necessary to perform a moment-curvature analysis on each element. For this, the constitutive relationship between concrete and steel bars is required, and the length of the plastic hinges in the structural elements.

Further, plasticity in structural components can be modelled in the form of non-linear hinges. These hinges can be modelled either as lumped plasticity (concentrated hinges) or distributed plasticity (as fiber hinges). When fiber hinges are used, the cross section is discretized into a series of axial fibers, which extend along the hinge length. Each of these fibers has a stress-strain relationship, and together these define the force-deformation and moment-rotation relationships for the frame section. Although the fiber hinge gives more accurate results, it is not used in the present analyses, since it is computationally more intensive.

Therefore, to model non-linearity in beams and columns, lumped plasticity approach was adopted. In this approach, the beams and columns are modelled by defining plastic hinges at both ends. It is assumed that due to rigid diaphragm action of concrete slabs, beams cannot deform along their axis. Plastic hinge behaviour of structural components has been defined (i.e., beams: M3 hinges; and columns: P-M2-M3 hinges) using default hinges present in SAP2000 software, calculated in accordance with ASCE 41. Beam-column joints were modelled as rigid joints using rigid end offsets in SAP2000 software. This approach of modelling the structural components has been the most widely adopted in literature which ensures seismic performance in terms of global response parameters (Inel and Ozmen, 2006; Surana *et al.*, 2018; Choudhury and Kaushik, 2018; Choudhury and Kaushik, 2019).

3.2.2 *Structural configurations*

The seismic behaviour of structures depends mainly on three important factors – distribution of mass, stiffness and strength. Reinforced concrete structures are made irregular for architectural and aesthetic requirements to meet certain functionalities viz., parking spaces, lighting, ventilation and other architectural demands etc. These irregularities cause behavioural changes in the structure, which are mainly responsible for vulnerability of RC building when subjected to a devastative event like earthquakes. The non-uniform distribution of mass, stiffness, and strength often leads to form structural weaknesses in buildings, and damages from the earthquakes are initiated from the locations where these structural weaknesses are present. Hence, it is necessary to study the effect of irregularities (distribution of mass, stiffness and strength) on the seismic behaviour of structures.

Irregular configuration, whether in plan or elevation, is recognised to be the main cause of failure in past earthquakes. A common type of vertical geometric irregularity in building structures is caused by the sudden decrease of the horizontal dimension of the building at a certain elevation level. A lot of research has been conducted to understand the behaviour of vertical irregular structures, and to determine ways to improve their performance (Varadharajan *et al.* 2012, Varadharajan *et al.* 2013, Bhosale *et al.* 2017, Bhosale *et al.*, 2018).

3.2.2.1 *Unreinforced masonry (URM) infill walls modelling*

URM infill walls are usually used as partitions in multi-storey RC frame buildings. These infills are generally treated as non-structural elements and the infill-frame interaction and behaviour is often ignored. Modelling and simulation of real behaviour of infilled frames is a difficult task, as they exhibit complex nonlinear behaviour due to infill-frame interaction. Two approaches can be used for modelling of URM infills: micro-models and macro-models. Micro-models are developed based on finite element method and capture the behaviour and interaction of infills with the frame in a very in depth, but they are computationally very expensive and are not appropriate for regular use. Macro-models are computationally simple and easy to be developed.

Haldar *et al.* (2013) have compared three different macro-models with the available experimental results and concluded that a single strut model is capable of representing the governing failure modes in most of the RC frame buildings. Further, it has been well reported in literature that infill wall contributes to higher initial strength and stiffness of the structural model, in elastic region of the structure. Once the model moves from elastic to inelastic/plastic state, there would be no contribution from the infill walls, as it fails/cracks at the end of linear/elastic state. Therefore, to study vulnerability effects due to local failure of the masonry infill and its interaction with the structural frame, usage of advanced modelling techniques i.e., with three diagonal struts are recommended. However, single diagonal strut modelling of infill has been adopted most widely in literatures owing to its simplicity in implementation and its ability to accurately represent the global behaviour of the RC frame with infill wall (Haldar *et al.*, 2013; Bhosale et al., 2017; Choudhury and Kaushik, 2018; Surana *et al.*, 2018; Choudhury and Kaushik, 2019; Choudhury and Kaushik, 2020).

Hence, equivalent single diagonal strut model with axial hinges at the centre has been adopted in this investigation, for simulating the infill wall interaction with the structural frame in accordance with IS 1893 (Part 1): 2016. The references of Kaushik *et al.* (2007) and Burton and Deierlein (2014) has been used in development of backbone curve for axial hinges. An in-house spreadsheet program has been developed for computation of respective infill wall properties required for modelling the infill wall in SAP2000 software.

In this investigation, modelling of infill is considered as ‘equivalent diagonal strut’, in which, the ends of the diagonal strut are pin-jointed with the RC frame. The thickness and modulus of elasticity of the equivalent strut are the same as that of the infill. The infill walls were modelled with masonry material possessing prism strength 4.1 MPa with an elastic modulus of 2255 MPa as equivalent diagonal struts, assuming to take axial load only as shown in Fig. 3.4. Infill walls are modelled using empirical equations given by IS 1893 (2016) and width of the diagonal strut is defined in Eqns. (3.1-3.2). The material properties and the nonlinearity in the masonry infill was characterized using the model proposed by Kaushik *et al.*

(2007). The inelastic behaviour of the strut elements is modelled using axial hinges provided at the centre of diagonal struts (Uva *et al.*, 2012; Haldar *et al.*, 2012; Burton and Deierlein 2014; Haldar *et al.*, 2013). The hysteretic behaviour in the equivalent diagonal strut was modelled using the Pivot hysteretic law, as depicted in Fig. 3.5 (Calveri *et al.*, 2014).

$$W_{ds} = 0.175\alpha_h^{-0.4} L_{ds} \tag{3.1}$$

$$\alpha_h = h\left(\sqrt[4]{\frac{E_m t \sin 2\theta}{4 E_c I_c h}}\right) \tag{3.2}$$

W_{ds} is Equivalent width of the diagonal strut; L_{ds} is Length of the diagonal strut; E_m is Modulus of elasticity of masonry; E_c is Modulus of elasticity of concrete; I_c is Moment of inertia of concrete member; h is Height of the wall; t is Angle of the diagonal strut with the horizontal; θ is Thickness of the infill wall

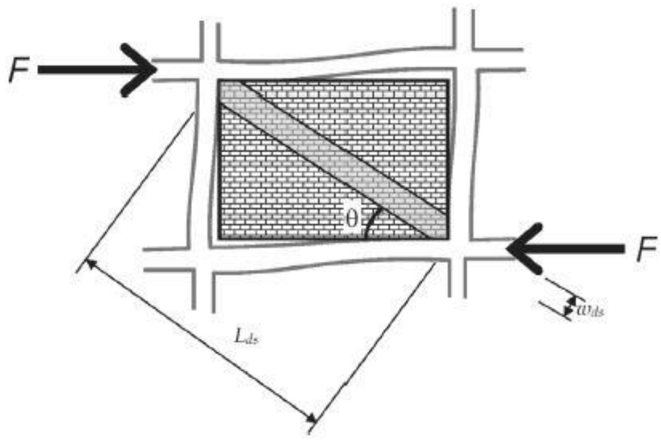


Fig. 3.4 Equivalent diagonal strut representation of URM infill wall

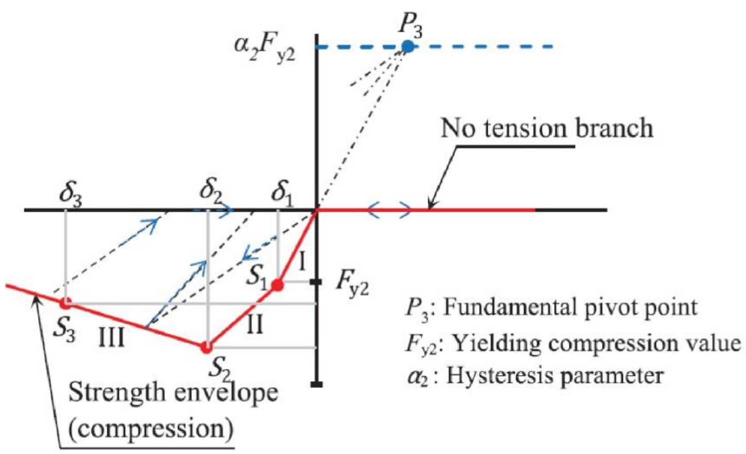


Fig. 3.5 Pivot hysteresis model (Calveri *et al.*, 2014)

3.3 Seismic analysis procedures

Most of the structures tend to behave non-linearly beyond some point of loading. In certain circumstances, linear analysis may be acceptable, but in others, the assumptions on which linear analysis is based may be violated in real-time structures, resulting in inaccurate results. Linear analysis is a subset of non-linear analysis, which is the most generalised form of analysis. If the loading causes a large change in stiffness, nonlinear analysis is required. However, linear analysis is commonly used to explain the basic behaviour of structures at first. Seismic analysis procedures can be broadly classified into four categories:

3.3.1 *Linear static analysis*

In linear static procedure, it is assumed that loads are applied gradually and remain constant. Other forces of inherent inertia and damping caused due to velocity and acceleration are neglected. Indian standard, IS 1893 (Part 1): 2016 adopts a linear static procedure to calculate the design lateral force, and distribute the base shear vertically along the height of the structure is given. The formula to calculate base shear is:

$$V_B = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \cdot W \tag{3.3}$$

Z – Zone factor; I – Importance factor; R – Response Reduction factor; S_a/g – Average response acceleration coefficient (depends on T – Undamped Natural period of the structure); W – Seismic weight of the building

This analysis procedure has certain limitations and cannot give accurate results for taller buildings (height > 15m), buildings having higher modal participation other than fundamental natural mode, irregular buildings having discontinuities in mass and stiffness in its geometrical configuration, and buildings located in higher seismic zones (zone III, IV and V).

3.3.2 *Linear dynamic analysis*

Linear dynamic procedure (also known as response spectrum method) considers multiple mode shapes of the building into account for analysis. Responses for each mode are taken from response spectrum, and are then combined to estimate the total response of the structure using various modal combination methods. Also, this method uses a uniform design spectrum that constitutes the average of numerous earthquake motions, and calculates only the maximum values of displacements and member forces in each mode.

This method of analysis has some limitations. It is an approximate method and cannot provide accurate results for MDOF systems. Non-linear behaviour of structures cannot be assessed in a response spectrum analysis. All forces and displacements obtained from a response spectrum analysis are maximum peak values and are all positive numbers.

3.3.3 *Non-linear static analysis*

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading or displacement is monotonically increased in accordance with a certain predefined pattern. With increase in magnitude of loading, weak links of the structures can be obtained. Pushover analysis estimates force and displacement capacity of structure along with sequential formation of hinges in the structure under analysis. The analysis is conducted until the structure fails, which helps determine the collapse load and ductility capacity. The outcome of pushover analysis is usually represented in the form of base force (or base shear) vs. roof displacement, popularly referred to as capacity curve of structure. FEMA 356 and ATC 40 present the procedure for conducting a simplified, nonlinear static analysis.

The purpose of pushover analysis is to estimate the strength and deformation requirements of the structural system in design earthquakes through non-linear static analysis, and compare these requirements with the available capacity of the performance level of interest to evaluate the expected performance of the structural system. This procedure can also be used to check the adequacy of the new structural design. It considers geometric non-linearity, material non-linearity, and the redistribution of internal forces. Pushover analysis also gives an estimate of maximum base shear that the structure can withstand and the corresponding inelastic displacement capacity. It is likely to provide a lot of information on response characteristics that cannot be obtained from a linear static or dynamic analysis.

Many publications have shown that traditional pushover analysis can be an extremely useful tool when used with carefulness and proper engineering judgement. However, it also has several limitations such as:

- The basic assumption of pushover analysis is that the response of a multi-degree-of-freedom structure is directly related to an equivalent single-degree-of-freedom system.

It only applies to cases where the natural mode of participation is dominant. This need not be true for structures where there is significant contribution from higher modes of vibration.

- This method also ignores the increasing stiffness degradation that occurs during the structure's cyclic non-linear seismic loading. This degradation results in changes in the period and modal characteristics of the structure, and these changes affect the loads attracted during the earthquake excitations.

3.3.4 *Non-linear dynamic analysis*

Seismic response of an inelastic structure can be most reliably estimated using nonlinear time-history (dynamic) analysis procedure. Non-linear dynamic analysis, also known as time history analysis is often considered as most accurate analysis procedure, though computationally expensive. In order to perform time history analysis, a representative ground motion acceleration time history is required for a structure being evaluated. In this analysis procedure, the structure is subjected to accelerations from earthquake time histories at the base of the structure. The method consists of a step-by-step direct integration over a time interval. In linear analysis, the stiffness characteristics of the structure are assumed to be constant for the entire duration of the earthquake, whereas in non-linear analysis, the stiffness is assumed to be constant only for a short increment of time.

3.3.4.1 Incremental Dynamic Analysis (IDA)

Incremental dynamic analysis (IDA) is the combination of a series of non-linear dynamic analyses to evaluate the non-linear response of building structures. The concept of IDA was first expressed by Bertero (1977), then by Nassar and Krawinkler (1991). It has however more recently gained popularity and widely used as a method to estimate the global capacity of structural systems by Vamvatsikos and Cornell (2002) and Vamvatsikos and Cornell (2004). The seismic performance characteristics of structures are assessed from the entire range of structural demands from linear elastic state, to yielding, then highly inelastic behaviour until the collapse state. Some of its basic objectives are:

- Full understanding of the response or demand of the structure in a wide range of different levels of ground motion records.
- Better understanding of the structural effects at different levels of the ground motion.

- Better understanding of changes of the behaviour of structural response with increase of the intensity of ground motion (e.g., changes in maximum displacements in height, first yielding, reduction in stiffness and strength).
- Evaluation of the non-linear dynamic capacity of the entire structural system.

In this approach, the spectrum-compatible accelerograms have to be scaled at different levels to estimate the capacity of the structure ranging from elastic to plastic state until it reaches the collapse state. The outcome of this analysis is an IDA curve termed as the non-linear capacity curve plotted as an Intensity Measure (IM) with respect to an Engineering Demand Parameter (EDP) of the structure. The crucial aspect of this NLD analysis lies in the selection of appropriate IM and EDP which depends on the target of analysis.

When a single ground motion is used, the IDA plot obtained is called single-record IDA. This is used extensively in this thesis to illustrate the effect of ground motion on the structural response due to variation in systemic parameters. Multi-record IDA is a plot of the IM versus DM of the system when it is subjected to a suite of ground motion accelerograms. This gives instant information regarding the relative potential of causing damage to the structure by each ground motion accelerogram.

3.3.4.1.1 Selection of Intensity Measure (IM)

The most commonly used IMs are peak ground acceleration (PGA), peak ground velocity (PGV), and first mode spectral acceleration (S_a (T_1, 5%)). However, in the case of RC multi-storey building frames with over three stories in height, the spectral acceleration estimated at first mode (S_a (T_1, 5%)) is treated to be an appropriate intensity measure unlike PGA (Shome and Cornell, 1999; Baker and Cornell, 2006; Maniyar *et al.,* 2009; Faggella *et al.*, 2013). In the case of structures possessing irregularities and also in NLD of multi-degree of freedom (MDOF) systems, the higher modes of vibration get manifested in the solution process.

3.3.4.1.2 Selection of Engineering Demand Parameter (EDP)

The structure's estimated performance (or damage pattern), corresponding to a given intensity of ground motion, is represented by the engineering demand parameters (EDP). The common EDPs in terms of displacements include roof displacement, inter-story drift, roof drift ratio, maximum inter-storey drift ratio, residual deformation, spectral displacement and maximum ductility demands. Another set of alternatives for EDPs in terms of forces may include maximum base shear, spring force, and hysteretic energy. The choice of EDP depends on the designer, who may be interested in measuring the demands in terms of either force or deformation.

3.3.4.1.3 Interpretation of IDA curve

The structural performance can be assessed with the help of IDA curve, by correlating it with structural damage limit states and their corresponding damage thresholds. The limit states can be defined and identified from the IDA curves. The yield state on an IDA curve can be identified as the point where the initial stiffness of the curve changes for the first time. The initial collapse is generally defined as the point where a small increment in IM, causes very large increase in EDP. The collapse state on an IDA curve can be identified or defined from two criteria, as depicted in Fig. 3.6:

- EDP-based criterion: Based on reaching a certain value of EDP, the lowest point can be termed as the capacity point (e.g., point A in Fig. 3.6). Maximum IDR is monitored as a 4% threshold was adopted to designate the collapse state of the structure, as recommended by FEMA 273.
- IM-based criterion: Similarly, on reaching a certain level of IM (when the slope becomes continuously flat) the region can be termed as collapse (e.g., point B in Fig. 3.6). When there is a flatline observed, that particular IM is taken as collapse capacity.

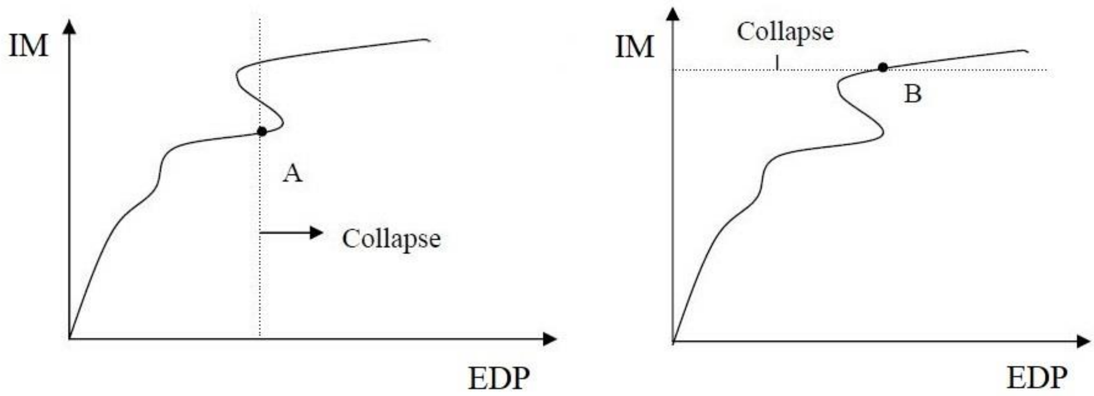


Fig. 3.6 Identifying collapse state on an IDA curve

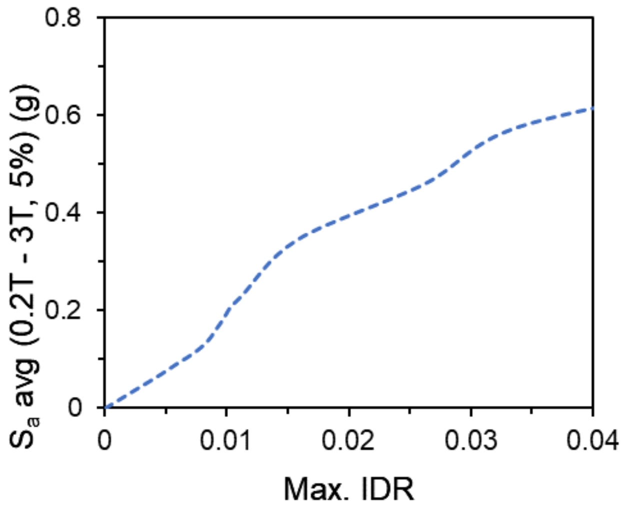


Fig. 3.7 An example IDA curve with chosen IM and EDP

Hence, in our investigation, the average spectral acceleration (S_a avg) is chosen as the IM, and the maximum inter-story drift ratio (IDR) is chosen as the EDP. In Fig. 3.7, a typical IDA curve is shown as an example with the chosen IM and EDP. This gives a better idea of the demand and capacity relationship that is useful for assessing the system at various performance levels.

3.4 Performance assessment parameters

3.4.1 *Displacement response parameters*

The performance of RC buildings under earthquakes using the incremental dynamic analysis (IDA) method is assessed by means of various displacements responses as follows:

- maximum lateral displacement vs. story levels
- storey drifts and maximum inter-storey drift ratio vs. storey levels
- residual displacements with respect to repeated ground motion
- maximum inter-story drift ratio (IDR) with respect to Sa avg
- Local Structural Damage (Hinge patterns)

3.4.2 *Performance levels*

The performance of any building frame is a combination of the performance of all its individual structural components. The performance levels are discrete damage states identified from a continuous range of possible damage states. The structural performance levels based on the drifts are as follows (FEMA 356; ASCE 41): Immediate occupancy (IO), Life safety (LS), and

Collapse prevention (CP). The non-linear procedures of ASCE 41 require definition of the non-linear load-deformation relation. Such a curve showing a typical load – deformation relation and target performance levels curve is shown in Fig. 3.8.

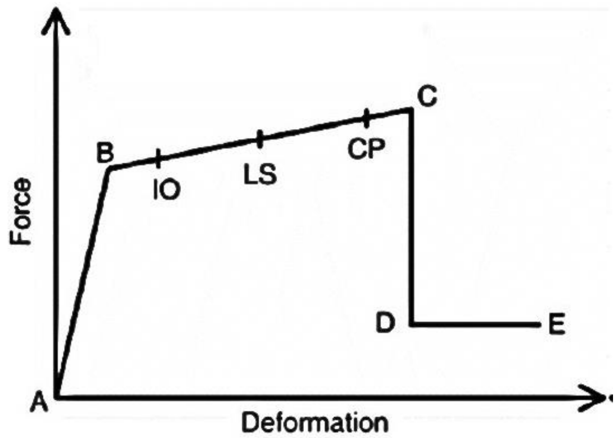


Fig. 3.8 Representative force-deformation curve with performance levels

The three levels are arranged according to decreasing performance of the lateral load resisting systems. The element performance levels are defined by values of the deformation of a structural element. Three performance levels are defined in the load versus deformation curve for the hinges of the element.

It is a curve defined by five points as explained:

- Point A corresponds to no load condition.
- Point B corresponds to the start of yielding.
- Point C corresponds to the ultimate strength.
- Point D corresponds to the residual strength. For computational stability, it is recommended to specify non-zero residual strength beyond C. In absence of the modelling of the descending branch of a load versus deformation curve.
- Point E corresponds to the maximum deformation capacity with the residual strength. To maintain computational stability, a high value of deformation capacity is assumed.

3.4.3 *Fragility curve development*

The development of fragility curves utilizing the guidelines specified by ATC 58 (1996) is essential to understand the vulnerability of RC building frames under sequential as well as isolated/individual earthquakes (Gautham and Krishna 2017, Kassem *et al.* 2019, Oggu *et al.*

2019). A fragility function expresses the probability of exceedance of damage for the whole building evaluated at a particular limit state in terms of ground motion intensity parameter as depicted in Fig. 3.9. The ground motion intensity parameter is generally expressed as peak ground acceleration, first mode spectral acceleration (S_a (T_1, 5%), or average spectral acceleration S_a avg (0.2T–3T, 5%). The lognormal distribution is most commonly used distribution for describing the vulnerability of RC buildings expressed in terms of fragility functions.

Fragility functions are proposed in general to be normal or lognormal distributions in which the ground motion characterisation is done with spectral acceleration or spectral displacement at fundamental elastic period of vibration. This approach takes in to account the frequency content of ground motion and fundamental period of vibration of building. It has been found that this approach shows greater correlation with ground motion input and damage. Hence, the average spectral acceleration ordinate in the period range corresponding to the RC building frames was used to characterise the ground motions for fragility curves. Also, in many published research works, it was reported that S_a is the most appropriate IM, as it is capable of representing the complete behaviour of structural response (Hosseinpour and Abdelnaby, 2017; Bhosale et al., 2017).

The fragility curves were developed utilizing the guidelines specified by ATC 58. The nonlinear analysis has been carried out, and then the mean and dispersion values of the seismic capacities of RC frame were calculated for developing the fragility curves. These fragility curves are defined by means of a lognormal distribution function. Therefore, the estimation of fragility parameters (mean and dispersion) and further development of fragility curves based on lognormal distribution function is carried out using an in-house spreadsheet program.

It can be observed that each fragility curve is defined by a median value of spectral displacement corresponding to the damage state and the associated variability. Therefore, the median spectral displacement is computed analytically. However, the estimation of variability is a complex process which requires statistical data. It can be further noted that this variability in general depends on the local conditions and construction practices adopted at that location.

Despite the fact that India has experienced several strong earthquakes in the past, this kind of systematic data for the Indian buildings is lacking. However, the aim of the present study is not to prescribe any standard fragility functions to be used for the Indian buildings. HAZUS (NIBS, 2003) has presented variability for the fragility estimation of American (i.e., Californian) buildings and has been widely adopted in literature for analysing the damage characteristics of buildings existing all over the world. Therefore, the values of variability specified in HAZUS for the relevant cases have been adopted in our study.

Hence, in this approach, the fragility parameters are estimated from the dynamic capacity curves generated for all the structural models using a spreadsheet program as per the following Eq. (3.4):

$$P[D \mid S_a] = \Phi\left[\frac{\ln(S_a) - \mu}{\sigma}\right] \tag{3.4}$$

Where: D is Damage state; 'Φ' is Cumulative distributive function; μ is Mean of the natural logarithm of IM; σ is Standard Deviation of the natural logarithm of IM; S_a is IM (average spectral acceleration).

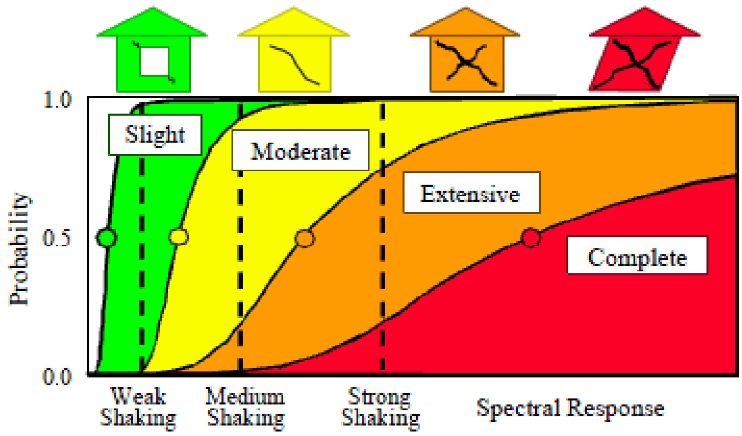


Fig. 3.9 Representative fragility curves for Slight, Moderate, Extensive and Complete damages (Source: Kircher *et al.*, 2006)

CHAPTER 4

Behaviour assessment of 3D RC frame buildings under single earthquake event

4.1 General

This chapter deals with seismic behaviour assessment of 3D RC frame multi-storeyed buildings. Certain RC building frames (OMRFs) perceived to mimic the configurations in Warangal city (regular and vertically irregular configurations) designed as per IS code provisions have been considered for this purpose. Incremental dynamic analysis has been performed to envisage the dynamic capacity and seismic response in terms of response parameters (maximum horizontal displacement, maximum inter-storey drift ratio) and spectral accelerations experienced at the location. Further, structural behaviour and fragility curve development for the chosen structural configurations are also discussed.

4.2 Description of structural models

Eight different ordinary moment-resisting frames (OMRFs), representing the building configurations pertaining to seismic zone III (PGA of 0.16g) with medium soil profile has been selected in this study (Location: Warangal city, Telangana State, India) (Dhir *et al.*, 2018). Most of the existing multi-storied RC building frames in this location are found to be a maximum of six stories above ground level and possess vertical setbacks to aid certain functional needs of the building (viz., natural ventilation, vehicle parking, etc.). These setbacks are known to possess reduced dimensions along the horizontal direction at a particular floor level and are categorized as vertical irregularity as per the code regulations. These irregularities are perceived to cause significant changes in dynamic characteristics of the RCMRFs (Varadharajan *et al.*, 2012; Varadharajan *et al.*, 2013; Bhosale *et al.*, 2017; Bhosale *et al.*, 2018). Further, these types of irregular structural configurations are proved to be detrimental during any seismic hazard. Hence, in this investigation, different hypothetical configurations of OMRFs (with and without infill contribution) including regular and vertically irregular (with setbacks introduced along the height of the building) has been selected to understand the seismic behaviour. These building configurations considered comprises two bays in both horizontal directions, with a bay width of 5m each, and possess a storey height of 3.2m along vertical direction, in addition to various vertical setbacks introduced along the height as depicted in Fig. 4.1.

These structural configurations were modelled using a commercial structural software SAP2000 (CSi, 2016) for gravity loads and Zone III seismic forces. Further, the seismic analyses were carried out for design loads as per regulations of IS 875 – Parts I & II (1987), IS 456 (2000) and IS 1893 (2016). The design details used for modelling are specified in Table 4.1. The dead load of the slab (inclusive of floor finish) was taken as 3.75 kN/m^2, and the slab live load was taken as 3 kN/m^2. The self-weight of the partition walls (230 mm thick) was applied onto the adjoining beams as a uniformly distributed load. Additionally, rigid diaphragms were assigned every storey level throughout the structure ignoring the flexibility of the floor. Further, respective material models for concrete and steel along with plastic hinge behaviour of the structural components are defined as discussed in Chapter 3.

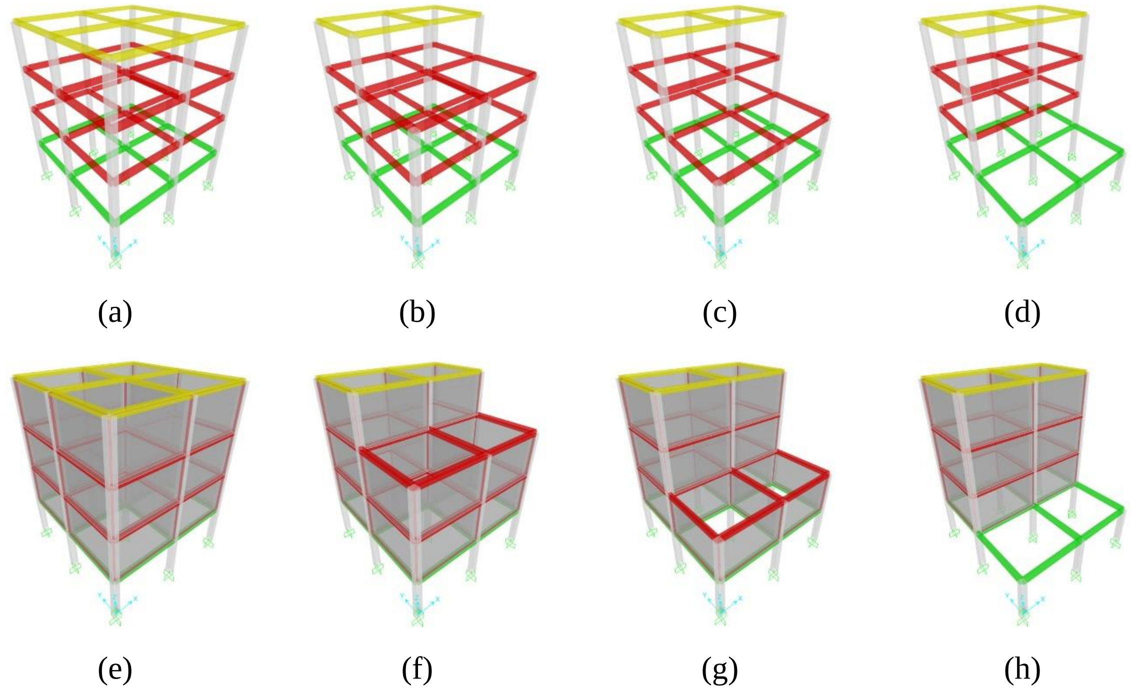


Fig. 4.1 3D Geometrical representation of the structural models investigated: (a). Bare Regular – B-R, (b). Bare Irregular – B-T, (c). Bare Irregular – B-M, (d). Bare Irregular – B-B, (e). Infill Regular – I-R, (f). Infill Irregular – I-T, (g). Infill Irregular – I-M and (h). Infill Irregular – I-B

Table 4.1 Cross-section and design details for beams and columns of the regular 4-storeyed building (Dhir *et al.*, 2018)

Member	Storey level	Breadth (mm)	Depth (mm)	Longitudinal steel rebar		Transverse steel rebar
				Top	Bottom	
Beam	1	250	450	5-20ϕ	4-20ϕ	8ϕ @ 150c/c
	2 & 3	250	400	4-16ϕ	3-16ϕ	8ϕ @ 150c/c
	4	250	300	4-12ϕ	3-12ϕ	8ϕ @ 150c/c
Column	1 to 4	420	420	8-16ϕ		8ϕ @ 175c/c

Table 4.2 Modal properties of bare frame models of different configurations

Structure	Modes	Time Period (sec)	Modal Participating Mass Ratios*			Cumulative Modal Participating Mass Ratios		
			U_X	U_Y	R_Z	$\sum U_X$	$\sum U_Y$	$\sum R_Z$
B-R	1st Mode	1.187	0.78	0	0	0.78	0	0
	2nd Mode	1.187	0	0.78	0	0.78	0.78	0
	3rd Mode	0.968	0	0	0.8	0.78	0.78	0.8
	4th Mode	0.358	0.14	0	0	0.92	0.78	0.8
	5th Mode	0.358	0	0.14	0	0.92	0.92	0.8
	6th Mode	0.298	0	0	0.13	**0.92**	**0.92**	**0.93**
B-T	1st Mode	1.134	0.77	0	0.01	0.77	0	0.01
	2nd Mode	1.133	0	0.78	0	0.77	0.78	0.01
	3rd Mode	0.912	0.01	0	0.78	0.79	0.78	0.8
	4th Mode	0.346	0	0.14	0	0.79	0.92	0.8
	5th Mode	0.342	0.13	0	0	0.92	0.92	0.81
	6th Mode	0.278	0	0	0.12	**0.92**	**0.92**	**0.93**
B-M	1st Mode	1.071	0.66	0	0.09	0.66	0	0.09
	2nd Mode	1.059	0	0.72	0	0.66	0.72	0.09
	3rd Mode	0.773	0.07	0	0.65	0.74	0.72	0.75
	4th Mode	0.391	0	0.18	0	0.74	0.9	0.75
	5th Mode	0.384	0.15	0	0.02	0.89	0.9	0.77
	6th Mode	0.318	0.01	0	0.13	**0.91**	**0.9**	**0.9**

B-B	1st Mode	1.126	0	0.66	0	0	0.66	0
	2nd Mode	1.084	0.67	0	0.05	0.67	0.66	0.05
	3rd Mode	0.825	0.01	0	0.55	0.68	0.66	0.6
	4th Mode	0.355	0	0.23	0	0.68	0.89	0.6
	5th Mode	0.340	0.21	0	0	0.89	0.89	0.61
	6th Mode	0.285	0.01	0	0.29	**0.9**	**0.89**	**0.9**

*U_X: Displacement along X-axis, U_Y: Displacement along Y-axis and R_Z: Rotation about Z-axis

Table 4.3 Modal properties of infill frame models of different configurations

Structure	Modes	Time Period (sec)	Modal Participating Mass Ratios*			Cumulative Modal Participating Mass Ratios		
			U_X	U_Y	R_Z	$\sum U_X$	$\sum U_Y$	$\sum R_Z$
I-R	1st Mode	0.58	0.48	0.48	0	0.48	0.48	0
	2nd Mode	0.579	0.48	0.48	0	0.97	0.97	0
	3rd Mode	0.516	0	0	0.97	**0.97**	**0.97**	**0.97**
I-T	1st Mode	0.565	0.79	0.16	0.017	0.79	0.16	0.017
	2nd Mode	0.563	0.15	0.81	0.002	0.95	0.97	0.02
	3rd Mode	0.498	0.02	0	0.95	**0.97**	**0.97**	**0.97**
I-M	1st Mode	0.537	0.82	0	0.14	0.82	0	0.14
	2nd Mode	0.524	0	0.96	0	0.83	0.96	0.14
	3rd Mode	0.435	0.14	0	0.82	**0.97**	**0.96**	**0.96**
I-B	1st Mode	0.543	0	0.92	0	0	0.92	0
	2nd Mode	0.531	0.81	0	0.15	0.81	0.92	0.15
	3rd Mode	0.428	0.13	0	0.75	**0.94**	**0.92**	**0.9**

*U_X: Displacement along X-axis, U_Y: Displacement along Y-axis and R_Z: Rotation about Z-axis

The Eigen value (modal) analysis has been performed on the structural models and are depicted in Tables 4.2-4.3. The modes of vibration to be considered for the analysis has to ensure the 90% cumulative mass participation. Therefore, it can be observed that six modes are to be considered for the solution process in case of bare frames, and three modes in case of infill frames. Further, as the irregularities get manifested in the RC frame, it can be observed that, the fundamental mode participation alone for arriving at dynamic response gets reduced. This necessitates adoption of multi-modal approaches to evaluate seismic response. Owing to this, limiting the analysis with fundamental mode alone cannot capture the actual behaviour of the structural system. Therefore, to alleviate this, it has been suggested in the literature that average spectral acceleration value (S_a avg), representing the geometric mean of 5% damped spectral accelerations over a range of time periods (i.e., 0.2T–3T; T is the fundamental time period of the structural model) can be considered to address the influence of lower and higher modal participation on RC building frame response, thereby reduce the dispersion in Intensity measure (IM). Therefore, S_a avg is suggested to be a more appropriate IM compared with S_a in capturing the effect of higher modes of vibration. Hence, keeping in view of this requirement, all the structural models considered are assessed with respect to S_a avg (0.2T–3T, 5%), and the maximum inter-storey drift ratio as the IM, and the Engineering Demand Parameter (EDP) respectively.

Bidirectional simultaneous earthquake forces are considered in the analysis in order to capture more accurate inelastic behaviour of building structures. Structural damping has been considered as Five percent and modelled as Rayleigh damping for all the structural models. Further, the geometric nonlinearity effects have been taken care of by considering the local P-Δ effects in the analysis. Newmark-β has been considered as the time integration algorithm. The IDA approach has been adopted with collapse prevention criteria as the limit state for analysis. Hence, inter storey drift ratio (IDR) of 4% is defined as performance limit for the EDP in accordance with ASCE 41 (2017).

4.3 Ground motion data

Since recorded ground motion data is not available at the considered location, ground motion records of certain real earthquakes with appropriately similar magnitude possible at the said location are considered from the available online databases viz., Pacific Earthquake Engineering Research (PEER) Center strong motion database and Consortium of Organizations for Strong-Motion Observation Systems (COSMOS) as listed in Table 4.4. The selected records are then made compatible with the elastic design spectrum corresponding to the site characteristics (Zone III and medium soil profile) using the SeismoMatch computer program, which runs a spectral matching algorithm designed by Al Atik and Abrahamson (2010). These scaled records are depicted in Fig. 4.2. In this investigation, seven ground motion records along both orthogonal directions were considered to generate a bi-directional earthquake force to envisage the non-linear behaviour of RCMRFs using IDA. This involves around 1200 NLD simulations using the IDA approach to arrive at non-linear response characteristics for the structural models considered.

