Effect of Irregular Placement of Infills on Seismic Performance and Fragility of URM Infilled RC Frame Buildings in India

Doctoral Thesis

by

Panna Lal Kurmi

(2016CEZ0002)

DEPARTMENT OF CIVIL ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY ROPAR

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Dedicated to my lovely wife, Priti

INDIAN INSTITUTE OF TECHNOLOGY ROPAR
ROPAR

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled "**Effect of Irregular Placement of Infills on Seismic Performance and Fragility of URM Infilled RC Frame Buildings in India**" in partial fulfilment of the requirements for the award of the Degree of Doctor of Philosophy and submitted in the Department of Civil Engineering of the Indian Institute of Technology Ropar is an authentic record of my own work carried out during a period from December, 2016 to September, 2023 under the supervision of **Dr. Putul Haldar,** Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Ropar.

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other Institute.

(PANNA LAL KURMI)

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

(PUTUL HALDAR)
Supervisor

Date: September 21, 2023

The Ph. D. Viva-Voce Examination of **Mr. Panna Lal Kurmi**, Research Scholar, has been held on **December 21, 2023**

Signature of Supervisor **External Examiner**

Chairman, DC

ABSTRACT

Reinforced Concrete (RC) framed buildings with Un-Reinforced Masonry (URM) infills are the most popular structural systems for multi-storey buildings in India and many other parts of the world. Infills are used as cladding at the exterior periphery walls and as partition in the interior of the building. Presence of regular solid infill between the frames contribute significantly in terms of lateral strength, stiffness, and energy dissipation capacity of the composite frame system, but reduces the fundamental time period, inelastic deformation capacity and thereby altering the failure modes as compared to its bare frame counterpart. These buildings have shown poor performance during past earthquakes and suffered severe damage or collapse, even under moderate earthquakes. Despite significant research effort dedicated to such buildings in the past decades, the understanding of seismic behavior of infilled frames is still not adequate and the uncertainty in infill-frame interaction results in complex modes of failure, rendering the simulation of seismic behavior of infilled frames a challenging task. The complex infill-frame interaction is intensified when functional openings in infills due to presence of doors and windows are introduced. Despite significant research efforts, there is still lack of consensus on role of size, shape, and combined effect of opening on seismic performance and consequent fragility of infilled RC frame buildings.

Owing to continuing urbanization coupled with ever decreasing available space for construction, a sizeable number of the residential buildings in India are being used for mixed occupancy, where the upper storey(s) are used for residential purpose and the ground storey is used for combination of purposes including parking, small to medium commercial use etc. The complex behaviour of infilled frames with functional openings under lateral loading gets further complicated when infills are placed irregularly in plan and/or elevation to maximize the usage of available space. The consequences of poor performance of RC buildings with irregular URM infills observed in past earthquakes have stirred up the concern regarding in depth study of the inelastic behavior of such buildings for short term and long term mitigation policies. Accordingly, a statistical exercise is carried out in this Thesis to classify the existing URM infilled RC building stock and to develop Model Building Types

(MBTs) with prevalent irregular configurations of URM infills and select a representative building plan, based on the analysis of 55 buildings selected from field surveys conducted in Indian cities. Based on the pilot surveys, URM infilled buildings are classified into 7 categories (WD, OGS, EPGS, EPGSIP1, EPGSIP2, EPGSIP3, POGS) depending on type of prevalent infill irregularity at ground storey which are further sub-divided based on the key parameters influencing seismic behaviour of such buildings i.e., framing system, design seismic force levels, detailing of reinforcement and height of buildings; and a total of 14 different MBTs are identified.

This Thesis is an attempt to develop modeling guidelines for URM infills, with and without functional openings, and develop a reliable, cost-effective methodology for seismic performance assessment of practical RC buildings compliant to Indian standards and construction practices. Non-linear Static Pushover analyses are performed to study the explicit effect of realistic combination of different sizes of doors and windows on uniformly infilled RC buildings. It is observed that lateral strength, stiffness, and ductility of uniformly infilled RC building reduce significantly with increasing opening ratio.

This Thesis work further highlights the combined effect of infill irregularity arising from functional and occupational requirement on the overall behaviour of infilled frame buildings under seismic excitation. Effect of functional openings in upper storey(s), due to presence of doors and windows, on seismic performance of one of the most common vertical irregular configurations of URM infills i.e., Open Ground Storey (OGS) building is studied. It is observed that the lateral stiffness and strength of mid-rise OGS buildings designed as per BIS (2002) provision is reduced to 55% and 65% respectively, when functional opening reached to 30%.

Taking a note of the widespread failure of buildings having irregularly placed infills in plan, the seismic behavior of the three most common configurations of RC frames with irregular placement of infills viz. Open Ground Storey (OGS), and Partially Open Ground Storey (POGS), and External Periphery of the Ground Storey without any interior partition walls (EPGS) are studied. To study the seismic performance of buildings with asymmetric placement of infills in the ground storey, Incremental Dynamic Analyses (IDA) are carried out using bi-directional ground motions with a wide range of source and site parameters. It is observed that all the plan irregular infill configurations considered in the study causes premature failure due to damage concentration in the irregular floor. It is further observed that EPGS MBT designed with older Indian seismic standards (BIS 1993, 2002) are comparatively higher vulnerable than the POGS MBT. OGSW building which is not designed for OGS design provision of BIS (2002) shows the lowest capacity and experience the maximum inter storey drift ratio concentration at the first storey level.

Fragility curves of the studied buildings are developed considering all the associated variabilities. Capacity of plan irregular infilled structure against "Collapse" damage state and variability in seismic demand are determined from the results of Incremental Dynamic Analysis. The comparative study of the fragility curves and Damage Probability Matrices (DPMs) suggest that functional openings in URM infills result in significant increase in seismic vulnerability of uniformly infilled RC frames. The vulnerability of infilled frames with functional openings is further increased due to irregular infill configurations in plan. The fragility parameters for Indian RC frame building with irregular configurations of URM infills derived in this Thesis are incorporated in the spreadsheet-based open-source seismic risk evaluation software tool 'SeisVARA' (Haldar et al. 2013).

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NOTATIONS

Symbols	Explanation
a	Equivalent width of the infill panel
A_g	Gross cross-sectional area
A_v	Area of the transverse reinforcement
C_0	Modification factor to relate spectral displacement of an equivalent single degree of freedom (SDOF) system to the roof displacement of the building
C_1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response
C_2	Modification factor to consider the effect of pinched hysteretic shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement
C_m	Effective mass factor to account for higher modal mass participation effects
d	Depth of column
ds	Particular damage state for a given spectral displacement
E_{fe}	Expected modulus of elasticity of frame material (concrete)
E_{me}	Expected modulus of elasticity of infill material
E_c	Modulus of elasticity of concrete
f'_c	Compressive strength of concrete
f_{ck}	Characteristic strength of concrete
f_{yv}	Yield strength of transverse reinforcement
g	Acceleration due to gravity
h_{col}	Column height between centerlines of beams
$h_{\inf}$	Height of infill panel
I_{col}	Moment of inertia of column
I_g	Moment of inertia of gross concrete cross section

Symbols	Explanation
$\overline{IM}$	median IM for collapse damage state
$L_{\inf}$	Length of infill panel
M_b	Nominal flexural strength of beam
M_c	Nominal flexural strength of column
M/V	Largest ratio of moment to shear under design loadings for the column
N	Number of ground motion records
P	Axial load on column
$r_{\inf}$	Diagonal length of infill panel
$R_{\mu d}$	Response reduction factor for ductility
S	Spacing of transverse reinforcement
S_a	Spectral acceleration
$S_{a,avg}$	Average spectral acceleration
S_{au}	Ultimate spectral acceleration
S_{ay}	Yield spectral acceleration
S_d	Spectral displacement
S_{dy}	Yield spectral displacement
S_{du}	Ultimate spectral displacement
S_s	Shear span
$\overline{S}_{d,\,ds}$	Median value of spectral displacement
$t_{\inf}$	Thickness of infill panel and equivalent strut
T	Arithmetic mean of the periods in the fundamental translational modes along the two orthogonal directions of the considered building
T_1	Fundamental period of a structure
T_e	Effective fundamental period
V	Base shear

Symbols	Explanation
V_c	Shear strength of column contributed by concrete
V_n	Shear strength of column
V_s	Shear strength of column provided by the hoops
V_y	Yield base shear corresponding to yield displacement
W	Effective seismic weight
W_i	Weight lumped at i^{th} element
α_m	Modal mass participation factor
β	Dispersion from the median of ground motion records
β_C	Variability associated with capacity curve
β_D	Variability associated with the demand spectrum
β_{ds}	Lognormal standard deviation parameter that describes the total variability of damage state
β_m	Modelling variability
$\beta_{M(ds)}$	Variability associated with the discrete threshold of the damage state
β_{RTR}	log- record-to-record variability
β_T	Total variability
δ_t	Target displacement at roof level
Δ_{eff} / h_{eff}	Drift ratio
Δ_{roof}	Roof displacement
ΔV_{RW}	Strength of infill in the above storey
Γ	Modal participation factor for the pushover mode
ϕ	Standard normal cumulative probability distribution function
ϕ_i	Modal shape coefficient for i^{th} floor
ρ_w	Area of flexural tension reinforcement
μ_d	Ductility

Symbols	Explanation
$\mu_{strength}$	Ratio of elastic strength demand to calculated yield strength
$\sum V_{ED}$	Sum of design lateral force in the storey

ABBREVIATIONS

Symbols	Explanation
ACI	American Concrete Institute
APA	Adaptive Pushover Analysis
ASCE	American Society of Civil Engineers
SEI	Structural Engineering Institute
ATC	Applied Technology Council
BIS	Bureau of Indian Standard
BR1/2/3	Bracings of type 1/2/3
BSC	Bulgarian Seismic Code
CDR	Capacity-Demand Ratio
CP	Collapse Prevention
CPWD	Central Public Works Department
CPWD	Central Public Works Department
DBD	Displacement-Based Design
DBE	Design Basis Earthquake
DM	Damage Measure
DMM	Displacement Modification Method
DOP	Door Openings
DPM	Damage Probability Matrix
DPMs	Damage Probability Matrices
EC	Eurocode
EERI	Earthquake Engineering Research Institute
EL	Equivalent Linearization
EPGA	Effective Peak Ground Acceleration
EPGS	External Periphery of the Ground Storey
FBD	Force-Based Design