Table 4.4 Details of ground motion records used for time history analysis

S. No.	Earthquake event	Year	Station	Magnitude	Source
1	Imperial Valley	1979	Holtville Post Office	6.53	PEER
2	Mammoth Lakes	1980	Convict Creek	5.69	PEER
3	Chalfant Valley	1986	Zack Brothers Ranch	5.77	PEER
4	Chamoli	1999	Gopeshwar	6.6	COSMOS
5	India-Burma Border	1988	Berlongfer	7.2	COSMOS
6	North-West China	1997	Jiashi	6.1	PEER
7	Whittier Narrows	1987	San Marino - SW Academy	5.9	PEER

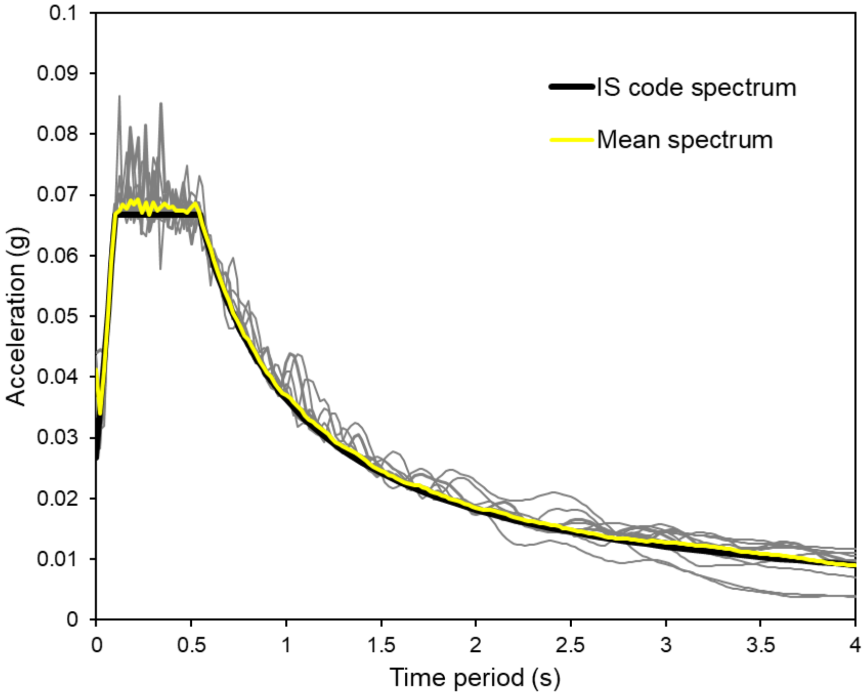


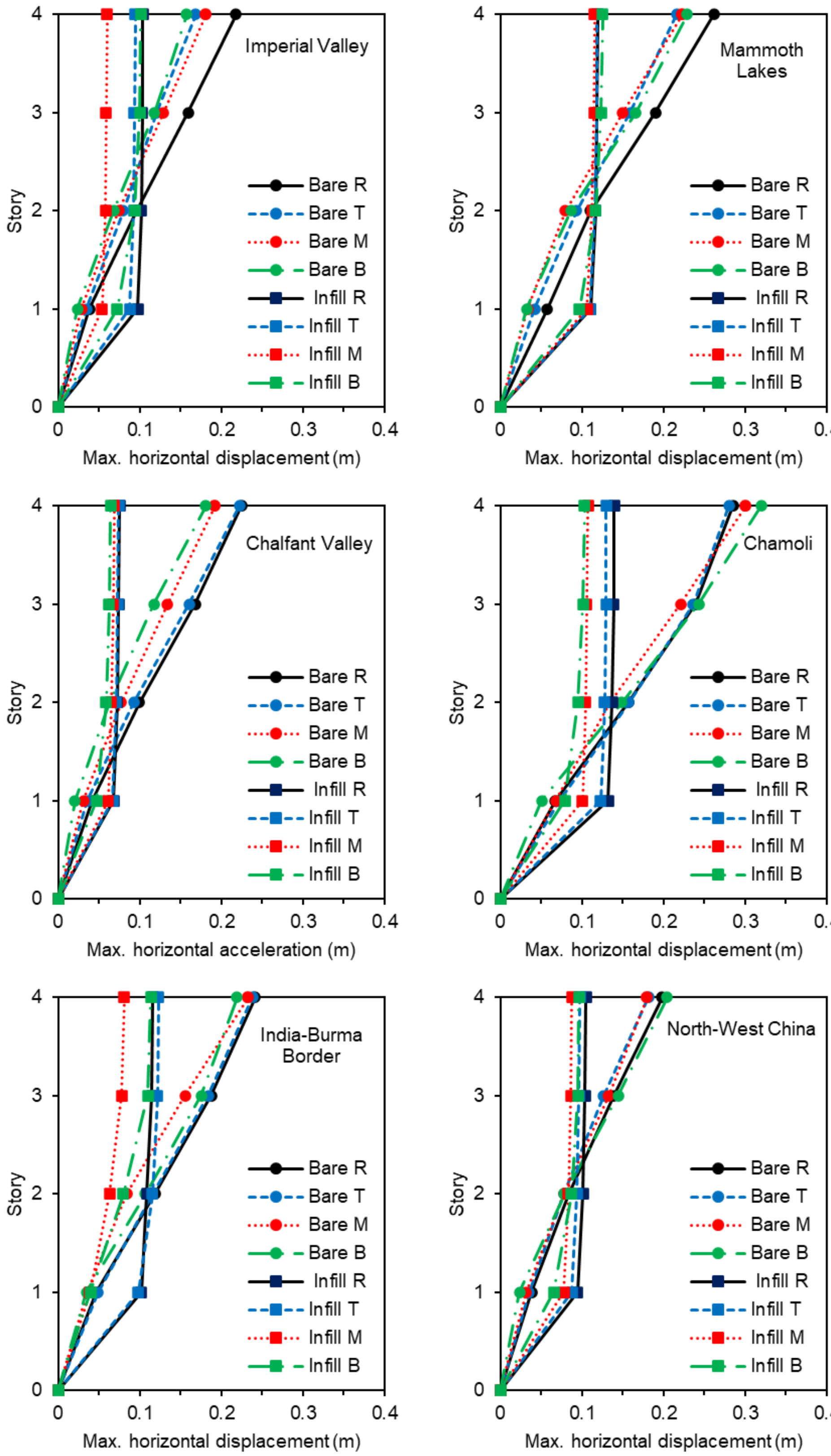
Fig. 4.2 Accelerograms compatible with the elastic target spectrum

4.4 Evaluation of seismic behaviour of RC MRFs

The seismic behaviour for the structural models considered has been described in terms of EDPs (lateral displacements and inter-storey drift ratios) for various earthquake ground motions pertaining to seismic zone III and medium soil profile.

4.4.1 Lateral displacements

Lateral displacement is the most commonly used displacement measure for evaluating the structural behaviour under a given seismic load. In this study, the absolute maximum horizontal displacement has been computed from the bidirectional non-linear seismic response of all the regular and vertical setback buildings across the height of the structure. This consideration has been made to visualize the maximum responses of the structures of the two orthogonal directions. The responses in each storey of the eight frames subjected to the seven ground motions considered has been plotted in Fig. 4.3. The maximum lateral storey displacements were extracted for the structural models (subjected to spectral acceleration of ~0.3g). It can be observed that horizontal roof displacements of the bare frame configurations are higher than corresponding infilled frame configurations i.e., 55%, 54%, 66%, 62% for R, T, M, and B models respectively. This pronounces the increased stiffness effect caused due to the interaction of infill with the bare RC frame on the overall structural response. Since OGS is most commonly observed structural configuration, the vulnerability of OGS buildings is clearly envisaged by means of increased horizontal displacement at first floor level. This is due to sudden drop in stiffness characteristics at the ground level components of the frame. Also, the influence of vertical setback RC buildings on the structural response can be visualized for both bare frame and infill frame structural models depicted in Fig. 4.3. in terms of horizontal displacements. Further, it can be observed that the horizontal displacements of the vertical setback RC frames are lower compared to that of the regular RC frame. These lower values of displacements can be attributed to the appropriate reduction in mass and stiffness characteristics along with the height of the building due to presence of setbacks along the vertical direction. This behaviour of the setback buildings changes the dynamic characteristics of the structure which significantly affects the inelastic capacity and needs to be accounted in estimation of seismic behaviour.


Imperial Valley
Mammoth Lakes
Chalfant Valley
Chamoli
India-Burma Border
North-West China
Story
Max. horizontal displacement (m)
Max. horizontal acceleration (m)
Bare R
Bare T
Bare M
Bare B
Infill R
Infill T
Infill M
Infill B
0
1
2
3
4
0.1
0.2
0.3
0.4

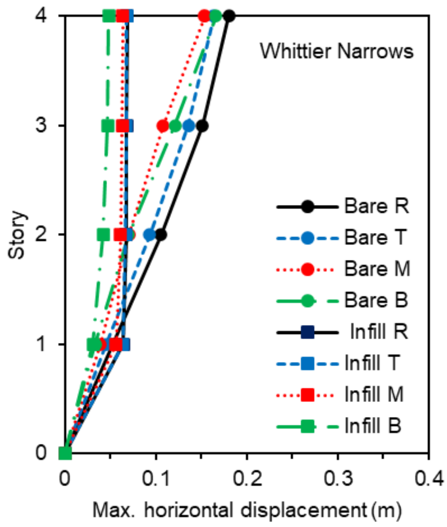
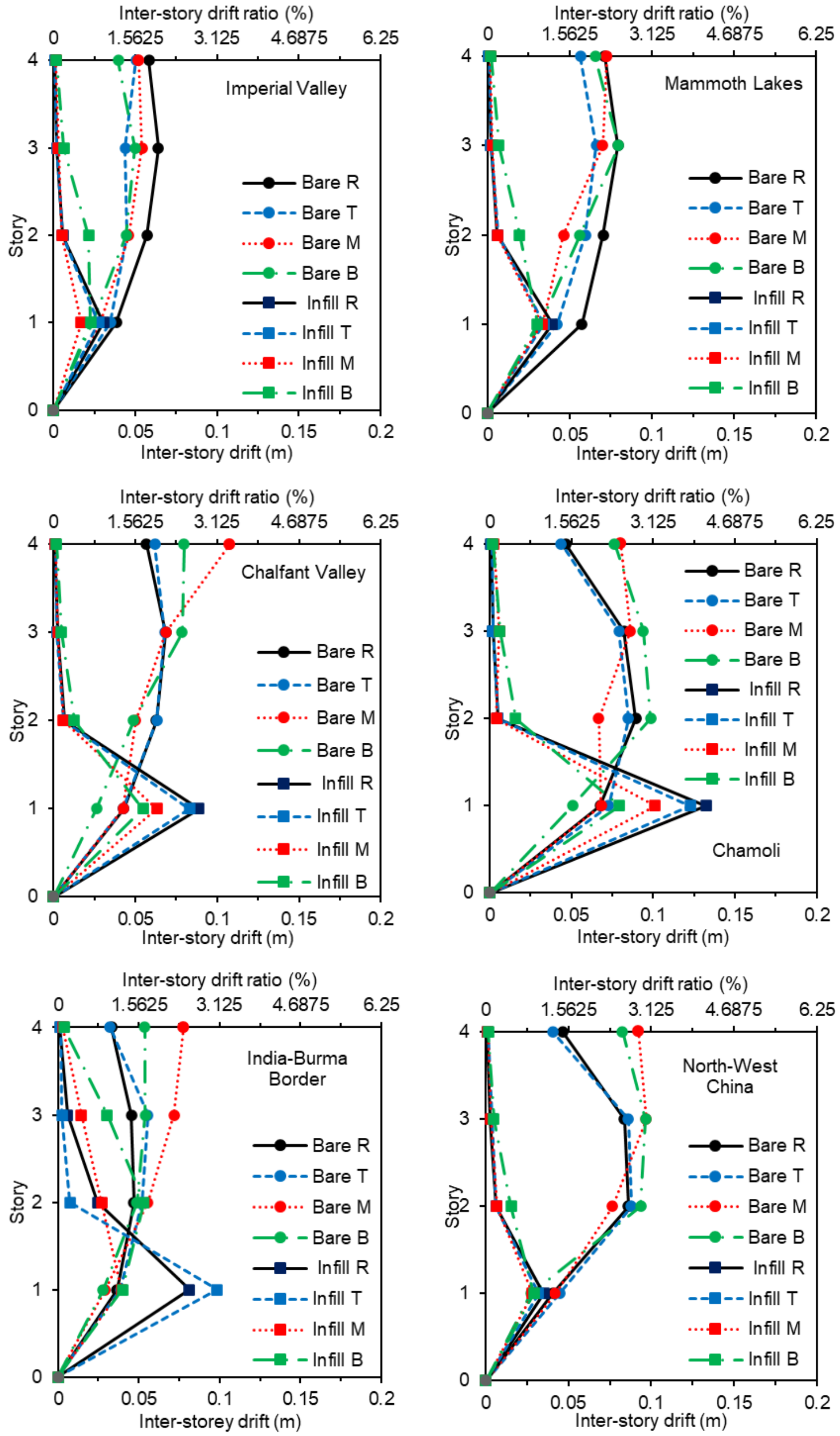


Fig. 4.3 Maximum horizontal storey displacements of all structural configurations under different ground motions

4.4.2 Inter-storey drifts

The responses (inter-storey drifts and inter-storey drift ratios) in each storey of all the eight building configurations subjected to the seven ground motions considered are shown in Fig. 4.4. The maximum lateral storey displacements were extracted for the structural models (subjected to spectral acceleration of ~0.3g). The first storey drifts of infill frame configurations are found to be much higher than that of corresponding bare frames as depicted in Fig. 4.4. This is similar in trend with horizontal displacements observed. This can be attributed mainly to the open ground storey influence, visualized even in case of horizontal displacements. This can be perceived as the weakness of the ground storey columns in withstanding the seismic force due to sudden reduction in stiffness characteristics at the ground level.

Also, it can be observed that IDRs of the bare frame configurations appear higher (almost 2 times) than corresponding infilled frame configurations above first storey level. This pronounces the increased stiffness effect due to presence of infill wall interacting with the RC frame above the ground storey. Similarly, it can be observed that IDR of the setback buildings along the height are lower than regular frame configurations. Further, the IDR is varied along the height of the RC building frame with respect to type of setbacks introduced along the height (i.e., R, B, M, T). These clearly contemplate the need to account for structural configuration changes in predicting the seismic response as it results in changed inelastic capacity.


Inter-story drift ratio (%)
0
1.5625
3.125
4.6875
6.25
Story
0
1
2
3
4
Inter-story drift (m)
0
0.05
0.1
0.15
0.2
Imperial Valley
Mammoth Lakes
Chalfant Valley
Chamoli
India-Burma Border
North-West China
Inter-storey drift (m)
Bare R
Bare T
Bare M
Bare B
Infill R
Infill T
Infill M
Infill B

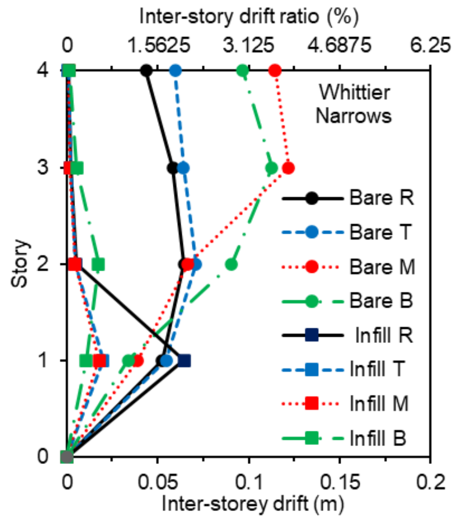
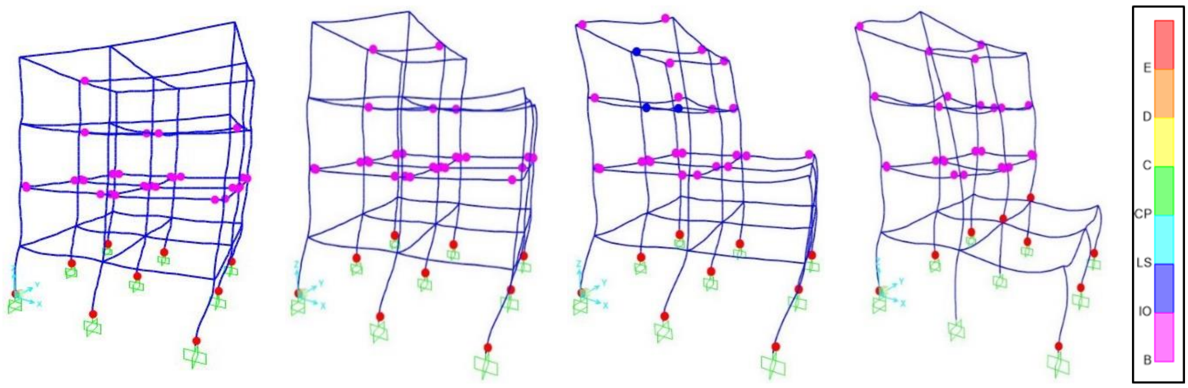


Fig. 4.4 Maximum horizontal storey displacements of all structural configurations under different ground motions

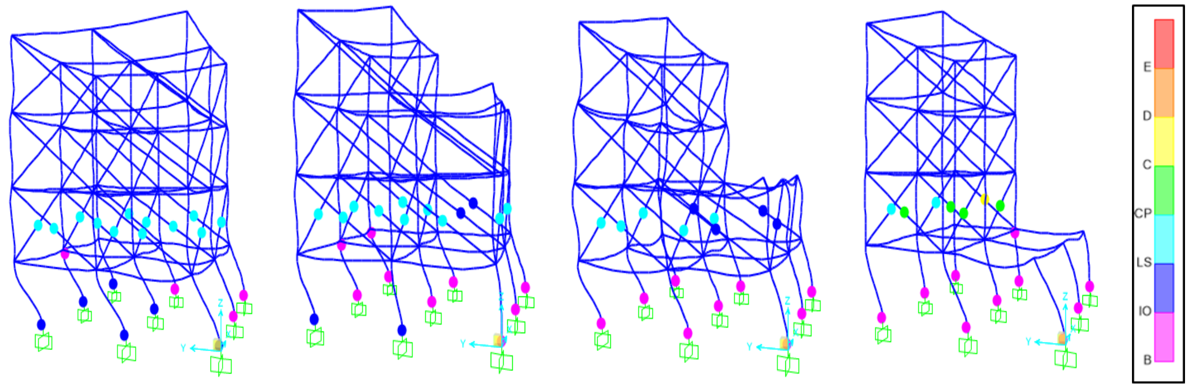
4.4.3 Local structural damage (Hinge Patterns)

The plastic hinge formation in an RC building serves as an indicator of structural damage induced when subjected to seismic events. The number of plastic hinges and the severity of those plastic hinge state can describe the performance level of that structure. The hinge patterns of the eight structural configurations at their corresponding yielding states and collapse states are depicted in Fig. 4.5 and Fig. 4.6 respectively. The legend present describes the various damage states of the structure with appropriate colours and labels in accordance with FEMA 356 (2000), i.e., IO: immediate occupancy, LS: life safety, CP: collapse prevention.

The purpose of these figures is to highlight the progression of local structural damages occurred in the buildings in the form of plastic hinge severity for the considered earthquake forces. However, effect of irregularity can also be seen to a certain extent in the formation of hinge patterns. From these Figs. 4.5-4.6, it can be observed that within a structural configuration, the higher number of structural components that reached the collapse state is found to be concentrated/restricted to lower two storeys than the above two storeys in case of infill frames, whereas distributed throughout in case of bare frames. This implies that the most of the energy dissipation in the form of inelastic/plastic deformations in the structural members has occurred due to the stiffness of the infill walls in case of infill frames, which is absent in case of bare frames in which the inelastic/plastic deformations in the structural members of higher storeys also contribute to the energy dissipation.

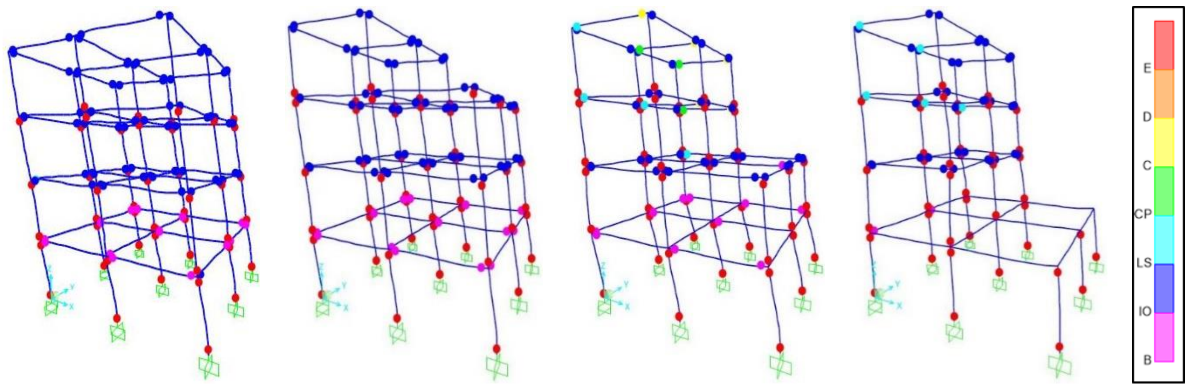


B-R B-T B-M B-B

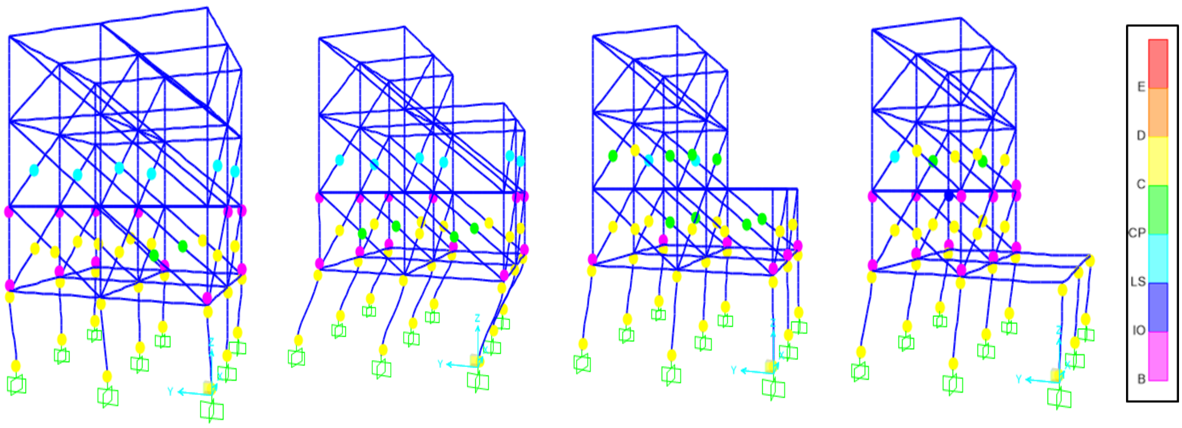


I-R I-T I-M I-B

Fig. 4.5 Hinges states for all structural models at yielding state



B-R B-T B-M B-B

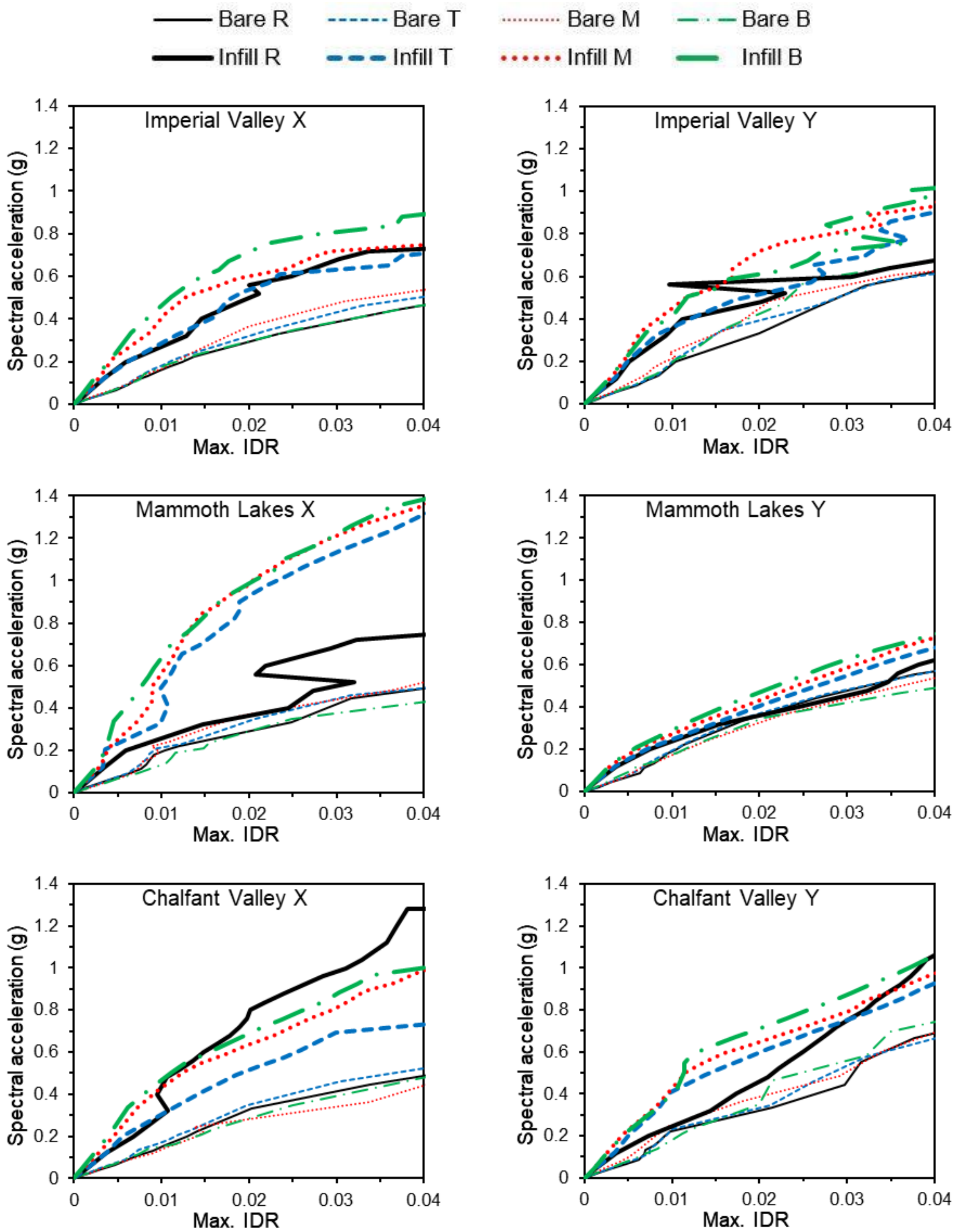


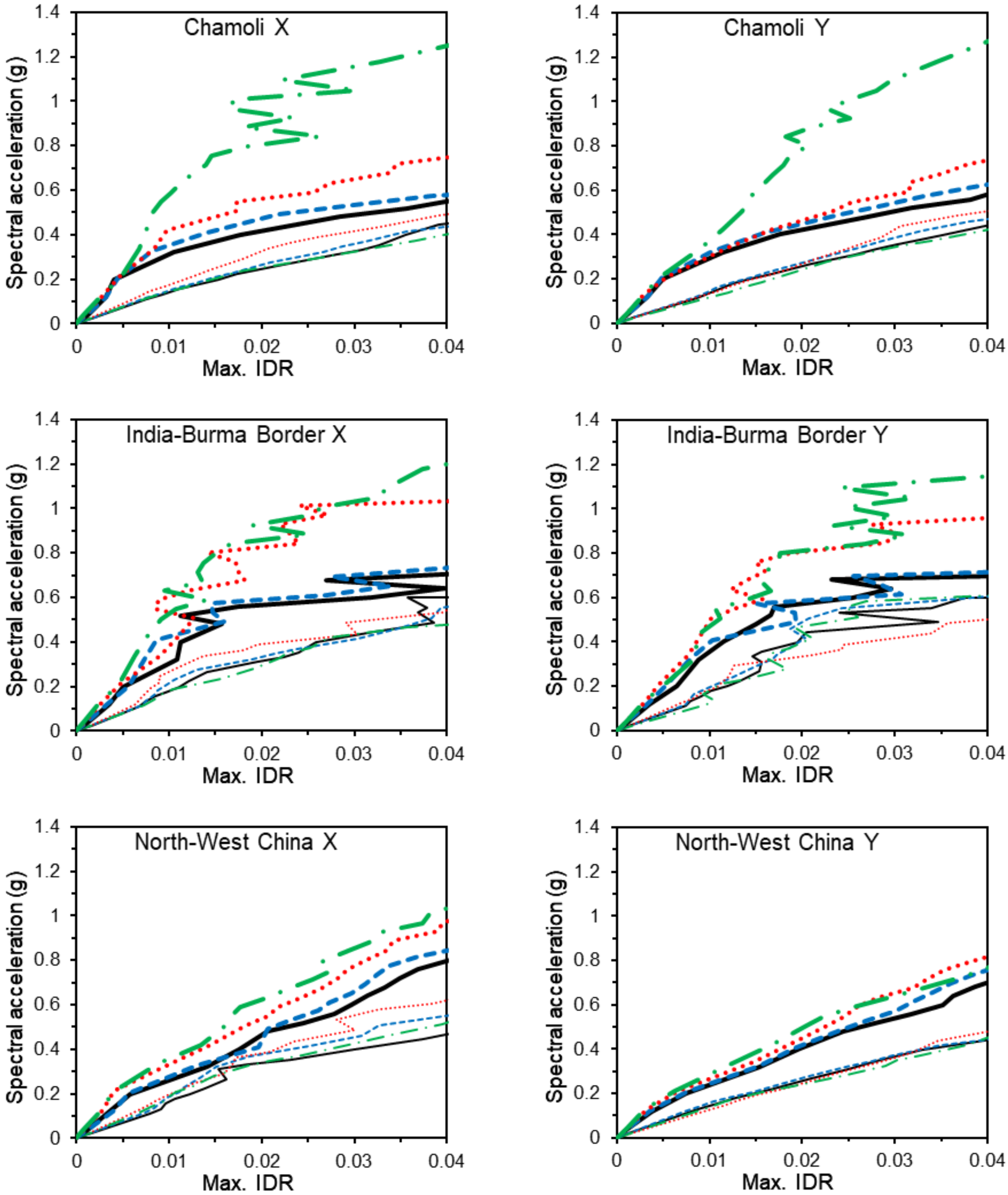
I-R I-T I-M I-B

Fig. 4.6 Hinges states for all structural models at collapse state

4.4.4 Dynamic structural capacity

The most commonly adopted EDP to describe the dynamic capacity of building structures are the Inter-storey Drift Ratio (IDR). Dynamic analyses are performed on eight different types of building configurations under eleven bi-directional ground motions resulting from around 1200 simulations of NLD analysis using the IDA approach. The dynamic capacity curves of structures subjected to three earthquake ground motions are depicted in Fig. 4.7 below. The outcome of these analyses is the dynamic capacity curve plots, represented in terms of S_a avg and maximum IDR as depicted in Fig. 4.7. It can be observed that dynamic capacity of the structural configuration changes due to the influence of irregularities present along the height of the RC building, resulting in changed dynamic characteristics. This is clearly evident even from the Eigen value analysis shown in Tables 4.2-4.3 which involves consideration of six modes for estimation of dynamic response. Also, this behaviour is clearly evident in the horizontal displacement and IDR values computed along the height of RC building models as discussed in sections 4.4.1 and 4.4.2. Further, it can be observed from these curves that bare frame building configurations reach collapse limit state at lower IM, compared to that of infill frames. In addition, it can be observed that spectral acceleration for infilled frame is higher than corresponding bare RC frame i.e., 71% & 35%; 84% & 41%; 76% & 70%; 108% & 70% for R, T, M, and B models respectively in X & Y directions. This pronounces the influence of infill wall interaction with corresponding bare RC frame in increasing the strength and stiffness during a seismic event.


Bare R
Bare T
Bare M
Bare B
Infill R
Infill T
Infill M
Infill B
Imperial Valley X
Imperial Valley Y
Mammoth Lakes X
Mammoth Lakes Y
Chalfant Valley X
Chalfant Valley Y
Spectral acceleration (g)
Max. IDR
0
0.2
0.4
0.6
0.8
1
1.2
1.4
0.01
0.02
0.03
0.04


Chamoli X
Chamoli Y
India-Burma Border X
India-Burma Border Y
North-West China X
North-West China Y
Spectral acceleration (g)
Max. IDR
0
0.2
0.4
0.6
0.8
1
1.2
1.4
0
0.01
0.02
0.03
0.04

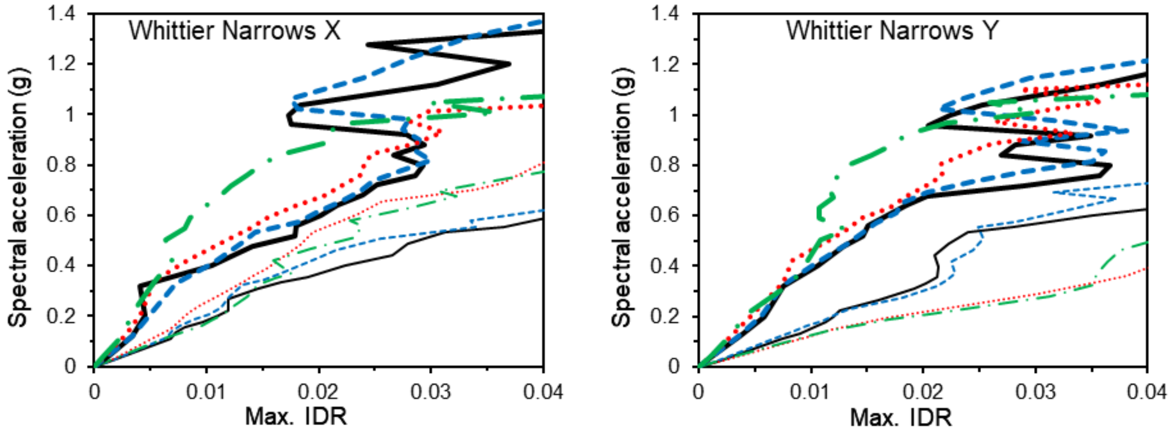


Fig. 4.7 Dynamic capacity (IDA) curves of all structural configurations under different ground motions

Furthermore, it can be observed that vertical setback buildings can resist higher spectral acceleration value compared to the regular RC frame. This higher resistance of setback buildings can be because of the lesser stiffness in the upper stories shows less negative impact than the positive impact of lesser mass in upper stories (Bhosale *et al.*, 2017; Bhosale *et al.*, 2018). This predominant feature is perhaps making the vertical setback RC buildings perform better than regular RC frame buildings. Therefore, this investigation emphasizes the need to account for configurational changes in estimation of seismic capacity and in predicting the inelastic behaviour of the structure.

4.4.5 Collapse fragility curves of bare and infill models subjected to earthquake

The probability of collapse for a particular intensity measure (S_a avg) is computed for both bare and infill frames subjected to seven earthquakes as depicted in Fig. 4.8. It can be concluded from the results that bare frames have a high probability of collapse at lower IM than compared to infill frames. This establishes the increase in the capacity of infill frames due to increased strength and stiffness when compared to that of bare frames. This is because that in case of infilled frame buildings, there is additional stiffness contribution from infill walls modelled as diagonal struts (Haldar and Singh, 2012). Due to this increased stiffness characteristics of infill frame buildings, the drifts demand of the structure reduces. This leads to increased median capacity in reaching the collapse state, thereby reducing the probability of collapse of the infilled frame buildings. This clearly emphasizes the importance of considering frame-infill interaction in seismic analysis of RC frame buildings.

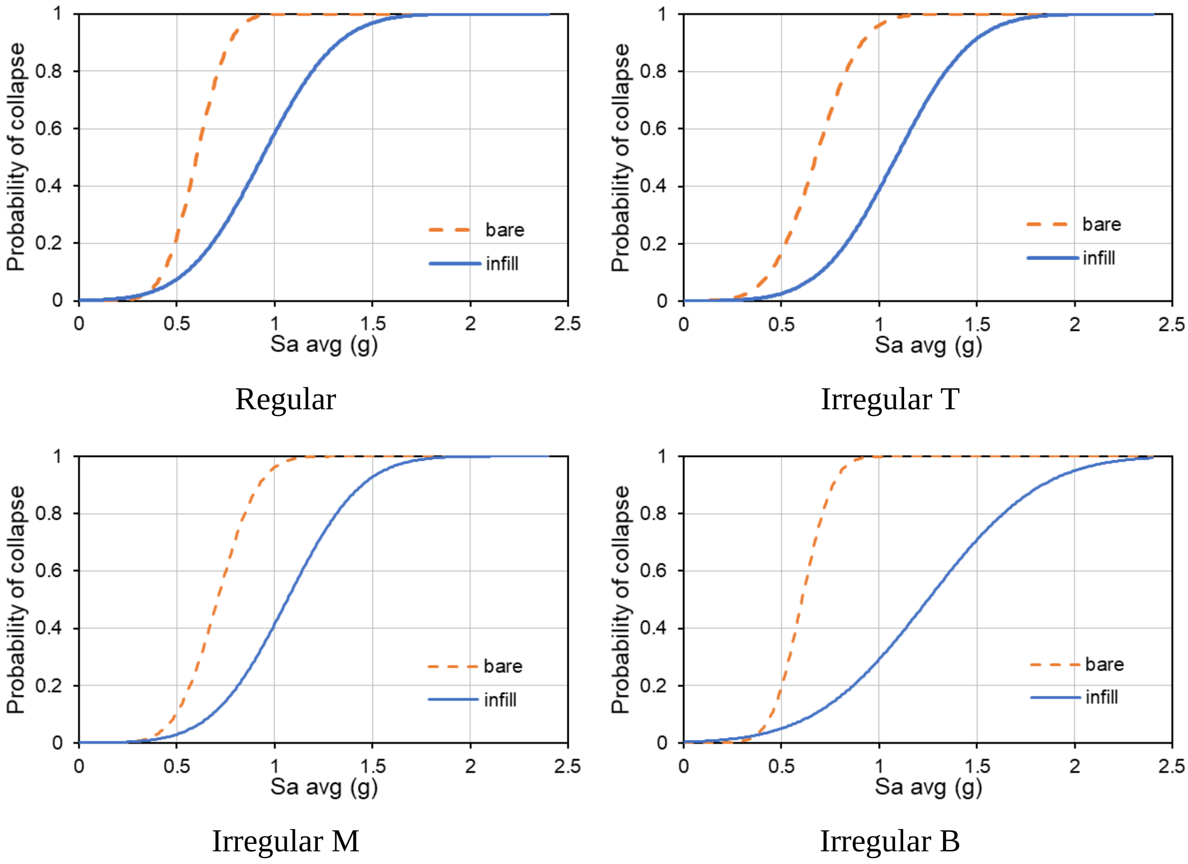


Fig. 4.8 Fragility curves of all structural configurations

4.5 Summary

The present chapter is focused on assessing the seismic behaviour of the RC building frame. Further, the changes in structural configurations which significantly alter the dynamic behaviour of the structure are usually not considered in seismic analysis and design procedures. Moreover, analysis of RC buildings for estimation of seismic design forces is usually carried out only on the moment-resisting frames (MRF), ignoring the interaction of the infill wall with the MRF. This results in the erroneous estimation of the seismic behaviour of the structure.

- From the analysis results depicting the seismic behaviour of buildings it can be observed that horizontal roof displacements experienced by bare frame configurations were found to be significantly higher than the corresponding infill frame configurations.
- This pronounces the increased stiffness effect caused due to the interaction of infill with the surrounding RC frame on the overall structural response.

- The horizontal displacements, IDR values computed along the height of the building vary with respect to presence of irregularities along the height. This emphasizes the need to account their behaviour in estimation of seismic response.
- Further, spectral accelerations experienced by the infill configurations at collapse limit state are higher than corresponding bare frame configurations. This advocates the influence of the infill wall contribution in significantly altering the dynamic characteristics of regular and vertical setback buildings.
- Furthermore, it can be observed that the horizontal displacements of the vertical setback RC frames are lower compared to that of the regular RC frame. These lower values of displacements can be attributed to the appropriate reduction in mass and stiffness characteristics along with the height of the building due to setbacks in the vertical direction.

CHAPTER 5

Behaviour assessment of 3D RC frame buildings under sequential earthquake event succeeding the first event

5.1 General

Earthquake occurrence at any location is randomly oriented and repeats itself number of times after a main earthquake event. These repeated earthquakes in many occasions are found to possess similar or higher energy than the main event. In reality, since these events occur in short duration of time, it impairs the repair/strengthening measures on existing structures resulting in damage accumulation, which finally lead to collapse. In view of this as discussed in Chapter 1, there is a necessity for seismic behaviour assessment under repeated earthquake forces. Hence, in this chapter seismic behaviour assessment of three-dimensional RC building frame models are carried out under simultaneous bi-directional sequential earthquake forces. This is carried out analogous to the inelastic capacity assessments made in chapter 4 in terms of response parameters (IM and EDP). In addition, the changes in structural behaviour due to single and repeated earthquake events are also discussed.

5.2 Sequential earthquake event

In general, seismic design of structures considers a single earthquake scenario (MCE/DBE) force to be evaluated for a particular performance criterion in PBD methodology. These comprises of Collapse prevention in general, for ordinary structures, life-safety criteria for certain important infrastructures, other than life line category, where in the structure is not expected to experience any damages during its life time. This criteria of evaluation of exclusively considering only isolated earthquake force termed as 'design earthquake ', do not address the influence of repeated nature of earthquake forces on the structure. It has been recommended in literature that the social and economic considerations have necessitated, multiple-load and target limit state seismic performance-based design as the current best practice for seismic analysis and design.

In this regard, it can be observed that various procedures for selecting sequential ground motion records have been followed in literature, including use of same record as both mainshock and aftershock, using historical mainshock and aftershock records or use of different ground motion records for mainshock and aftershock. Further, conflicting results have been reported regarding the effects of ground motion selection in mainshock-aftershock studies. Therefore, it is generally recommended in literature to use some source recorded seismic sequences as more optimal approach, and not to use pairs of mainshock-mainshock records without any altering of ground motion characteristics (e.g., magnitude, frequency content, etc.). Hence, in this study real/ actual seismic sequences chosen from ground motion databases (PEER, COSMOS) are adopted. Regarding scaling of aftershock ground motion: we to the best of our knowledge reiterate that there is no information/clarity regarding target spectrum to which aftershock records needs to be scaled. However, in order to analyse the seismic behaviour, the ground motion records are to be made spectrum compatible in accordance with the design spectrum specified by codal provisions at chosen site location. Hence, in this study the chosen ground motion sequences (one mainshock and one aftershock for each earthquake) are made spectrum compatible with IS 1893 (Part 1): 2016, Seismic zone III medium soil profile to suit the conditions of Warangal city, Telangana state.

The repeated/sequential earthquake events represented in terms of ground motion records are usually composed of:

- one main mainshock (i.e., the event with the largest earthquake magnitude) and multiple aftershocks,
- two earthquakes sequence (mainshock plus one aftershock),
- three earthquakes sequence (mainshock plus two aftershocks), etc.