Symbols	Explanation
FEMA	Federal Emergency Management Agency
GMPEs	Ground Motion Prediction Equations
IBC	International Building Code
ICC	International Code of Council
IDA	Incremental Dynamic Analysis
IDR	Inter-storey Drift Ratio
IM	Intensity Measure
IO	Immediate Occupancy
ISC	Israel Seismic Code
ISE	Inelastic Spectrum Estimation
LS	Life Safety
MBT	Model Building Type
MCE	Maximum Considered Earthquake
MF	Multiplication Factor
MPA	Modal Pushover Analysis
NBC	National Building Code
NCR	National Capital Region
NDP	Non-linear Dynamic Procedure
NSP	Non-linear Static Procedure
NZS	Standards New Zealand
NZSEE	New Zealand Society for Earthquake Engineering
OGS	Open Ground Storey
OOGS	OGS building conforming to SCWB but without MF
OGSW	OGS building not conforming BIS2002 OGS design requirement
PBD	Performance-Based Design
PEER	Pacific Earthquake Engineering Research Center
PGA	Peak Ground Acceleration
POGS	Partially Open Ground Storey

Symbols	Explanation
RC	Reinforced Concrete
SCWB	Strong-Column Weak-Beam
SD	Standard Deviation
SDOF	Single Degree of Freedom
SeisVARA	**Sei**smic **V**ulnerability **A**nd **R**isk **A**ssessment
SI	Seismic Code of Israel
SMRF	Special Moment Resisting Frame
SRSS	Square Root of Sum of Squares
TIA	Total Infill Area
TOA	Total Opening Area
UI	Uniform Infills
UI	Uniformly Infilled
URM	Un-Reinforced Masonry
WD	Windows and Doors
WIDF	Weak-Infill Ductile-Frame
WOP	Windows Opening

Chapter 1
Introduction

1.1 Background

Un-Reinforced Masonry (URM) infills are widely utilized for cladding at the exterior periphery walls and partition in the interior of the Reinforced Concrete (RC) frame buildings in India and across the world. URM infills are typically constructed by adjoining bricks with cement mortar, make them inhomogeneous in nature and exhibit complex behaviour due to interaction with surrounding RC frames during seismic events resulting in highly unpredictable failure mechanism (Smith 1967; Pauley and Priestley 1992; Mehrabi et al. 1996). However, URM infill is the most preferred partition material due to its availability in local region, effective thermal, moisture, and acoustic insulation properties, ease of construction, and cost effectiveness.

It has been experienced from past earthquakes (Jain et al. 2002; Murty et al. 2006; Johansson et al. 2007; Goda et al. 2015), laboratory experiments (Dhanasekhar and Page 1986; Mehrabi et al. 1996; Buonopane and White 1999; Al-Chaar et al. 2002; Cavaleri and Di Trapani 2014; Basha and Kaushik 2016), and analytical studies (Mehrabi and Shing 1997; Chrysostomou et al. 2002; Asteris 2003; El-Dakhakhni et al. 2003; Fardis and Panagiotakos 2007; Asteris et al. 2011; Chrysostomou and Asteris 2012; Haldar et al. 2013) that presence of infills in the RC frames have significant influence on the overall seismic response and collapse mechanism of the infilled frame structure. Infills interact with the adjacent RC frame members, increases its lateral strength and stiffness, reduces deformability and fundamental time period leading to alteration of seismic demand as compared to its bare frame counterpart (Fardis and Panagiotakos 1997; Dolšek and Fajfar 2008a; Chrysostomou and Asteris 2012; Haldar 2013; Asteris et al. 2015; Kurmi and Haldar 2022). Unfortunately, the complex bounding frame-infill interaction, stiffness and strength contribution of infills to the structural system is ignored in general, during structural analysis and design by the practicing engineers owing to associated modeling complexities, uncertainty in degree of infill-frame interaction and lack of proper guidelines in national design standards (Bulgarian seismic code 1987; NBC-201 1995; SI-413 1995; BIS 2002; Eurocode-8 2004).

The complex infill-frame interaction is intensified when functional openings in infills are introduced due to presence of doors and windows (Demetrios and Karayannis 2007; Kakaletsis and Karayannis 2008; Mondal and Jain 2008; Mohammadi and Nikfar 2013; Yekrangnia and Asteris 2020) causing undesirable seismic behavior leading to extensive damage to bounding RC frame and infills (Ozturkoglu et al. 2017; Repapis and Zeris 2019). The complex behavior of infilled frames with functional openings under lateral loading gets further complicated when infills are placed irregularly in plan and/or elevation to maximize the usage of available space.

(a) (b)

(c)

Fig. 1.1 Typical RC buildings with irregular placement of infills to serve the purpose of (a) vehicle parking only; (b) commercial stores only; and (c) combination of both vehicle parking and commercial stores at the ground storey while upper storey(s) used for residential purpose

One of the most common vertical irregular configurations of URM infills is Open Ground Storey (OGS) building (Fig. 1.1 (a)) in which infills are completely absent in the ground storey. OGS buildings has always remained vulnerable to earthquakes due to irregular distribution of storey strength and stiffness at the ground storey and performed poorly during seismic events owing to soft-weak storey mechanism formation (DolŠEk and Fajfar 2008b; Haldar et al. 2016; Mazza et al. 2018; Pavel and Carale 2019; Borsaikia et al. 2021; Das et al. 2023; Lal and Remanan 2023) and suffered severe damage to complete collapse in past earthquakes (Jain et al. 2002; Mayorca and Leon 2007; Sharma et al. 2013; Goda et al. 2015).

In order to avoid severe consequences of poor performance of OGS buildings, International Building Code ICC IBC (2012), NZSEE (2006), ASCE/SEI 7 (2010) prohibit extremely irregular buildings in seismically active areas. However, to allow the functional advantage of the open storey for all practical purposes, compensation of storey stiffness and strength deficiency is essential at the design stage. Considering the widespread failure of open ground storey buildings (Fig. 1.2 (a)) during Bhuj earthquake of January 26, 2001, the Indian seismic design standard have been revised in 2002 (BIS 2002) included an amendment requiring the beams and columns of the open ground storey to be designed for 2.5 times the design base shear for corresponding uniformly infilled frame buildings.

(a) Bhuj, India (Jain et al. 2002) (b) L'Aquila, Italy (Gattulli et al. 2013)

Fig 1.2 Damage to RC buildings due to the open ground storey

Several other national standards like Bulgarian seismic code (1987), Israel seismic code (SI-413 1995); Eurocode-8 (2004) have suggested the use of multiplication factors to increase the design force in the open and even adjacent storey members. Moreover, in the revised Indian seismic design standard (BIS 2016a), the OGS design provision of BIS (2002) have been removed and open storey strengthening measures like RC structural wall and bracings have been recommended in the selected open bay and left to the intelligence of designer in-charge. However, the efficacy of these OGS design interventions needs to be examined in order to check its feasibility for practical application, effectiveness on the seismic performance, as well as ease of OGS building design for all practical purposes. Moreover, several important aspects of controlling parameters related to seismic hazard, design, and detailing have been modified, removed or introduced in the latest revision of Indian seismic design standards (BIS 2016b, 2016a).

A sizeable number of the residential buildings in India are also being used for mixed occupancy, where the ground storey is used for commercial purpose (Fig. 1.1 (b)) or both commercial and parking (Fig. 1.1 (c)); and the upper storey(s) for residential purpose. The increasing urbanization in country like India with ever decreasing available land for construction compelling for buildings with irregular configuration of infills not only in elevation in the form of OGS but also irregularity exists in plan to encompass the occupational and functional demand together. The commercial usage demands for larger free spaces without partitions, and open front and/or sides. However, RC buildings with irregular URM infills in plan alter displacement and ductility demand resulting in high damage indices and worse seismic behavior leading unacceptable collapse mechanism such as excessive torsion under earthquake and consequential premature failure through the flexible side of building as observed in past earthquakes (Fig. 1.3).

(a) Sikkim earthquake (Sharma et al. 2013)

(b) Peru earthquake (Mayorca and Leon 2007)

Fig. 1.3 Collapse of ground storey on the flexible side due to irregular placement of infills

The widespread failure of URM infilled RC frame buildings and consequent extensive physical and social losses, during the 2001 Bhuj earthquake, the first large earthquake in India affecting urban areas, highlighted the need for realistic seismic performance and associated fragility assessment of the huge existing stock of such buildings. As systematic data on damage of such buildings during past earthquakes, is lacking, and accurate analytical investigation of these infilled RC buildings need reliable structural modeling and analysis in order to formulate collapse risk mitigation guidelines of these huge stock of building typologies from future earthquakes. In spite of intensive research effort of several decades (Ockleston 1955; Klinger and Bertero 1978; Paulay and Priestley 1992; Mehrabi and Shing 1994; Al-Chaar 2002; Asteris et al. 2011) to understand the behavior of RC frames with URM infills, assessing the seismic performance of masonry-infilled RC frames remains a challenging task, because of difficulties in modeling the complex infill-frame interaction along with functional openings. Accurate estimation of nonlinear force-deformation behavior of URM infills being solid panel and with functional openings with governing failure mode, strength and stiffness degradation under lateral loading, make realistic assessment of seismic response of infilled frames even more challenging. The present study has attempted to propose a simplified macro modeling approach for URM infills with and without functional opening based on the result of experimental studies to estimate the in-plane nonlinear response and failure mechanism of infilled RC frames efficiently. Present study also classifies the existing URM infilled RC building stock and developed Model Building Types (MBTs) with prevalent irregular configurations of URM infills for in-depth understanding of their seismic performance and consequent failure mechanism to pave the path for seismic mitigation policies to be undertaken.

Capacity curves and fragility functions have been generated for RC buildings with irregularly placed URM infills in both plan and elevation associated with range of functional openings, through non-linear static and dynamic analysis. The design and detailing levels are selected as per Indian standards (BIS 1993, 2000, 2002, 2016a, 2016b), prevalent in Indian constructions. The present study also examines various OGS design interventions recommended in several national design standards, for a feasible solution to OGS design by practicing engineers with ease in terms of seismic performance.

1.2 Need for the Study

RC buildings with URM infills are leading building typology in urban areas of India. Moreover, with modern occupational and functional demand, infills are placed in irregular manner in both elevation and plan of the building. Knowing the threat of past seismic events, seismic safety and risk mitigation of these buildings in seismically active regions has always gained importance in national and international agendas. The composite behaviour and governing failure mechanism of infilled frames not only dependent on the properties of the frame and infill individually, but also on their relative strength and stiffness, degree of infill frame interaction and is highly sensitive to the irregularity of infills configuration. Therefore, it is essential to classify the existing URM infilled RC building stock and develop Model Building Types (MBTs) with prevalent irregular configurations of URM infills for in-depth understanding of their seismic performance and consequent failure mechanism to pave the path for seismic mitigation policies to be undertaken.

An exhaustive examination of the existing literature highlights a conspicuous gap in the global research landscape, with a scarcity of comprehensive studies addressing the seismic performance and fragility analyses of RC buildings featuring irregularly placed infills. This dearth underscores the imperative for further investigation in order to enhance our understanding of this critical aspect of structural engineering. India have witnessed many past earthquakes (1897 Great Assam earthquake, 1991 Uttarkashi earthquake, 1993 Killari earthquake, 1997 Jabalpur earthquake, 1999 Chamoli earthquake, 2001 Bhuj earthquake, 2005 Kashmir earthquake, 2011 Sikkim earthquake), unfortunately, systematic information on building damage during past earthquakes for development of empirical fragility functions for Indian buildings is not available. Furthermore, the daunting challenge of conducting experimental investigations on full-scale multi-storey buildings is exacerbated by the absence of adequate laboratory facilities within the country, coupled with the prohibitive costs involved. Given these constraints, the adoption of analytical modeling emerges as a pragmatic and resource-efficient alternative for comprehensively understanding the seismic response of such buildings. This approach not only mitigates the logistical challenges associated with experimental studies but also offers a more economically viable and feasible avenue for advancing our knowledge in this critical domain.

This Thesis is an attempt to develop modeling guidelines for URM infills, with and without functional openings, and develop a reliable, cost-effective methodology for seismic performance assessment of practical RC buildings compliant to Indian standards and construction practices. Classification of MBTs based on prevalent irregular configuration of URM infills in RC frame buildings and evaluation of capacity and fragility curves of RC buildings with irregular configuration of URM infills due to occupational demand, along with range of functional openings due to doors and windows. Moreover, feasibility and ease of design for practicing engineers have been examined for OGS RC buildings, considering OGS design intervention of various national design standards around the world.

1.3 Research Objectives

The primary aim of this Thesis is to develop and identify a reliable analytical modeling solution for URM infilled RC buildings, with infills being solid (without opening) and with functional opening for range of doors and windows. The analytical model would be later used to study the effect of irregular configuration of URM infills in plan, elevation or combined in plan and elevation on the seismic performance and associated seismic fragilities of Indian RC buildings.

The specific objectives of the study are as following:

1. To review the available analytical models and selection of a simplified macro model for simulating the effect of infill in URM infilled RC frame building under seismic excitation.

2. Identification of the efficient analytical model to simulate functional openings for realistic assessment of URM infilled RC frame buildings.

3. To study the effect of functional openings due to doors and windows on seismic behaviour of URM infilled RC frame buildings with functional openings.

4. To study the effect of irregular placement of URM infills in elevation viz. Open Ground Storey (OGS) on seismic behaviour of URM infilled RC frame buildings with functional openings.

5. To examine the adequacy of available prescriptive design interventions for Indian RC frame buildings with irregular URM infills in elevation.

6. To classify the existing URM infilled RC building stock and to develop Model Building Types (MBTs) with prevalent irregular configurations of URM infills.

7. To study the seismic behaviour of Indian RC buildings with prevalent irregular configurations of URM infills in plan.

8. To develop fragility functions for representative Indian RC frame buildings with irregular configurations of URM infills.

1.4 Organization of the Thesis

The work reported in the present Thesis is organized in the following chapters:

Chapter-1 briefly describes several complications associated with seismic behavior and failure mechanism of RC frame buildings with irregular configuration of infills. This Chapter also describes the need, objectives and scope of the present study.

Chapter-2 presents a comprehensive review of available elastic and inelastic modeling techniques for RC frame members, solid URM infills, and URM infills with opening. Chapter 2 also discusses the linear static and incremental dynamic analysis procedures adapted in this Thesis work. A simplified macro-model for solid URM infills has been developed based on ASCE-41 (2007) to simulate the in-plane nonlinear response of experimentally tested RC infilled frames. This chapter further identified efficient analytical model to simulate the effect of opening in infills for realistic assessment of URM infilled RC buildings. As the composite behaviour and governing failure mechanism of infilled frames is highly sensitive to the irregularity of infills configurations, therefore, a pilot survey is conducted in Indian cities to classify the existing URM infilled RC building stock and identify Model Building Types (MBTs) with prevalent irregular configurations of URM infills. Based on statistical evaluation of the survey data, a generic building plan is also developed to represent the wide characteristics of existing Indian RC buildings.

Chapter-3 examines the effect of functional openings due to doors and windows on seismic behaviour of uniformly infilled RC frame buildings. Extensive analytical studies has been conducted on a set of mid-rise and high-rise Indian infilled RC frame buildings considering various realistic combinations of openings by varying size and shape of doors and windows suggested in CPWD manual (CPWD 2006) for Indian residential building.

Chapter-4 examines the effect of irregular placement of URM infills in elevation on seismic behaviour of URM infilled RC frame buildings with functional openings. The seismic response and governing failure mechanism of representative Open Ground Storey buildings with varying seismic design levels as per relevant Indian standards (BIS 1993, 2000, 2002).

Chapter-5 evaluates the effectiveness and practicability of various design solutions that are currently accessible and recommended by different national design standards to eliminate strength/stiffness irregularities in OGS structures. The impact of the design recommendations made by various national seismic design standards on sets of mid-rise and high-rise infilled RC frame structures with the OGS has been compared. This Chapter also examines the efficacy of prescriptive design provision of open storey of BIS (2002) to eliminate the strength and stiffness irregularity due to open storey which has been removed from the subsequent revised Indian seismic design standard BIS (2016) and suitable measures to remove strength and stiffness irregularity of the open storey is left to the intelligence of designer in-charge.

Chapter-6 evaluates effect of prevalent plan irregular configurations of URM infills on seismic behaviour of Indian RC buildings with URM infills. The existing URM infilled RC building stock has been classified for this purpose and Model Building Types (MBTs) with prevalent irregular configurations of URM infills have been identified. Incremental Dynamic Analyses (IDA) have been employed on a set of RC buildings compliant to Indian seismic design standards (BIS 1993, 2002, 2016a, 2016b) with varying degree of irregular configuration of URM infills in plan to evaluate seismic performance in terms of dynamic capacity curves and identify consequent prevalent failure mechanisms.

Chapter-7 develops fragility functions for representative Indian RC frame buildings with prevalent irregular configurations of URM infills identified during the pilot survey. Fragility curves have been developed for different damage states using HAZUS methodology (FEMA 1999, 2003, 2006) while collapse fragilities have been estimated as per Haselton et al. (2010). As the various identified MBTs have different dynamic properties, therefore, Damage Probability Matrices (DPMs) have also been estimated in terms of discrete damage probabilities with respect to different values of Effective Peak Ground Acceleration (EPGA), conventionally used in the design codes as zone factor, for the purpose of direct comparison of damage probability of the identified MBTs.

Chapter-8 summarizes the major conclusions drawn from the present study and presents the scope for future research.

Chapter 2
Modeling of URM Infilled RC Frames with Opening

2.1 Introduction

Realistic simulation of inelastic behavior of a structure requires reliable estimation of member properties, such as effective stiffness, strength, ductility, and strength and stiffness degradation under cyclic loading (Kim 2005). This Chapter takes a stock of the available models for estimating various member properties for RC frames, URM infills and opening in infills. A comprehensive review on modeling of stiffness, strength and ductility of RC beams, columns, beam-column joints, URM infills and opening in infills is presented. A simplified macro modeling approach for modeling of URM infills with and without opening have been identified based on the adequacy of simulating the observed behavior during experimental investigation of infilled RC frames. As the composite behaviour and governing failure mechanism of infilled frames is highly sensitive to the irregularity of infills configurations, therefore, a pilot survey is conducted in Indian cities to classify the existing URM infilled RC building stock and identify Model Building Types (MBTs) with prevalent irregular configurations of URM infills. Based on statistical evaluation of the survey data, a generic building plan is also developed to represent the wide characteristics of existing Indian RC buildings. The identified simplified yet efficient analytical models verified with the observed behavior in available experimental investigations and post-earthquake damage reports, have been used in the subsequent Chapters for realistic assessment of seismic performance of URM infilled RC frame buildings with openings.

2.2 Modeling of Stiffness of RC Members

Under seismic loading, RC members are expected to undergo large deformation beyond elastic limit resulting cracking of RC members. In the context of Force-Based Design (FBD) as well as Performance-Based Design (PBD), reliable estimation of effective stiffness of RC member under cyclic loading is a very important and a complex issue to be addressed. In traditional FBD philosophy followed in design standards, dynamic characteristic of the structure (time period, mode shape), design force and spectral acceleration depends on the choice of effective stiffness. Damage (indicated by displacement, inter-storey drifts, and plastic rotations in members) in the structure is largely administrated by the realistic choice of effective stiffness in case of PBD philosophy. The choice of selection of effective stiffness varies highly among the design standards and research community (Kumar and Singh 2010). It is well understood from the available literature that effective stiffness of RC members depends on several factors viz. axial loads (FEMA-356 2000; ASCE-41 2007, 2013, 2017), eccentricity ratio (Mirza 1990; Khuntia and Ghosh 2004), yield strength of longitudinal reinforcement (NZS-3101:Part2 2006; Elwood and Eberhard 2009), bond slip of reinforcement bars (Elwood and Eberhard 2009), and shear span (Mirza 1990; Elwood and Eberhard 2009) of the member. Considering the uncertainty in estimation of effective stiffness of RC members, several national standards (BIS 2002; NZS-1170.5 2004; ASCE/SEI 7 2010) recommend a capping on the design period of buildings, ensuring design for a minimum base shear as a safeguard against unrealistic stiffness estimates. Table 2.1 represents overview of stiffness modeling guidelines for RC members. Eurocode-8 (2004) recommends 50% of gross moment of inertia as effective for RC members. While, ACI 318 (2008); ACI 318 (2014) recommends 35% and 70% of gross moment of inertia for beams and columns, respectively and same has been adopted in revised edition of Indian seismic design standard BIS (2016a).