The scenario of two earthquakes sequence has been more commonly employed in many of the available studies (Goda and Taylor 2012; Zhai *et al.* 2014; Ruiz-García and Aguilar 2015; Zhang *et al.* 2017; Yang *et al.* 2019). The observations presented in these studies, suggested that consistent information about the influence of aftershock can be obtained from consideration of two earthquakes sequence. Therefore, one main shock plus one aftershock sequence-type ground motion has been adopted in this study. The research in this direction has been recently initiated, with very few studies being reported in the literature in terms of consideration of multiple earthquake events for seismic resilient design.

5.3 Ground motion data

Ground motion records for sequential earthquake forces at the chosen location in this study (Warangal represented by Seismic Zone III with medium soil profile) has been developed by joining the spectrum-compatible accelerograms related to chosen earthquake at a particular station. These accelerograms are selected from the available online data bases based on site condition and made spectrum compatible according to the regulations of the seismic code using SeismoMatch program as discussed in Chapter 3, and shown in Fig. 5.1. The accelerograms to be coupled, are separated by a time interval of 100 seconds by padding zero ordinates between them, thereby generating a seismic sequence. The separation time has been added to stop the movement of the structure initiated by the first shock, by means of inherent damping to attain rest position (Hatzigeorgiou and Liolios 2010, Zhang *et al.* 2017).

The seismic sequences generated and used in this study (i.e., Imp, Mam, Chal, Ch, ibb, NWC and Wh) from their corresponding single accelerograms (i.e., Imp1 & Imp2, Mam1 & Mam2, Chal1 & Chal2, Ch1 & Ch2, ibb1 & ibb2, NWC1 & NWC2 and Wh1 & Wh2) in both the perpendicular directions (X and Y) are shown in Fig. 5.2. A total of 21 pairs (Single and Sequence) of accelerograms (orthogonal directions) were used in this analysis as listed on Table 5.1.

Table 5.1 Details of seismic ground motion data used in this study

S. No.	Earthquake name	Station name	Date	Magnitude	Denoted as	Source
1.	Imperial Valley 01	Holtville Post Office	10/15/1979	6.53	Imp1	PEER
2.	Imperial Valley 02		10/15/1979	5.01	Imp2	
3.	Mammoth Lakes 01	Convict Creek	5/25/1980	5.69	Mam1	PEER
4.	Mammoth Lakes 02		5/25/1980	5.91	Mam2	
5.	Chalfant Valley 01	Zack Brothers Ranch	7/20/1986	5.77	Chal1	PEER
6.	Chalfant Valley 02		7/20/1986	6.1	Chal2	
7.	Chamoli 01	Gopeshwar	3/29/1999	6.6	Ch1	COSMOS

8.	Chamoli 02		3/29/1999	5.4	Ch2	
9.	India-Burma Border 01	Berlongfer	8/6/1988	7.2	ibb1	COSMOS
10.	India-Burma Border 02		1/10/1990	6.1	ibb2	
11.	North-West China 01	Jiashi	4/11/1997	6.1	NWC1	PEER
12.	North-West China 02		4/15/1997	5.8	NWC2	
13.	Whittier Narrows 01	San Marino - SW Academy	10/1/1987	5.9	Wh1	PEER
14.	Whittier Narrows 02		10/4/1987	5.3	Wh2	

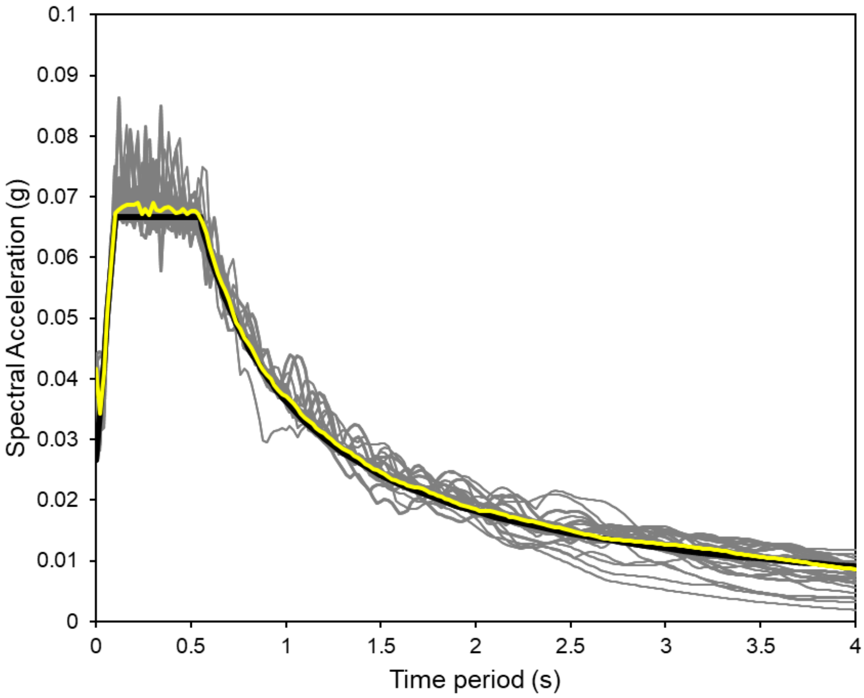
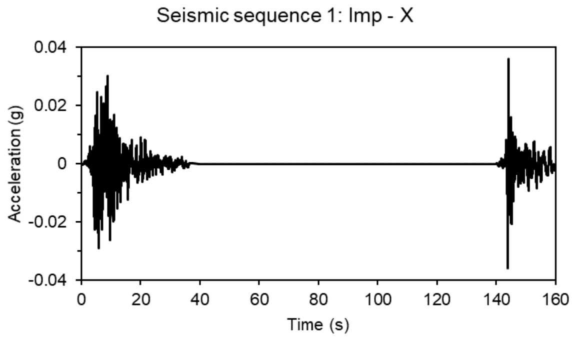
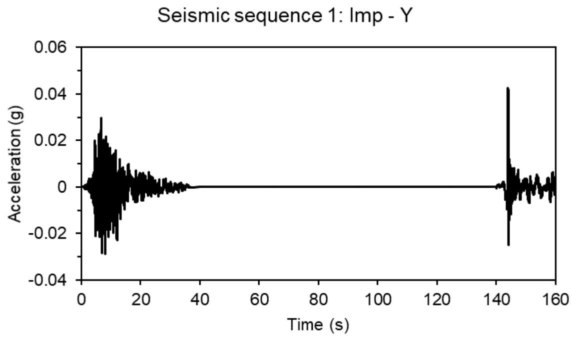
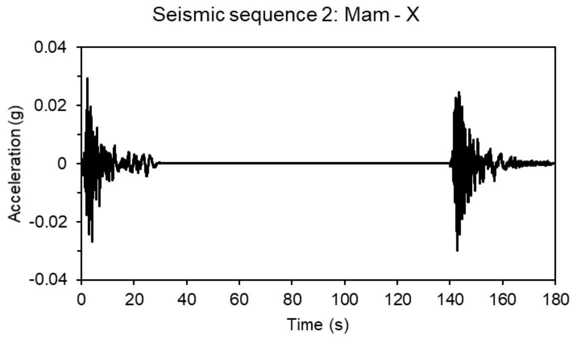
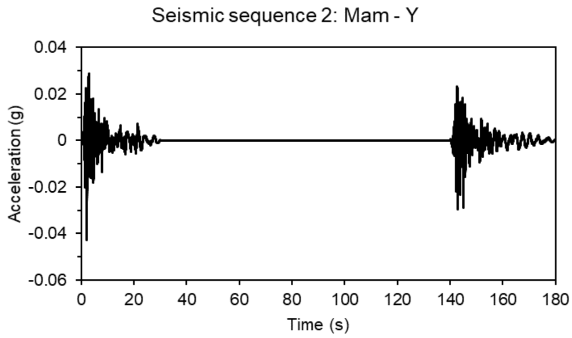
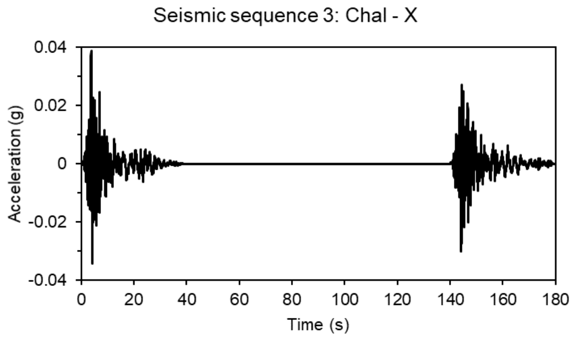
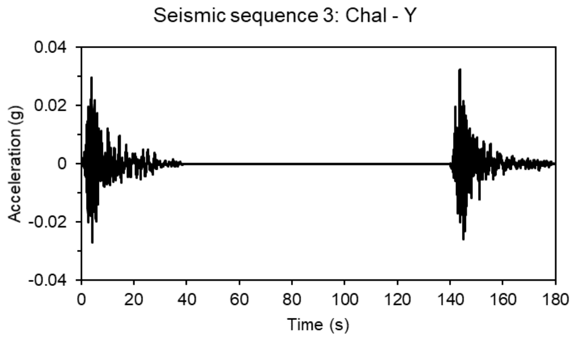
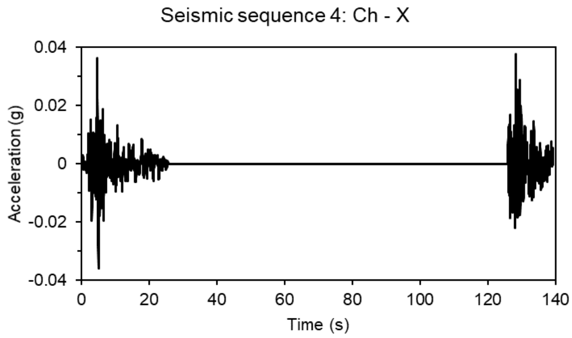
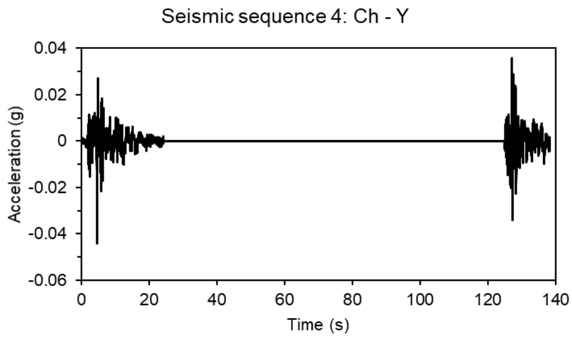


Fig. 5.1 Ground motions scaled to the design target spectrum


Seismic sequence 1: Imp - X
Acceleration (g)
Time (s)

Seismic sequence 1: Imp - Y
Acceleration (g)
Time (s)

Seismic sequence 2: Mam - X
Acceleration (g)
Time (s)

Seismic sequence 2: Mam - Y
Acceleration (g)
Time (s)

Seismic sequence 3: Chal - X
Acceleration (g)
Time (s)

Seismic sequence 3: Chal - Y
Acceleration (g)
Time (s)

Seismic sequence 4: Ch - X
Acceleration (g)
Time (s)

Seismic sequence 4: Ch - Y
Acceleration (g)
Time (s)

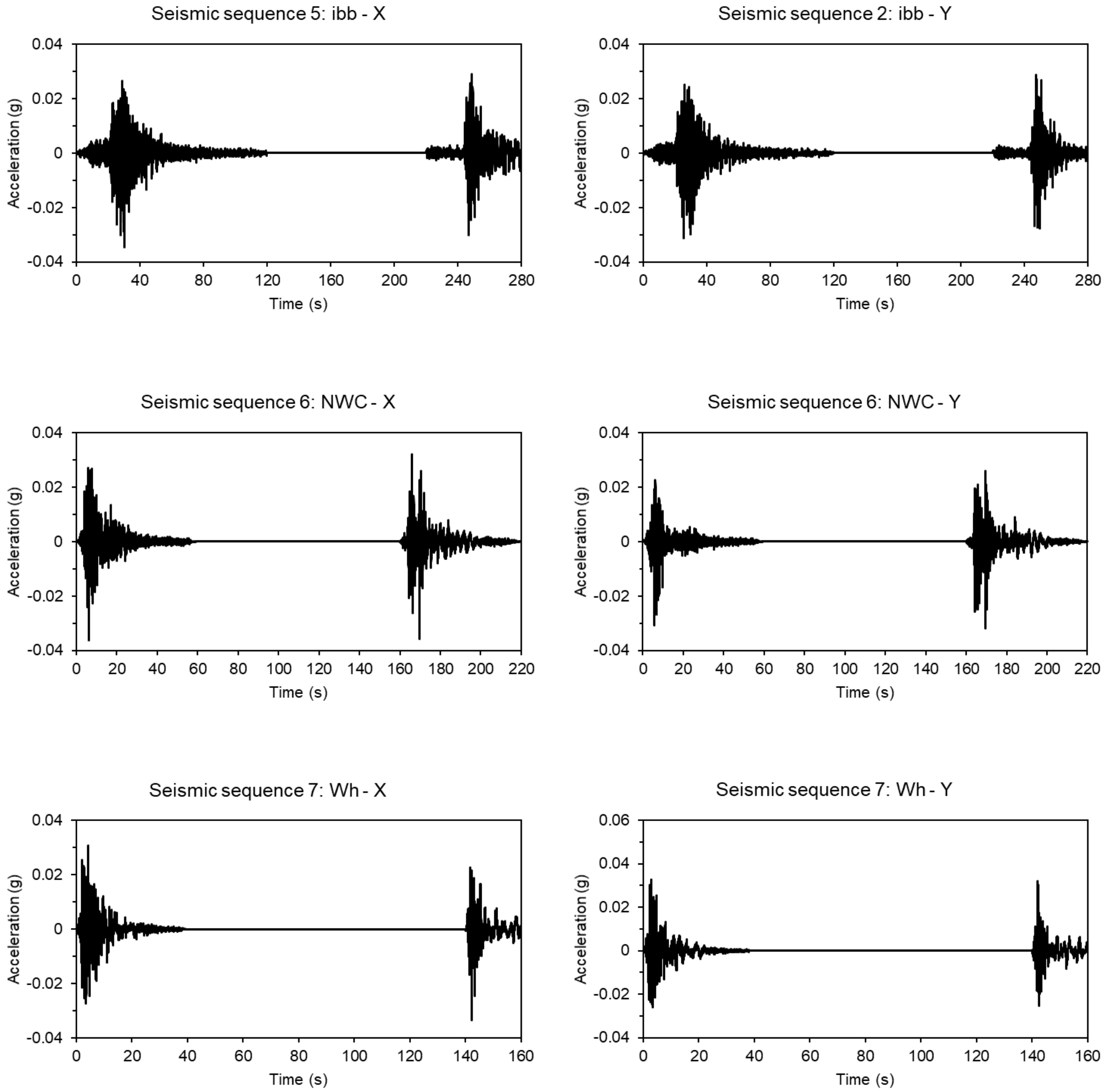


Fig. 5.2 Seismic sequences used as input for non-linear dynamic analysis

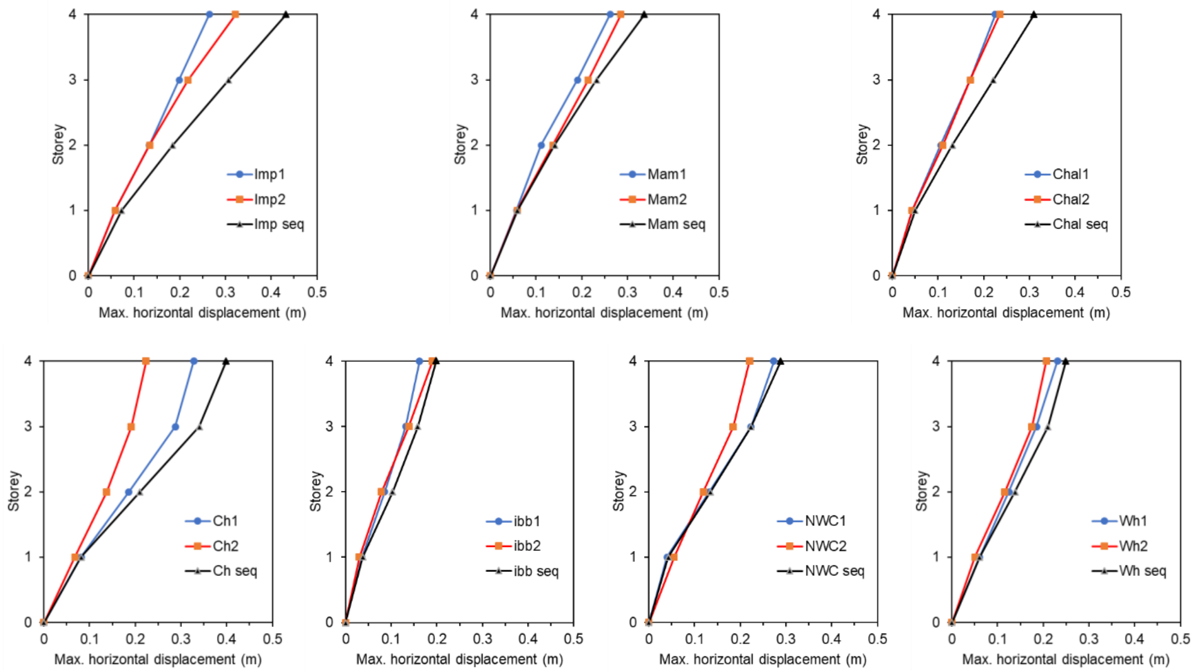
5.4 Seismic behaviour of RC MRF models under sequential earthquakes vs single earthquakes

The non-linear dynamic analyses in the form of IDA are carried out on RC building frame models under sequential earthquake forces analogous to the discussions represented in Chapter 4 for single or isolated earthquake forces. This process involves around 4000 NLD simulations using the IDA approach to arrive at non-linear response characteristics for the structural models considered. Further, response parameters obtained from both sequential and isolated / single earthquakes are compared to visualize the effect of sequential repeated events on seismic behaviour.

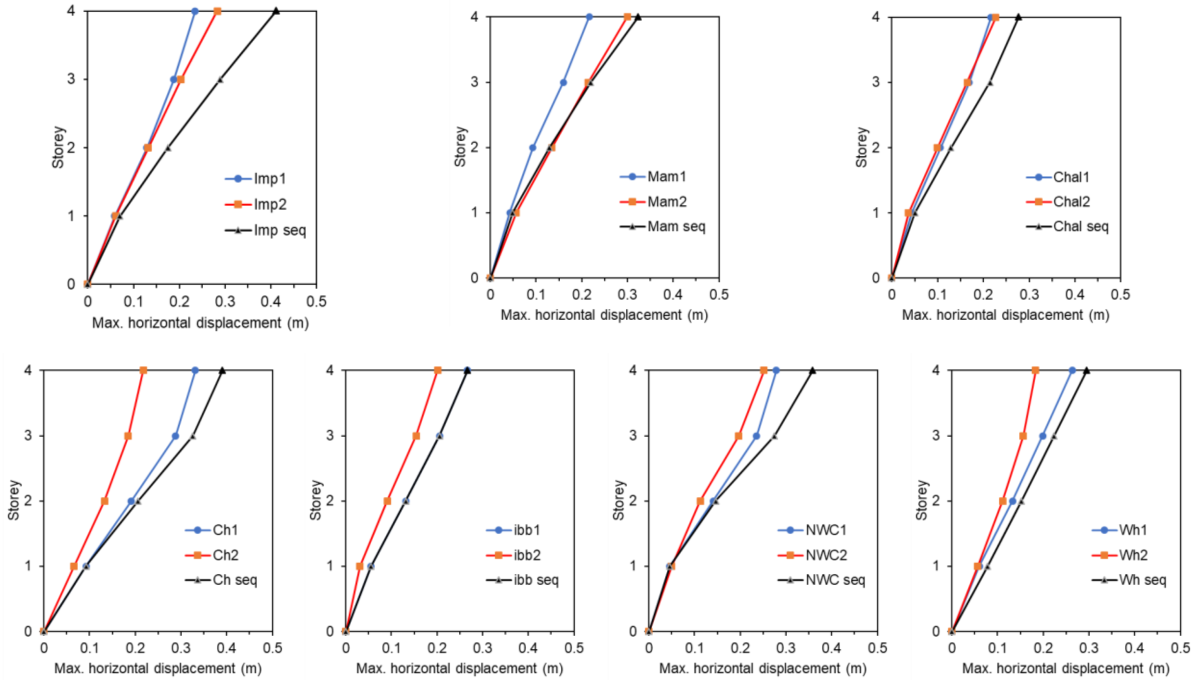
5.4.1 Seismic behaviour of RC bare frame models

5.4.1.1 Maximum lateral story displacements

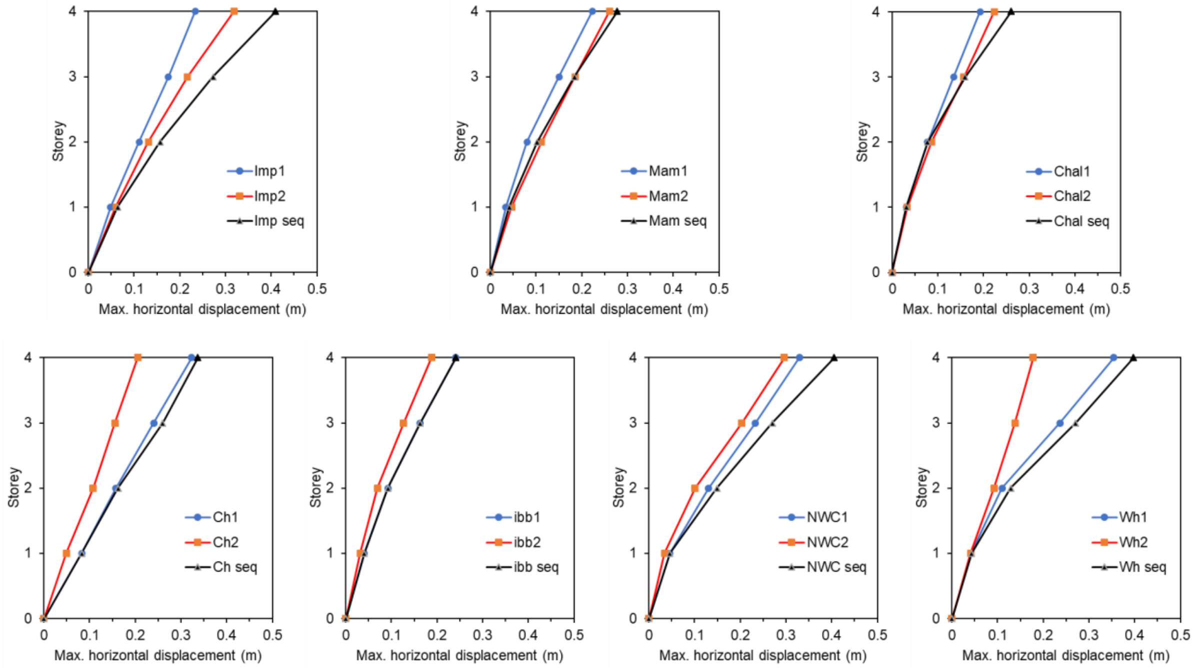
The absolute maximum horizontal displacements have been computed from the bi-directional non-linear seismic response of all the structural models throughout the height of the structure. This is extracted under sequential earthquake forces similar to the one presented in Chapter 4 for isolated earthquakes. The maximum story displacements were extracted for all the structural models (subjected to spectral acceleration of ~0.3g), computed along the lateral directions for a specific 'IM' are depicted in Fig. 5.3. It can be observed that the story displacements under sequential earthquakes are found to increase significantly unlike the corresponding individual isolated earthquakes for all the structural configurations. This increase was found to be around 35%, 41%, 41%, and 31% for the B-R, B-T, B-M, and B-B models respectively.



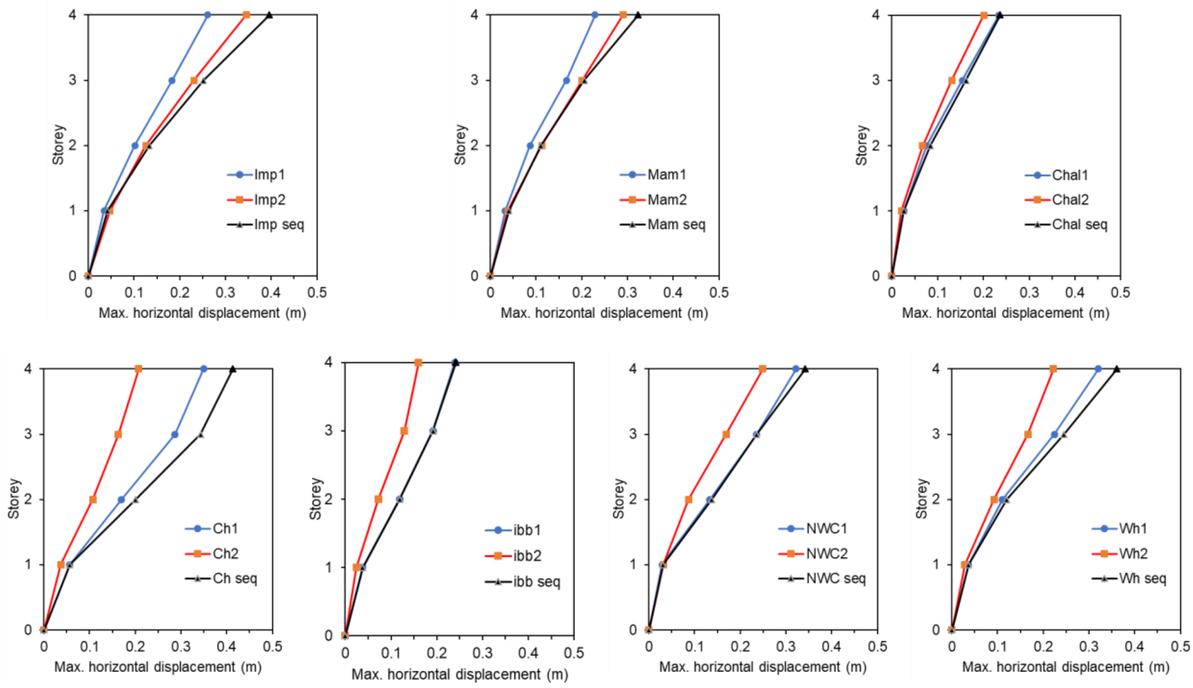
(a)


Storey
Max. horizontal displacement (m)
Imp1
Imp2
Imp seq
Mam1
Mam2
Mam seq
Chal1
Chal2
Chal seq
Ch1
Ch2
Ch seq
ibb1
ibb2
ibb seq
NWC1
NWC2
NWC seq
Wh1
Wh2
Wh seq

(b)


Storey
Max. horizontal displacement (m)
Imp1
Imp2
Imp seq
Mam1
Mam2
Mam seq
Chal1
Chal2
Chal seq
Ch1
Ch2
Ch seq
ibb1
ibb2
ibb seq
NWC1
NWC2
NWC seq
Wh1
Wh2
Wh seq

(c)



(d)

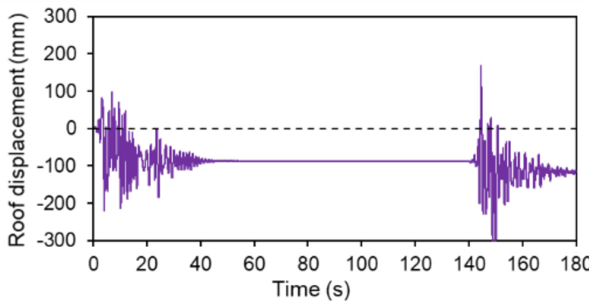
Fig. 5.3 Maximum lateral storey displacements of structural models: (a). B-R, (b). B-T, (c). B-M, (d). B-B subjected to sequential earthquakes

5.4.1.2 Permanent structural damage (Residual displacements)

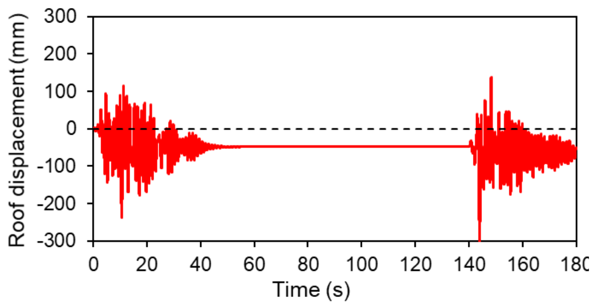
The permanent damage gets manifested in RC MRFs in the form of residual displacements under sequential earthquakes. This phenomenon can be observed when the structure remains in a plastic state after the first earthquake and before being subjected to further seismic events. In general, when the RC building MRFs is subjected to sequential events in a short duration the dynamic characteristics of the structure get changed due to degradation in stiffness and strength characteristics leading to impaired structural performance.

Any structure when subjected to earthquake excitation vibrates, and comes to rest position due to the presence of structure's inherent damping. In case, the structure does not return to the initial position at the end of vibration, certain permanent deformations get manifested within the structure. These permanent deformations in the structure are termed as residual displacements and can be computed for any structure. In order to investigate the formation of residual displacements, an extra time history data for 100s of zero acceleration was provided at the end of first/ main shock, which is sufficient for structure to come to rest position within that time duration (Hatzigeorgiou and Liolios 2010, Zhang *et al.* 2017). After performing analysis for that entire time history (actual time history and extra 100s), the displacement values at the end of the time history are extracted at all the storey levels. Those displacements are used to compute and plot the residual displacements of the structure using spreadsheet program.

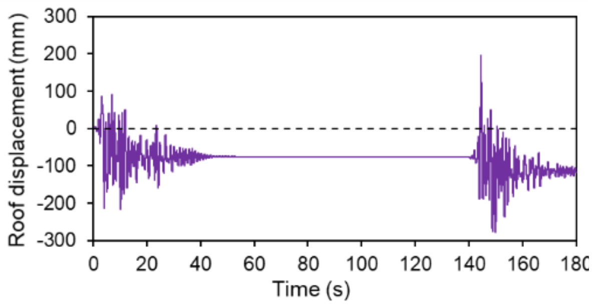
The structural response in terms of roof displacements has been computed for all the structural models at a specific spectral acceleration value. The response for all the sequences is found to be of a similar order, hence for brevity, responses computed for sequence #3 (i.e., Chalfant Valley) and sequence #4 (i.e., Chamoli) are presented in Figs. 5.4-5.5. From the results, it can be concluded that the accumulation of damages can be visualized in both orthogonal directions in terms of residual displacements under the sequential earthquake forces. This can be attributed to the structural weakness of RC building frames in resisting sequential earthquake events after getting damaged due to the first one, described in terms of displacement response. This feature can also be observed in the fragility curves developed and presented in Fig. 5.11.



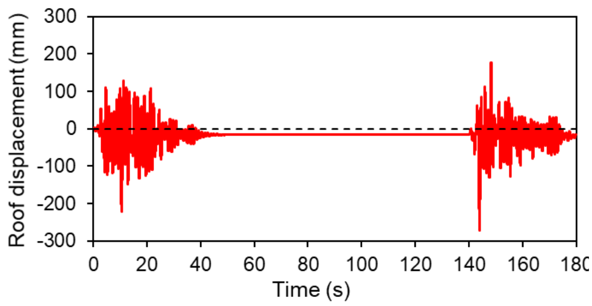
(a)



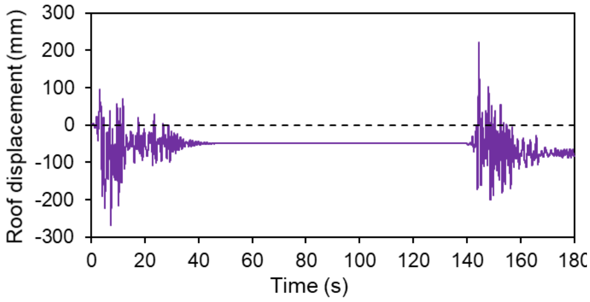
(b)

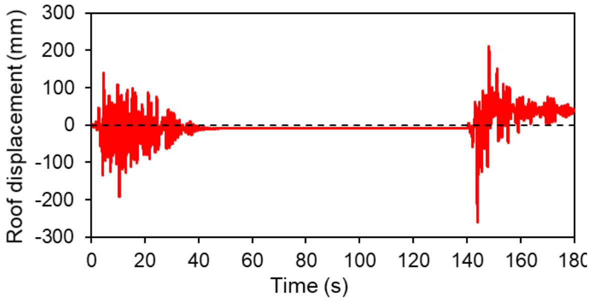


(c)

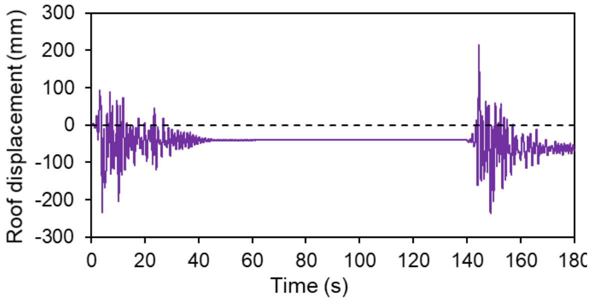


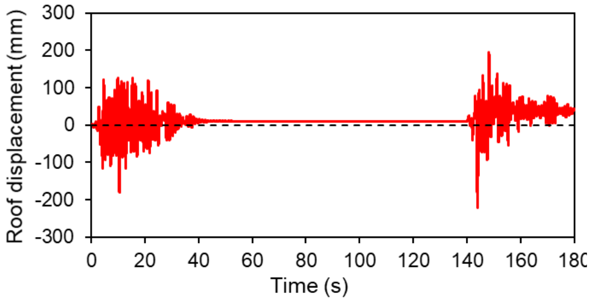
(d)





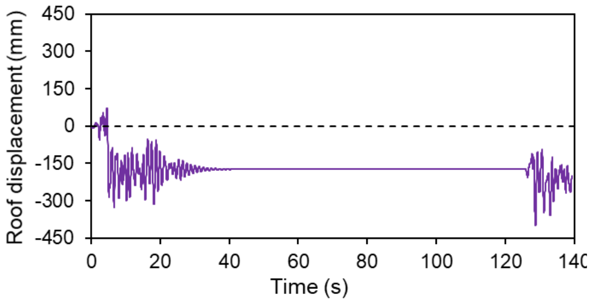
(e) (f)

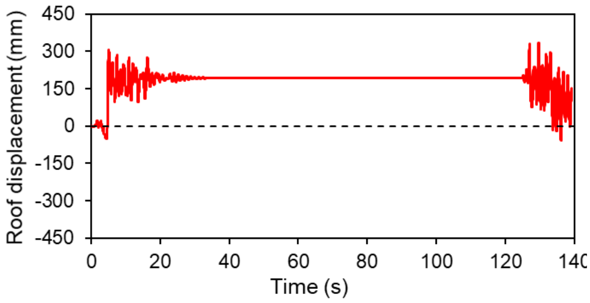




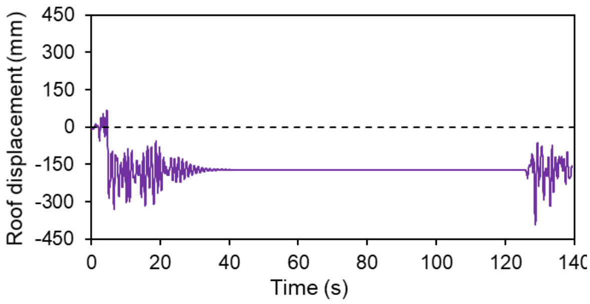
(g) (h)

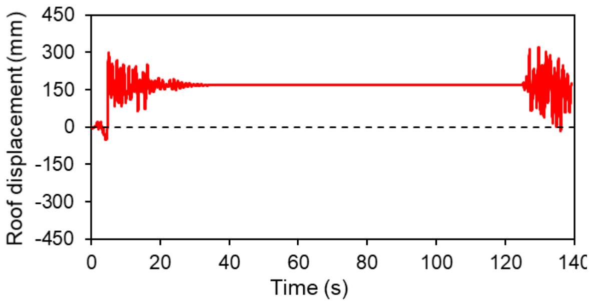
Fig. 5.4 Roof displacements in both X & Y directions of structural models subjected to sequential earthquake Chalfant Valley: (a). B-R – X (b). B-R – Y (c). B-T – X (d). B-T – Y (e). B-M – X(f). B-M – Y (g). B-B – X (h). B-B – Y





(a) (b)





(c) (d)

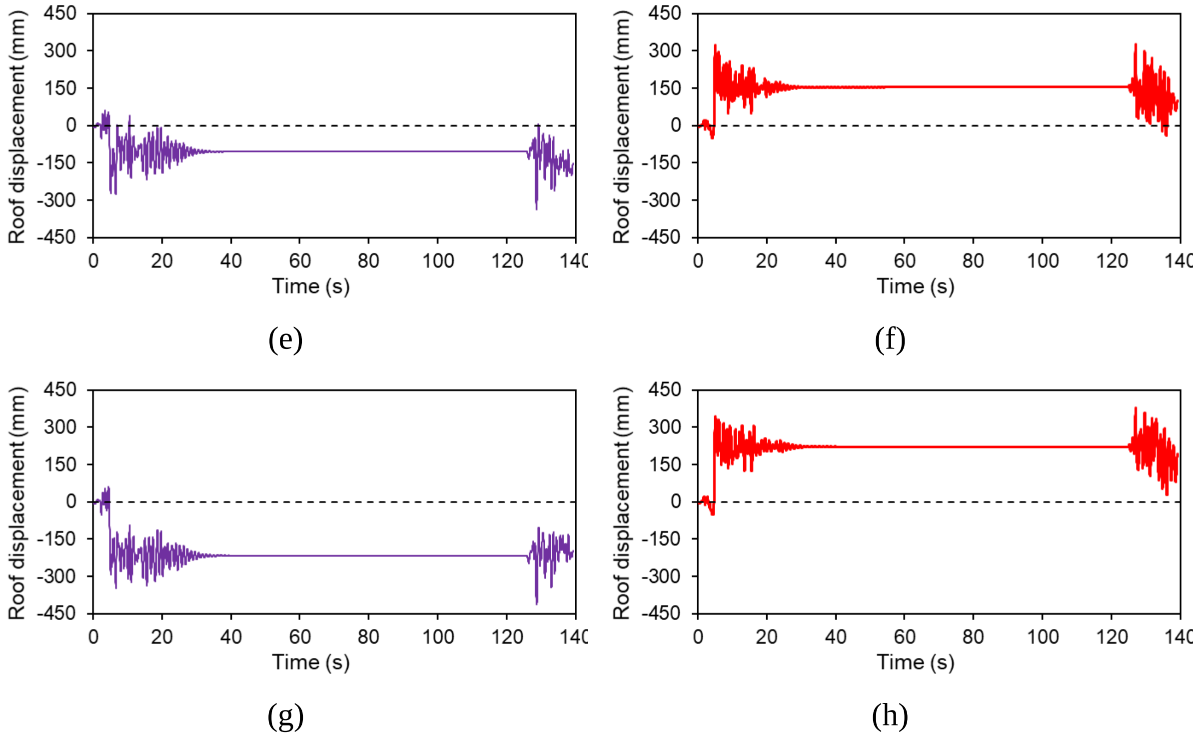
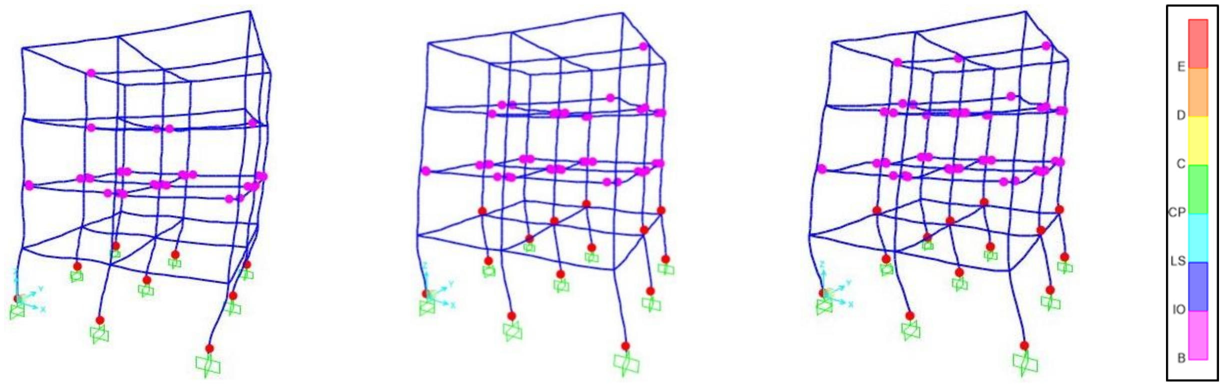


Fig. 5.5 Roof displacements in both X & Y directions of structural models subjected to sequential earthquake Chamoli: (a). B-R – X (b). B-R – Y (c). B-T – X (d). B-T – Y (e). B-M – X (f). B-M – Y (g). B-B – X (h). B-B – Y

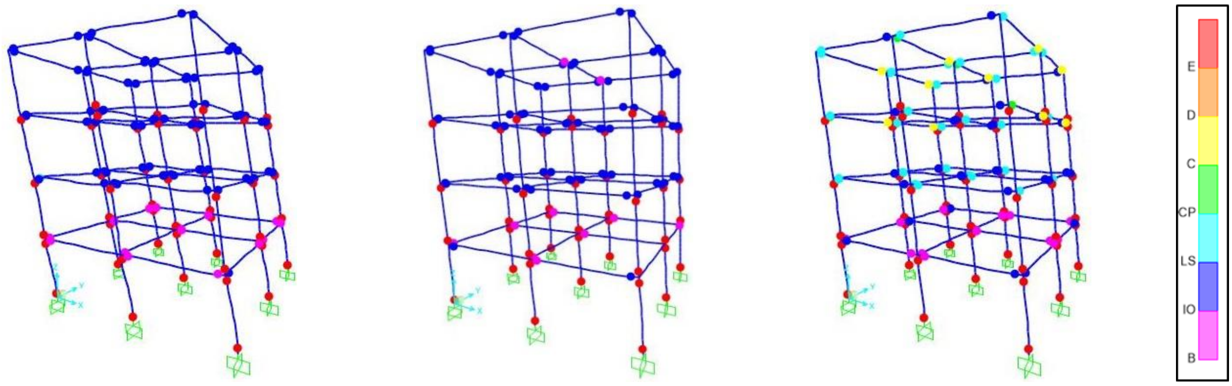
5.4.1.3 Local Structural Damage (Hinge patterns)

In general, during seismic analysis procedure (NLS or NLD) the local structural damages of structural components in an RC MRF can be visualized in the form of plastic hinge formation. The number of plastic hinges and the severity of the plastic hinge state can describe the performance level of any structure as discussed in Chapter 4 for isolated earthquake force. Similarly, to envisage the local structural damages, the plastic hinge states for the Chalfant Valley sequential earthquake are depicted in Figs. 5.6-5.9 at both yielding state and just before collapse limit states.