Table 2.1 Overview of effective stiffness recommended for RC beams and columns

<table>
<tr><th>RC Member</th><th>Eurocode-8 (2004)</th><th>ACI 318 (2008); ACI 318 (2014)</th><th>FEMA-356 (2000)/ASCE-41 (2007)</th><th>ASCE/SEI-41 Supplement-1 (2007); ASCE-41 (2013, 2017)</th><th>BIS (2016a)</th></tr>
<tr><td>Non-prestressed Beam</td><td rowspan="4">$0.5E_cI_g$</td><td>$0.35E_cI_g$</td><td>$0.5E_cI_g$</td><td>$0.3E_cI_g$</td><td>$0.35E_cI_g$</td></tr>
<tr><td>Columns with design gravity loads ≥$0.5A_gf_c'$</td><td rowspan="3">$0.7E_cI_g$</td><td>$0.7E_cI_g$</td><td>$0.7E_cI_g$</td><td rowspan="3">$0.7E_cI_g$</td></tr>
<tr><td>Columns with design gravity loads ≤$0.3A_gf_c'$</td><td>$0.5E_cI_g$</td><td>Linear interpolation</td></tr>
<tr><td>Columns with design gravity loads ≤$0.1A_gf_c'$ or with tension</td><td>-</td><td>$0.3E_cI_g$</td></tr>
</table>

where, E_c is Modulus of elasticity of concrete, $I_g$is moment of inertia of gross concrete section, $A_g$is gross cross-sectional area, f_c' is compressive strength of concrete, M_c and M_b are nominal flexural strength of column and beam, respectively.

ASCE-41 (2007); FEMA-356 (2000) considers effect of axial loads on the effective stiffness of columns, and the same effective stiffness properties are adopted by BIS (2013). ASCE-41 (2013, 2017) adopted effective stiffness properties updated based on the study of Elwood et al. (2007). Haselton (2007) recommended effective stiffness estimation based on the yield point of the component. It can be observed from Table 2.1 that significant variation in effective stiffness recommendation among the national standards can be attributed to experimental database which is used to develop these equations. In this Thesis, effective stiffness adapted in BIS (2016a) has been considered for all the buildings.

2.3 Non-linear Modeling of RC Members

Three different categories are available for non-linear modeling of RC frame members are: (i) Continuum model; (ii) Distributed plasticity model; and (iii) Lumped plasticity model. Continuum model generally consists of finite element modeling of component and follows the individual material constitutive law for concrete and reinforcements. Continuum models do not require the definition of member strength, stiffness and deformation capacities as these effects are captured inherently through material properties. The distributed plasticity model captures some behaviour explicitly such as integration of flexural stress and strain through the cross section and other effects implicitly as a function of confinement. The continuum and distributed plasticity model can effectively capture the cracking of concrete, yielding of reinforcement but unable to capture the strength degradation such as rebar buckling, bond-slip and shear failure, which are more important for collapse assessment of structure.

2.3.1 Lumped Plasticity Model

In lumped plasticity model, inelasticity is concentrated at a predefined point and simulated in the components force-deformation behaviour. It is assumed that yielding takes place only at generalized plastic hinges of zero length, and the member between these hinges is assumed to be linearly elastic. Plastic rotation capacity of member is used to define damage and performance limits. Lumped plasticity models are particularly suitable for analysis of building frames under seismic loading, because plastic action in such structures is usually confined to small lengths at beam and column ends. The lumped plasticity models simplify the computational effort significantly without compromising with accuracy (Chen and Powell 1982; Powell and Chen 1986). Lumped plasticity model has gained popularity due to its simplicity and ability to represent non-linear behaviour of structural components in the framework of performance-based earthquake engineering. The non-linear force-deformation behaviour of structural and non-structural component is represented through back-bone curve and presented in many documents (FEMA-273 1997; FEMA-356 2000; ASCE-41 2007, 2013, 2017). Fig. 2.1 represents back-bone curve to represent force-deformation behaviour of component.

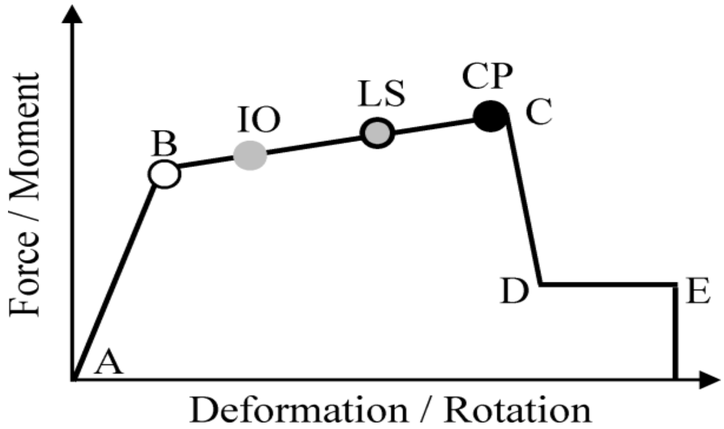


Fig. 2.1 Generalized force-deformation behavior of a typical RC member to define performance limit states under flexure as per ASCE-41 (2017)

The first branch (AB) represents the elastic behaviour of component, and the slope from A to B represents the effective elastic stiffness of the member at yield. It is generally represented by the secant stiffness at first yield (Priestley 2003; Priestley et al. 2007). The second branch (BC) represents post-yield behaviour of component, Point B represents the expected yield strength of the member and until this point, no deformation occurs in the plastic hinge. The expected yield strength is obtained from equivalent bi-linearization of the moment-curvature curve for the RC section (Priestley 2003). The line BC represents strain hardening and the slope from B to C is generally considered such that the ultimate capacity at point C is 0-10% higher than the yield capacity. The line CD represents the initial failure of the component which may occur due to fracture of longitudinal reinforcement, spall of concrete or shear failure. The line DE represents residual strength of member where point E is considered as failure of the member. However, the resistance to the lateral load beyond point C is usually unreliable and ignored. However, the CD branch representing sudden vertical drop of strength is highly unrealistic and may cause numerical instability in non-linear analysis performed in commercial structural analysis program. In order to avoid numerical instability, the strength drop branch (CD) is modified for collapse assessment of structures (PEER/ATC-72-1 2010; FEMA-P58 2011) as shown in Fig. 2.2.

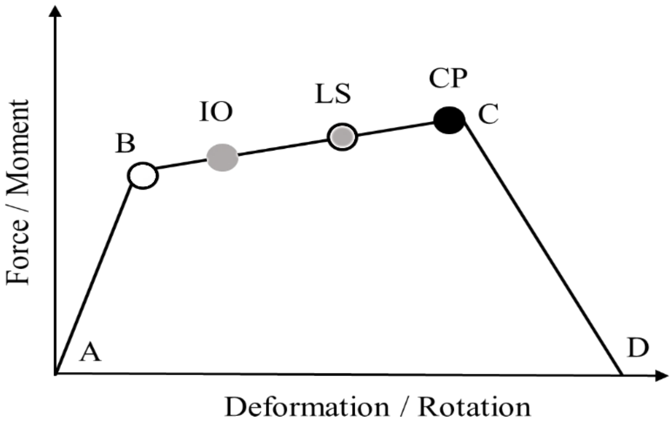


Fig. 2.2 Generalized force-deformation behavior of a typical RC member to define performance limit states under flexure (PEER/ATC-72-1 2010)

In the present study, the flexural capacity of the RC beams and columns has been calculated using section analysis, considering the expected strengths of concrete and steel. The Indian RC design standard (BIS 2000) defines the nominal strength (termed as characteristic strength, f_{ck}) as 95% confidence level cube crushing strength. Therefore, the expected cylinder strength for concrete has been considered as $0.8(f_{ck}+1.64\sigma)$, where, σ is the standard deviation of the cube strength, with values given in BIS (2000). The expected strength of the reinforcing steel has been considered as 1.25 times the nominal or minimum specified strength, according to ASCE-41 (2017) as values of standard deviation in strength of steel manufactured in India are not available. Three performance levels of members, namely Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) is considered in generalized force-deformation behaviour of RC members. The acceptance criteria for plastic rotations corresponding to the three performance levels have been considered as per ASCE-41 (2017) based on design axial and shear forces at the critical section, longitudinal reinforcement ratio, and spacing of transverse (confining) reinforcement. Flexural (M) hinges are assigned at both ends of beams, whereas axial force-bi-axial moment interaction hinges (P-M-M) are assigned to columns.

2.3.2 Modeling of Shear Failure of Columns

It has been witnessed in past earthquakes that column losses its axial load carrying capacity due to shear failure under large lateral deformation and may cause vertical collapse of structure (Bertero and Collins 1973; EERI 1994; Saatcioglu et al. 2001; GSI 2003; Özcebe et al. 2003; Paul et al. 2004). Columns of structure which are not designed for earthquake force and seismic detailing provisions are more prone to shear failure. Shear failure is a brittle in nature and recognised as force-controlled mode of failure (FEMA-273 1997; FEMA-356 2000; ASCE-41 2017).

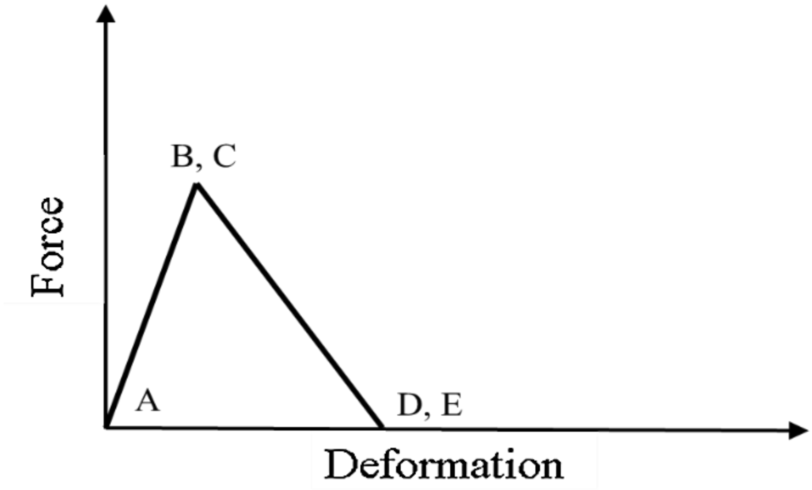


Fig. 2.3 Generalized force-deformation behavior of a typical RC member

Fig. 2.3 represents generalized force-deformation behavior of a typical RC member implying the after reaching shear strength capacity of the member, the strength degrades gradually to zero without any plastic deformation coinciding point B and C.

Contribution of concrete in shear strength is rather complex and is influenced by several factors including axial compressive force, column aspect ratio and deformation ductility demand (Priestley et al. 1994; Sezen and Moehle 2004; Erduran and Yakut 2007). Extensive research on this front over the past decades has revealed that the shear strength (V_n) of a column can be considered to have distinct contributions from concrete (V_c) and transverse reinforcement (V_s). A number of models are available for evaluation of shear strength of RC columns. Table 2.2 summarizes a few of the available models, and in the present study shear strength model recommended by ASCE-41 (2017) has been considered.

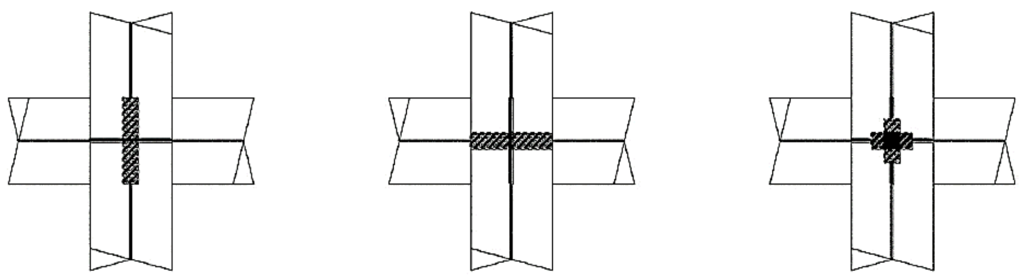
Table 2.2 Overview of shear strength models of RC columns considered in the present study

Model reference	V_c	V_s
FEMA-356 (2000)	$k\left(\frac{6\sqrt{f_c'}}{M/V_d}\sqrt{1+\frac{0.74\,P}{\sqrt{f_c'}\,A_g}}\right)0.8\,A_g$ $2<M/V_d<3$ k = 1.0 for low ductility region and 0.7 for high ductility region	$\frac{A_v\,f_{yv}\,d}{s}$
ACI 352R-02 (2002)	$k\left(\frac{0.5\sqrt{f_c'}}{S_s/d}\sqrt{1+\frac{P}{0.5\sqrt{f_c'}\,A_g}}\right)0.8\,A_g$	$k\frac{A_v\,f_{yv}\,d}{s}$
ACI 318 (2005)	$0.17\left(1+\frac{P}{14\,A_g}\right)\sqrt{f_c'}\,A_g;\ if\ P\ \geq 0$ $0.17\left(1+\frac{29\,P}{A_g}\right)\sqrt{f_c'}\,A_g\ >0\,;\ if\ P\ <0$	$\frac{A_v\,f_{yv}\,d}{s}$ $\leq 0.66\sqrt{f_c'}\,A_g$
Sezen and Moehle (2004)	$\left(1+\frac{3\,P}{A_g f_c'}\right)(0.07+10\,\rho_w)\sqrt{f_c'}\,A_g$ $0.08\sqrt{f_c'}<(0.07+10\,\rho_w)\sqrt{f_c'}\ <0.2\sqrt{f_c'}$	$\frac{A_v\,f_{yv}\,d}{s}$
ASCE-41 (2017)	$k\,\lambda\left(\frac{0.5\sqrt{f_c'}}{M/V_d}\sqrt{1+\frac{P}{0.5\sqrt{f_c'}\,A_g}}\right)0.8\,A_g$ k = 1.0 for ductility less than 2; 0.7 for ductility greater than 6 and varies linearly in between. λ is 0.75 and 1 for light weight and normal weight aggregate concrete	$\alpha\frac{A_v\,f_{yv}\,d}{s}$

where, *M/V* is the largest ratio of moment to shear under design loadings for the column, P is axial load on column, S_s is shear span, d is depth of column, ρ_w is area of flexural tension reinforcement, and A_v, S, and f_{yv} are area, spacing, and yield strength, respectively, of the transverse reinforcement.

2.3.3 Non-linear Modeling of Beam-Column Joints

Seismic response of reinforced concrete beam-column joints is a complex phenomenon.



(a) (b) (c)

Fig. 2.4 Beam-column joint model as per ASCE-41 (2017) for effective stiffness considered in the present study, when ratio of flexural strength of columns and beams framing into joint is: (a) greater than 1.2. (b) less than 0.8, and (c) in between 0.8 and 1.2

A number of design parameters affect the strength, stiffness and deformation capacity, and eventually the damage of the joint (Pagni and Lowes 2004). Several approaches, including lumped plasticity models (Otani 1974), multi-spring models (Biddah and Ghobarah 1999), and finite element simulations (Lowes and Altoontash 2003) have been proposed for modeling of joints in RC frames. The present study mainly concentrates on Special Moment Resisting Frames (SMRF) and joint flexibility is implicitly modeled as per ASCE-41 recommendations. Based on the ratio of flexural strength of beams and columns framing into the joint ASCE/SEI-41 Supplement-1 (2007); ASCE-41 (2017) provides a simple centre line model of beam-column joints with semi-rigid joint offsets, as shown in Fig. 2.4, which accounts for joint shear flexibility and can be very easily implemented in available commercial structural analysis software. Therefore, in the present study, the guidelines of ASCE-41 (2017) for effective stiffness of RC beams, columns and beam-column joints have been considered for their simplicity and reasonable accuracy.

2.4 Modeling of URM Infills

Modeling of infills is an important step in assessment of accurate seismic behaviour of URM infilled RC frame buildings. The fact has been recognized for long, and extensive research on analytical modeling of masonry infills has been carried out since early 1960's due to the difficulties and limitations as well as the high costs associated with the laboratory testing. As simulation of the actual behavior of infilled frame is a complex task, because of infill-frame interaction, many different modeling techniques for the simulation of the infilled frames viz. 'Micro' models and 'Macro' models are available in literature. Micro-models are based on finite element depiction of each infill panel and thus are able to account for the local infill-frame interaction and to capture the behavior in a detailed manner. However, these are computationally very expensive, whereas macro-models are based on physical understanding of the behavior of the infill panel as a whole and therefore are able to simulate the gross behavior of infill efficiently, though approximately.

2.4.1 Micro-Models

The finite element method of modeling infill panels was first suggested by Mallick and Severn (1967) and has been widely adopted since. Infill panels are represented by linear elastic rectangular finite elements with two degrees of freedom at each of the four corner nodes. Different approaches have been used to simulate the interface conditions between infill and frame. In order to represent the interaction between rocks, Goodman et al. (1968) modified a four-nodded plane strain rectangular element of predefined length and zero width such that it has resistance to compressive force and has no resistance to tensile force perpendicular to its length. This concept has been used by many researchers (King and Pandey 1978; Page 1978; Lofti and Shing 1994; Mehrabi and Shing 1994) to simulate infill-frame interaction with refinements over the years. Axley and Bertero (1979) suggested two finite element approaches, exact scheme and constraint scheme, to find the stiffness contribution of infill panel to infill-frame system. Liauw and Kwan (1984) proposed a plastic theory of infilled frame in which the infill-frame system was idealized as either integral, or semi-integral, or non-integral frame, depending on the interface conditions. Rivero and Walker (1984) developed a nonlinear model with reduced degrees of freedom, suitable for dynamic analysis of infilled frames. The model was divided into three parts representing un-cracked elastic behavior of infill panel, infill-frame interface, and cracking in the infill panel. Rots (1991) implemented three basic approaches to model the mechanical properties of masonry infills numerically, using finite element code DIANA (Coenraads 1991). The first approach (one-phase material model) is the least refined, where infills are assumed to be homogeneous material and joints are represented by continuum elements. The one-phase material model was adopted by many researchers (Dhanasekar 1985; Gambarotta and Lagomarsino 1997; Zhuge et al. 1998) to reduce the problem for dynamic analysis. However, local failure of masonry at weak joints cannot be simulated by continuum joint model and therefore, applicability of one-phase material model is limited to the large structures, not requiring detailed stress analysis. In the second approach, based on two-phase material model, masonry units were represented by continuum elements, but joints were represented by dis-continuum elements and separate mechanical properties were assigned for brick and mortar. This refined two-phase material model was first implemented by Page (1978) and followed by many researchers (Ali and Page 1987; Lofti and Shing 1994; Lourenco 1996; Gambarotta and Lagomarsino 1997) over the years. This model was successfully implemented in commercial software ABAQUS (HKS: Hibbitt et al. 1997). In the finest third approach, masonry was assumed as an anisotropic composite where masonry units were represented by continuum elements and mortar joints were modeled with interface elements (Lofti and Shing 1994; Mehrabi and Shing 1994).

This model was further developed to consider an important feature of unreinforced masonry infills, i.e., cracking. The smeared crack model was used by Mosalam et al. (1993) to model overall cracking within an area rather than tracing individual crack. However, Schnobrich (1985) brought out the high sensitivity of this approach to mesh refinement and later Shing et al. (1992) concluded about its incapability in capturing brittle shear failure of infill panel and suggested use of interface elements in the discrete crack approach.

2.4.2 Macro-Models

Extensive experimental investigation has established the fact that non-integral infills under lateral loading separates from surrounding frame at the unloaded corners and behaves as diagonal strut as shown in Fig. 2.5. The load carrying capacity and governing failure modes of infill depend on the mechanical and geometric properties of the infill and the surrounding frame. This observed physical behaviour of infill enabled it represent through a diagonal strut element with appropriate geometric and mechanical characteristics and led to development of diagonal strut model to represent the infill became widely popular as macro-model.