The Figs. 5.6-5.9, (a, b, c) represent hinge states at yielding, and (d, e, f) represent hinge states at collapse for (Chal1, Chal2, Chal) accelerograms respectively. Of those, (a & d) represent hinge patterns when subjected to one individual earthquake as discussed in Chapter 4, (b & e) represent hinge patterns when subjected to subsequent individual earthquake, and (c & f) represent hinge patterns when subjected to sequential earthquake (one event followed by another event). The legend describes the various damage states of the structure with appropriate colours and labels in accordance with FEMA 356 (2000), i.e., IO: immediate occupancy, LS: life safety, CP: collapse prevention. From the results, it can be observed that the total number of structural components within a structural configuration that reached the collapse state is found to be higher under sequential earthquakes, unlike isolated earthquake forces.

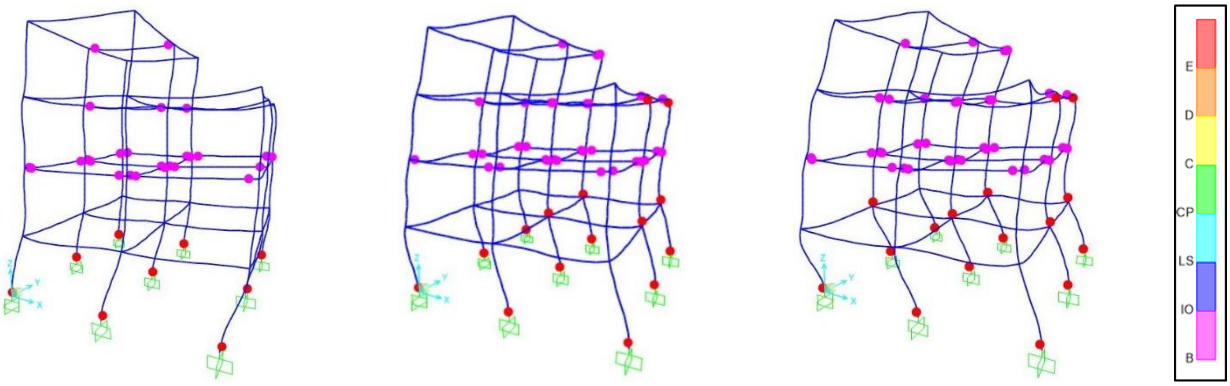


(a) (b) (c)

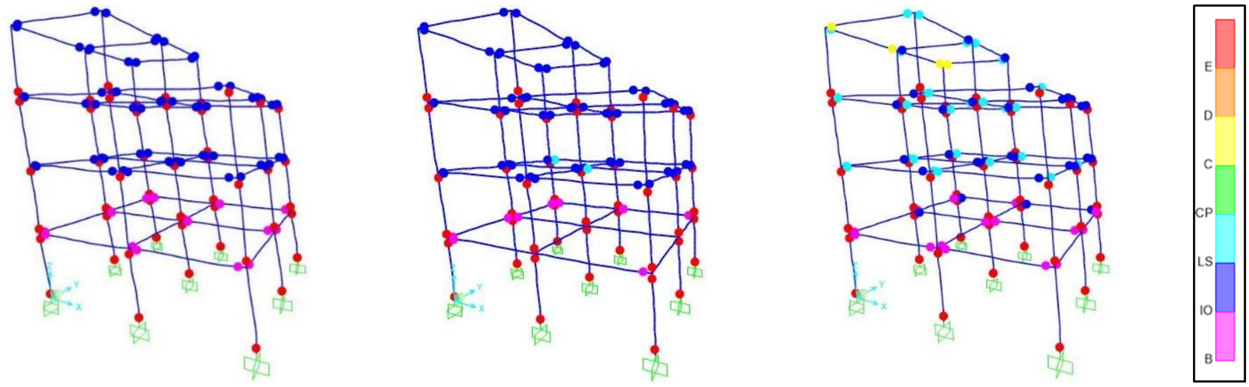


(d) (e) (f)

Fig. 5.6 Hinges states for structural model B-R

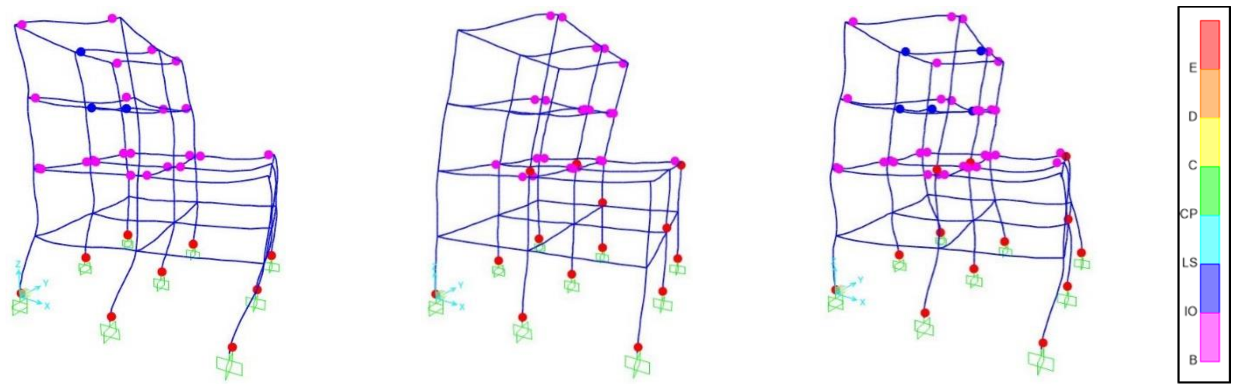


(a) (b) (c)

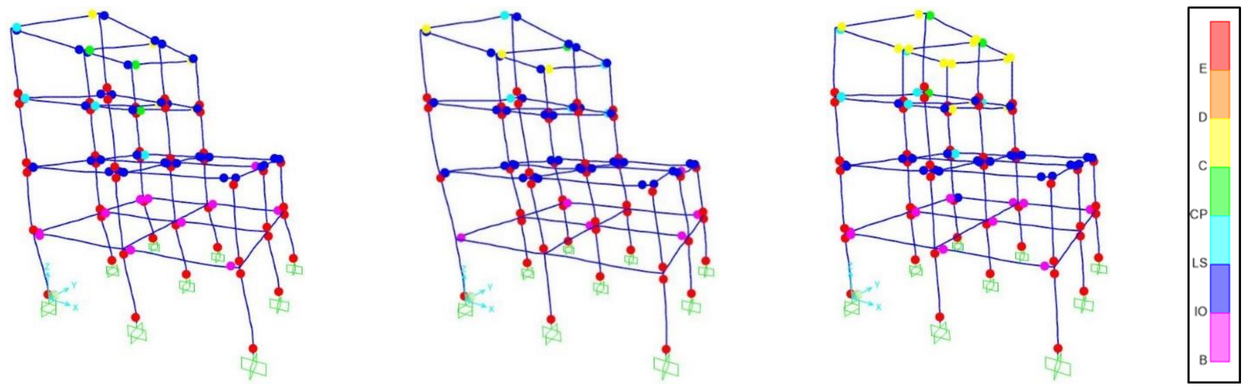


(d) (e) (f)

Fig. 5.7 Hinges states for structural model B-T



(a) (b) (c)



(d) (e) (f)

Fig. 5.8 Hinges states for structural model B-M

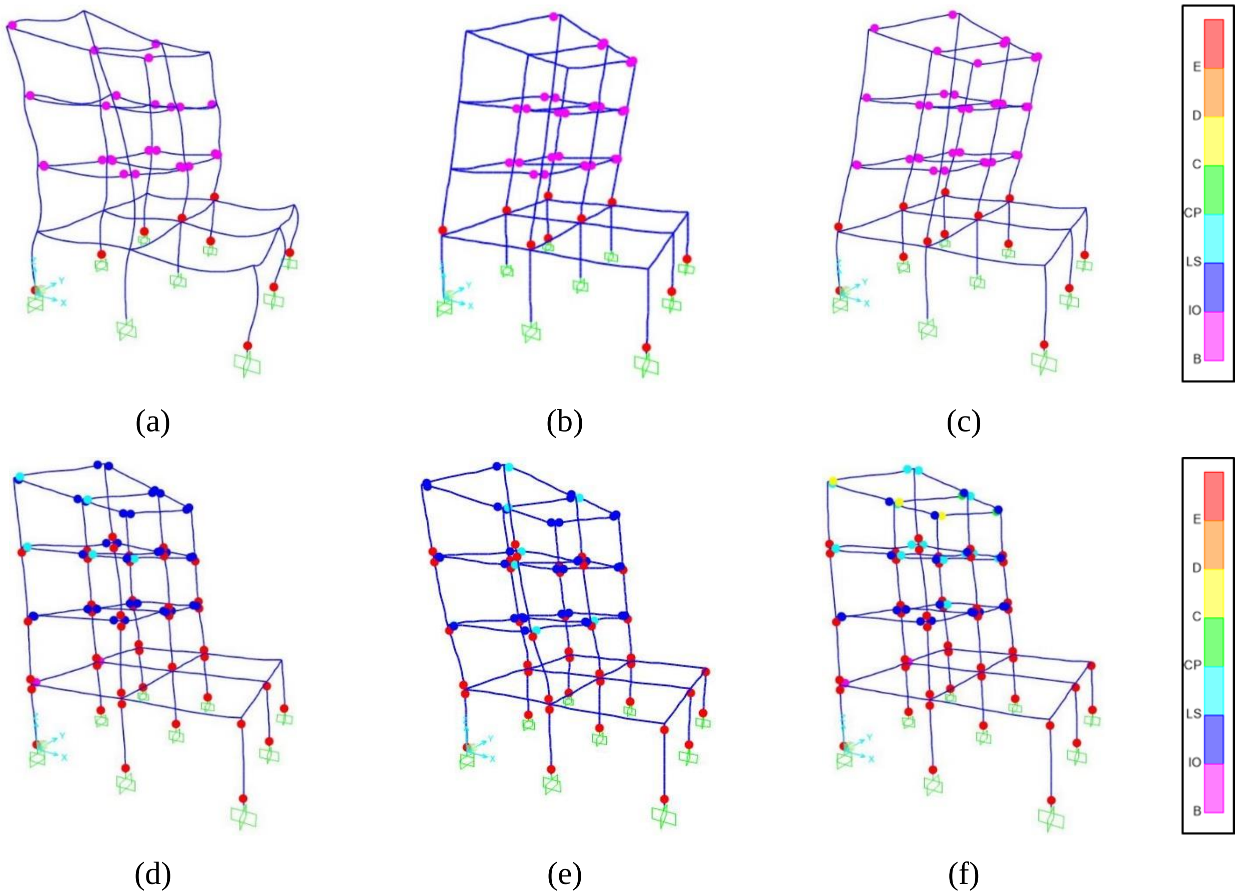


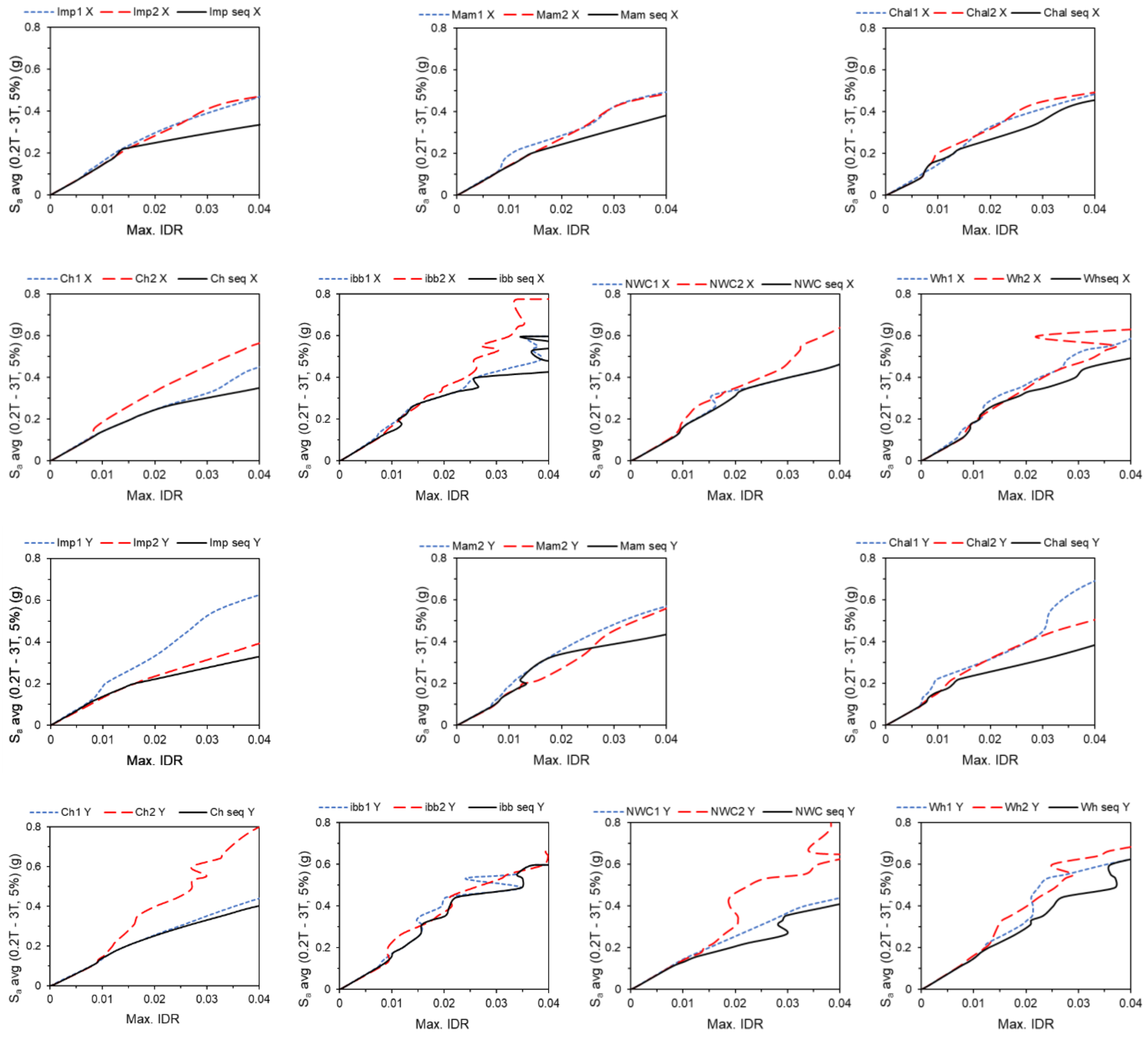
Fig. 5.9 Hinges states for structural model B-B

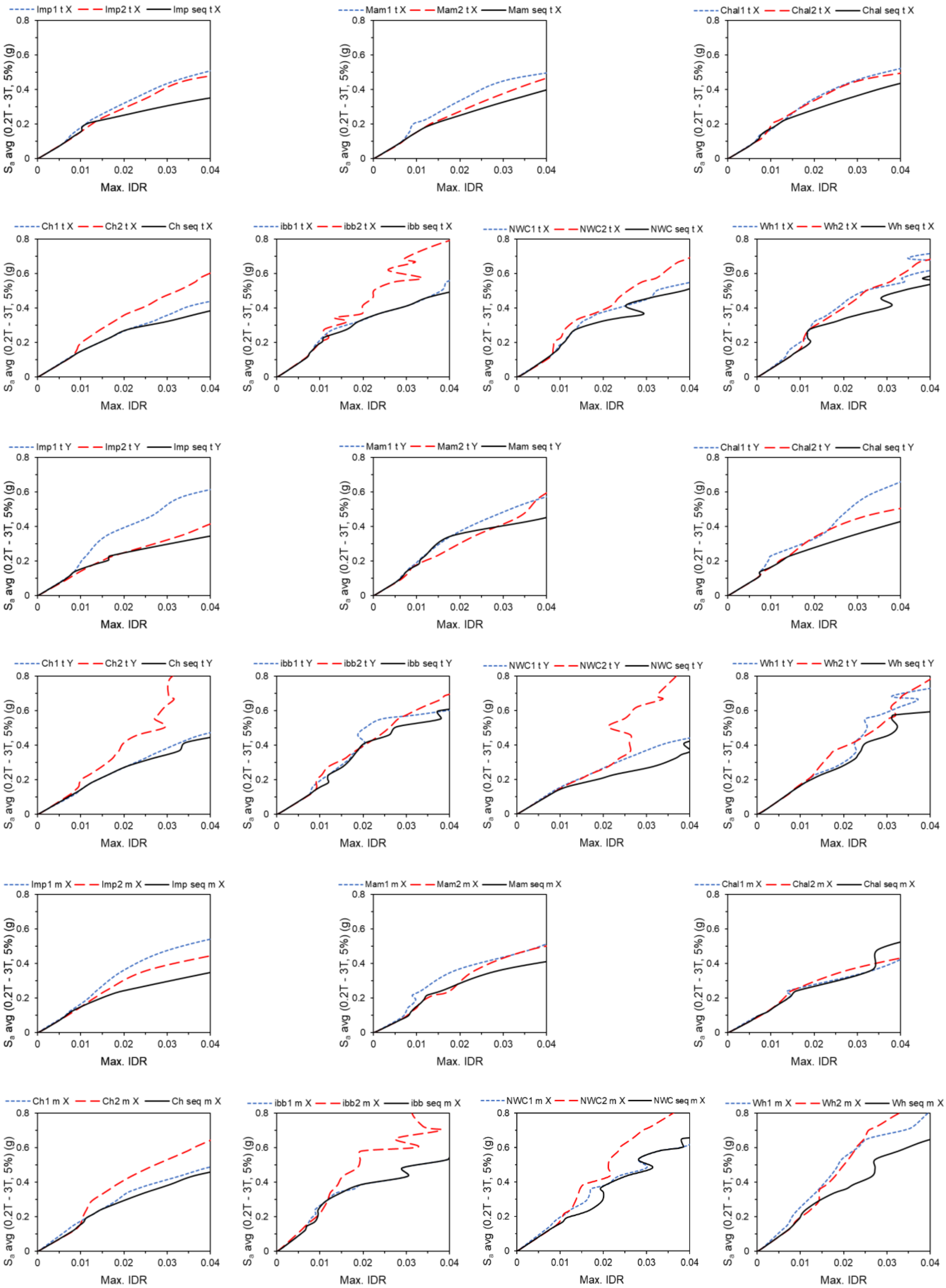
The hinge pattern in RC building frame models clearly envisage the progression of local structural damages occurred in the buildings in the form of plastic hinge severity for the considered earthquake forces. However, effect of irregular configuration can also be seen to a certain extent in the formation of hinge patterns. This phenomenon advocates the local vulnerability of structural models in resisting the sequential earthquakes compared to isolated single earthquakes

5.4.1.4 Structural dynamic capacity curves

The maximum Inter-story Drift Ratio happens to be the most commonly used engineering demand parameter to develop dynamic capacity curves and fragility curves using the IDA method. The dynamic NL capacity curve is developed for all the four structural configurations using the IDA approach (with around 1900 NL simulations) for the chosen seven sequential bi-directional earthquake forces. The resulting capacity curve is presented in Fig. 5.10, which is plotted between S_a avg and maximum inter-story drift ratios (IDR) parameters. Structural collapse is characterized to be the most severe catastrophe when subjected to natural hazards. Hence, the collapse limit state characterized in terms of 4% IDR should be the default performance limit for evaluating general structures with importance factor 1(lowest importance factor). Hence, collapse prevention is used as performance level for evaluating the structural behaviour.

The results depicted in Fig. 5.10 below shows that the structural models reach collapse limit state at lower spectral acceleration (IM) under sequential earthquakes, unlike isolated individual earthquake forces. This can be attributed to the reduction in the capacity of the structural system due to the accumulation of damages under sequential earthquakes. Hence, it can be emphasized that sequential earthquake forces can be perceived as the worst-case force causing maximum damage rather than the strongest isolated individual earthquake forces.




Imp1 t X
Imp2 t X
Imp seq t X
Mam1 t X
Mam2 t X
Mam seq t X
Chal1 t X
Chal2 t X
Chal seq t X
Ch1 t X
Ch2 t X
Ch seq t X
ibb1 t X
ibb2 t X
ibb seq t X
NWC1 t X
NWC2 t X
NWC seq t X
Wh1 t X
Wh2 t X
Wh seq t X
Imp1 t Y
Imp2 t Y
Imp seq t Y
Mam1 t Y
Mam2 t Y
Mam seq t Y
Chal1 t Y
Chal2 t Y
Chal seq t Y
Ch1 t Y
Ch2 t Y
Ch seq t Y
ibb1 t Y
ibb2 t Y
ibb seq t Y
NWC1 t Y
NWC2 t Y
NWC seq t Y
Wh1 t Y
Wh2 t Y
Wh seq t Y
Imp1 m X
Imp2 m X
Imp seq m X
Mam1 m X
Mam2 m X
Mam seq m X
Chal1 m X
Chal2 m X
Chal seq m X
Ch1 m X
Ch2 m X
Ch seq m X
ibb1 m X
ibb2 m X
ibb seq m X
NWC1 m X
NWC2 m X
NWC seq m X
Wh1 m X
Wh2 m X
Wh seq m X
S_a avg (0.2T - 3T, 5%) (g)
Max. IDR

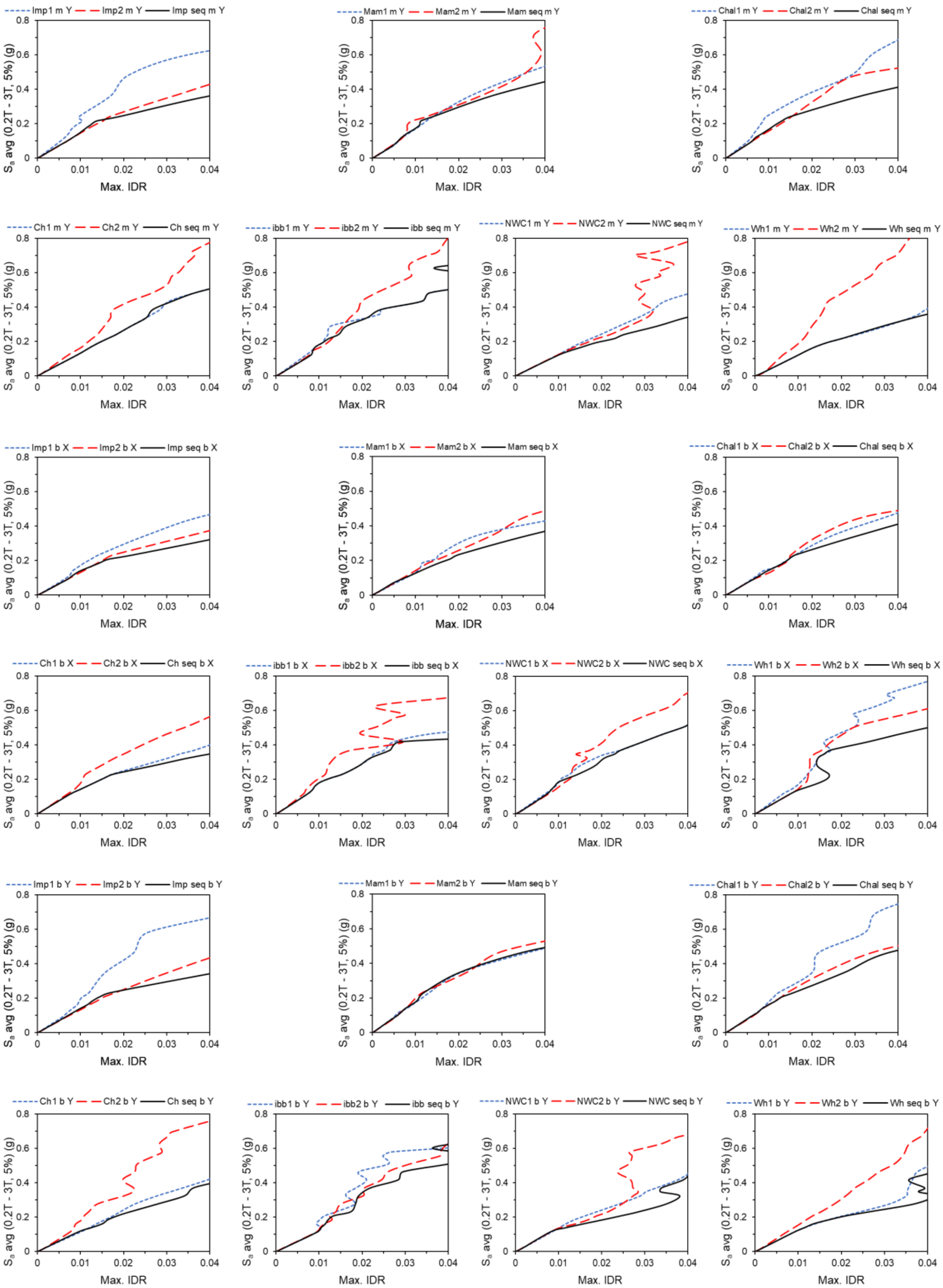


Fig. 5.10 Dynamic capacity curves of all the structural models for seven seismic sequences

5.4.1.5 Structural Fragility Estimation

Development of fragility curve for the chosen limit state envisages the probability of exceedance of that limit state with respect to IM. The probability of collapse for a particular intensity measure (S_a avg) under sequential and Isolated earthquakes are depicted in Fig. 5.11.

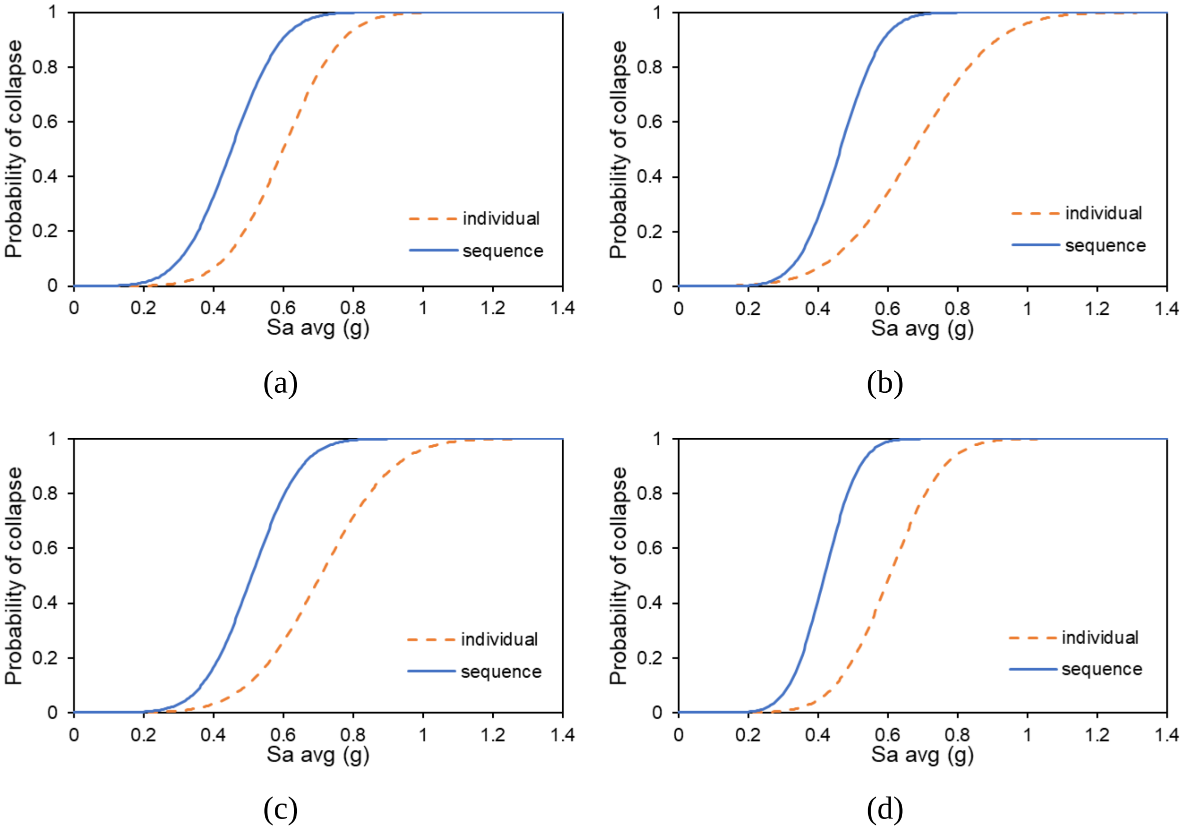


Fig. 5.11 Fragility curves at collapse-state for different structural models: (a). B-R, (b). B-T, (c). B-M, (d). B-B, for worst individual earthquake and sequential earthquake

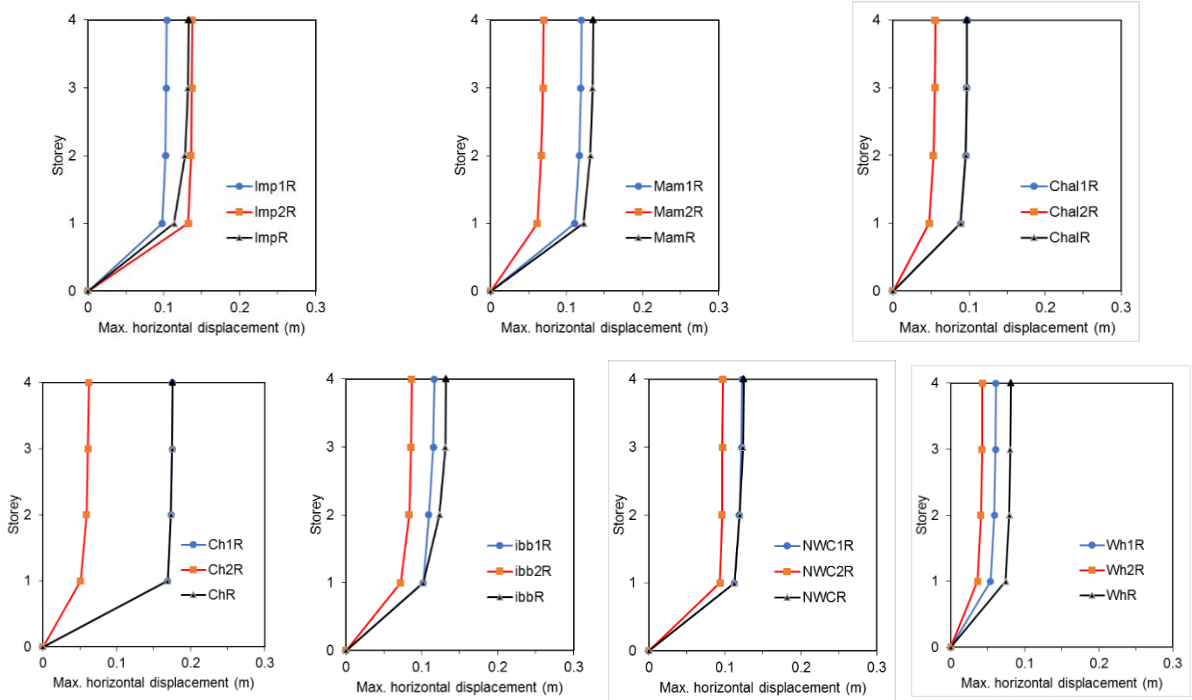
It can be concluded from the results that RC MRFs subjected to sequential forces have a high probability of collapse at lower IM unlike subjected to individual earthquake forces. This clearly advocates the decrease in the capacity of RC MRFs in resisting subsequent earthquakes after getting damaged due to the first earthquake. Hence, this investigation portrays the deficiency of new and existing RC building frames designed for isolated worst-case earthquake scenario in facing sequential earthquake forces.

5.4.2 Seismic behaviour of RC infilled frame models

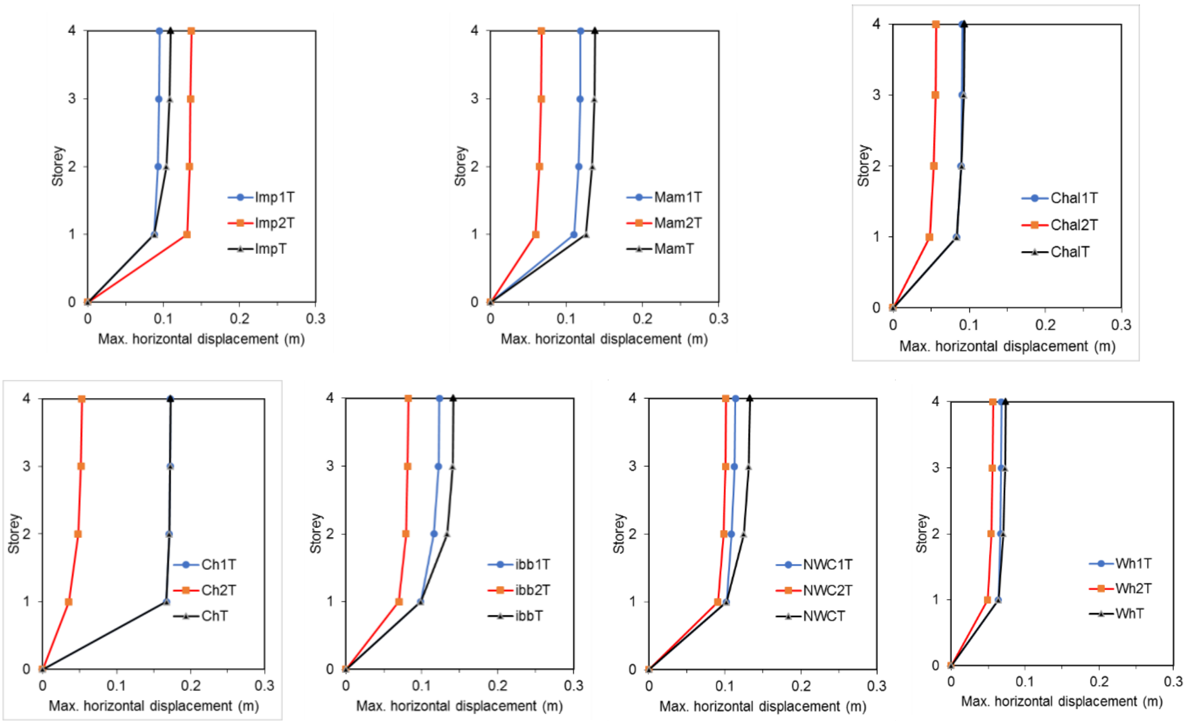
The interaction of infill frame with RC MRF is modelled in accordance with regulations specified in IS 1893 (Part 1) 2016 for all the structural models discussed in 5.4.1 and described in detail in Chapter 3. Around 2000 simulations of non-linear dynamic analyses (IDA) are carried out on RC infill frame models under sequential earthquakes. These are represented in the form of different response parameters and compared with corresponding behaviour under isolated earthquake forces, which are a part of sequential forces to visualize the effect of sequential repeated events.

5.4.2.1 Maximum lateral story displacements

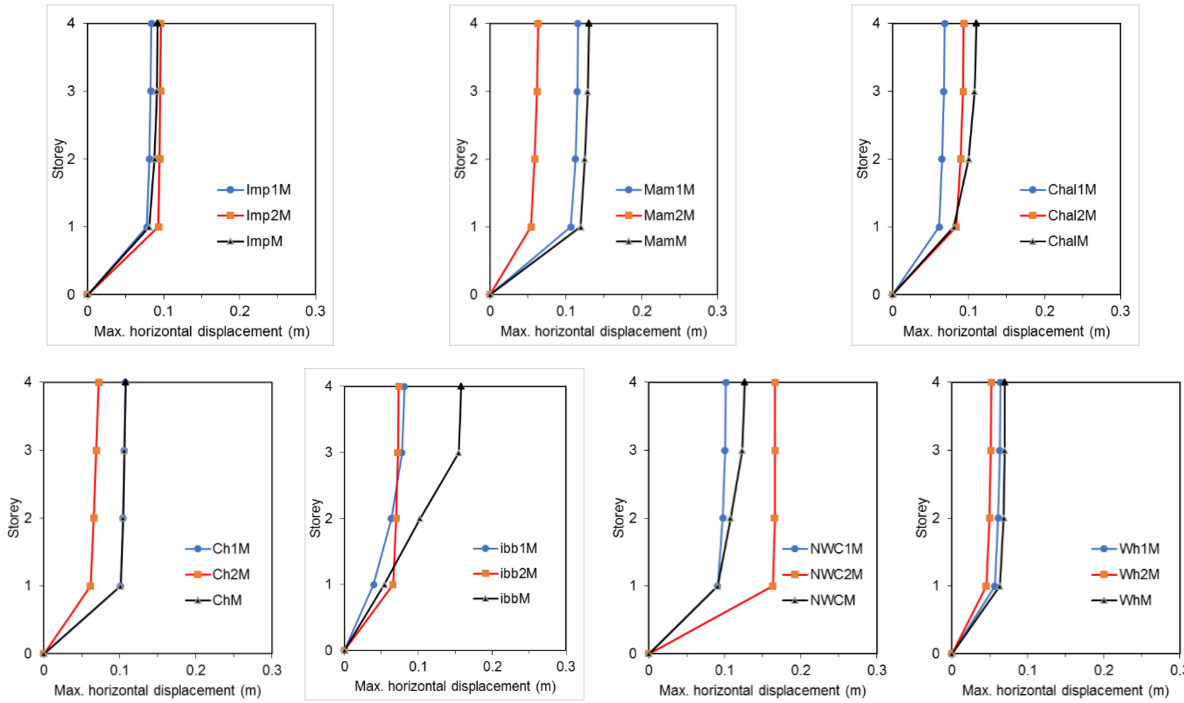
In order to understand the effect of seismic sequences, the maximum lateral story displacements extracted for all the structural models are compared for both individual and corresponding repeated ground motions at a particular intensity measure as depicted in Fig. 5.12. The average increase in roof displacements in the case of seismic sequences was found to be of the order of 53%, 32%, 57% and 59% for the I-R, I-T, I-M, and I-B configurations respectively, in comparison with individual ground motions. Further, the influence of OGS also can be clearly envisaged from the results depicted in Fig. 5.12.



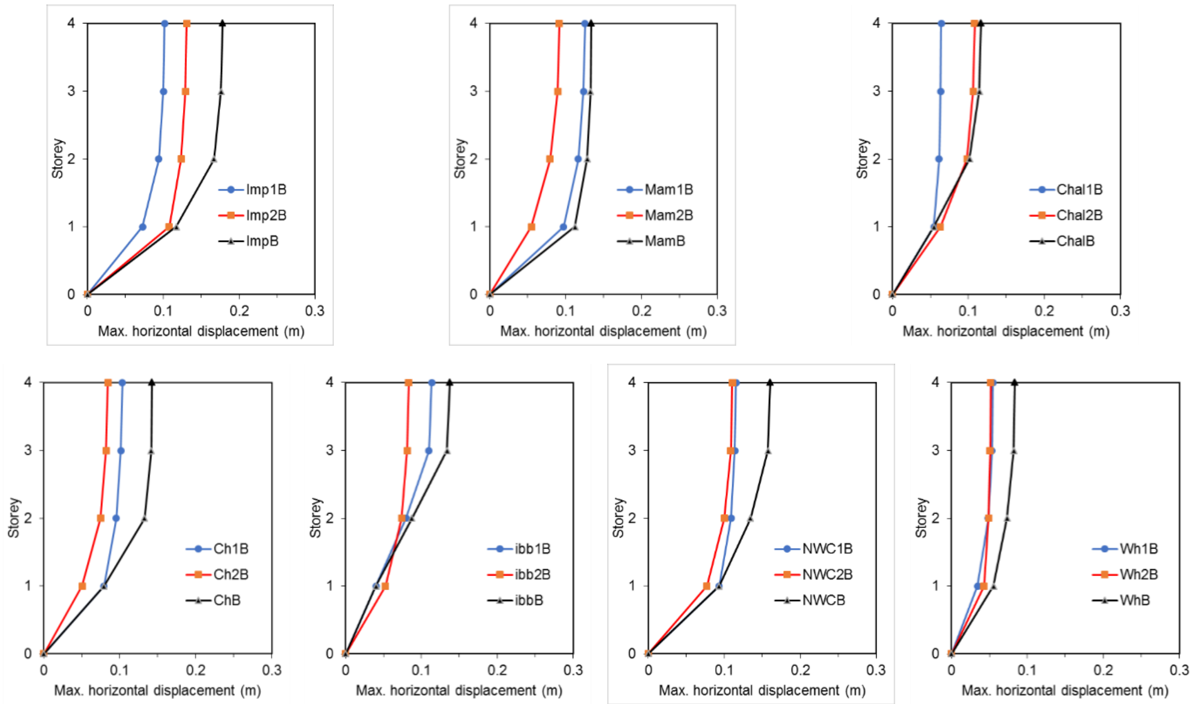
(a)


Storey
Max. horizontal displacement (m)
Imp1T
Imp2T
ImpT
Mam1T
Mam2T
MamT
Chal1T
Chal2T
ChalT
Ch1T
Ch2T
ChT
ibb1T
ibb2T
ibbT
NWC1T
NWC2T
NWCT
Wh1T
Wh2T
WhT

(b)


Storey
Max. horizontal displacement (m)
Imp1M
Imp2M
ImpM
Mam1M
Mam2M
MamM
Chal1M
Chal2M
ChalM
Ch1M
Ch2M
ChM
ibb1M
ibb2M
ibbM
NWC1M
NWC2M
NWCM
Wh1M
Wh2M
WhM

(c)

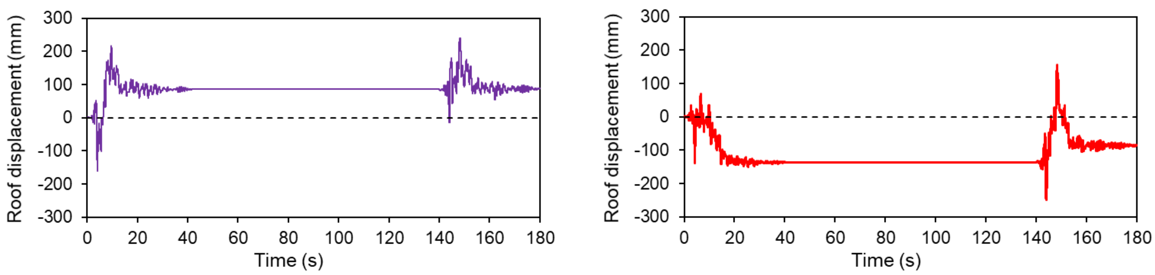


(d)

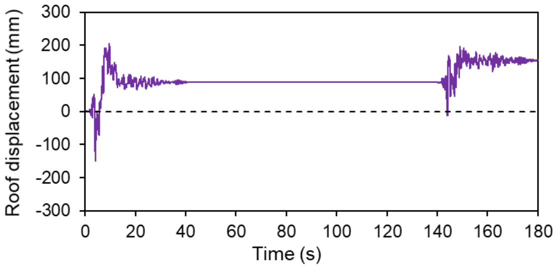
Fig. 5.12 Maximum lateral storey displacements of different building configurations: (a). I-R, (b). I-T, (c). I-M, (d). I-B for seven seismic sequences

5.4.2.2 Permanent structural damage (Residual displacements)

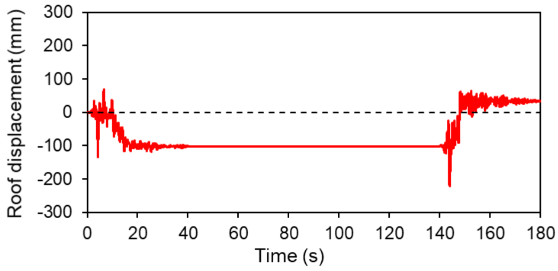
The structural response for seismic sequence #3 (i.e., Chalfant Valley) and sequence #4 (i.e., Chamoli) at a particular spectral acceleration value for the considered configurations are described in Figs. 5.13-5.14. Further, it can be observed in this study that the accumulation of damages for the repeated earthquake forces, after the first earthquake event were found to be significant along both directions.



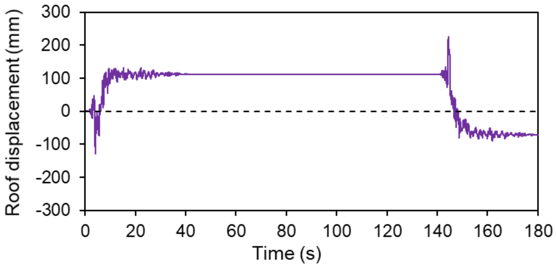
(a) (b)



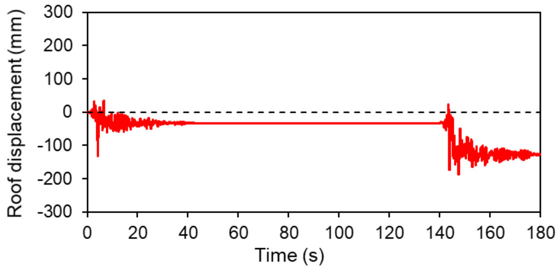
(c)



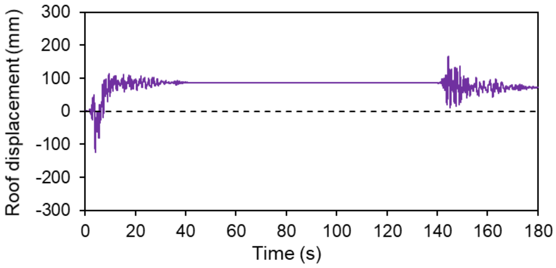
(d)



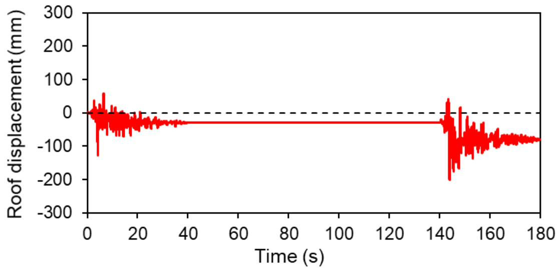
(e)



(f)

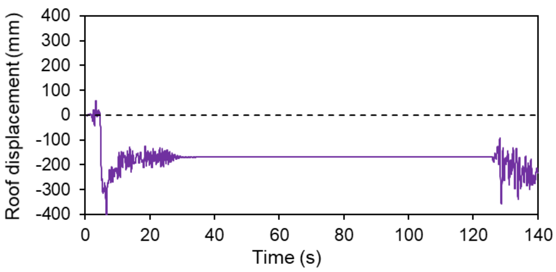


(g)

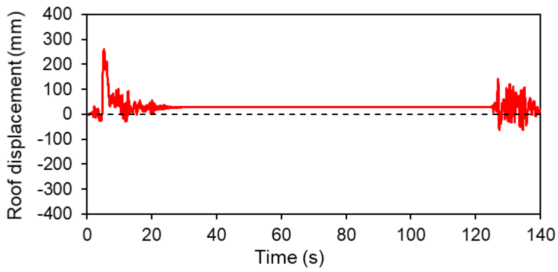


(h)

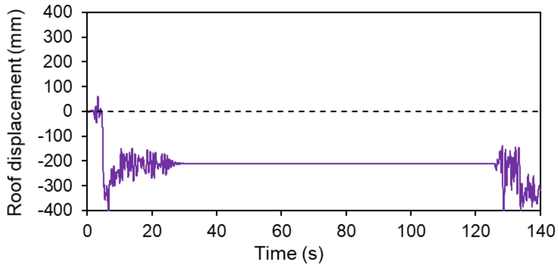
Fig. 5.13 Roof displacements of different building configurations for seismic sequence: Chalfant Valley: (a). I-R – X direction (b). I-R – Y direction (c). I-T – X direction (d). I-T – Y direction (e). I-M – X direction (f). I-M – Y direction (g). I-T – X direction (h). I-T – Y direction



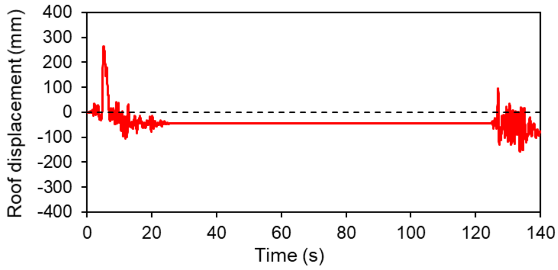
(a)



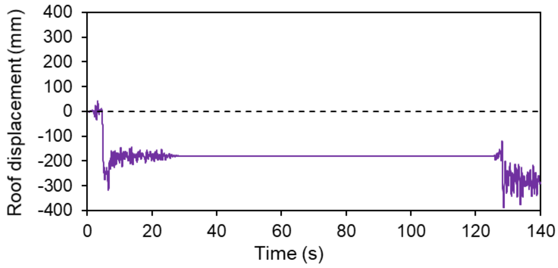
(b)



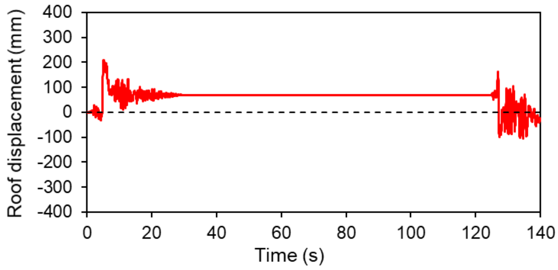
(c)



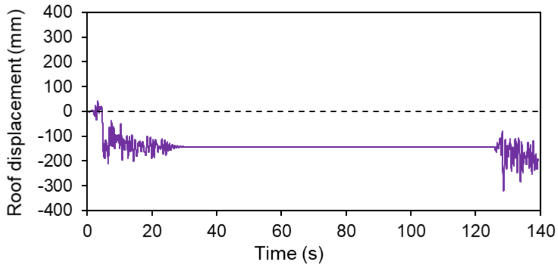
(d)



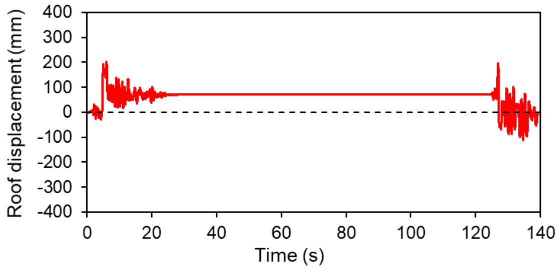
(e)



(f)



(g)



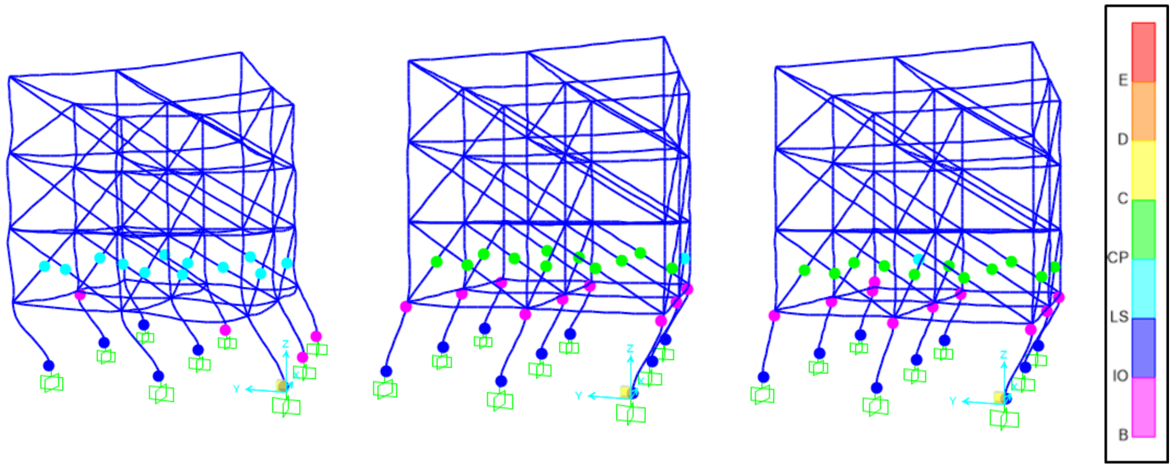
(h)

Fig. 5.14 Roof displacements of different building configurations for seismic sequence: Chamoli: (a). I-R – X direction (b). I-R – Y direction (c). I-T – X direction (d). I-T – Y direction (e). I-M – X direction (f). I-M – Y direction (g). I-T – X direction (h). I-T – Y direction

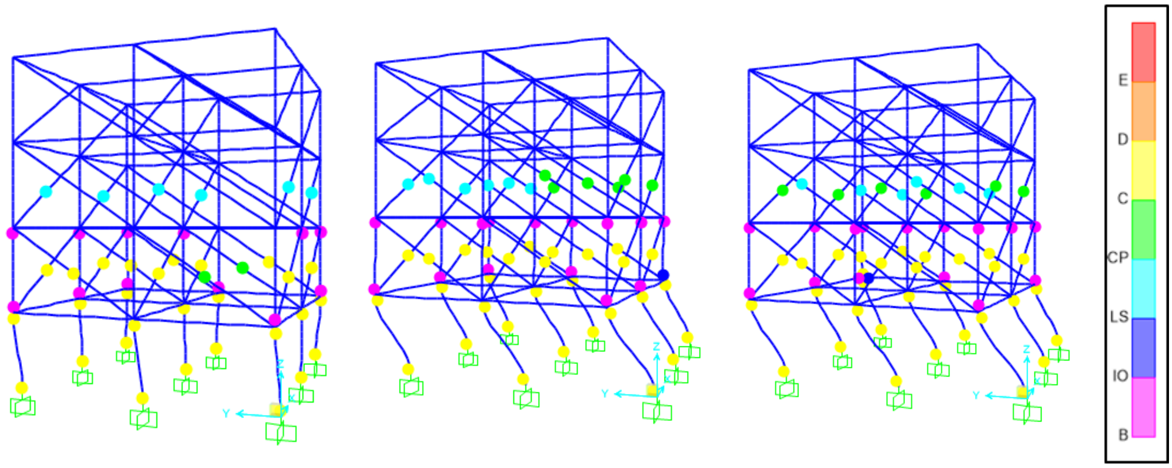
The quantity of residual displacement indicates the weakness of RC building models in resisting repeated earthquake events described in terms of displacement response. Further, this phenomenon can be also envisaged in the fragility curves developed as shown in Fig. 5.20.

5.4.2.3 Local Structural Damage (Hinge patterns)

The severity of plastic hinge pattern for the structural models under seismic forces – Chal1 (a & d), Chal2 (b & e) and Chal (c & f) are depicted at yielding (a, b, c) and just before collapse state (d, e, f) in Figs. 5.15-5.18. The legend in these figures describes different damage states of plastic hinges (i.e., IO: immediate occupancy, LS: life safety, CP: collapse prevention) with appropriate labels and colours, as per ASCE 41 (ASCE, 2017).

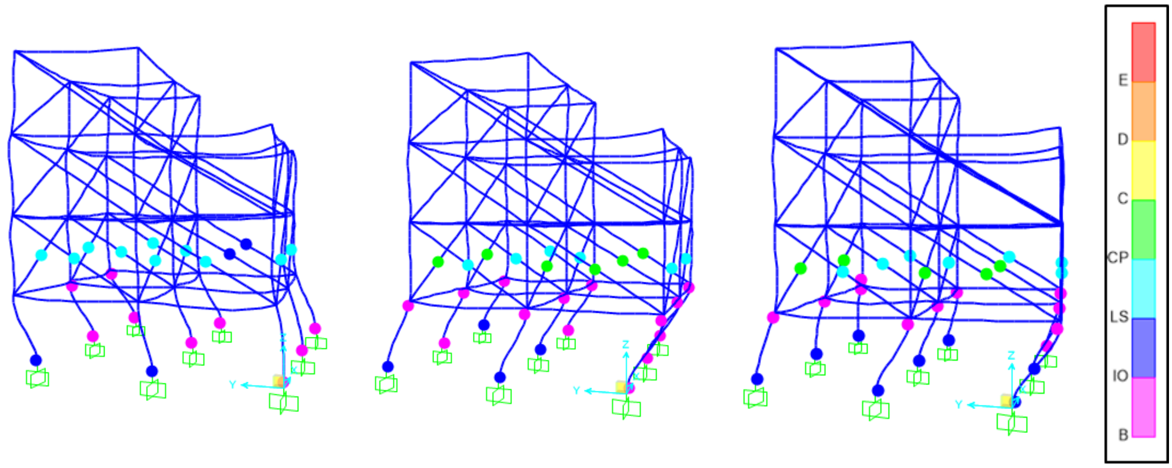


(a) (b) (c)

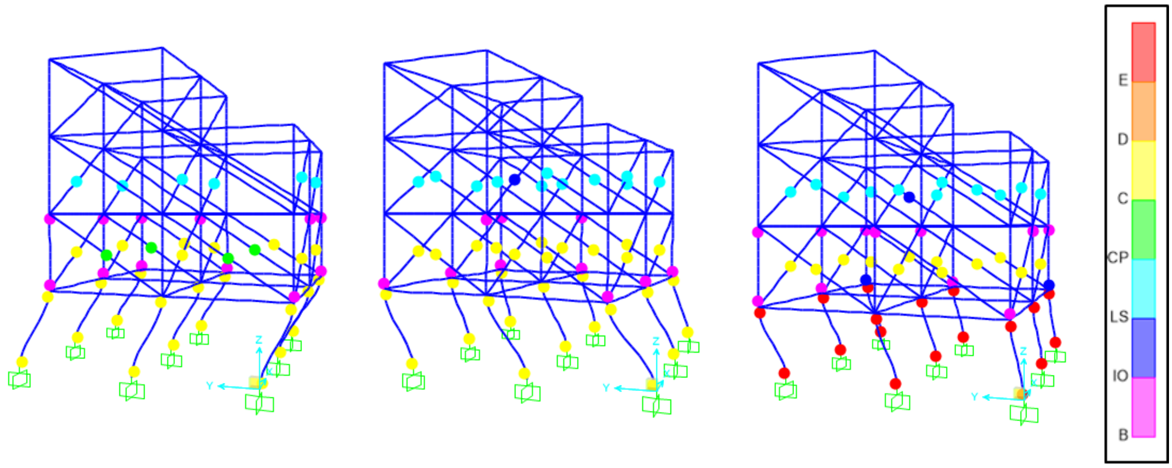


(d) (e) (f)

Fig. 5.15 Hinges pattern for building configuration I-R

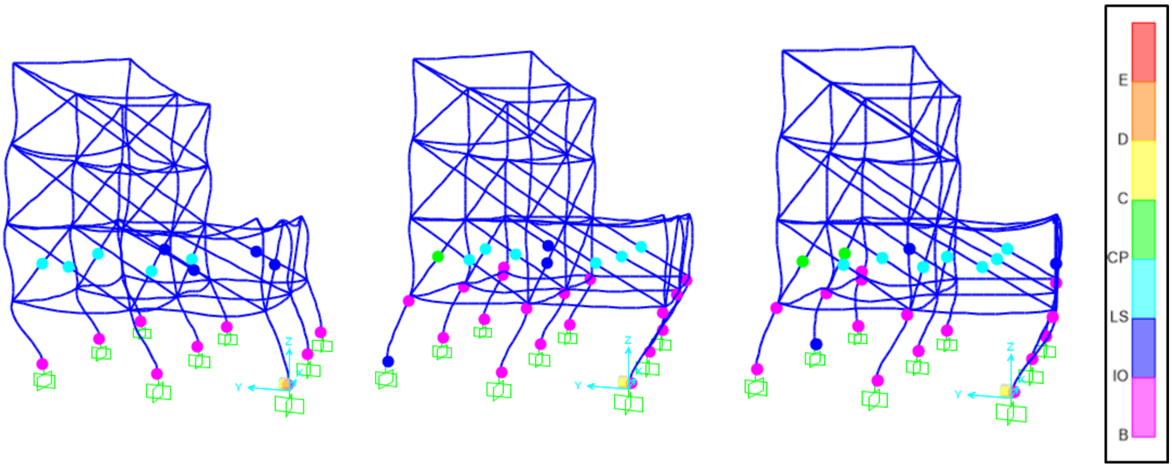


(a) (b) (c)

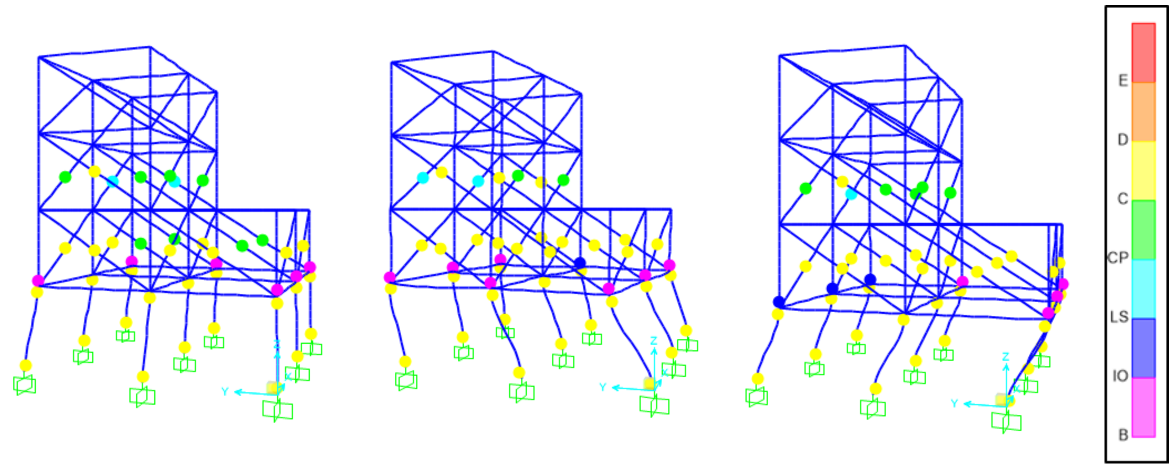


(d) (e) (f)

Fig. 5.16 Hinges pattern for building configuration I-T

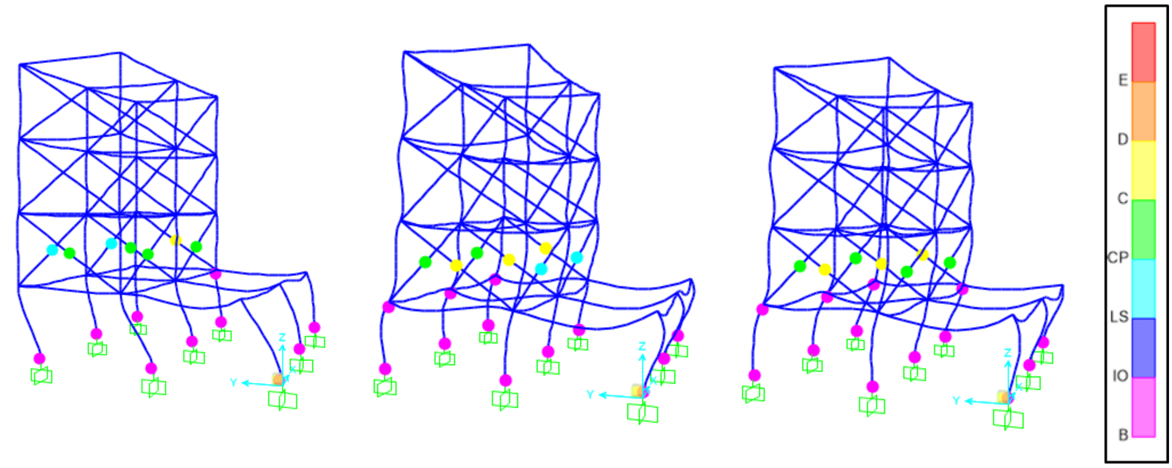


(a) (b) (c)

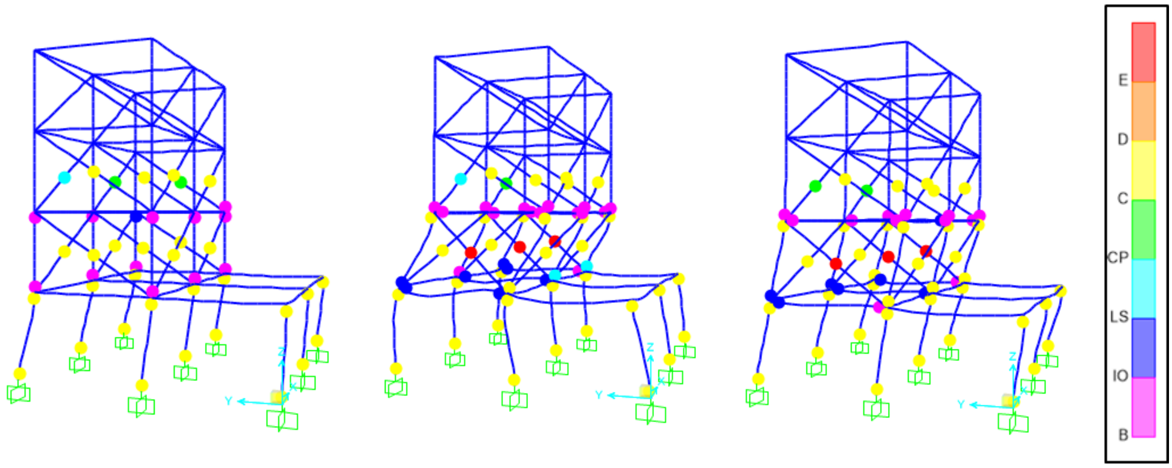


(d) (e) (f)

Fig. 5.17 Hinges pattern for building configuration I-M



(a) (b) (c)



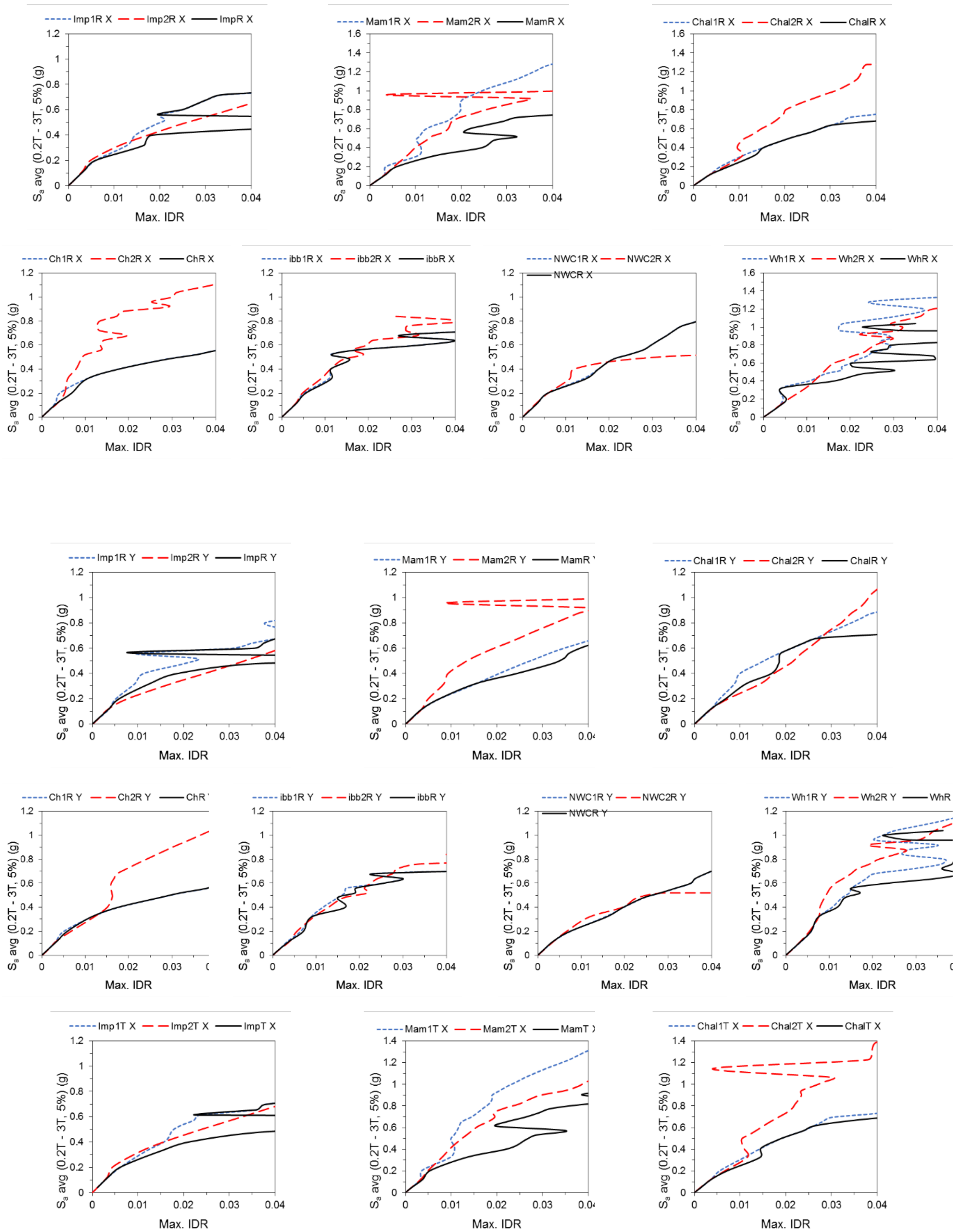
(d) (e) (f)

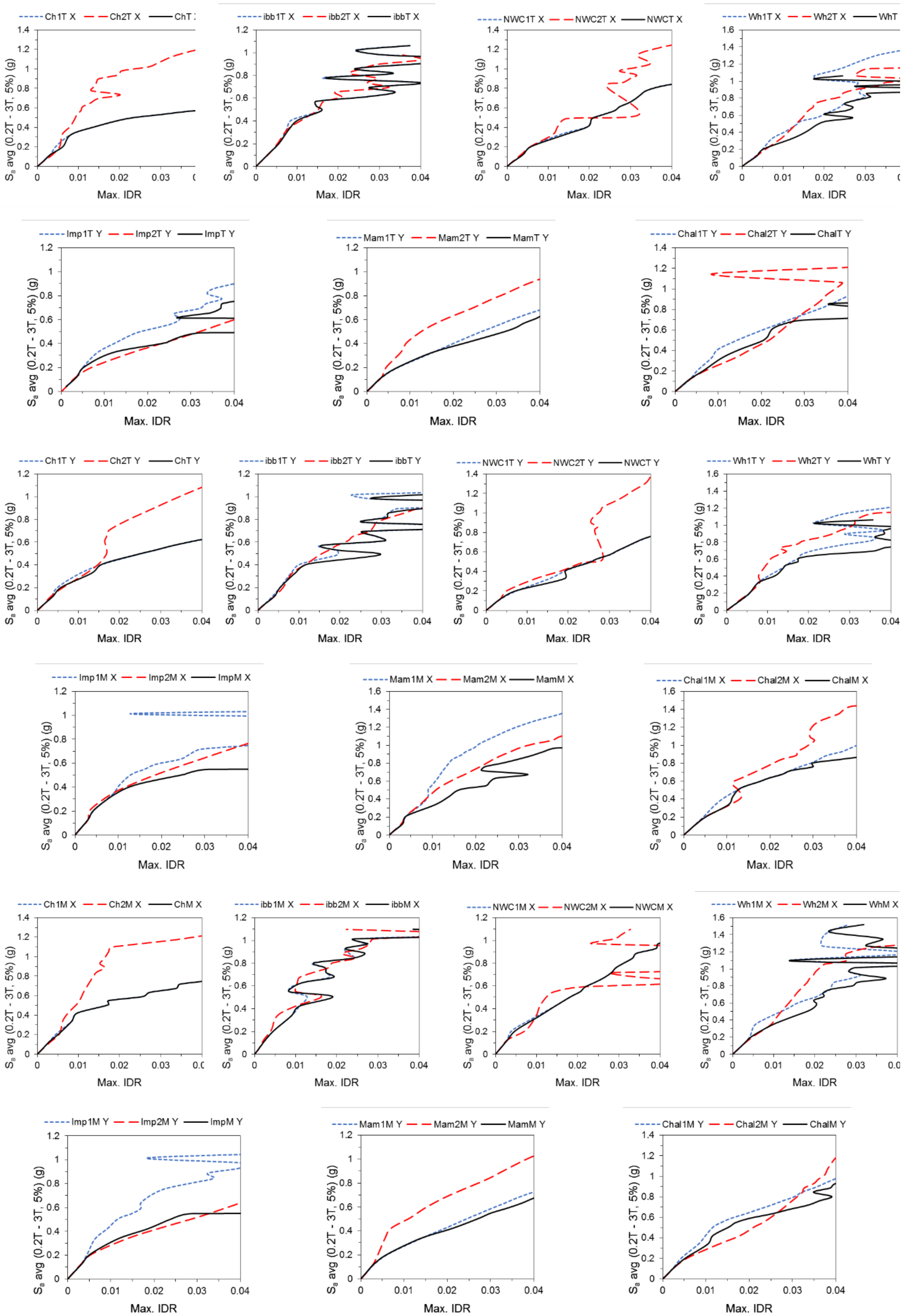
Fig. 5.18 Hinges pattern for building configuration I-B

In all building configurations, it can be observed that an increased number of structural components reached the plastic state when subjected to repeated earthquakes than isolated individual earthquakes. This pronounces the vulnerability of structural models in resisting the repeated earthquake events.

5.4.2.4 Structural dynamic capacity curves

The IDA curves are plotted between S_a avg and maximum inter-story drift ratios (IDR) for all the structural models considering the infill interaction with RC MRF are shown in Fig. 5.19. From these graphs, it is evident that building reaches collapse limit state (4% IDR in our study) at lower IM, i.e., at lower spectral acceleration value under repeated earthquake force compared to that of corresponding individual earthquake forces.




Ch1T X, Ch2T X, ChT X
ibb1T X, ibb2T X, ibbT X
NWC1T X, NWC2T X, NWCT X
Wh1T X, Wh2T X, WhT
Imp1T Y, Imp2T Y, ImpT Y
Mam1T Y, Mam2T Y, MamT Y
Chal1T Y, Chal2T Y, ChalT Y
Ch1T Y, Ch2T Y, ChT Y
ibb1T Y, ibb2T Y, ibbT Y
NWC1T Y, NWC2T Y, NWCT Y
Wh1T Y, Wh2T Y, WhT Y
Imp1M X, Imp2M X, ImpM X
Mam1M X, Mam2M X, MamM X
Chal1M X, Chal2M X, ChalM X
Ch1M X, Ch2M X, ChM X
ibb1M X, ibb2M X, ibbM X
NWC1M X, NWC2M X, NWCM X
Wh1M X, Wh2M X, WhM X
Imp1M Y, Imp2M Y, ImpM Y
Mam1M Y, Mam2M Y, MamM Y
Chal1M Y, Chal2M Y, ChalM Y
Sa avg (0.2T - 3T, 5%) (g)
Max. IDR

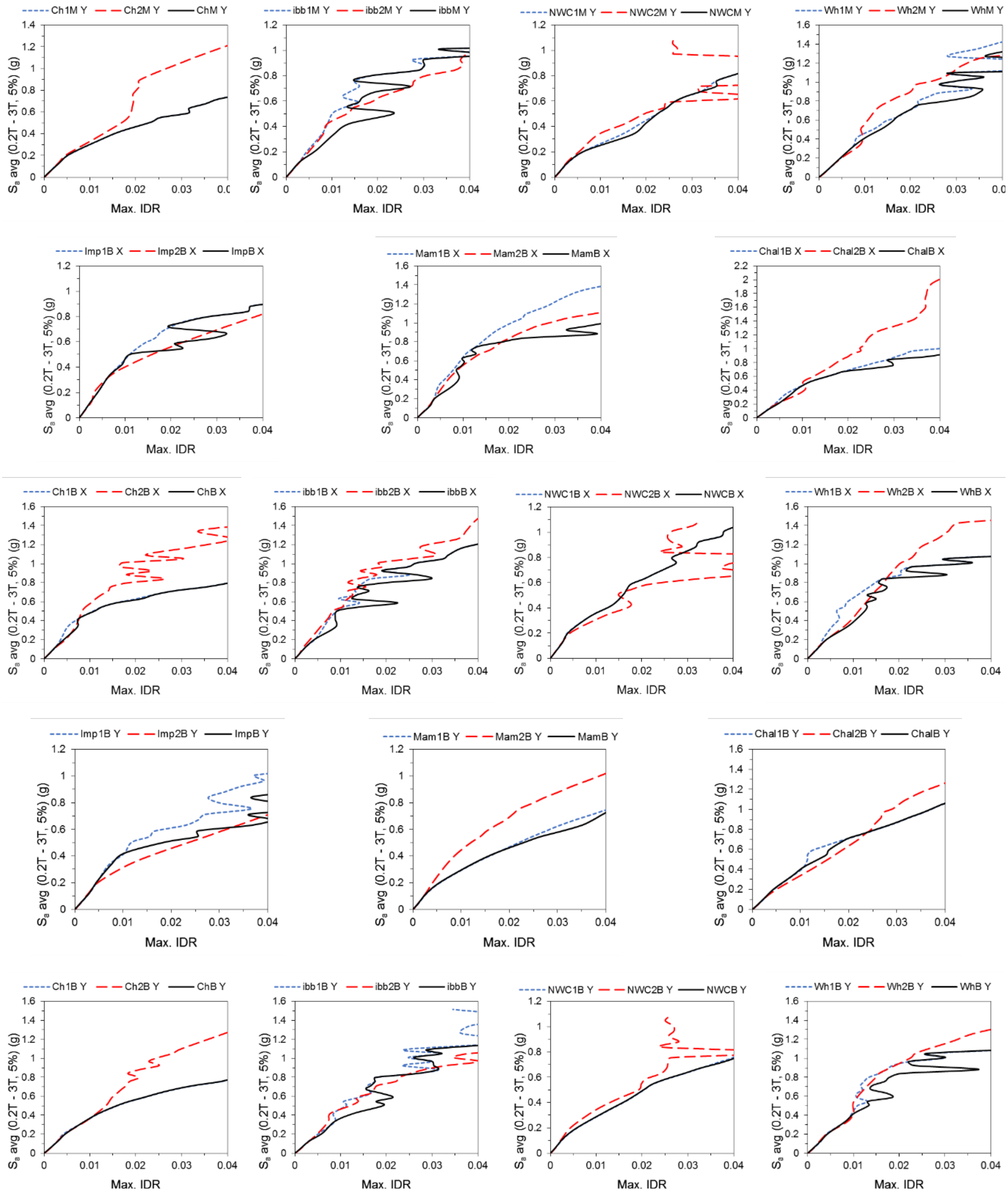


Fig. 5.19 IDA curves of all building configurations for seven seismic sequences

This observed behaviour clearly envisages the accumulation of damages due to repeated earthquake forces on the building. Also, it can be observed that consideration of repeated earthquake force happens to be a worst-case scenario than isolated individual earthquakes.

5.4.2.5 Structural fragility estimation

The plot of the probability of collapse for a given intensity measure (S_a avg) is shown in Fig. 5.20. It can be observed from the analysis that the probability of collapse occurs at much lower spectral acceleration under repeated earthquake forces compared to single or individual earthquake force. This is attributed to the considerable reduction in the capacity of the buildings while facing a second or subsequent earthquake after getting damaged when subjected to the first one.