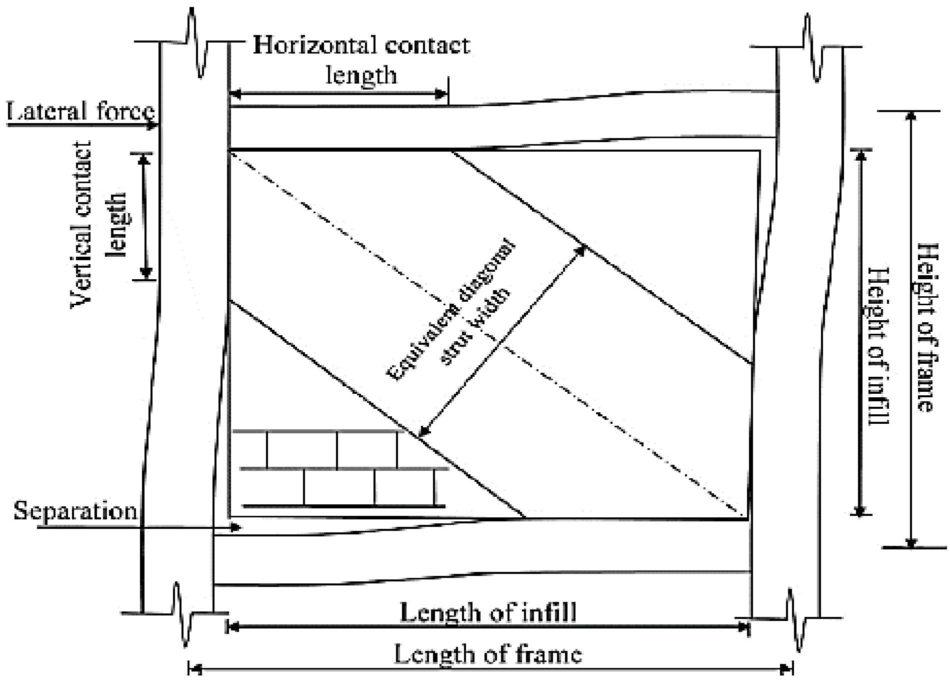


Fig. 2.5 Behaviour of infilled frame under lateral loading (Haldar et al. 2013)

Simplicity of macro-model bank on its ability to represent the behaviour of infill lesser computational effort with sufficient accuracy and preferred for practical structures as its highly unlike micro-modeling of infill which require constitutive relationship for each masonry material, resulting time intensive and computationally complex finite element model barred its applicability to solve the practical design and structural analysis of structures. Based on the diagonal compression behaviour of an infill within a frame system, the idea of a strut model was first presented by Polyakov (1960). In this concept, the global effect of an infill panel is represented by a single or multiple compressive diagonal strut(s) within the frame, having an equivalent width with same thickness same as that of the infill panel. Holmes (1961) proposed width of diagonal strut can be estimated as one third of its diagonal length. Smith (1966) proposed the width of the equivalent diagonal strut as a function of relative stiffness of infill and frame and considered the width approximately as one fourth of diagonal length of infill panel. This concept of diagonal struts was further investigated by many researchers (Smith 1962; Liauw and Kwan 1984; Paulay and Priestley 1992) and a variety of macro-models based on different empirical formulations of diagonal width, strength, and stiffness properties of the strut, were developed over the decades. Mainstone (1974) proposed empirical equation for the calculation of the equivalent strut width on the basis of experimental and analytical data and adopted by FEMA-273 (1997); FEMA-356 (2000). This popular equivalent strut model is used by majority of researchers around the world for analysis of infilled frame because of its simplicity. NZS-4230 (2004) adopted width of infill strut as one fourth of diagonal length for modeling of infills. Paulay and Priestley (1992) pointed out that higher strut width may result stiffer structure and higher seismic response and proposed conservative value of diagonal strut for seismic design of masonry infilled frame. On the basis of parametric finite element studies and empirical fitting of results, expressions of calculation of strut width have been proposed by several other researchers (Bazan and Meli 1980; Liauw and Kwan 1984). Angel (1994) investigated the behavior of RC frames with masonry infills and concluded that the in-plane stiffness can be better approximated using equivalent diagonal strut with a width equal to one eighth of its length. Doudoumis and Mitsopoulou (1986) considered the initial lack of fit between infill and surrounding frame due to shrinkage and proposed a new hysteretic model for equivalent diagonal strut, where the stiffness decreases gradually due to cracking along the compressed diagonal till corner crushing of infills.

Some researchers also raised concern about accuracy on prediction of response quantities in the frame members using single diagonal strut connecting the two loaded corners (Saneinejad and Hobbs 1995; Buonopane and White 1999). Thiruvengadam (1985) proposed multiple strut model of infill panel by considering the reciprocal stiffening effect. The model consists of a moment resisting frame with a number of pin-jointed diagonal and vertical struts. Hamburger and Chakradeo (1993) proposed a rather complicated multi-strut configuration that can account opening in infill. The same configuration is also adopted in FEMA-273 (1997); FEMA-356 (2000) for simulation of perforated infills. Chrysostomou (1991) developed six-strut model of simulation of infill panel that takes both strength and stiffness degradation of infill, an essential development for response assessment of infilled frame under dynamic loading. Madan et al. (1997) have proposed an analytical macro-model that takes into account the strength and stiffness degradation with slip pinching effect incorporating it in nonlinear program IDARC2D for non-linear analysis and damage evaluation of buildings under combined dynamic, static, and quasi-static loading.

Crisafulli (1997) investigated the influence of different multi-strut models on the structural response of infilled RC frame in terms of stiffness and forces induced in the frame. The author concluded that triple strut model is superior in precision, but stiffness may change significantly based on the separation between the struts, single diagonal strut underestimates bending moment in the frame due to truss action, but sufficient to predict overall response. Crisafulli and Carr (2007) proposed four nodded element system with two parallel struts connected with shear spring for accounting compression and shear behaviour of masonry panel. The model can adequately represent the lateral stiffness and strength of masonry panel when shear failure along mortar joints or diagonal tension failure is anticipated. International standards like FEMA-273 (1997); FEMA-356 (2000); ASCE-41 (2007) recommended the diagonal strut model of infill with deformation-controlled action and specified strength and deformation properties. Bed joint sliding shear is considered as the controlling action and drift of the infill as corresponding deformation parameter. The revised Indian seismic design standard BIS (2016a) also adopted modeling of URM infill through single equivalent diagonal concentric strut in structural analysis and design if the structural plan density of the masonry infill exceeds 20% of the building plan.

However, remains silent on estimation of governing infill strength which is essential for non-linear analysis as well as on deformation parameters and back-bone envelope.

2.5 Selection of Simplified Macro-Modeling Approach for Infills

Haldar et al. (2013) demonstrated that efficacy of 1-strut, 2-strut, and 3-strut macro modeling approaches for simulation of infill. The authors have identified that using a single eccentric equivalent diagonal strut, the failure of RC columns observed due to exceedance to shear capacity under the lateral action of infill observed during the experiential studies can be simulated efficiently. Researchers (Stavridis et al. 2017; Bose and Stavridis 2018) have highlighted that strut models are based on case-specific data, and their accuracy in predicting the lateral behavior of infilled frames tends to vary significantly and widely used equivalent strut analogy underestimates the strength and stiffness of the infilled frame. The authors have also highlighted that infill modeling guidelines recommended in ASCE-41 (2007); ASCE-41 (2013) similar to FEMA-273 (1997); FEMA-356 (2000) are not validated with experimental results. In this section, a simplifed macro-model for URM infill is proposed capable of estimating the in-plane resposne of infilled RC frames. The proposed macro modeling approach of solid URM infills follows the guidelines of ASCE-41 (2007) and is capable of predicting the initial stiffness, peak strength, and overall load-deformation behavior of URM infilled RC frame observed during experimental investigations (Mehrabi and Shing 1994; Cavaleri and Di Trapani 2014; Bose and Rai 2016) with higher accuracy as compared to ASCE-41 (2007) model.

2.5.1 Development of URM Infill Model

Macro-model to simulate the solid URM infill prescribed in ASCE-41 (2007) has been modified based on the findings of Haldar et al. (2013), and experimental investigations by Mehrabi et al. (1994), Bose and Rai (2016). The geometry and material properties of all test specimens considered in the present study are summarized in Tables 2.3 and 2.4. The complete set of test specimens of URM infilled RC frames used for experimental investigations by Mehrabi et al. (1994), Bose and Rai (2016) are being analytically modeled using 3D line element for RC members where solid URM infills are being modeled using a pin-jointed eccentric diagonal compressive strut as per ASCE-41 (2007) guidelines.

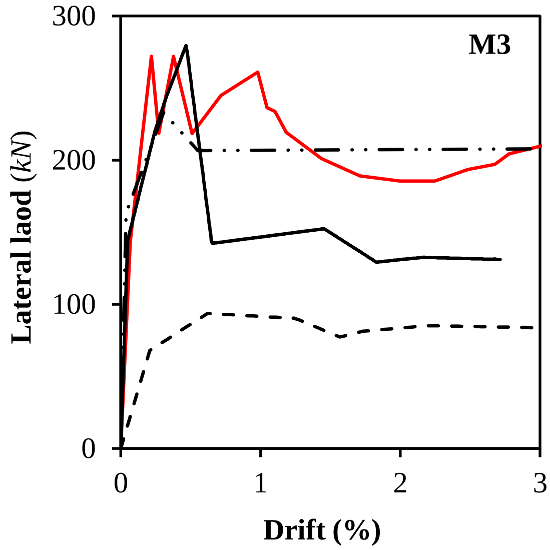
Table 2.3 Geometry of test specimens of URM infilled RC frames (Mehrabi and Shing 1994; Cavaleri and Di Trapani 2014; Bose and Rai 2016)

Specimen	L_w	L_{inf}	h_w	h_{inf}	t_{inf}	A_s	A_v	s	h_c	b_c	b_b	h_b
	mm	*mm*	*mm*	*mm*	*mm*	*mm²*	*mm²*	*mm*	*mm*	*mm*	*mm*	*mm*
M3	2311	2134	1537	1422	92	1013	63	64	178	178	152	229
M4	2311	2134	1537	1422	33	1013	63	64	178	178	152	229
M5	2311	2134	1537	1422	92	1013	63	64	178	178	152	229
M6	2337	2134	1537	1422	33	1583	63	38	203	203	152	229
M7	2337	2134	1537	1422	92	1583	63	38	203	203	152	229
M8	2311	2134	1537	1422	33	1013	63	64	178	178	152	229
M9	2311	2134	1537	1422	92	1013	63	64	178	178	152	229
M10	3124	2946	1537	1422	33	1013	63	64	178	178	152	229
M11	3124	2946	1537	1422	92	1013	63	64	178	178	152	229
M12	3124	2946	1537	1422	92	1013	63	64	178	178	152	229
AAC	2400	2200	1430	1330	125	628	57	50	200	200	200	200
S1A1	1800	1600	1800	1600	210	314	57	100	200	200	200	400