The behaviour observed pronounces the weakness of most of the existing and new buildings designed as per the current seismic provisions, considering only one isolated earthquake force during design phase. In view of fore mentioned observations, it is imperative to consider repeated earthquake forces in modelling and analysis of earthquake resistant design of structures to achieve seismic structural resilience of buildings

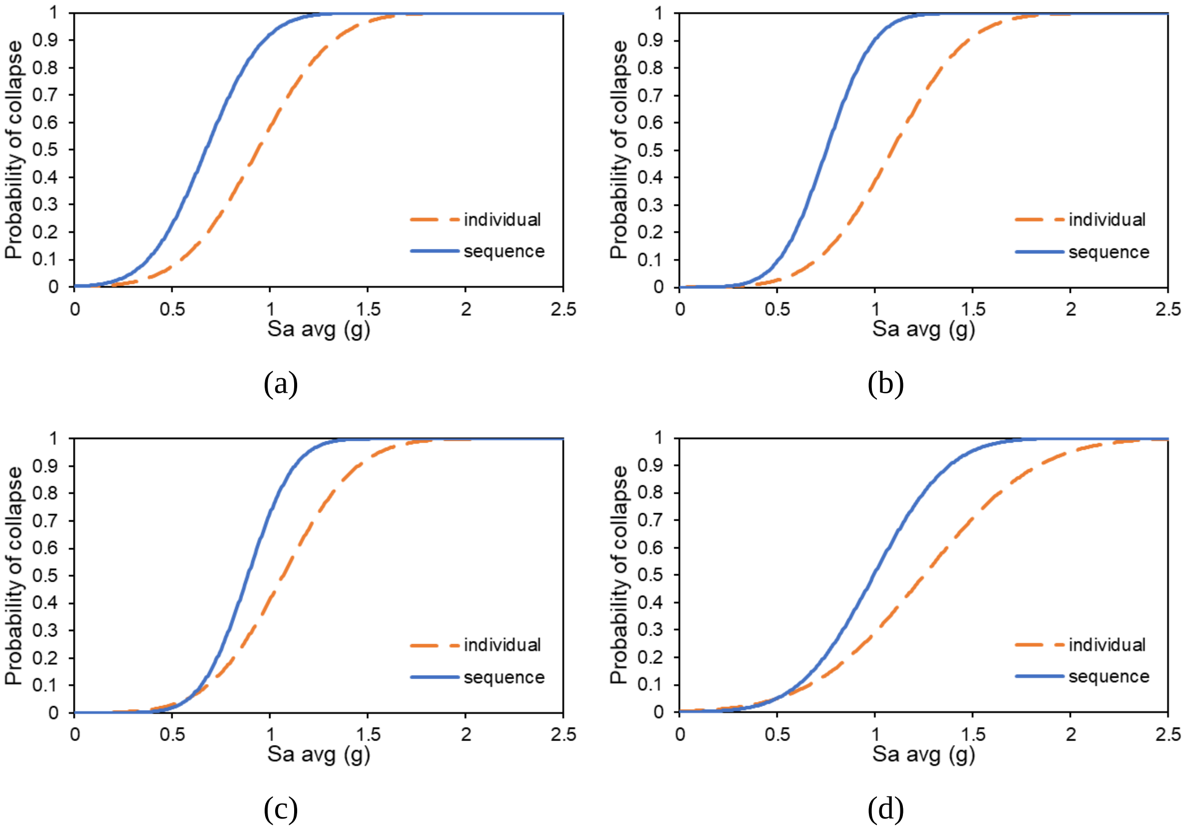


Fig. 5.20 Collapse fragility curves of different building configurations: (a). I-R, (b). I-T, (c). I-M, (d). I-B, for worst individual scenario and seismic sequence

5.5 Summary

The present study mainly focused on the seismic performance of 3D RC building frames of medium-rise configuration as discussed in Chapter 4 with and without vertical irregularities (setbacks) under repeated earthquakes. In this investigation, IDA is performed for bidirectional repeated earthquake sequence loading to investigate the structural behaviour analogous to that of isolated earthquake force, (for regular and vertical setback models) in terms of several response parameters such as maximum horizontal displacement vs. story level, residual displacements with respect to repeated ground motion, variation of maximum inter-story drift ratio (IDR) with respect to S_a avg, and finally the probability of collapse in terms of S_a avg. From the results presented in section 5.4, the following can be observed:

- The influence of repeated earthquake forces is clearly pronounced on the vulnerability characteristics of the building structures designed as per the seismic provisions of Indian code.
- Story displacements along lateral direction characterized in terms of maximum horizontal displacements significantly increase under sequential earthquakes compared to that of isolated individual earthquakes.
- Residual displacements computed under sequential earthquakes advocate the permanent damage experienced by RC building frames.
- The plastic hinge formulation during NLD analysis just before the collapse limit state advocates the vulnerability of RC building frames in resisting the sequential earthquakes after getting damaged due to the first earthquake.
- Structural capacity described in terms of maximum IDR envisages the weakness of RC building frames under sequential forces. Here the structural models considered were reaching the collapse state at lower spectral acceleration value for sequential earthquakes than that of the individual earthquakes.
- The fragility curves developed from the non-linear capacity curves also reinforce the weakness of RC building frames in facing the sequential earthquakes.
- Considerable reduction in the capacity of the buildings is observed while facing a second or subsequent earthquake, after getting damaged due to first one. This clearly signifies the weakness of most of the existing and new buildings designed as per the seismic provisions considering only one earthquake force for the design.

Hence, from this investigation, it can be concluded that there is an imperative need for consideration of repeated earthquake forces during the analysis / design phase itself for both new as well as existing buildings in order to arrive at a seismic resilient structure. Consideration of interaction of infill with RC MRF is necessary for arriving at more appropriate structural behaviour. This is in general ignored during structural analysis to arrive at design configurations. This investigation reveals consideration of interaction of infill has substantial impact on the response characteristics under seismic events, hence needs consideration.

Hence, the sequential earthquake force was identified to be the destructive force which is to be considered during the design of building structures as per the seismic provisions of Indian code for various hazard levels. In order to convert this destructive earthquake force to seismic design force for incorporating in the conventional design procedure, an attempt was made to look into the seismic design parameters which influence the calculation of seismic design force. Among all seismic design parameters required for estimation of design force, only independent parameter which links the elastic and inelastic response is the Response reduction factor (R).

CHAPTER 6

Importance of Response reduction factor (R) in seismic behaviour Assessment of RC buildings

6.1 General

This chapter presents the importance of response reduction factor (R) factor in seismic analysis and design of RC buildings and its analytical estimation using nonlinear analysis (NLA) procedures for a given RC building design. The estimated R value is then compared with respective code specified R value for a comprehensive understanding of seismic capacity of selected RC building frame. In view of this, certain medium-rise 3D RC MRFs are selected and are subjected to simultaneous bi-directional earthquake ground motions using NLA procedures as discussed in chapter 4 and 5. The outcome of these analysis is development of inelastic structural capacity curve, which aids in subsequent assessment of sufficiency of code specified R value and its influence on seismic behaviour

6.2 Response reduction factor (R)

Response reduction factor/Response modification factor/Behaviour factor is generally designated as ‘R’ in most of the existing seismic codes of practice around the world. Since, most of the existing seismic design codes all over the world adopt a force-based design approach, the non-linear inelastic response of the RC moment-resisting frames (MRFs) is accounted using an implicit representation of constant scale factor referred to as response reduction factor (R) in a linear elastic design. This R value is specified to account for non-linear behaviour and deformation characteristics in a linear elastic design. Therefore, the elastic forces are reduced by a response reduction factor under DBE hazard level to arrive at design base shear for the given earthquake at the chosen location. However, it has been observed that the ‘R’ specified by most of the design codes (in particular the IS 1893) does not address the changes in structural configurations of RC MRFs (viz., building height, number of bays present, bay width, irregularities arising out of mass and stiffness changes, etc.). These changes in structural configuration for a chosen structural type results in changed dynamic characteristics of the structural system. Therefore, understanding the influence of ‘R’ on inelastic capacity of a structural type helps in more accurate assessment of its seismic behaviour. The response of building frames to the earthquake forces depends on various factors that influence the design (viz., ductility, over strength factor, damping etc. leading to estimation of response reduction factor). Hence, in order to ensure minimum stability of these building frames to remain functional during repeated earthquake forces, the accurate estimation and influence of response reduction factor (R) also need to be considered in the design calculations.

As the structure moves into inelastic phase due to applied lateral forces, the structural lateral force resisting elements should be designed to absorb and resist without collapse the seismic energy emanated due to earthquakes. Therefore, the existing seismic provisions, arrive at the design lateral force for any structural type, mainly to make use of inherent inelastic capacity prevalent in a given structural type. In this perspective, existing seismic codes specify a constant factor (behaviour factor, response modification factor or response reduction factor) to reduce the maximum lateral force to arrive at design lateral force. However, analytical estimation of this factor provides a qualitative understanding of inelastic response of a code-compliant building for a design earthquake (DBE). In addition, R value is defined to exploit the structures inherent over strength and ductility characteristics prevalent as per the given design to remain functional in accordance with seismic design philosophy. Hence, to arrive at design lateral force of a given structural type under DBE at a chosen location the following expressions specified by IS 1893 is considered:

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \tag{6.1}$$

$$V_B = A_h \cdot W \tag{6.2}$$

Here, A_h: Design horizontal seismic coefficient, Z: Zone factor, I: Importance factor, R: Response reduction factor, Sa/g: Average response acceleration coefficients for corresponding soil types, W: Seismic weight of the structure, V_B: Design seismic base shear.

6.3 Analytical Estimation of Response reduction factor (R)

The analytical estimation of response reduction factor (R) can be expressed as a function of various parameters of the structural system, such as strength, ductility, damping and redundancy as per Whittaker et al. (1999) shown below.

$$R = R_S \cdot R_\mu \cdot R_R \cdot R_\xi \quad (6.3)$$

(a) Overstrength factor (R_S): It is a measure of the reserve strength in the structure. It is developed because the maximum lateral strength of a structure always exceeds its design strength.

(b) Ductility factor (R_μ): It is a measure of the deformation capacity of the structure.

(c) Redundancy factor (R_R): It can be assumed as unity following the ASCE7 guidelines.

$$R_R=1$$

(d) Damping factor (R_ξ): It balances the effect of supplementary viscous damping and is mainly applicable in the case of structures with additional energy dissipating devices. In the absence of such devices the damping factor is generally assumed as 1.0.

$$R_\xi=1$$

Therefore, computation of the response reduction factor is carried out as the product of over strength factor (R_s), ductility factor (R_μ), damping factor (R_ξ), and a redundancy factor (R_R) for any structural type. Since the structural models considered here (also discussed in-detail in chapters 4 & 5) do not have any damping energy dissipation devices, therefore, the damping factor is considered to be 1. Moreover, it has been widely reported that 'R' is invariant with the number of bays and spans of the bays in a building frame, therefore, the redundancy factor is also considered to be unity (ATC 19, 1995, ATC 34, 1995). Hence, the critical factors for the estimation of 'R' boil down to R_s and R_μ as depicted in Fig. 6.1. R_s is constant for a particular structural model at a chosen design level and does not vary with different loading scenarios unlike R_μ, which significantly changes under isolated and sequential earthquake forces. The NL capacity curves computed from NLS for all the structural models considered are depicted in Fig. 6.2. Moreover, the parameters necessary to be considered for estimation of R from Fig. 6.1 are - design base shear (V_d), yield base shear (V_y), roof displacement at yield point (Δ_y), maximum elastic base shear (V_e), displacement at elastic base shear (Δ_e), and maximum displacement (Δ_{max}). From these parameters, the overstrength factor (R_s) is defined as the ratio of the yield base shear (V_y) to the design base shear (V_d) of the frame as given by the Eq. (1). Similarly, R_μ is estimated using the relationship proposed by Newmark and Hall [50] shown in Eqs. (6.5-6.8).

$$R_s = \frac{V_y}{V_d} \tag{6.4}$$

$$R_\mu = 1 \quad \text{for } T < 0.2 \text{ s} \tag{6.5}$$

$$R_\mu = \sqrt{2\mu - 1} \quad \text{for } 0.2 \text{ s} < T < 0.5 \text{ s} \tag{6.6}$$

$$R_\mu = \mu \quad \text{for } T > 0.5 \text{ s} \tag{6.7}$$

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{6.8}$$

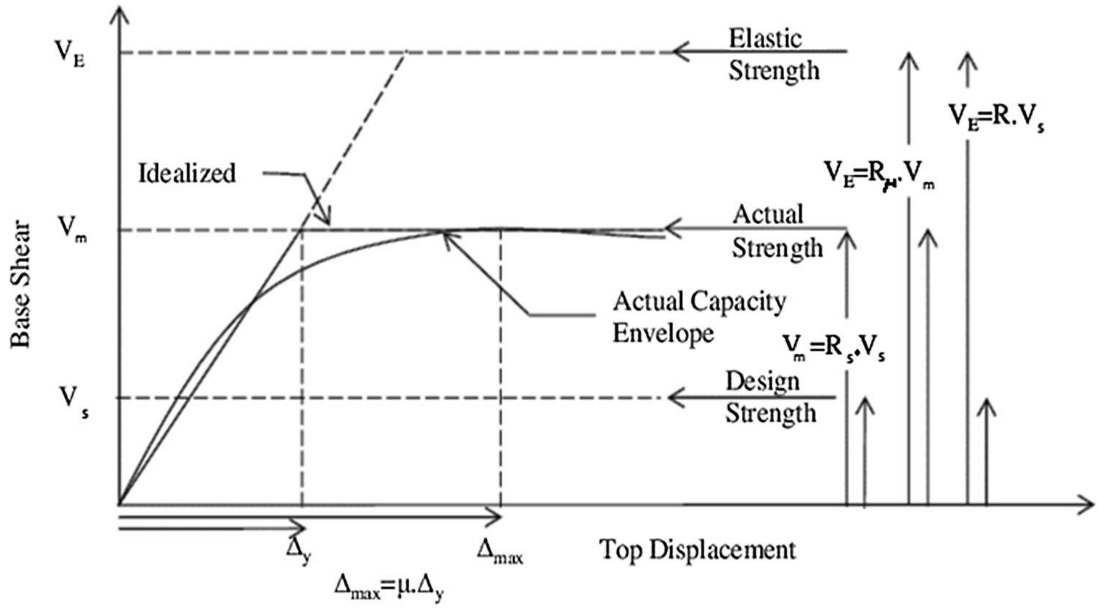


Fig. 6.1 A typical capacity curve of a structure

Finally, the computed values of 'R' for all the structural models utilising the Eqns. (1-6) are depicted in Tables 6.1-6.5.

6.3.1 R-factors for RC MRFs subjected to single earthquakes

R-factors are computed from the capacity curves obtained from non-linear static (NLS) and non-linear dynamic (NLD) analysis on all the structural models described in chapter 4 & 5. The pushover curves for all the eight structural models along both orthogonal directions are depicted in Fig. 6.2. Here, ultimate / failure displacement (Δ_{max}) of a building corresponds to the collapse state (i.e., a threshold of 4% max. inter-story displacement). This is considered in accordance with FEMA 273 regulations, as IS 1893 do not address the performance state of any designed RC MRF. Further, yield displacement of a building is extracted from the bilinear capacity curves generated for the structural models considered. Yield displacement (Δ_y) is considered at a point where the building deviates from linear elastic behaviour and enters plastic state.

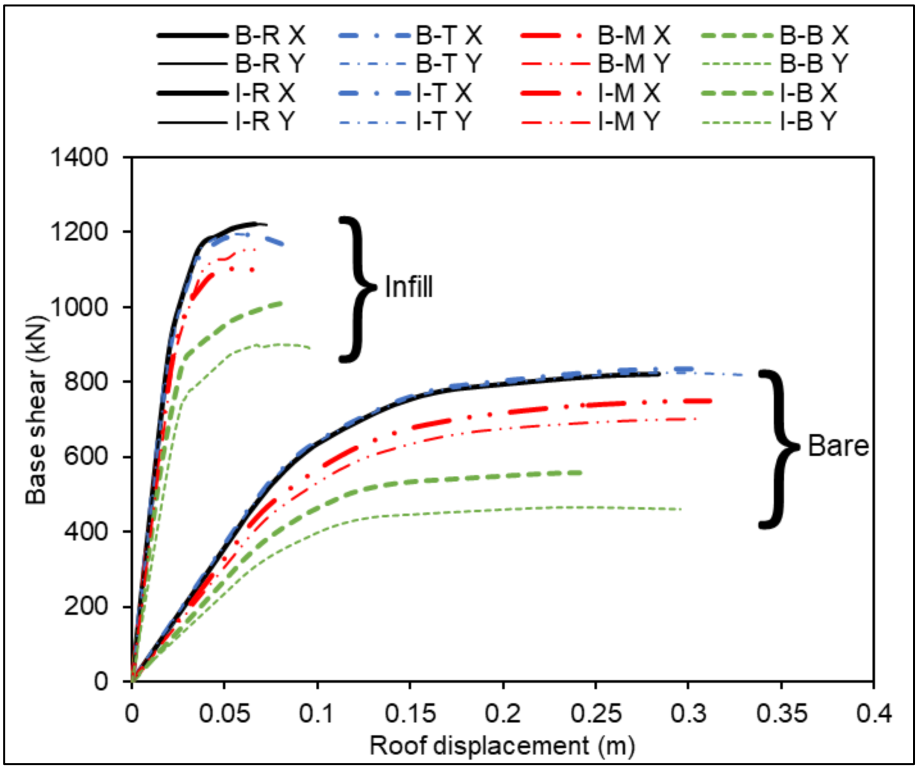


Fig. 6.2 Pushover curves of eight building configurations along both orthogonal axes

Table 6.1 Overstrength factors for different structural models obtained from NLS analysis

	R	T	M	B
Bare	2.3	2.4	2.4	1.8
Infill	3.5	3.5	3.7	3.5

Table 6.2 Ductility factors for different structural models obtained from NLS analysis

	R	T	M	B
Bare	2.9	2.9	3.1	2.7
Infill	2.7	2.6	2.5	2.9

Table 6.3 ‘R’ values for different structural models obtained from NLS analysis

	R	T	M	B
Bare	6.7	7.1	7.3	4.9

Infill	9.4	9.2	9.4	10.4

Table 6.4 Ductility factors for different structural models obtained from IDA

Seismic ground motion	Bare				Infill			
	B-R	B-T	B-M	B-B	I-R	I-T	I-M	I-B
Imperial Valley	4.1	4.7	4.4	3.9	3.8	3.9	4.7	2.5
Mammoth Lakes	4.2	5.0	5.7	1.5	5.0	5.0	4.7	4.7
Chalfant Valley	3.3	2.8	4.2	3.0	2.9	5.1	3.7	3.9
Chamoli	2.0	2.7	1.3	1.9	4.6	4.3	2.9	2.9
India-Burma Border	5.0	5.0	3.1	3.4	4.4	3.4	2.8	5.5
North-West China	2.5	2.4	1.4	2.1	3.6	5.4	5.0	4.4
Whittier Narrows	3.9	2.1	2.1	3.5	5.8	6.0	3.7	7.9
Average	**3.6**	**3.5**	**3.2**	**2.8**	**4.3**	**4.7**	**3.9**	**4.5**

Table 6.5 'R' values for different structural models obtained from IDA

Seismic ground motion	Bare				Infill			
	B-R	B-T	B-M	B-B	I-R	I-T	I-M	I-B
Imperial Valley	9.6	11.4	10.5	7.2	13.0	13.7	17.4	8.9
Mammoth Lakes	9.7	12.2	13.4	2.7	17.3	17.6	17.4	16.6
Chalfant Valley	7.6	6.8	9.9	5.5	10.0	18.1	13.6	13.6
Chamoli	4.5	6.5	3.1	3.5	15.8	15.1	10.7	10.2
India-Burma Border	11.7	12.2	7.4	6.3	15.2	12.0	10.4	19.5
North-West China	5.9	5.8	3.3	3.9	12.6	18.9	18.6	15.5
Whittier Narrows	9.0	5.2	4.8	6.4	19.9	21.1	13.7	27.8
Average	**8.3**	**8.6**	**7.5**	**5.1**	**14.8**	**16.6**	**14.5**	**16.0**

6.3.1.1 Effect of infill on the estimation of 'R'

The analytically computed 'R' value is found to vary with the structural configuration and also with the interaction of the infill wall on the structural frame. These variations are depicted in Tables 6.1-6.5. Moreover, it can be observed that R_s for infill frames models are found to be higher than corresponding bare frame models (i.e., 48%, 45.44%, 64.90%, 93.54% for R, T, M, B configurations respectively). However, R_μ factors for infill frames models computed from pushover analyses are almost similar to the corresponding bare frame configurations. In addition, in the case of IR-B configuration, a significant increase in R_μ factors of the order of 25% can be noticed for the infilled frame, unlike the bare frame model. Moreover, R_μ factors for all infill frame structural models computed using IDA are found to be higher than corresponding bare frame models (i.e., 32%; 39%; 32%; 70% for R, T, M, and B configurations respectively). Similarly, the overall R factors for infill frames models computed using IDA are higher compared to the corresponding bare frame models (i.e., 78%; 93%; 93.3%; 213% for R, T, M, and B configurations respectively).

Hence, the increase in the 'R' value indicates that the structure has higher reserve strength in the form of ductility to absorb and dissipate seismic energy. Further, in the case of consideration of interaction of infill with RC frame, higher energy dissipation due to the strength and stiffness of infill walls compared to that of the bare frame structural model at a particular displacement can be observed. From these observations, it can be concluded that the computation of the 'R' value should account the stiffness contribution of the infill wall also in addition to its load for appropriate estimation of seismic design forces.

6.3.1.2 Adequacy of code-specified ‘R’ value under single earthquake event

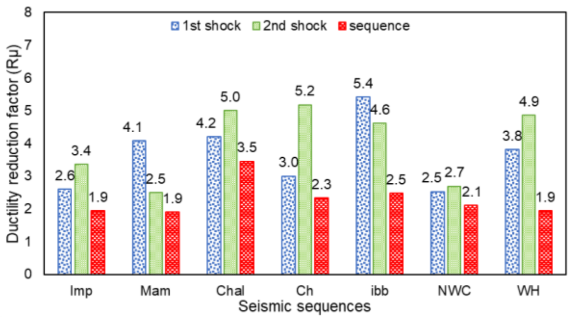
The ‘R’ values evaluated for all the structural configurations considered in this study with NLS and NLD analysis, utilizing the Newmark-Hall relationship are found to be much higher than the code-specified ‘R’ value (R=3 for OMRFs) for a chosen RC frame (OMRF). This signifies that RC MRFs conforming to IS code possess higher inelastic capacity expressed in terms of ductility and overstrength factors, albeit the structural changes. Further, it can also be attributed to the varied utilization factor, used for structural design of a code conforming RC building frame. This portrays the conservativeness of code specified constant ‘R’ value in the estimation of seismic demand during any seismic event. Here, the computation of analytical R value is attempted for collapse prevention performance criteria. This is performed in view of the seismic design philosophy specified in IS 1893 wherein the structure is not expected to collapse even under MCE but can experience certain damages during its life time.

6.3.1.3 Effect of using dynamic analysis in comparison with static analysis

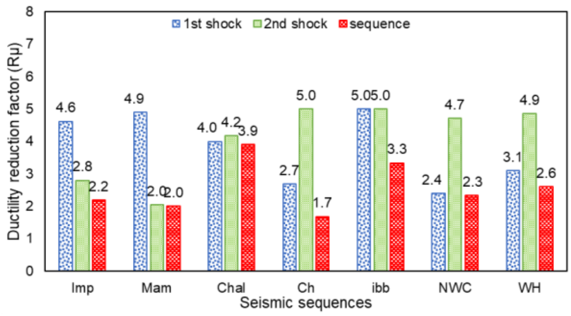
In general, for NLS analysis, the building frame is pushed with predefined (response spectrum) load pattern from elastic state to inelastic state, beyond yield till collapse; whereas in case of NLD, real earthquake ground motions are used to perform IDA, scaling the accelerograms in such a way that building frame responds from elastic to inelastic state till collapse. Further, conventional pushover analysis relies on the idealization of a multi-degree of freedom (MDOF) system into an equivalent single degree of freedom (SDOF) system thereby assuming fundamental mode as the most dominant mode contributing to the structural response. This assumption leads to inaccurate results for various building configurations, necessitating higher modal participation. Furthermore, in pushover analysis, the frame is pushed monotonically in a particular direction, whereas in NLD, the frame is subjected to cyclic loading, thereby inherently accounts for the hysteretic behaviour and dynamic characteristics of the frame which are usually ignored in the static analysis. This results in varied estimation of the ‘R’ value of the frame. From the results depicted in Tables 6.1-6.5, it can be observed that ‘R’ values obtained from NLS (pushover) analyses are comparatively lower than that obtained from NLD (time history) analyses. Further, (R_μ) factors for infill frames models computed using NLD increases in comparison with corresponding bare frame models, in contrast to the NLS analysis. This pronounces the superiority of NLD analysis in an accurate estimation of dynamic characteristics over NLS analysis procedures. Hence, NLD is always a preferred alternative to provide a more realistic inelastic seismic capacity. This aids to estimate a more appropriate ‘R’ value leading to a precise estimate of seismic demand on the structures considered.

6.4.2 R-factors for RC MRFs subjected to sequential earthquakes

The ductility factors and response reduction factors (R) are evaluated for RC MRFs considered for single and sequential earthquakes. These values are depicted in Figs. 6.3-6.6.



B-R



B-T

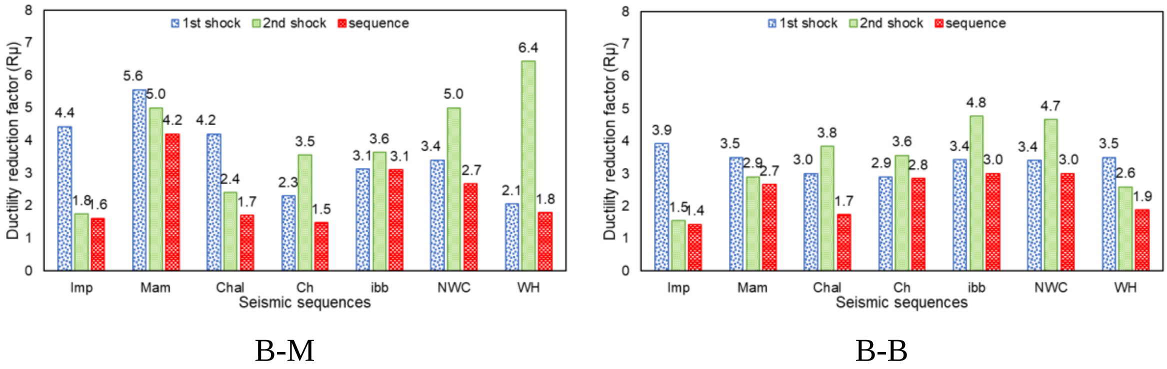


Fig. 6.3 Ductility reduction factors for bare frame models

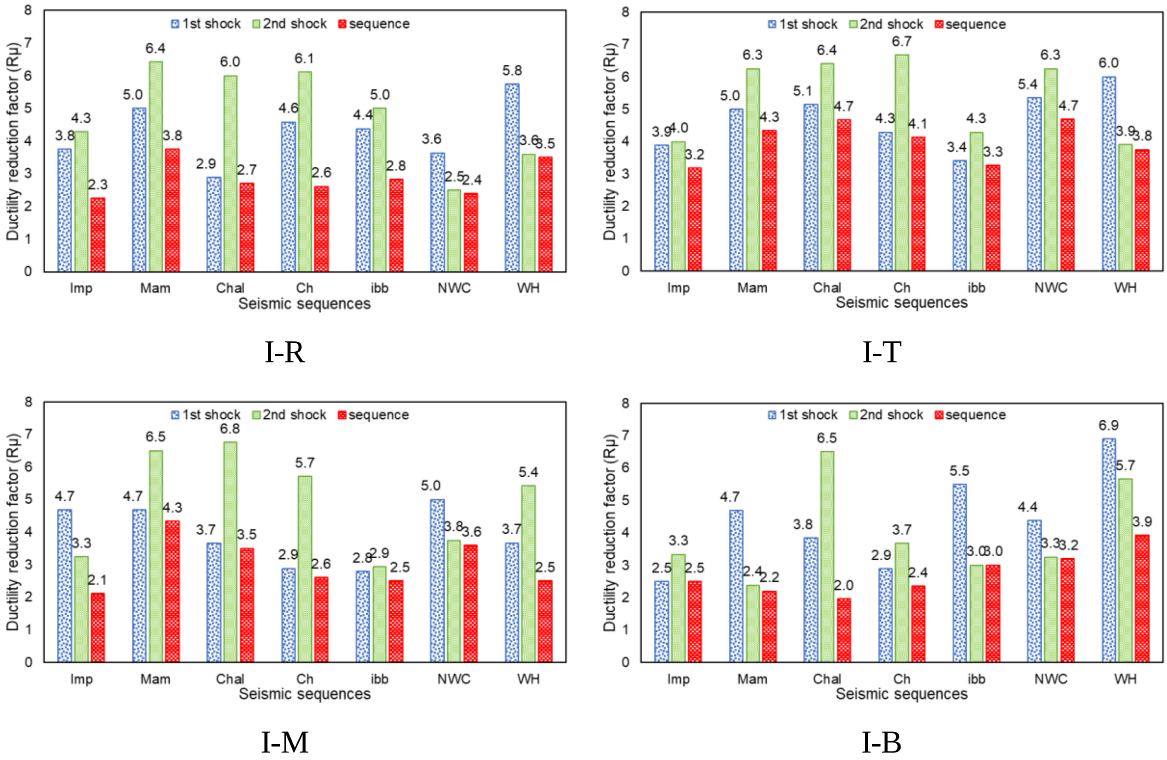


Fig. 6.4 Ductility reduction factors for infill frame models

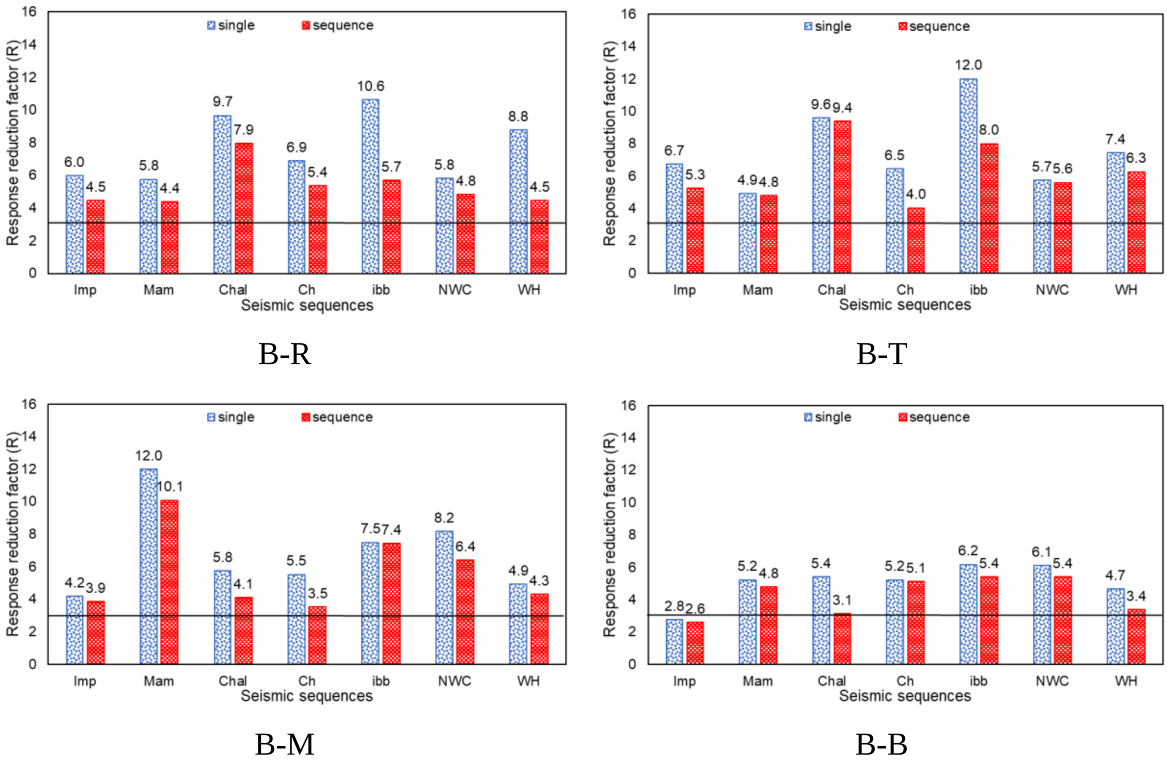


Fig. 6.5 Response reduction factors for bare frame models

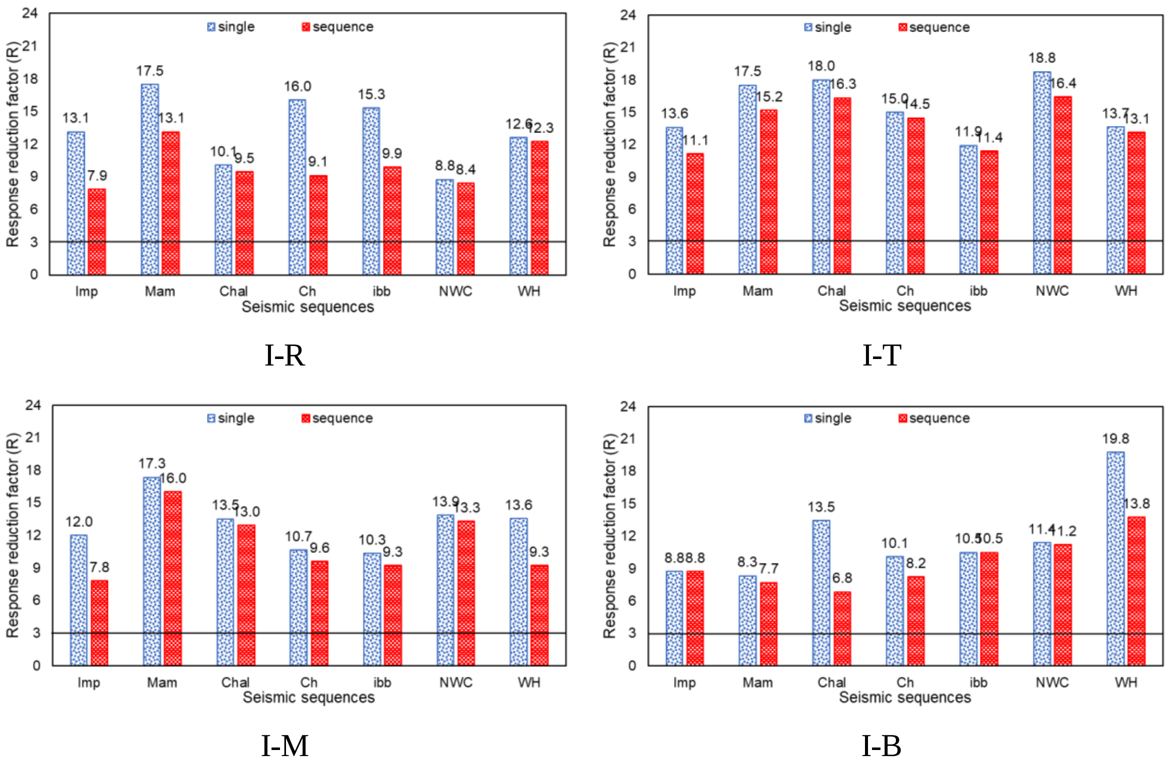


Fig. 6.6 Response reduction factors for infill frame models

6.4.2.1 Effect of sequential earthquakes on 'R'

From the results depicted in Figs. 6.5-6.6, it can be observed that the average value of response reduction factors evaluated using NLD for all the structural models is found to be smaller under sequential earthquakes than under single isolated earthquakes. The 'R' values computed for R, IR T, IR M, and IR B bare frame configurations under seismic sequences are found to be 30%, 18%, 17%, and 16% lower than the corresponding isolated single strongest earthquake. The 'R' values computed for R, IR T, IR M, and IR B infill frame configurations under seismic sequences are found to be 24%, 9%, 14%, and 18% lower than the corresponding isolated single strongest earthquake. Further, the computed base shear for all these structural models is found to increase by the same amount under sequential earthquakes. This serves as an indicator of damage that has been resulted due to first earthquake event, and has been further accumulated due to sequential event (i.e., as the strength got reduced, the ductility demand has also been affected). This phenomenon can be attributed to the weakness or inability of structural components in facing sequential earthquakes after getting damaged by the first earthquake. Hence, the structures designed in accordance with base shear computed using 'R' for an isolated single strongest earthquake scenario cannot remain functional under seismic sequence earthquakes, leading to unsafe design.

6.4.2.2 Adequacy of code-specified 'R' under Sequential Earthquake events

The adequacy of code-specified 'R' is investigated adopting the Newmark-Hall relationship to calculate ductility demands along with structural capacity parameters derived using IDA on structural configurations. The overstrength for any designed structural configuration remains same, irrespective of whether the structure experiences single or sequential earthquake events. Therefore, structural overstrength is estimated from NLS analysis.

It can be observed that 'R' values evaluated for all the structural configurations are significantly higher than code-specified 'R' for a particular OMRF (R=3 for OMRFs), depicted in Figs. 6.3-6.6 under both isolated individual and sequential earthquakes. This can be attributed to erroneous representation of seismic demand on the structure leading to very high inherent inelastic capacity, and higher reserve strength of the RC MRFs expressed in terms of ductility and overstrength factors. Hence, there is a need to address the code-specified constant 'R' to account for the changed dynamic characteristics of RC building configurations (viz., sequential earthquakes, irregularities in RC frame, etc.). This facilitates accurate estimation of seismic demands in arriving at a safe and economical design configuration that remains functional during its serviceable life.

6.5 Summary

This chapter presents the importance of R factor on non-linear response characteristics of medium-rise 3D RC building frames possessing vertical setbacks under bi-directional single and sequential earthquakes. Further, the adequacy of code specified constant 'R' in an accurate representation of dynamic characteristics in linear elastic design is also discussed in section 6.3.

Therefore, hypothetical models of RC MRFs of medium-rise configuration located at Warangal city, Telangana state, India (Seismic Zone III, medium soil profile) have been considered in this study. The vertical setbacks provided for attaining certain functional benefits and prevalent in the chosen location are also considered in this study. Further, the IDA approach has been adopted considering an ensemble of seven seismic sequences developed in addition to isolated individual earthquakes, to investigate the bi-directional effects on 3D structural models considered and described in detail in chapters 4 & 5. The 'R' values for the structural models considered are computed in this chapter considering the Newmark-Hall relationship for both individual and sequential earthquakes. This approach of arriving at R factor accounts for the changed dynamic characteristics of the structural system. It can be observed from the results that:

- The overstrength and ductility factors computed for structural models with infill contribution is found to be higher than corresponding bare frame models. The increase in the 'R' value in the case of infill frames is because there will be higher energy dissipation due to the strength and stiffness of infill walls compared to that of the bare frame structural model at a particular displacement.
- The higher values of R can be observed from IDA than NLS analysis in view of accurate estimation of dynamic characteristics/behaviour of the structure. Further, these R values are observed to be significantly higher than those specified by IS 1893 (Part 1): 2016 for the models considered, stipulating the higher inherent reserve inelastic capacity of the Indian code designed RC frame.

- It can also be observed that ‘R’ computed for sequential earthquake forces are smaller compared to ‘R’ values computed for individual earthquakes under IDA.
- Further, the constant ‘R’ suggested by IS code appears erroneous in estimating the design base shear both under individual as well as sequential earthquakes.

Hence, the analytical investigation concludes that estimation of ‘R’ should be carried out during the analysis & design phase for RC building frames. Further, it should encompass consideration of repetitive nature of earthquake forces as well as, appropriate representation of inelastic capacity of the structure. This should include even the interaction of infill wall with the MRF for appropriate evaluation of seismic behaviour. Moreover, NLD approach appears to be the only feasible alternative for adequate estimation of design lateral forces under simultaneous bi directional earthquake forces, albeit at a higher computational cost. Hence, further research is necessitated in this direction for the development of an appropriate empirical model resulting in quick and accurate estimation of ‘R’ value to complement the findings of this investigation.

CHAPTER 7

Formulation of modified R-factor for RC buildings based on structural capacity

7.1 General

This chapter is mainly focussed on proposed formulation for arriving at modified value of R factor for a given design configuration of RC building. This is computed considering the seismic inelastic capacity of RC building type for the chosen limit state/performance level. In addition, the necessary background required is presented and application of this formulation for midrise RC buildings is also discussed.

7.2 Background

The existing seismic codal provisions around the world still considers the force-based design methodology for earthquake resistant design of structures. Most of the existing codes, the IS 1893 in particular, do not address the specific performance level for which the structure has been designed. However, in accordance with the seismic design philosophy, the structure designed in conformity with IS code can experience minor/major damages under DBE / MCE but can never collapse. Therefore, it can be perceived to be collapse prevention as the default performance level considered in the IS code design. Moreover, the structural Utilisation Factor (UF) is often not accounted for in the conventional seismic design procedure. This usually results in an inherent over strength and leads to varied seismic capacity, for a particular code designed building type. Therefore, in most of the RC buildings, in addition to structural characteristics discussed in chapter 6, the utilization of structure's design capacity also needs to be accounted during the analytical estimation of R. In order to alleviate the varied capacities of a code-designed RC MRF, an UF ~ 0.9 has also been adopted for all the structural components of the chosen building type in this study. Providing sufficient energy dissipation capacity is the primary goal in designing the lateral load resistance systems for the given design seismic load. Currently, the research is more focussed on, a target limit state/performance level based seismic evaluation and design under the single/multiple-loads.

In general, the response of building frames to the earthquake forces depends on various factors that influence the design viz., ductility, over strength factor, damping etc. leading to estimation of response reduction factor. Hence, in order to ensure minimum stability of these building frames to remain functional during single/repeated earthquake forces, the accurate estimation and influence of response reduction factor (R) also need to be considered in the design calculations. Hence, in the present study the seismic performance of code-compliant RC buildings is assessed under single and sequential earthquake forces at a targeted limit state using a performance-based design framework. This approach is being used to arrive at an optimal value of code specified R, termed as modified R factor.

7.2.1 Previous studies on estimation of 'R' for IS code-designed building frames

- Mondal *et al.* (2013) assessed R-factors for RC SMRFs of different heights (2-, 4-, 7-, and 12-storey frames) designed as per Indian standards. The frames were assumed to be located in higher seismic zone (zone IV). In this study, a deterministic framework was used, and non-linear static analysis was performed on all the models. From the results it was observed that the design of the frames using code-specified R was inadequate to ensure life safety performance limit. This was understood from the obtained R-factors which were lower than the code-specified value of 5. The structural behaviour is not validated by any nonlinear time-history analysis.
- Badal and Sinha (2020) investigated archetypical buildings of different performance groups (2- to 12-storey frames) designed as per Indian standards for performance limit corresponding to Collapse Prevention at MCE hazard level. It was reported that the effect of the number of bays and bay-width of an RC frame building on R is not significant, and do not influence their seismic performance. Also, the low- and mid-rise buildings configurations located in seismic zone-V are not able to meet the expected seismic performance.

7.3 Methodology for estimating modified R-factor

The methodology for arriving at the R factor considering all structural characteristics, UF and even the single and sequential nature of earthquakes, for a given RC building design is depicted in Fig. 7.1. The designed RC building configurations are analysed using non-linear analysis for arriving at inelastic capacity for both single and sequential events.

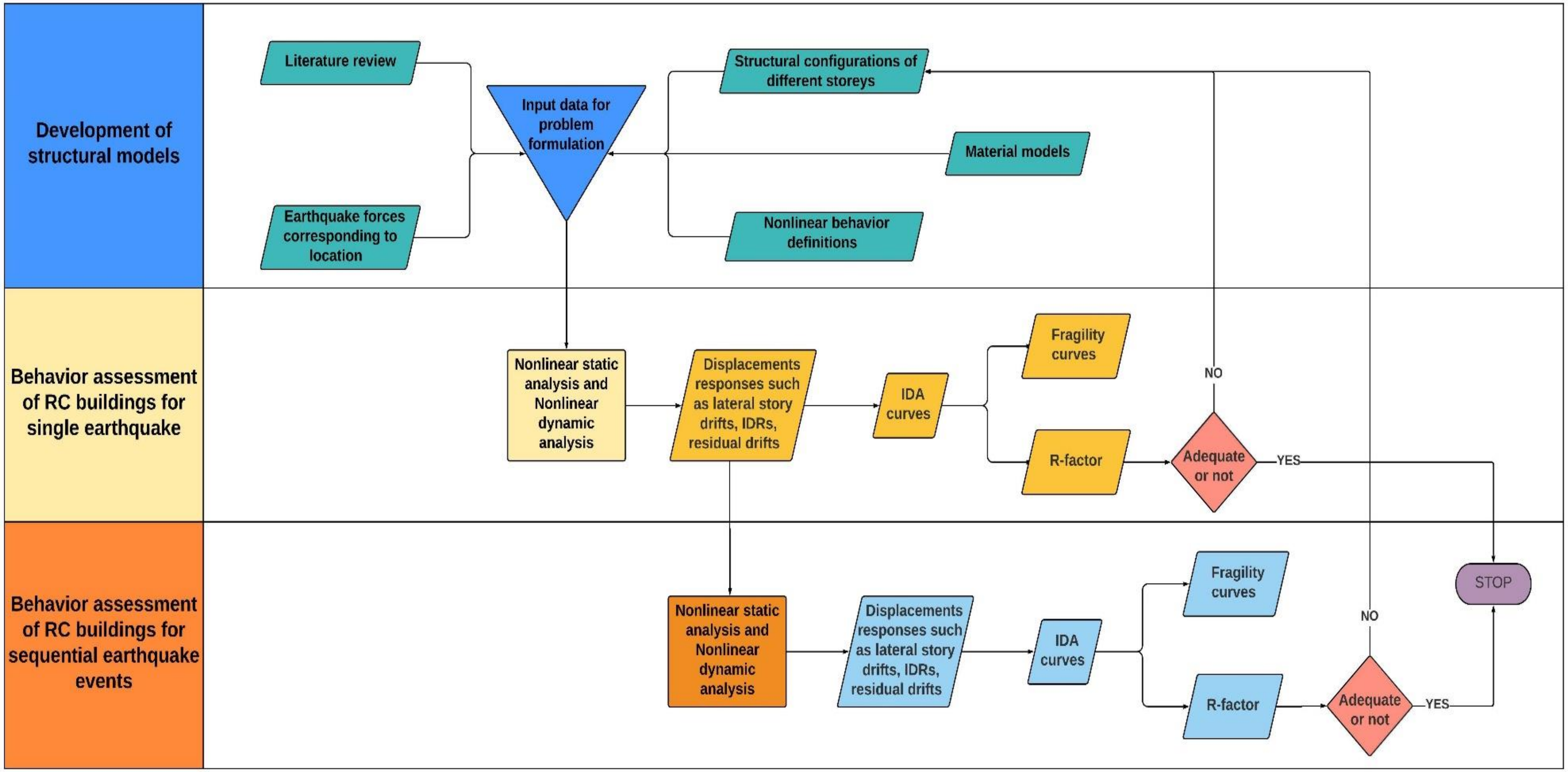


Fig. 7.1 Graphical representation of the methodology for estimating modified R-factor

7.3.1 Development of Structural models

1. A large number of distinct building configurations can be found for a specific lateral load-resisting system at any chosen location. Based on the choice of the location, a set of index archetypical buildings that adequately captures the key design variables are required for assessing the seismic performance of a specific building typology (FEMA P695, 2009).
2. Also, the selection of archetypical building configurations depends on the objective and restrictions posed by the nature of the study (Gaetani d'Aragona *et al.*, 2019; Kircher *et al.*, 2010; Badal and Sinha, 2020).
3. In view of these selection criteria, several structural models that mimic the real building configurations in the chosen region of interest i.e., Warangal city, Telangana state, India (benchmark space frames as the archetype configuration) were selected in this study as depicted in Figs. 7.2-7.4.
4. As per recommendation of latest seismic design code IS 1893 (Part 1): 2016, OMRFs are not allowed in seismic zone III. Therefore, All the buildings should be designed to be ductile as Special moment-resisting frames (SMRFs). So SMRFs building configurations have been considered in the study. This comprises of a set of 14 building configurations designed and detailed per Indian standards. These multi-storeyed RC building configurations are most commonly observed in any urban environment in India.
5. The design and detailing of the reinforced concrete members confirm with IS 456 (2000), IS 1893 (Part 1) 2016 and IS 13920 code provisions and also adopt a Structural Utilizing Factor ~ 0.9 for all structural components. Modelling and analysis of all the models was done in SAP2000 as discussed in chapter 3. Since, the UF is ensured to be around 0.9, it accounts for uniform utilisation of structural configurations of a code-compliant RC building.
6. In order to represent the effect of variation of time periods on the structural response, different building configurations of SMRF commonly found in any urban habitat and varying in height are chosen (3 -, 4-, 5-, 6-, 7-, 8-, and 9-storey buildings with and without infill walls).

7. In case of infill frame models, more commonly witnessed open ground storey (OGS) buildings are adopted in which, the OGS irregularity addressed in the analysis and design phase itself as per the provisions of IS 1893 guidelines. This facilitates to envisage the seismic behaviour more appropriately nullifying the effect of OGS irregularity.
8. The ranges of average time periods for all the structural models (both bare frame and infill frame) considered in this study are shown by marking on the response spectrum plot depicted in Fig. 7.5.

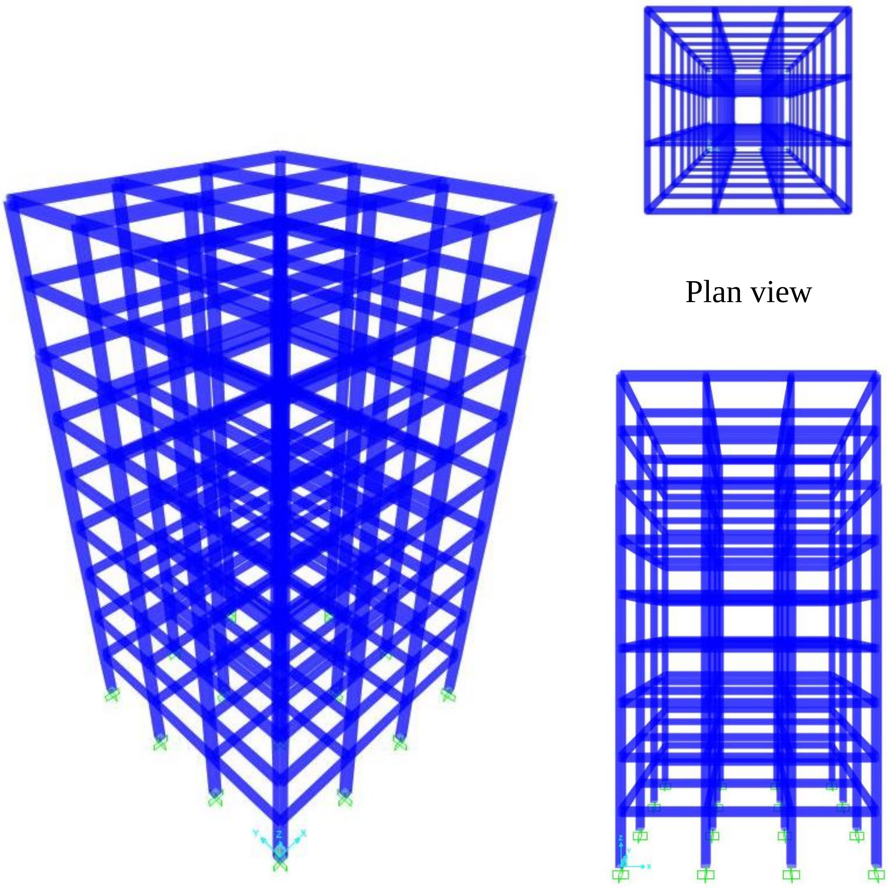


Fig. 7.2 Typical 9-storey RC frame model

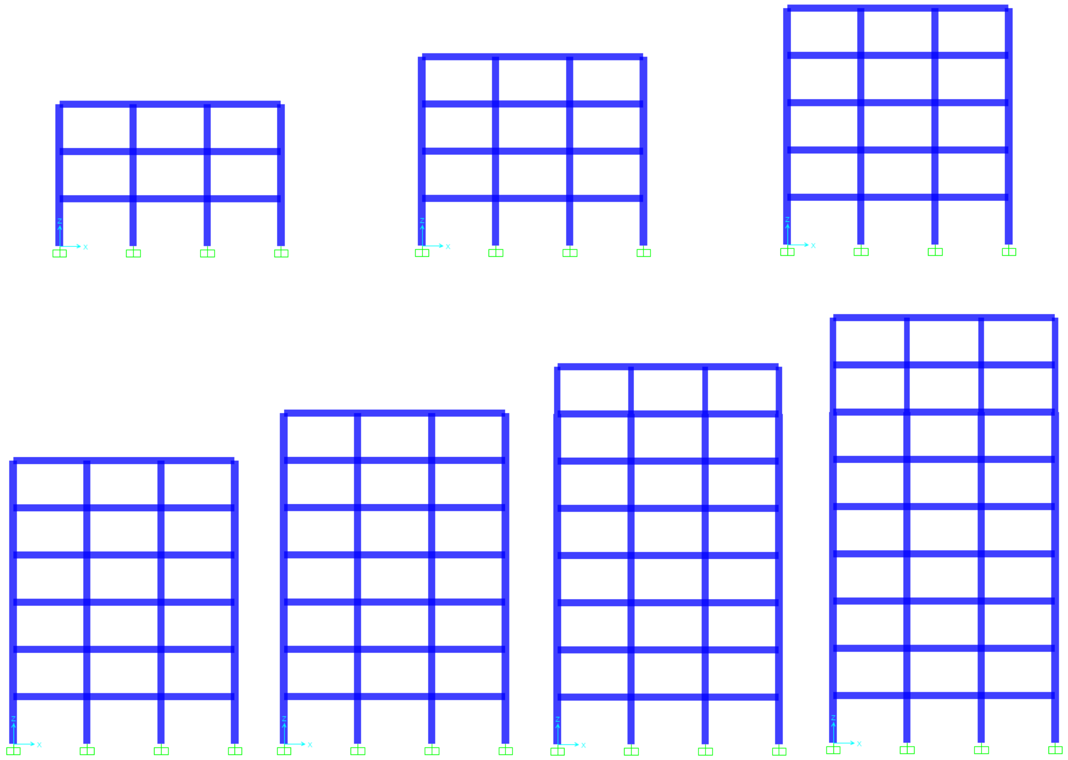


Fig. 7.3 Bare frame models of different heights

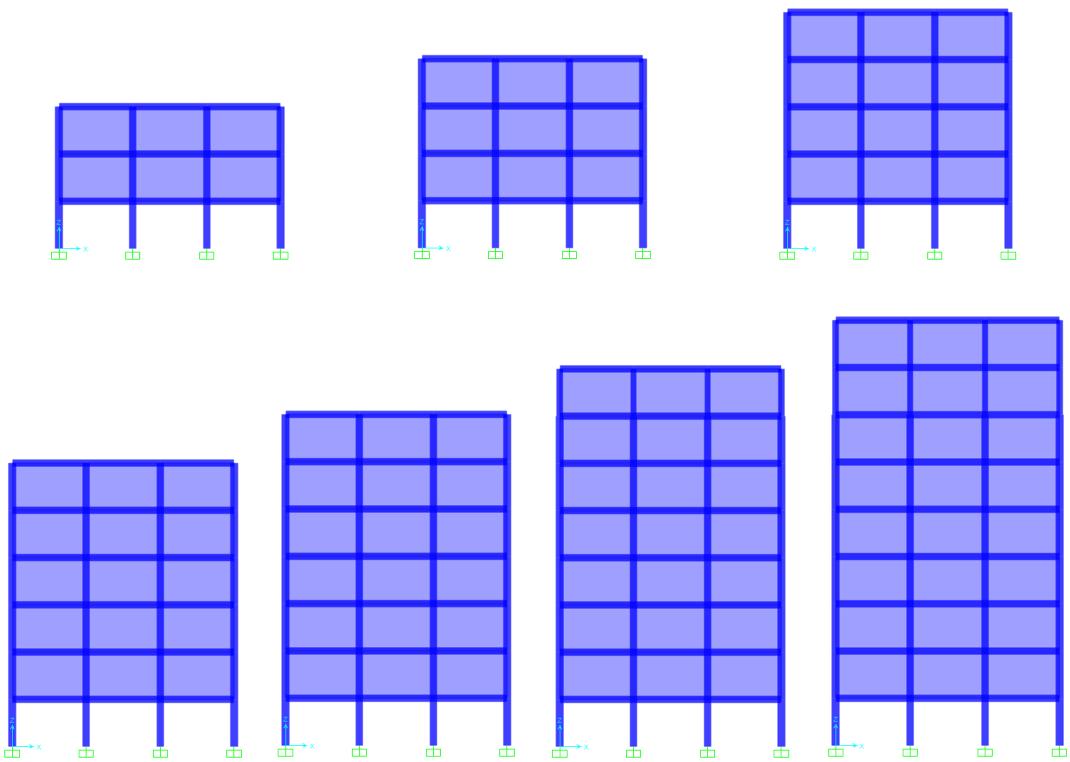


Fig. 7.4 Infill frame models of different heights

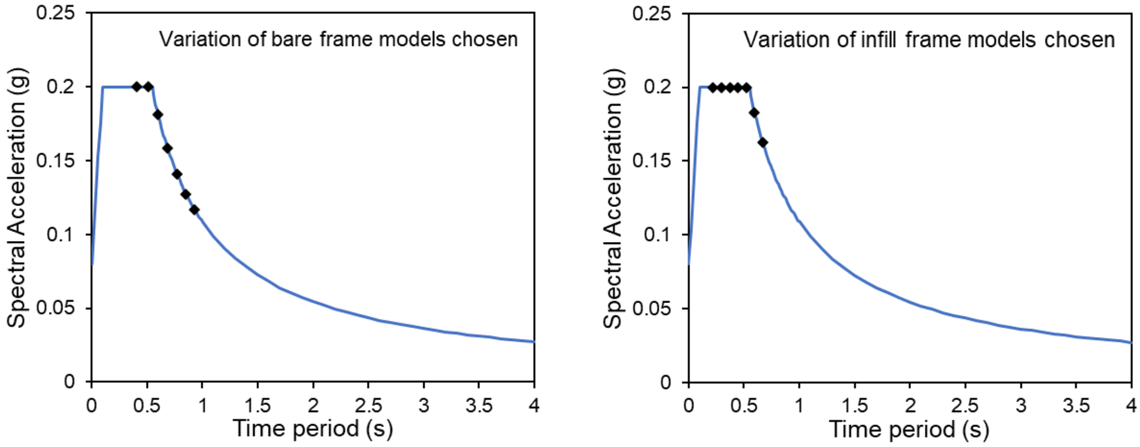


Fig. 7.5 Average Time periods of the models chosen for analysis plotted on the response spectrum IS 1893 (Part 1): 2016

7.3.2 Development of ground motion data

Since ground motion data is not available at the chosen location, spectrum compatible accelerograms have to be generated. Since this investigation deals with formulation for R value, elastic spectrum of IS 1893 (Part 1): 2016 has been chosen as a reference spectrum for generating the spectrum compatible accelerogram, i.e., the value of R is taken as unity. This implies consideration of maximum lateral force (base shear, V_B) experienced by the selected RC building type at the chosen location for both MCE and DBE Hazard levels. The equations for evaluation of base shear for both MCE and DBE hazard levels are given by the following expressions 7.1-7.2:

$$V_{MCE} = W \cdot \left(Z \cdot \frac{I}{R} \cdot \frac{S_a}{g}\right) \tag{7.1}$$

$$V_{DBE} = W \cdot \left(\frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g}\right) \tag{7.2}$$

Rest all the procedure in arriving at spectrum compatible accelerograms using SeismoMatch programme remains the same. The accelerograms, earthquake details and loading data necessary for this study are shown in Tables 7.1-7.2. Further, sequential ground motion data is also developed similarly as discussed in chapter 5. Twenty-one accelerograms summing up the Single and Sequential earthquake accelerograms were used along both orthogonal directions for in this analysis.

Table 7.1 Details of ground motion data used in this study

S. No.	Earthquake name	Station name	Date	Magnitude	Denoted as	Source
1.	Imperial Valley 01	Holtville Post Office	10/15/1979	6.53	Imp1	PEER
2.	Imperial Valley 02		10/15/1979	5.01	Imp2	
3.	Mammoth Lakes 01	Convict Creek	5/25/1980	5.69	Mam1	PEER
4.	Mammoth Lakes 02		5/25/1980	5.91	Mam2	
5.	Chalfant Valley 01	Zack Brothers Ranch	7/20/1986	5.77	Chal1	PEER
6.	Chalfant Valley 02		7/20/1986	6.1	Chal2	
7.	Chamoli 01	Gopeshwar	3/29/1999	6.6	Ch1	COSMOS
8.	Chamoli 02		3/29/1999	5.4	Ch2	
9.	India-Burma Border 01	Berlongfer	8/6/1988	7.2	ibb1	COSMOS
10.	India-Burma Border 02		1/10/1990	6.1	ibb2	
11.	North-West China 01	Jiashi	4/11/1997	6.1	NWC1	PEER
12.	North-West China 02		4/15/1997	5.8	NWC2	
13.	Whittier Narrows 01	San Marino - SW Academy	10/1/1987	5.9	Wh1	PEER
14.	Whittier Narrows 02		10/4/1987	5.3	Wh2	

Table 7.2 Loading data used for design and analysis

S. No.	Particulars	Description
1	Dead Load	Self-weight
2	Live Load	3 kN/m^2 (IS 875 (Part 2): 1987)
3	Slab and floor finishes	3.75 kN/m^2 (IS 875 (Part 1): 1987)
4	Wall thickness	230 mm
5	Seismic Load	IS 1893 (Part 1): 2016
6	Importance Factor	1
7	Zone	III (PGA = 0.16g)
8	Soil Type	Medium soil
9	Response Reduction Factor	5 for SMRF

7.3.3 Performance evaluation criteria

The RC building configurations considered in this study are perceived to be utilised for residential purposes, hence, Life Safety (LS) and Collapse Prevention (CP) limit states are chosen as the performance criterion for evaluation in this study.

Existing assessment studies for ordinary buildings present in literature are based on collapse prevention damage state for maximum considered earthquake (MCE) ground motion. This performance level has an imprecisely-specified low probability of exceedance. However, it can be observed that, in case of moderate seismic regions in India (like that of Warangal city, Telangana State considered here), complete collapse of code-conforming buildings due to earthquakes are relatively rare and therefore, not generally expected. Moreover, the capability of these structural systems to meet the intended seismic performance levels under specified hazard levels are also not addressed explicitly in the IS code. Therefore, DBE and MCE have been chosen as the hazard levels in this study. The performance limits adopted here are based on global displacement parameter, i.e., maximum inter-story drift ratio, MIDR (FEMA 273, 2000; ASCE 41, 2013) i.e., 2% for Life Safety (LS) and 4% for Collapse Prevention (CP). A typical seismic behaviour of an RC buildings subjected to different levels of shaking/earthquakes for various damage/performance states is depicted in Fig. 7.6 (Source: ASCE 41).

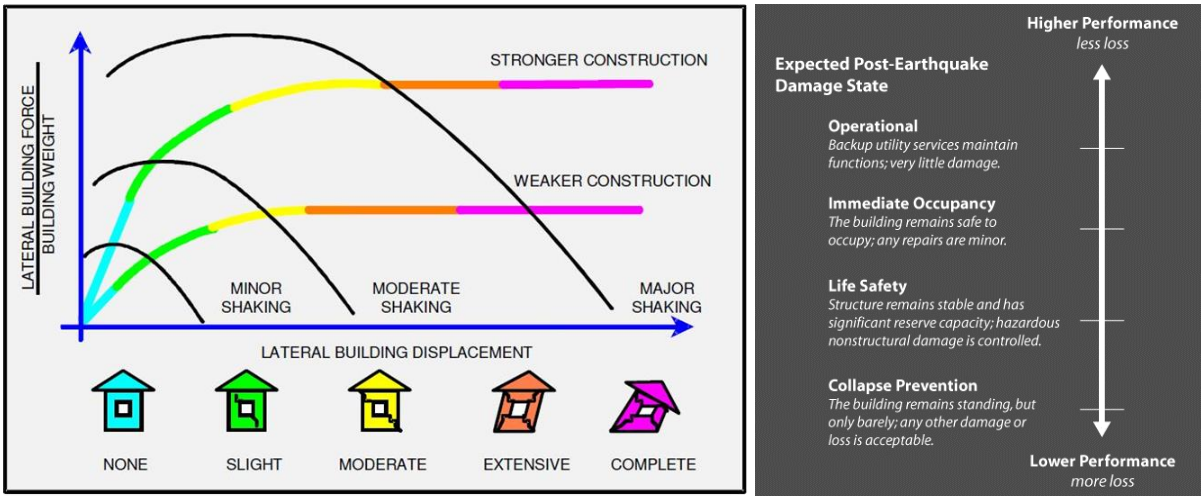


Fig. 7.6 Seismic behaviour of RC building for different damage/performance levels

7.4 Results and discussion

7.4.1 IDA curves

About 6000 simulations of NLD analyses using IDA approach for the chosen time histories mentioned in Table 7.1 were performed on all the 14 structural models considered in this study. The outcome of this analysis is the dynamic capacity curves representing the elastic and inelastic behaviour of RC MRF. Certain representative dynamic capacity curves of 5 storey bare and infill RC building frames from IDA are shown in Figs. 7.7-7.8.

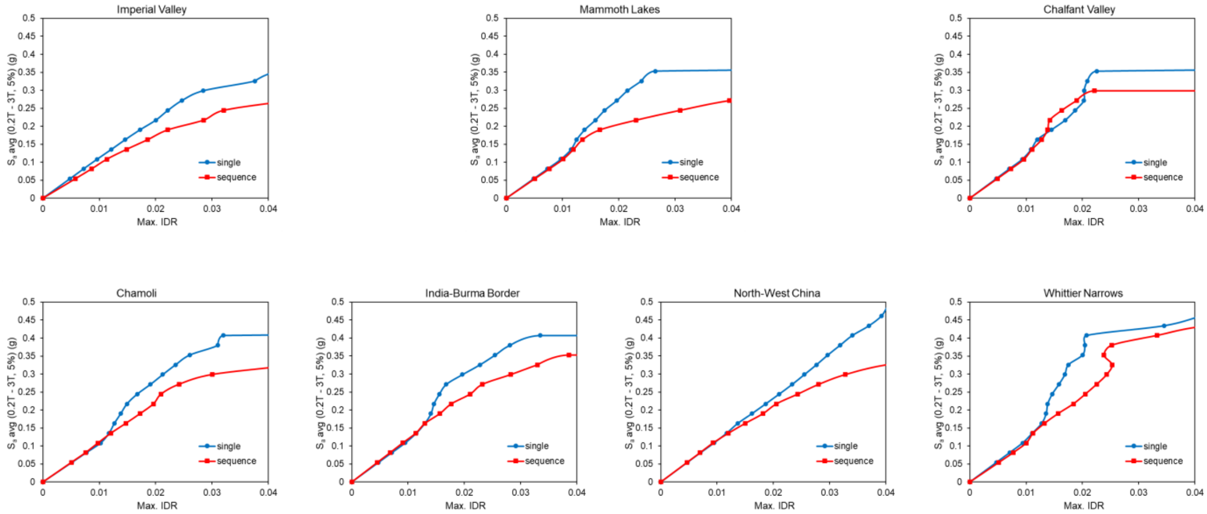


Fig. 7.7 IDA curves of 5-storey bare frame

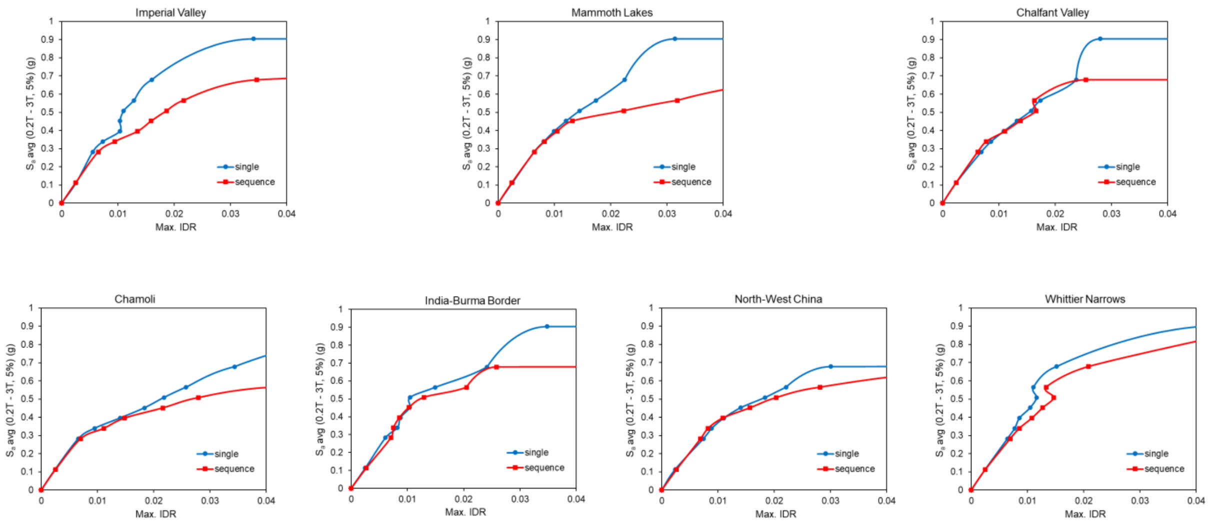


Fig. 7.8 IDA curves of 5-storey infill frame

From the inelastic capacity curves obtained from IDA, the corresponding median inelastic capacities of the frames have been computed for both LS and CP performance levels under different hazard levels (DBE and MCE). These capacities are plotted against number of storeys of the building frames, and depicted in Figs. 7.9-7.10.

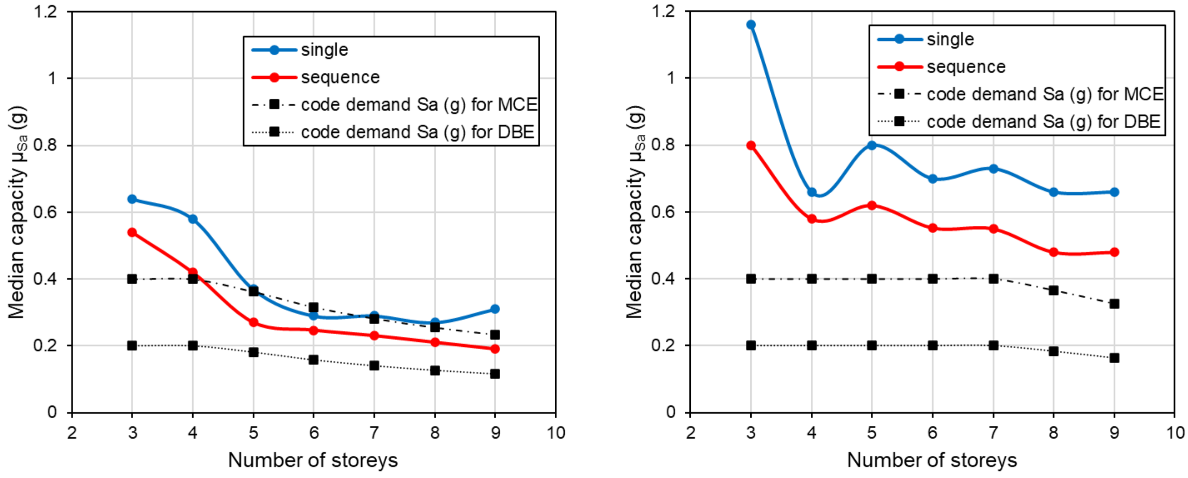


Fig. 7.9 Median capacities of frames for CP level for bare and infill frames

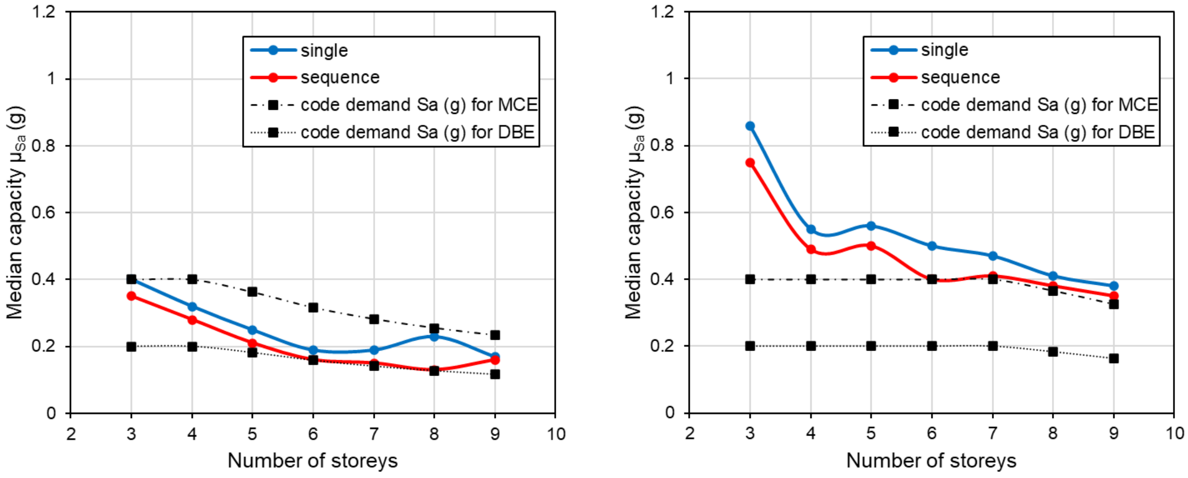


Fig. 7.10 Median capacities of frames for LS level for bare and infill frames

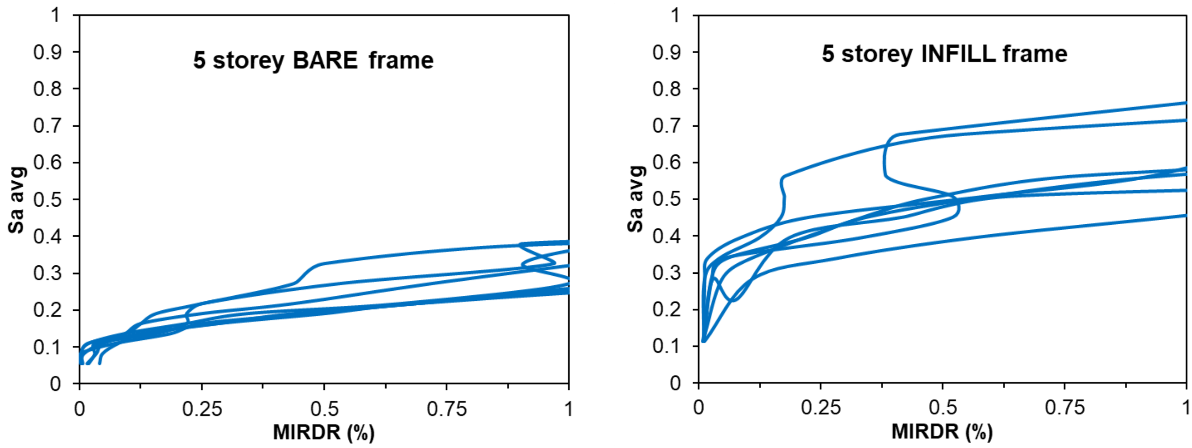


Fig. 7.11 Max. inter-storey residual drift ratios (MIRDR) of 5-storey infill frame

7.4.2 Findings

1. It can be observed from the median capacities plotted in Figs. 7.9-7.10, the effect of sequence earthquake forces on RC MRFs can be envisaged in terms of their reduced capacities compared to single earthquake forces.
2. The significance of performance-based design approach compared to the conventional force-based approach can also be visualised in terms of the variations in the capacities of frames for different performance levels LS and CP.
3. The relatively higher median capacities of infill frames compared to that of bare frames can be attributed to the inherent higher strength and stiffness contributed by the frame-infill interaction. It can also be visualised in terms of lower and delayed residual displacements in case of infill frame models.
4. From Fig. 7.11, it can be observed that residual displacements/max. inter-storey residual drift ratios get manifested at higher Sa avg in case of infill frames compared to that in case of bare frames. This implies and reiterates the importance of considering frame-infill interaction for estimation of inelastic capacities of RC MRFs, thereby affecting the analysis and design of RC MRFs.

7.5 Proposed formulation for modified R-factor

7.5.1 Concept of Safety-Margin-Ratios (SMRs)

The targeted performance criteria (viz., LS or CP) under specified hazard level is decided for defining the Safety Margin Ratios (SMRs). Safety margin ratio is defined to quantify the inelastic reserve capacity and computed considering the code specified demands for the respective moment-resisting frame. Therefore, SMR for a particular RC building design is defined as the ratio of its median collapse capacity (μ_{S_a}) evaluated at a given performance level to its respective code-demand ($S_{a,\,T}$), obtained from response spectrum based on the characteristics of RC building frame under a specific hazard level as shown by the following expression 7.3:

$$\mathrm{SMR} = \frac{\mu_{S_a}}{S_{a,T}} \tag{7.3}$$

This SMR factor specifies the available/deficient intrinsic inelastic capacity of the RC frame at the chosen performance level. This facilitates the designer to arrive at a decision of how much inelastic capacity is necessary for the structure at the chosen location, given the performance level to arrive at safe and economical functional configuration for the serviceable life time.

The SMRs computed for all the structural models analysed in this study for the two different performance levels – Life Safety (LS) and Collapse Prevention (CP) under single as well as sequential forces. Therefore, the SMR values are computed for all RC building types within a range of time periods varying in height in seismic zone III with medium soil profile for both single as well as sequential earthquake forces at LS and CP performance levels. These computed values are presented in Tables 7.3-7.6. In addition, the adequacy of code specified constant R value can also be envisaged directly from the SMR values computed at the specified location.

Table 7.3 Safety Margin Ratios for CP level for different bare frames structures

Model	code T (s)	T_{avg} (s)	code demand Sa (g) for DBE	code demand Sa (g) for MCE	Median capacity Sa (g)				SMR MCE		SMR DBE	
					single		sequence		single	sequence	single	sequence
					median	CoV %	median	CoV %				
3-storey	0.41	1.00	0.20	0.40	0.64	13	0.54	10	1.60	1.35	3.20	2.70
4-storey	0.51	1.16	0.20	0.40	0.58	16	0.42	8	1.45	1.05	2.90	2.10
5-storey	0.60	1.48	0.18	0.36	0.37	10	0.27	15	1.02	0.74	2.04	1.49
6-storey	0.69	1.80	0.16	0.32	0.29	23	0.25	16	0.92	0.78	1.83	1.56
7-storey	0.77	1.90	0.14	0.28	0.29	20	0.23	32	1.03	0.82	2.06	1.63
8-storey	0.85	2.19	0.13	0.25	0.27	25	0.21	35	1.06	0.82	2.12	1.65
9-storey	0.93	2.49	0.12	0.23	0.31	34	0.19	24	1.33	0.81	2.66	1.63

Table 7.4 Safety Margin Ratios for CP level for different infill frames structures

Model	code T (s)	T_{avg} (s)	code demand Sa (g) for DBE	code demand Sa (g) for MCE	Median capacity Sa (g)				SMR MCE		SMR DBE	
					single		sequence		single	sequence	single	sequence
					Median	CoV %	Median	CoV %				
3-storey	0.22	0.44	0.20	0.40	1.16	17	0.80	18	2.90	2.00	5.80	4.00
4-storey	0.30	0.53	0.20	0.40	0.66	13	0.58	13	1.65	1.45	3.30	2.90
5-storey	0.37	0.63	0.20	0.40	0.80	12	0.62	15	2.00	1.55	4.00	3.10
6-storey	0.45	0.68	0.20	0.40	0.70	9	0.55	11	1.75	1.38	3.50	2.76
7-storey	0.52	0.78	0.20	0.40	0.73	11	0.55	10	1.83	1.38	3.65	2.75
8-storey	0.59	0.88	0.18	0.37	0.66	5	0.48	15	1.80	1.31	3.61	2.62
9-storey	0.67	0.98	0.16	0.33	0.66	19	0.48	16	2.03	1.48	4.06	2.95

Table 7.5 Safety Margin Ratios for LS level for different bare frames structures

Model	code T (s)	T_{avg} (s)	code demand Sa (g) for DBE	code demand Sa (g) for MCE	Median capacity Sa (g)				SMR MCE		SMR DBE	
					single		sequence		single	sequence	single	sequence
					median	CoV %	median	CoV %				
3-storey	0.41	1.00	0.20	0.40	0.40	13	0.35	10	0.65	0.60	1.30	1.20
4-storey	0.51	1.16	0.20	0.40	0.32	13	0.28	7	0.80	0.70	1.60	1.40
5-storey	0.60	1.48	0.18	0.36	0.25	15	0.21	9	0.69	0.58	1.38	1.16
6-storey	0.69	1.80	0.16	0.32	0.19	13	0.16	13	0.60	0.51	1.20	1.01
7-storey	0.77	1.90	0.14	0.28	0.19	14	0.15	8	0.67	0.53	1.35	1.06
8-storey	0.85	2.19	0.13	0.25	0.23	14	0.13	18	0.90	0.51	1.80	1.02
9-storey	0.93	2.49	0.12	0.23	0.17	25	0.16	16	0.73	0.69	1.46	1.37

Table 7.6 Safety Margin Ratios for LS level for different infill frames structures

Model	code T (s)	T_{avg} (s)	code demand Sa (g) for DBE	code demand Sa (g) for MCE	Median capacity Sa (g)				SMR MCE		SMR DBE	
					single		sequence		single	sequence	single	sequence
					Median	CoV %	Median	CoV %				
3-storey	0.22	0.44	0.20	0.40	0.86	14	0.75	12	2.15	1.88	4.30	3.75
4-storey	0.30	0.53	0.20	0.40	0.55	6	0.49	11	1.38	1.23	2.75	2.45
5-storey	0.37	0.63	0.20	0.40	0.56	12	0.50	12	1.40	1.25	2.80	2.50
6-storey	0.45	0.68	0.20	0.40	0.50	10	0.40	14	1.25	1.00	2.50	2.00
7-storey	0.52	0.78	0.20	0.40	0.47	9	0.41	11	1.18	1.03	2.35	2.05
8-storey	0.59	0.88	0.18	0.37	0.41	13	0.38	10	1.12	1.04	2.24	2.08
9-storey	0.67	0.98	0.16	0.33	0.38	11	0.35	13	1.17	1.08	2.34	2.15

It can be noted that, the obtained SMR for a building configuration quantifies the inherent inelastic capacity of RC building frame. This implies, for instance if the SMR >>1.0, it indicates that the structure possesses vary high inelastic capacity and the code-based R provides a very conservative design forces leading to uneconomical design configuration. Hence, can be modified to obtain modified design forces leading to a more economical configuration than existing due to reduced forces. In case SMR <1.0, it means the structure have deficient inelastic capacity at that performance level therefore, the code-based R value is inadequate and needs modification to arrive at appropriate design forces necessary to be considered for analysis and redesign. In this case, appropriate structural configuration required to resist the expected earthquake forces can be obtained. In either case, modified R value $(\overline{\mathrm{R}})$ can be obtained as a product of code specified R with SMR obtained as specified below:

$$\overline{\mathrm{R}} = \mathrm{R} \cdot \mathrm{SMR} \tag{7.4}$$

Since the structural utilization factor for designing is taken as 0.9, this approach of arriving at modified R factor results in defining a minimum value that is adequate for any structure to remain safe due to an eventuality at the chosen location. Therefore, the proposed methodology as shown in Fig. 7.13, facilitates in arriving at optimal level of design forces for the structure to possess adequate inelastic capacity to remain safe and functional during its lifetime.