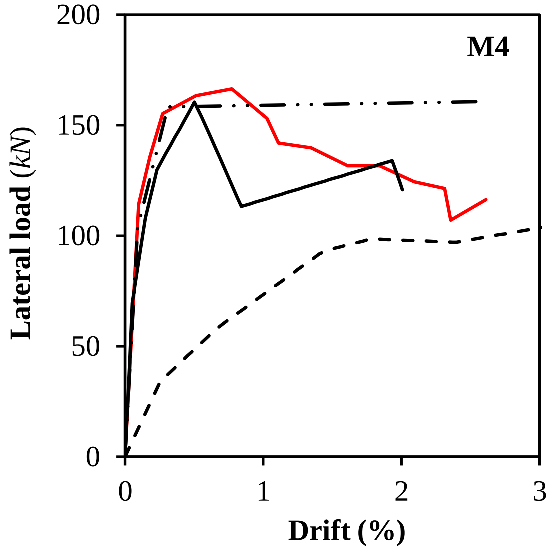
Table 2.4 Material properties and vertical load of test specimens (Mehrabi and Shing 1994; Cavaleri and Di Trapani 2014; Bose and Rai 2016)

Specimen	P_T	f_m	f_c	f_y	f_{yv}	E_m	E_c	E_s	C
	kN	*N/mm²*	*N/mm²*	*N/mm²*	*N/mm²*	*N/mm²*	*N/mm²*	*N/mm²*	
M3	293.6	15.1	30.9	420.6	367.5	9522	21925	199948	0.34
M4	293.6	10.6	26.8	420.6	367.5	4599	17237	199948	0.34
M5	293.6	13.6	20.9	420.6	367.5	8949	18064	199948	0.34
M6	293.6	10.1	25.9	420.6	367.5	4199	19857	199948	0.34
M7	293.6	13.6	33.4	420.6	367.5	9074	18616	199948	0.34
M8	293.6	9.5	26.8	420.6	367.5	5102	17237	199948	0.34
M9	293.6	14.2	26.8	420.6	367.5	8239	17237	199948	0.34
M10	293.6	10.6	26.9	420.6	367.5	3944	20133	199948	0.34
M11	293.6	11.4	25.7	420.6	367.5	9604	18133	199948	0.34
M12	440.4	13.9	26.9	420.6	367.5	7336	20133	199948	0.34
AAC	110	2.38	37.6	417.6	417.6	2400	27600	222000	0.31
S1A1	400	2.67	25	450	450	3933	25500	200000	0.73

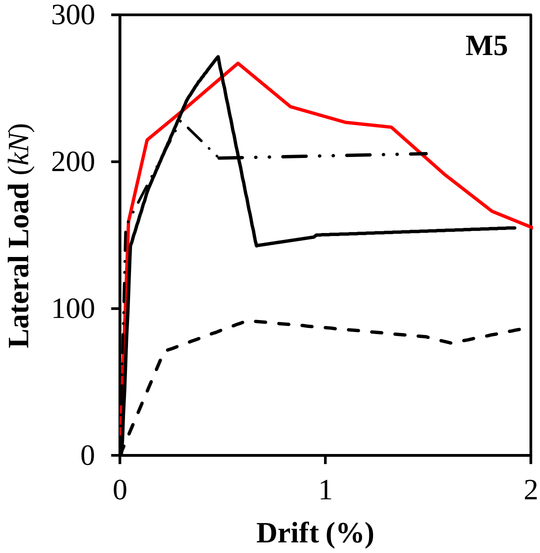
Flexural hinges (M3) and interacting (P-M-M) hinges are assigned at both the ends of beams and columns, respectively, as per ASCE-41 (2017). Axial hinge assigned at the soffit of beam column joint to simulate the possible shear failure of column due to infills. Capacity curves developed through nonlinear static pushover analysis is represented as ASCE-41 (2007) model in Fig. 2.6 and compared with the experimental observations. A closer look at Fig. 2.6 reveals that ASCE-41 (2007) model does not capture the experimental observations and underestimates the initial stiffness, peak strength significantly. In search of an analytical model to represent URM infills with sufficient accuracy, the capacity curves obtained as per ASCE-41 (2007) guidelines are further capped with a factor estimated from the ratio of effective stiffness and peak strength observed from the experimental test specimens to the ASCE-41 (2007). The proposed macro-model is further verified with Martin and Stavridis (2018).