7.5.2 Proposed empirical model/equations

In view of this an attempt has been made to develop an empirical model specifying SMR for RC building types in zone III with medium soil profile, possessing an UF of 0.9 for various time periods. Around 14 structural models as discussed have been considered to arrive at establishing a relation between SMR and time period of structural configuration.

The corresponding expression depicting the plot of SMR with time period for bare frame as well as with infill frame RC building models are depicted in Fig. 7.12 below. This representation of SMR with time period is analogous to the response spectrum plot specified by various codes and serves as a ready reckoner chart to estimate SMR value for an RC building type characterised by time period (T), given the performance level. However, the present empirical model is restricted to only seismic zone III medium soil profile and characterising for only LS and CP performance levels under single and sequential earthquake forces. This chart can be extended in future, for all categories of building structures and its various performance levels to arrive at SMR value based on fundamental time period of the building. Hence, this approach of arriving at modified R value based on SMR for the chosen RC building type provides appropriate design forces and quantifies the inelastic capacity for the intended performance level. Furthermore, considering the choice of stakeholders (viz., Municipal Corporations, Builders, Contractors, Town Planners, Banking Insurance companies, House owners etc), the performance level are to be decided at any location and the corresponding SMRs needs to be evaluated to arrive at an estimated inelastic capacity of the structure from Table 7.7.

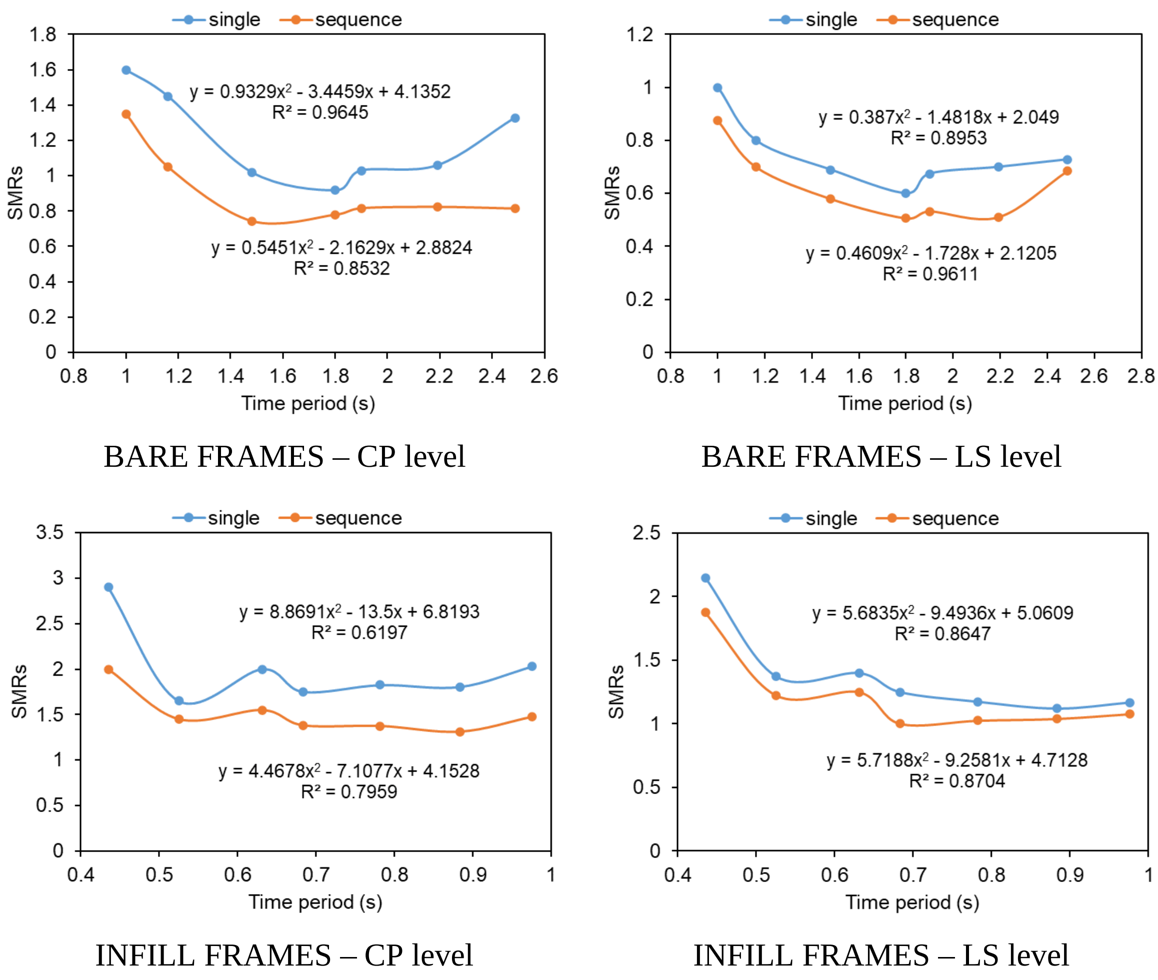


Fig. 7.12 Variation of SMRs w.r.t. Average Time periods

Table 7.7 Proposed empirical equations to compute SMRs for different structural configurations and performance levels

BARE FRAMES		
	CP	**LS**
single	$SMR = 0.9329T^2 - 3.4459T + 4.1352$	$SMR = 0.387T^2 - 1.4818T + 2.049$
sequence	$SMR = 0.5451T^2 - 2.1629T + 2.8824$	$SMR = 0.4609T^2 - 1.728T + 2.1205$
INFILL FRAMES		
	CP	**LS**
single	$SMR = 8.8691T^2 - 13.5T + 6.8193$	$SMR = 5.6835T^2 - 9.4936T + 5.0609$
sequence	$SMR = 4.4678T^2 - 7.1077T + 4.1528$	$SMR = 5.7188T^2 - 9.2581T + 4.7128$

7.5.3 Integration of Performance-based design (PBD) into Force-based design (FBD)

The proposed modified R-factor helps in integrating performance-based design procedure into conventional force-based design procedure.

Elastic strength-based design (or FBD)	**Inelastic deformation-based design (or PBD)**
• Chose design code and earthquake loads • Design check parameters – forces / stresses • Calculate stresses – load factors • Get allowable stresses • Calculate stress ratios	• Chose performance level and design loads – ASCE 41 • Demand capacity measures – storey drifts / base shears • Get deformation and force capacities • Calculate deformation and force demands • Calculate D/C ratios – limit states/performance levels

7.6 Application of the proposed methodology

The importance of the proposed methodology in arriving at appropriate design forces necessary for the structure to remain functional has been presented. In order to demonstrate the application of the proposed methodology, for the chosen performance criteria at different hazard levels a case study has been presented below.

7.6.1 Case study

The structural model considered has been evaluated in accordance with the flowchart depicted in Fig. 7.13. A nine-storeyed RC building model perceived to be located in moderate seismic zone III with medium soil profile and designed with UF of 0.9 has been considered. After computing the time period of the 9-storey bare frame model, we can arrive at the SMR for the target performance level (CP here) as per procedure discussed in section 7.5, and depicted in Table 7.8. SMR values obtained serves as indicator of inelastic capacity of the RC building frame type, whether it represents adequate (A) or deficient (D) to meet the seismic demands of the code relevant at the time of evaluation and at the chosen location. Further, fragility curves have been developed from inelastic curves as discussed in chapter 3 to complement the findings of SMR values, and depicted in Fig. 7.14.

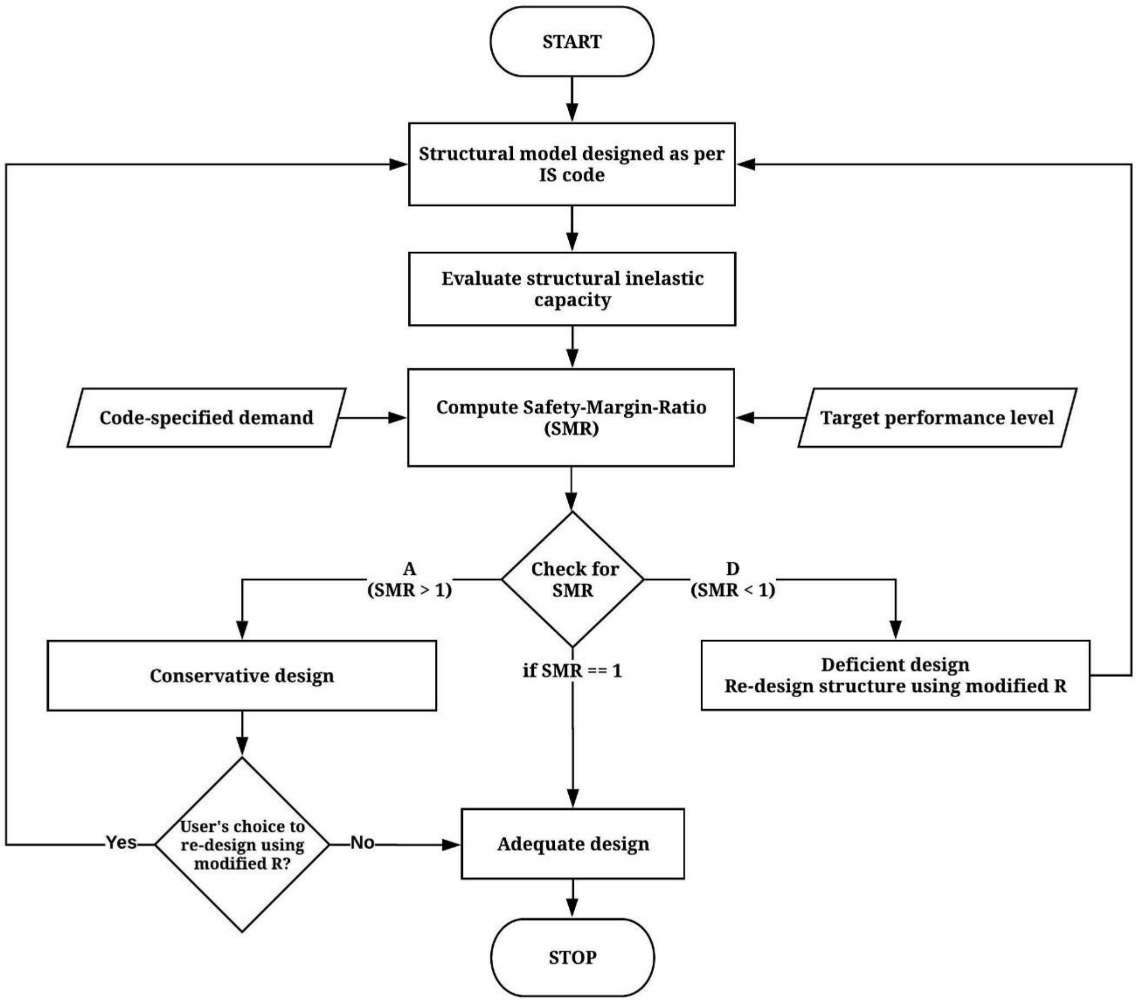


Fig. 7.13 Flowchart representing the application of modified R to arrive at adequate structural design

7.6.1.1 Computation of modified R value

Modified R values are computed utilising the SMRs computed in accordance with equations given in Table 7.7 for the considered 9-storey model, and are shown in Table 7.8. The decision whether or not to use the modified R-factor ($\overline{\mathrm{R}}$) can be made based on the following scenarios:

[1]. When the $\overline{\mathrm{R}}$ is similar in value to code specified R, then the designed RC building can be perceived to be compliant with seismic provisions at the selected place at the evaluation time and this structure doesn't need any redesign.

[2]. In case of $\overline{\mathrm{R}}$ > R (specified by code), then it implies, providing R will result in estimation of significantly higher lateral forces than required for seismic design. In this case the $\overline{\mathrm{R}}$ specified represents lateral forces adequate enough to ensure stability and functionality of the structure for the chosen performance level and results in economical configuration than existing.

[3]. In case of $\overline{\mathrm{R}}$ < R, then it implies, providing R will result in inadequate estimation of lateral forces required for seismic design and imminently needs modification to avoid a disaster. In this case also the $\overline{\mathrm{R}}$ specified represents lateral forces adequate enough to ensure stability and functionality of the structure for the chosen performance level.

Table 7.8 Calculation of SMRs and modified R-factors for case study model

Performance Level	T_{avg}	SMR values	$\overline{\mathrm{R}}$	Remarks*
CP	2.49 sec	Single: 1.33	6.65	A
		Sequential: 0.81	4.05	D
***Remarks:**	Adequate (A): Provides conservative estimate of design forces Deficient (D): Inadequate to meet the current seismic demands			

7.6.1.2 Check for modified R value

Modified R value obtained for the 9-storey model under sequential earthquake forces for CP performance level has been used for redesign. Further, the redesigned structural model has been subsequently analysed under sequential earthquake forces for CP performance level to arrive at the inelastic capacity curve. Further, fragility curves have been developed as depicted in Fig. 7.14. From the analysis results obtained, the following observations are made:

a. The median inelastic capacity of the frame with R and subjected to sequential earthquake forces were found to be less than the prescribed code-demand (Code-demand is represented by means of Black-coloured dotted lines in Fig. 7.14).
b. The median inelastic capacity of the frame with $\overline{R}$ and subjected to sequential earthquake forces has increased significantly, and is found higher than the prescribed code-demand.

The fragility curves developed for both R and $\overline{R}$ under single as well as sequential earthquake forces reinforces the observations a and b, in terms of probability of exceedance. In addition, it can be observed that probability of failure or probability of exceedance of damages for a given damage state (represented in terms of S_a) has significantly reduced after adopting modified R ($\overline{R}$), for RC structural model under both single and sequential earthquake forces as depicted in Fig. 7.14.

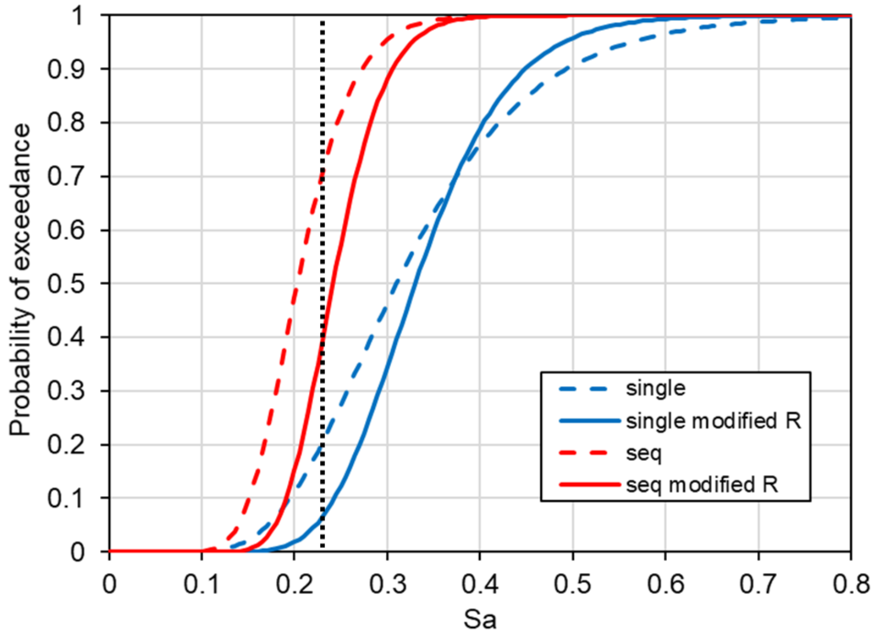


Fig. 7.14 Fragility curves of 9-storey bare frame using modified R-factor for CP performance level under MCE hazard

This case study clearly portrays the importance of the proposed methodology in arriving at safe design lateral loads by specification of appropriate modified $\overline{R}$ necessary for the considered structural configuration based on its inelastic capacity.

7.7 Summary and conclusions

This chapter is primarily focussed on proposing a formulation for arriving at R factor based on structural capacity for the chosen performance criteria. This has been carried out for both LS and CP performance characteristics under single and sequential earthquake events for the hazard levels (MCE and DBE). It can be observed that:

- R-factor depends upon the design force, structural utilization factor, and ductility demand, in addition to the dynamic characteristics of the structural system.
- Hence, the value of R need not be constant for a structural type. Given, a structural type, R value depends on the purpose (configuration and importance) and required target performance level for which the structure is to be designed.
- Interaction of infill wall with the RC frame affects the ductility demand of the frame significantly, hence, needs to be considered while formulating R-factor.
- In order to model infill framed buildings, in addition to the separate fundamental time period, modified R should be appropriately calculated with the proposed methodology in order to attain a safe and economical design.
- The methodology described above shown in Fig. 7.1 and evaluated for the case study mentioned, for arriving at the modified R-factor as depicted in Fig. 7.13 can be extended to any structural typology to arrive at appropriate design lateral forces.

This ensures functionality of the analysed structure for its serviceable life time and contributes to arrive at more appropriate design lateral forces in case of new structures. Further, this approach aids in integrating performance-based design procedure into conventional force-based design procedure thereby ensuring performance of the chosen structure for the considered criteria.

CHAPTER 8

Summary and Conclusions

8.1 Summary

The present study mainly focused on the assessment of behaviour and non-linear response characteristics of 3D RC building frames of medium-rise configuration with and without vertical irregularities (setbacks) under bi-directional sequential earthquakes. Given this scenario, RC MRFs of medium-rise configuration located at Warangal city, Telangana state, India (Seismic Zone III, medium soil profile) have been considered in this study. The vertical setbacks provided for attaining certain functional benefits and prevalent in the chosen location are also considered. Moreover, analysis of RC buildings for estimation of seismic design forces is usually carried out only on the moment-resisting frames (MRF), ignoring the interaction of the infill wall with the MRF. This results in the erroneous estimation of the seismic behaviour of the structure.

In this investigation, IDA is performed for bidirectional individual and repeated earthquake sequence loading to investigate the structural behaviour of regular and vertical setback models. The non-linear response and structural damages have been analysed in terms of evaluation of several parameters such as story displacements, permanent damage (residual displacements), local structural damage (plastic hinge formation), structural capacity evaluation (dynamic capacity curves with respect to S_a avg), and collapse fragility estimation in terms of S_a avg. A considerable reduction in the capacity of the buildings is observed while facing a second or subsequent earthquake, after getting damaged due to first one. This clearly signifies the weakness of most of the existing and new buildings designed as per the seismic provisions considering only one earthquake force for the design. Further, from the results obtained, the influence of repeated earthquake forces is clearly pronounced on the vulnerability characteristics of the building structures designed as per the seismic provisions of Indian code. Hence, there is a need to account for sequential earthquake forces during the design phase in order to build a seismic resilient structure. NLD appears to be the only feasible alternative for adequate estimation of design lateral forces under simultaneous bi-directional earthquakes, albeit at a higher computational cost. In general, the response of building frames to the earthquake forces depends on various factors that influence the design. Hence, in order to ensure minimum stability of these building frames to remain functional during sequential earthquake forces, the accurate estimation and influence of response reduction factor (R) also need to be considered in the design calculations.

Further, the adequacy of code-specified constant ‘R’ in an accurate representation of dynamic characteristics in linear elastic design is also presented. The ‘R’ values for the structural models considered are computed from nonlinear analysis procedures for both individual and sequential earthquakes addressing the changed dynamic characteristics of the structural system. It can be observed that ‘R’ computed for sequential earthquake forces are smaller compared to ‘R’ values computed for individual earthquakes under IDA. This clearly serves as an indicator for damage accumulation under sequential forces. Further, the constant ‘R’ suggested by IS code appears erroneous in estimating the design base shear both under individual as well as sequential earthquakes. Hence, it is evident from the results that, the estimation of ‘R’ should encompass sequential earthquake forces with appropriate representation of the dynamic characteristics of the building configurations, during the design phase itself. This facilitates in arriving at a safe and seismic resilient configuration. This includes even the interaction of infill wall with the MRF for appropriate evaluation of seismic behaviour.

Hence, a methodology for estimation of modified R-factor was proposed for RC MRFs subjected to single and sequential earthquake forces for DBE and MCE hazard levels. The target performance levels considered in our study were Life Safety (LS) and Collapse Prevention (CP). In general, the structure’s capacity is not fully utilized, which lead to varied structural capacities, for a particular code designed building type. In order to alleviate these varied capacities for code-conforming design, UF ~ 0.9 has been adopted during structural design. Non-linear analysis is performed in order to arrive at the inelastic capacity of the structural model. Modified R values are computed using Safety Margin Ratios (SMRs) in accordance with the methodology depicted in section 7.6. This modified R-factor can be utilised to obtain a safe and economical design configuration.

8.2 Conclusions

The detailed itemised conclusions drawn from the overall seismic behaviour assessment of RC building frames under single/sequential earthquake forces are listed below:

8.2.1 Behaviour assessment of 3D RC frame buildings under single earthquake event

- It can be observed that the horizontal displacements of the vertical setback RC frames are lower compared to that of the regular RC frame. These lower values of displacements can be attributed to the appropriate reduction in mass and stiffness characteristics along with the height of the building due to setbacks in the vertical direction.
- Effect of considering bi-directional earthquake forces on seismic response of buildings is evident from the results, especially in case of irregular buildings.
- The effect of interaction of infill with the surrounding RC frame is pronounced on the overall structural response due to the increased stiffness in the elastic region.

8.2.2 Behaviour assessment of 3D RC frame buildings considering sequential earthquake event succeeding the first earthquake event

- There is a considerable reduction in the collapse capacity of the buildings while facing a second or subsequent earthquake, after getting damaged when subjected to the first one. This clearly signifies the weakness of most of the existing and new buildings designed as per the seismic provisions of IS 1893 (Part 1): 2016 considering only one earthquake force for the design.
- The fragility curves developed from the non-linear capacity curves also reinforce the weakness of RC building frames in facing the sequential earthquakes.
- This is in line with the behaviour of seismic-resistant buildings designed as per various international codes of practice reported in the literature. Hence, this investigation emphasizes the need to account for sequential earthquake forces during the design phase in order to build a seismic resilient structure, as sequential earthquake forces represent a damaging earthquake force compared to a single earthquake force for DBE hazard level.
- Therefore, the capabilities of IS code-designed RC frames for a target performance level needs to be understood, as IS codes do not address these capacities explicitly.

8.2.3 Importance of Response reduction factor (R) in seismic behaviour Assessment of RC buildings

- It can be observed that R values evaluated for all the structural configurations are significantly higher than code specified R for a particular OMRF (R=3). This can be attributed to erroneous representation of seismic capacity of the structure leading to very high inherent inelastic capacity, and higher reserve strength of the RC MRFs expressed in terms of ductility and overstrength factors.
- R value depends on the structural configuration and also varies with the interaction of the infill wall on the structural frame. Hence in the estimation of R, it is imperative to address the strength and stiffness characteristics in addition to the mass of infill walls. Therefore, there is a need to address the code specified constant 'R' to account for the changed dynamic characteristics of RC building configurations.
- In order to account these diverse structural systems in providing a safe and economical solution, it is essential to develop a formulation for evaluating R-factor considering the dynamic characteristics of the structural system.

8.2.4 Formulation of modified R-factor for RC buildings based on structural capacity

- R-factor depends upon the design force, structural utilization factor, and ductility demand, in addition to the dynamic characteristics of the structural system. Hence, the value of R cannot be constant for a structural type. Within a structural type, R value depends on the purpose (configuration and importance) and target performance level for which the structure is to be designed.
- Interaction of infill wall with the RC frame affects the initial strength capacity of the frame significantly, hence, needs to be considered while formulating R-factor.
- In order to model infill frame buildings, in addition to the separate fundamental time period, modified R should be appropriately calculated with the proposed methodology in order to attain a safe and economical design.
- In this way, performance-based/displacement-based design objectives can be achieved using conventional force-based design procedure.

8.3 Significant contributions from the study

- The novelty of this study lies in estimation of damaging earthquake demand for IS code designed RC MRF, considering the repetitive nature of earthquakes usually not accounted in the response spectrum specified at any chosen location.
- Checking the adequacy of code-specified R-factor in accounting the inelastic capacity of IS code designed RC building configurations.
- Development of a methodology for computation of modified R-factor considering the inelastic capacities of RC buildings in moderate seismic zone and medium soil profile (Zone III and Type II soil) under sequential earthquake events (for both MCE and DBE hazard levels) for target performance levels.
- The proposed methodology for arriving at the modified R-factor is applicable for all building configurations and can be utilised at any location with appropriate consideration of seismic parameters (hazard level (Z), Importance factor (I), soil type).
- It has been attempted to integrate performance-based design criteria in conventional force-based design approach using this formulation.

8.4 Limitations of the study

- The findings from this study are at present limited to certain types of RC building frames designed according to IS codes and needs to be generalised for all structural building types.
- The confidence in the observations made in this study is limited by the facts that the building plan is regular, and certain deterministic parameters (gravity loads, cross sectional details etc.) are used in this work, although in reality their statistical variations are significant enough, which requires a reliability-based framework for evaluation of structural response.
- Regarding the ground motion sequences, it has been observed that in certain instances, the aftershock or the sequential ground motion also have been found with similar or even higher amplitude and duration (time) than mainshock. Since seismicity of different locations are variable, the confidence regarding the observations arrived in this investigation need to be complemented with adequate reliability study in future research. This facilitates to arrive at an optical combination of sequences in terms of ground motion characteristics (amplitude, frequency and duration) necessary for a particular location.

8.5 Scope for further study

The study can be further extended by means of experimental and probabilistic study incorporating all the uncertainties arising in the seismic analysis and design procedure to evaluate the following:

- ❖ Different structural typologies – structural wall systems, dual systems, flat slab systems
- ❖ Buildings with discontinuous mass and stiffness distributions
- ❖ Buildings in different locations – zones, soils, hilly slopes etc.

References

Al-Ahmar, R., and Al-Samara, M. (2015), "An evaluation of the seismic response modification factor R for RC special moment-resisting frame system", *International Journal of Structural Engineering*, *6*(4), 368-387.

Arslan, M. H., Ceylan, M., Kaltakci, Y. M., Ozbay, Y., and Gulay, F. G. (2007), "Prediction of force reduction factor (R) of prefabricated industrial buildings using neural networks", *Structural Engineering and Mechanics*, 27(2), 117-134.

ASCE 41–13 (2013), *Seismic evaluation and retrofit of existing buildings*. American Society of Civil Engineers, Reston, USA.

ASCE 7–10 (2010), *Minimum design loads for buildings and other structures*, American Society of Civil Engineers; Reston, USA.

ATC-19 (1995), *Structural response modification factors*, Applied Technology Council; Redwood, USA.

Badal, P. S., and Sinha, R. (2020), "A framework to incorporate probabilistic performance in force-based seismic design of RC buildings as per Indian Standards", *Journal of Earthquake Engineering*, 1-28.

Barbat, A. H., Pujades, L. G., and Lantada, N. (2008), "Seismic damage evaluation in urban areas using the capacity spectrum method: application to Barcelona", *Soil Dynamics and Earthquake Engineering*, 28(10-11), 851-865.

Bhosale, A.S., Davis, R. and Sarkar, P. (2017), "Vertical irregularity of buildings: Regularity index versus seismic risk", *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering*, 3(3), 04017001.

Bhosale, A., Davis, R. and Sarkar, P. (2018), "New Seismic Vulnerability Index for Vertically Irregular Buildings", *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering*, 4(3), 04018022.

Burton, H. and Deierlein, G. (2014), "Simulation of seismic collapse in nonductile reinforced concrete frame buildings with masonry infills", *Journal of Structural Engineering*, 140(8), A4014016.

Cavaleri, L. and Di Trapani, F. (2014), "Cyclic response of masonry infilled RC frames: Experimental results and simplified modeling", *Soil Dynamics and Earthquake Engineering*, 65:224-242.

Ceylan, M., Arslan, M. H., Ceylan, R., Kaltakci, M. Y., and Ozbay, Y. (2010), "A new application area of ANN and ANFIS: determination of earthquake load reduction factor of prefabricated industrial buildings", *Civil Engineering and Environmental Systems*, 27(1), 53-69.

Chaulagain, H., Rodrigues, H., Spacone, E., Guragain, R., Mallik, R. and Varum, H. (2014), "Response reduction factor of irregular RC buildings in Kathmandu valley", *Earthquake Engineering and Engineering Vibration*, 13(3), 455-470.

Choudhury, T., and Kaushik, H. B. (2015), "Vulnerability analysis of buildings for seismic risk assessment: A review", *Bridge and Structural Engineer, The journal of the Indian National Group (ING) of International Association for Bridge and Structural Engineering (IABSE)*, 45(1).

Choudhury, T., and Kaushik, H. B. (2018), "Seismic fragility of open ground storey RC frames with wall openings for vulnerability assessment", *Engineering Structures*, 155, 345-357.

Choudhury, T., and Kaushik, H. B. (2019), "Treatment of uncertainties in seismic fragility assessment of RC frames with masonry infill walls", *Soil Dynamics and Earthquake Engineering*, 126, 105771.

Computer and Structures Inc. 2016, SAP2000, Analysis reference manual, Berkeley, USA.

COSMOS (Consortium of Organizations for Strong Motion Observation Systems) https://strongmotioncenter.org/vdc/scripts/earthquakes.plx.

Dhir, P.K., Davis, R. and Sarkar, P. (2018), "Safety assessment of gravity load–designed reinforced concrete–framed buildings", *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering*, 4(2), 04018004.

Di Sarno, L. (2013), "Effects of multiple earthquakes on inelastic structural response", *Engineering Structures*, 56, 673-681.

Earthquake Disaster Risk Index Report (2019), *A publication of the National Disaster Management Authority, Government of India*, National Disaster Management Authority; New Delhi, India.

ECP-201 (2012), *Egyptian code for calculating loads and forces in structural work and masonry*, Housing and Building National Research Center; Cairo, Egypt.

ECP-203 (2007), *Egyptian code of practice for design and construction of concrete structures*, Research Center for Housing and Construction; Cairo, Egypt.

Eurocode 8 (2004), *Design of structures for earthquake resistance, Part 1: General rules, seismic actions, and rules for buildings*, European Committee for Standardization; Brussels, Belgium.

FEMA 273 (1997), *NEHRP guidelines for the seismic rehabilitation of buildings*, Federal Emergency Management Agency; Washington DC, USA.

FEMA 356 (2000), *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Agency; Washington DC, USA.

FEMA P695 (2009), *Quantification of Building Seismic Performance Factors*, Federal Emergency Managment Agency, Washington, DC, USA.

Haldar, P., and Singh, Y. (2009), "Seismic performance and vulnerability of Indian code designed RC frame buildings", *ISET journal of Earthquake Technology*, *46*(1), 29-45.

Haldar, P., Singh, Y. and Paul, D.K. (2012), "Effect of URM infills on seismic vulnerability of Indian code designed RC frame buildings", *Earthquake Engineering and Engineering Vibration*, 11(2), 233-241.

Haselton, C.B., Baker, J.W., Liel, A. and Deierlein, G.G. (2009), "Accounting for ground-motion spectral shape characteristics in structural collapse assessment through an adjustment for epsilon", *Journal of Structural Engineering*, 137(3), 332-344.

Hatzigeorgiou, G.D. and Beskos, D.E. (2009), "Inelastic displacement ratios for SDOF structures subjected to repeated earthquakes", *Engineering Structures,* 31(11), 2744-2755.

Hatzigeorgiou, G.D. and Liolios, A.A. (2010), "Nonlinear behaviour of RC frames under repeated strong ground motions", *Soil Dynamics and Earthquake Engineering*, 30(10), 1010-1025.

Hatzivassiliou, M. and Hatzigeorgiou, G.D. (2015), "Seismic sequence effects on three-dimensional reinforced concrete buildings", *Soil Dynamics and Earthquake Engineering*, 72, 77-88.

Hosseinpour, F., and Abdelnaby, A. E. (2017), "Fragility curves for RC frames under multiple earthquakes", *Soil Dynamics and Earthquake Engineering*, *98*, 222-234.

Inel, M., and Ozmen, H. B. (2006), "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings", *Engineering structures*, 28(11), 1494-1502.

IS 1893 (2016), *Criteria for earthquake resistance design of structures - Part 1*, Bureau of Indian Standards; New Delhi, India.

IS 456 (2000), *Plain and reinforced concrete - code of practice*, Bureau of Indian Standards; New Delhi, India.

IS 875 (1987), *Code of practice for design loads other than earthquake for buildings and structures—Part I. Dead loads—unit weights of building*, Bureau of Indian Standards, New Delhi, India.

IS 875 (1987), *Design loads (other than earthquake) for buildings and structures—Part II. Imposed loads—code of practice*, Bureau of Indian Standards, New Delhi.

Jalayer, F., and Ebrahimian, H. (2017), "Seismic risk assessment considering cumulative damage due to aftershocks", *Earthquake Engineering & Structural Dynamics*, *46*(3), 369-389.

Kaushik, H.B., Rai, D.C. and Jain, S.K. (2007), "Stress-strain characteristics of clay brick masonry under uniaxial compression", *Journal of Materials in Civil Engineering*, 19(9), 728-739.

Kircher, C. A., Whitman, R. V., and Holmes, W. T. (2006). HAZUS earthquake loss estimation methods. *Natural Hazards Review*, *7*(2), 45-59.

Liu, J., and Zhou, Y. (2012), "Predicting earthquakes: The Mw9. 0 Tohoku Earthquake and historical earthquakes in northeastern Japan", *International Journal of Disaster Risk Science*, *3*(3), 155-162.

Loulelis, D., Hatzigeorgiou, G. D., and Beskos, D. E. (2012), "Moment resisting steel frames under repeated earthquakes", *Earthquake and Structures*, 3(3-4), 231-248.

Mander, J.B., Priestley, M.J. and Park, R. (1988), "Theoretical stress-strain model for confined concrete", *Journal of Structural Engineering*, 114(8), 1804-1826.

Miranda, E., and Bertero, V. V. (1994), "Evaluation of strength reduction factors for earthquake-resistant design", *Earthquake spectra*, 10(2), 357-379.

Mondal, A., Ghosh, S. and Reddy, G.R. (2013), "Performance-based evaluation of the response reduction factor for ductile RC frames", *Engineering Structures*, 56, 1808-1819.

Morfidis, K., and Kostinakis, K. (2017), "The role of masonry infills on the damage response of R/C buildings subjected to seismic sequences", *Engineering Structures*, 131, 459-476.

Mwafy, A. M., and Elnashai, A. S. (2002), "Calibration of force reduction factors of RC buildings", *Journal of Earthquake Engineering*, 6(02), 239-273.

Oinam, R. M., Sugumar, R., and Sahoo, D. R. (2017), "A comparative study of seismic performance of RC frames with masonry infills", *Procedia engineering*, 173, 1784-1791.

Oyguc, R., Toros, C., and Abdelnaby, A. E. (2018), “Seismic behavior of irregular reinforced-concrete structures under multiple earthquake excitations”, *Soil Dynamics and Earthquake Engineering*, *104*, 15-32.

PEER (Pacific Earthquake Engineering Research Centre) Strong motion database. https://ngawest2.berkeley.edu/site.

Sarkar, P., Prasad, A. M., and Menon, D. (2010). “Vertical geometric irregularity in stepped building frames”, *Engineering Structures*, 32(8), 2175-2182.

SeismoSoft SeismoMatch www.seismosoft.com.

Saatcioglu, M. (2013). Structural damage caused by earthquakes. *Encyclopedia of Natural Hazards; Springer: Dordrecht, The Netherlands*.

Surana, M., Singh, Y., and Lang, D. H. (2018), “Floor spectra of inelastic RC frame buildings considering ground motion characteristics”, *Journal of Earthquake Engineering*, *22*(3), 488-519.

Surana, M., Pisode, M., Singh, Y., and Lang, D. H. (2018), “Effect of URM infills on inelastic floor response of RC frame buildings”, *Engineering Structures*, *175*, 861-878.

Takeda, T., Sozen, M.A. and Nielsen, N.N. (1970), “Reinforced concrete response to simulated earthquakes” *Journal of the Structural Division*, 96(12), 2557-2573.

Uva, G., Raffaele, D., Porco, F. and Fiore, A. (2012), “On the role of equivalent strut models in the seismic assessment of infilled RC buildings”, *Engineering Structures*, 42, 83-94.

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

Vamvatsikos, D. and Cornell, C.A. (2004), “Applied incremental dynamic analysis”, *Earthquake Spectra*, 20(2), 523-553.

Varadharajan, S., Sehgal, V.K. and Saini, B. (2012), “Review of different structural irregularities in buildings”, *Journal of Structural Engineering*, 39(5), 393-418.

Varadharajan, S., Sehgal, V.K. and Saini, B. (2013), “Determination of inelastic seismic demands of RC moment resisting setback frames”, *Archives of Civil and Mechanical Engineering*, 13(3), 370-393.

Vona, M. and Mastroberti, M. (2018), “Estimation of the behavior factor of existing RC-MRF buildings”, *Earthquake Engineering and Engineering Vibration*, *17*(1), 191-204.

Yang, F., Wang, G., and Ding, Y. (2019), “Damage demands evaluation of reinforced concrete frame structure subjected to near-fault seismic sequences”, *Natural Hazards*, *97*(2), 841-860.

Zhang, Y., Jun, C. and Chaoxu, S. (2017), “Damage-based strength reduction factor for nonlinear structures subjected to sequence-type ground motions”, *Soil Dynamics and Earthquake Engineering*, 92, 298-311.

Publications

Journals

1). **Oggu, P.**, Pithadiya, M. and Gopikrishna, K. (2019) "Influence of Real Ground Motion Records in Performance Assessment of RC Buildings", ***International Journal of Engineering***, 32(12), 1745-1752. https://dx.doi.org/10.5829/ije.2019.32.12c.07 (ESCI/Scopus indexed)

2). **Oggu, P.**, and Gopikrishna, K. (2020). "Assessment of three-dimensional RC moment-resisting frames under repeated earthquakes", ***Structures*** (Elsevier), 26, 6-23. https://doi.org/10.1016/j.istruc.2020.03.039 (SCI indexed)

3). **Oggu, P.**, Gopikrishna, K., and Sewaiwar, S. (2020) "Seismic assessment of existing gravity load-designed RC framed building: a case study from Warangal, India", ***SN Applied Sciences***, 2, 1-13. https://doi.org/10.1007/s42452-020-2690-7 (Scopus & ESCI indexed)

4). **Oggu, P.**, Gopikrishna, K. and Jha, S. (2020) "Importance of 'DAF' in evaluating structural adequacy of gravity-load designed RC buildings", ***Materials Today: Proceedings,*** 32, 810-818. https://doi.org/10.1016/j.matpr.2020.04.018 (Scopus indexed)

5). **Oggu, P.**, and Gopikrishna, K. (2021) "Effect of sequential earthquakes on evaluation of non-linear response of 3D RC MRFs", ***Earthquakes & Structures***, 20(3), 279-293. https://doi.org/10.12989/eas.2021.20.3.279 (SCI indexed)

6). **Oggu, P.**, Gopikrishna, K., and Nagariya, A. (2021). "Seismic behavior and response reduction factors for concrete moment-resisting frames", ***Bulletin of Earthquake Engineering***, 19, 5643–5663. https://doi.org/10.1007/s10518-021-01184-z (SCI indexed)

Conferences

1). **Oggu, P.,** Kumar, V.K. and Gopikrishna, K. "Seismic fragility analysis of Vertical Setback RC Structures", In the Proceedings of the ***International Conference on the Trends and Recent Advances in Civil Engineering*** – *TRACE 2016),* Amity University, Noida, India, August 11- August 12, 2016.

2). **Oggu, P.** and Gopikrishna, K. "Seismic behaviour and its influence on damage probabilities of irregular R.C. buildings", In the Proceedings of the ***International Conference on Composite Materials and Structures*** *– ICCMS 2017*, IIT Hyderabad, India, 27-29th December 2017.

3). **Oggu, P.** and Gopikrishna, K. "Sensitivity analysis for uncertainty estimation in the seismic performance of reinforced concrete frame structures", *SEC18: Proceedings of the* ***11th Structural Engineering Convention*** *– 2018,* Jadavpur University, Kolkata, India, December 19 - 21, 2018.

4). **Oggu, P.**, Gopikrishna, K. and Jha, S. "Importance of 'DAF' in evaluating structural adequacy of gravity-load designed RC buildings", In the Proceedings of the ***3rd International Conference on Innovative Technologies for Clean and Sustainable Development*** *– ITCSD 2020,* NITTTR Chandigarh, India, 19-21 February 2020.