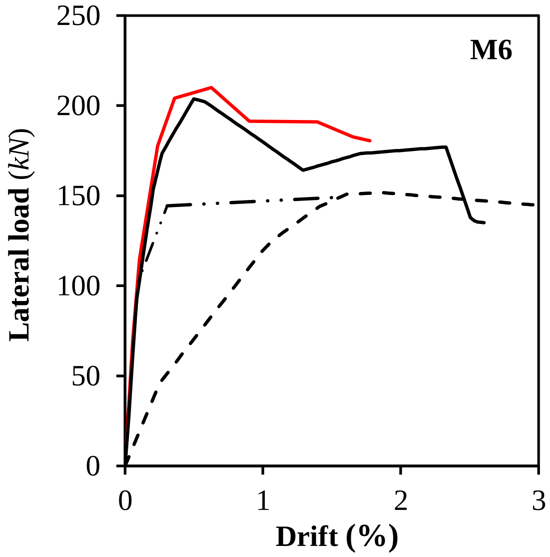
(a) Test specimen M3



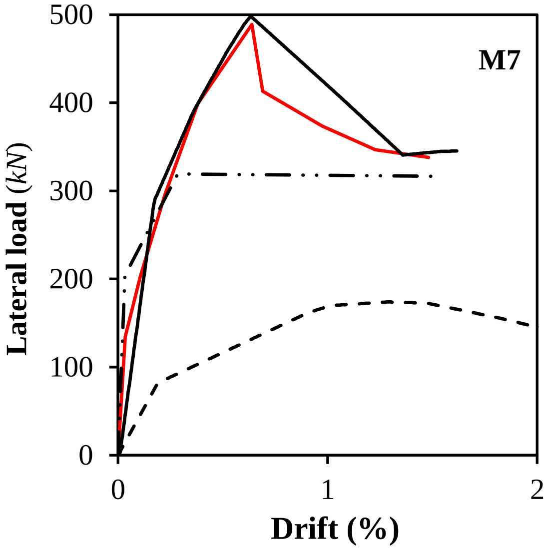
(b) Test specimen M4



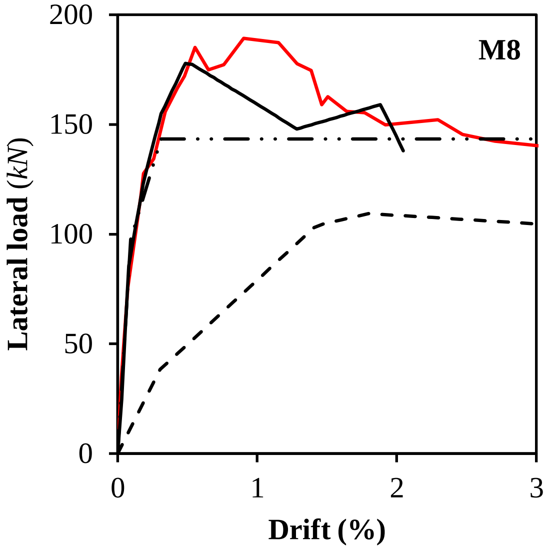
(c) Test specimen M5



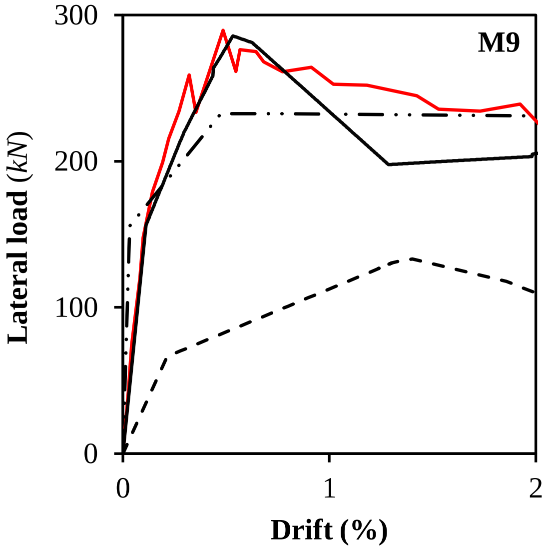
(d) Test specimen M6



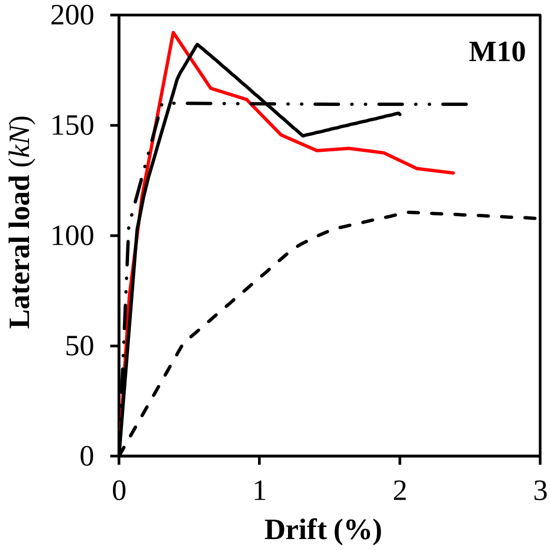
(e) Test specimen M7



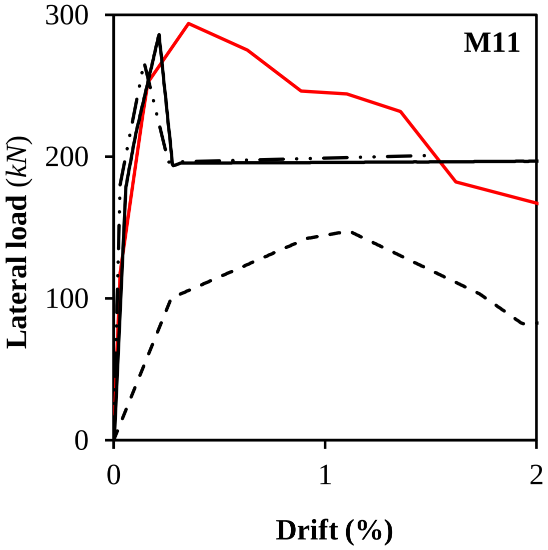
(f) Test specimen M8



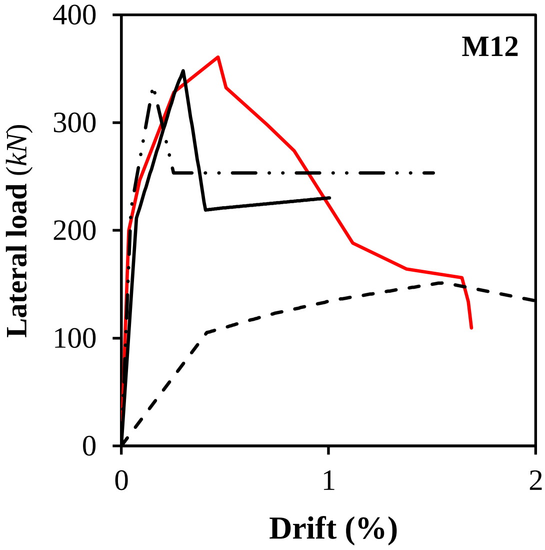
(g) Test specimen M9



(h) Test specimen M10



(i) Test specimen M9



(j) Test specimen M10

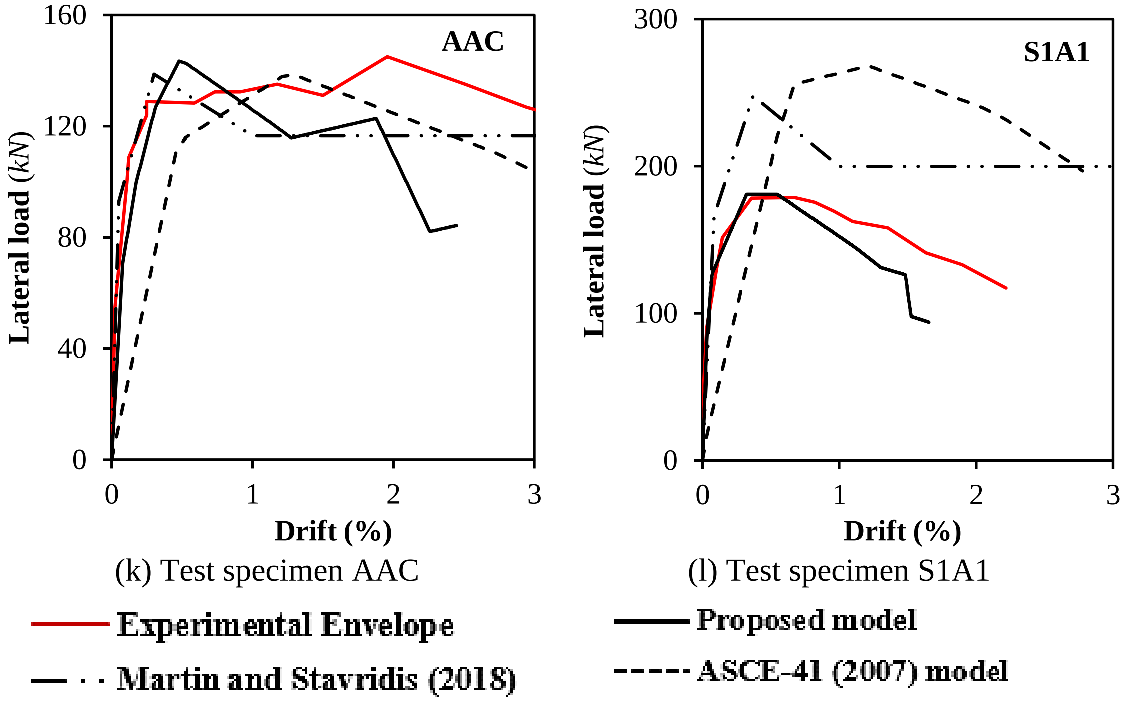


Fig. 2.6 Comparison of the back-bone envelope of experimental load-deformation behaviour, Martin and Stavridis (2018) method, capacity curves obtained using ASCE-41 (2007) infill model and proposed model of URM infill

The adequacy of the proposed model for URM infills in predicting the peak strength of infilled RC frame observed during experimental investigations, and by Martin and Stavridis (2018) is reported in Table 2.5. The proposed model gives the best match of peak strength with experimental results as compared to the analytical method of Martin and Stavridis (2018) and ASCE-41 (2007) model. The failure modes of the proposed model are also in good agreement with the failure mechanism observed during the experimental studies. The specimens M4, M6, M7, M8, and M10 are classified as Weak-Infill Ductile-Frame (WIDF) and hence relatively ductile failure pattern is anticipated indicating sliding along the bed joints distributed along the height of the infill, crushing of the infill near the frame corners, and columns failing in flexure, and no shear failure of column is observed in the test as reported by Mehrabi et al. (1994). The failure mode observed in the proposed model is the failure of diagonal strut due to exceedance and sliding shear capacity of infill, and the formation of plastic hinges at the column ends.

Table 2.5 Comparison of proposed model with experimental data along with Martin and Stavridis (2018) and ASCE-41 (2007) model

Specimen	**Max peak test load (kN)**	**Martin and Stavridis (2018)**	**Infill-frame classification**	**Peak load (kN)**		**Error (%)**		**Failure mechanism***	
				Proposed model	**ASCE -41 (2007)**	**proposed model**	**Martin and Stavridis (2018)**	**Experiment**	**Proposed model**
M3	277.7	236.7	SIWF	279.5	93.6	+0.64	-14.76	B	B
M4	162.4	159.8	WIDF	160.5	105.3	-1.16	-1.6	A	A
M5	266.9	230.4	SIWF	271.5	91.7	+1.69	-13.67	B	B
M6	207.3	152.6	WIDF	203.7	151.7	- 1.73	-26.38	A	A
M7	488.9	313.8	WIDF	498.4	174	+1.9	-35.8	A	A
M8	189.9	143.2	WIDF	177.8	109.4	-6.37	-24.59	A	A
M9	292.5	234.3	SIWF	285.7	133.1	-2.32	-19.89	B	B
M10	191.2	159.8	WIDF	186.7	110.6	-2.35	-16.43	B	B
M11	293.9	277.0	SIWF	286.1	148.2	-2.65	-5.75	B	B
M12	360.8	335.3	SIWF	348.1	151.3	-3.51	-7.06	B	B
AAC	145	139	WIDF	143.4	138.5	-1.1	-4.14	A	A
S1A1	178.7	248	SIWF	180.9	198.9	+1.21	+27.9	B	B

***Failure mechanism A:** This failure mechanism indicates sliding along the bed joints distributed along the height of the infill, crushing of the infill near the frame corners, and columns failing in flexure. No shear failures of columns were observed in the test. The failure mechanism obtained from the proposed model shows the failure of diagonal strut and the formation of plastic hinges at column ends.

***Failure mechanism B:** This failure mechanism indicates failure mode dominated by major shear cracks in the infill and RC columns observed during the test. The failure mechanism obtained from the proposed model shows the failure of the diagonal strut, formation of plastic hinges at column ends, and exceedance of plastic shear capacity at column ends.

Compared to the experimental backbone envelope, the post-peak non-linear response of the proposed model is observed in the conservative side. However, the drift at which the proposed model reaches the peak strength is in fair agreement compared to ASCE-41 (2007).

2.5.2 Modeling of Stiffness of URM Infills

The stiffness of URM infills can be modeled as per widely accepted methodology of ASCE-41 (2007). According to this method, the thickness and modulus of elasticity of the equivalent strut are considered to be the same as those of the infill material, whereas the equivalent width, *a* of the infill panel prior to cracking is defined as

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \tag{2.1}$$

where, $$\lambda_1 = \left[\frac{E_{me} t \sin 2_{inf}}{4E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}}$$

h_{col}= column height between centerlines of beams
h_{inf}= height of infill panel
E_{fe}= expected modulus of elasticity of frame material (concrete)

E_{me}= expected modulus of elasticity of infill material
I_{col}= moment of inertia of column
L_{inf}= length of infill panel
r_{inf}= diagonal length of infill panel
t_{inf}= thickness of infill panel and equivalent strut

2.5.3 Modeling of Strength of URM infills in Various Failure Modes

Extensive experimental and analytical investigation on infilled frames in past that failure of composite infilled frame occurs either due to failure of infill or bounding frame. The usual failure from occurs due to yield formation in beams or columns through due to shearing or tensional actions.

Table 2.6 Overview of identified failure modes of infills

Sl. no.	Reference	Identified failure modes of infill panels			
		Sliding shear failure	Diagonal tension	Diagonal compression	Corner crushing
1	Smith (1967)	○	●	●	○
2	Smith and Carter (1969)	●	●	●	○
3	Mainstone (1971)	○	●	○	●
4	Wood (1978)	●	●	○	●
5	Liauw and Kwan (1984)	○	○	●	●
6	Smith and Coull (1991)	○	○	○	●
7	Priestley and Calvi (1991)	●	●	●	○
8	Paulay and Priestley (1992)	●	○	●	○
9	Saneinejad and Hobbs (1995)	●	●	●	●
10	Flanagan and Bennett (1999)	○	○	○	●
11	Al-Chaar (2002)	●	○	●	○
12	ACI 530 (2005)	●	○	●	●
13	ASCE-41 (2007)	●	○	○	○
14	Basha and Kaushik (2016)	●	○	○	○

○ – Failure mode not considered; ● – Failure mode considered

However, if the frame is suffieciently strong to prevent its failure, the weak infill bounded between the frame fails due to increasing lateral load. Based on the extensive investigations (Smith 1967; Smith and Carter 1969; Mainstone 1971; Liauw and Kwan

1984; Pauley and Priestley 1992; Saneinejad and Hobbs 1995; Mehrabi et al. 1996; Buonopane and White 1999; Fardis et al. 1999; Al-Chaar et al. 2002; Ghosh and Amde 2002; El-Dakhakhni et al. 2003; Basha and Kaushik 2016) four distinct failure modes of the infill panels viz.,bed-joint sliding shear failure, cracking due to diagonal tension, failure due to diagonal compression, and corner crushing of infills, have been identified and several models have been proposed for evaluating strength of the equivalent diagonal strut in these failure modes. Table 2.6 presents an overview of identified failure modes of infills.

Haldar (2013) studied the strength of URM infill under various failure modes for panel of 230 *mm* and 110 *mm* thickness considering fair quality of masonry as per ASCE-41 (2007) with compressive strength (4.1 *MPa*). The strength of infills under distinct failure modes have been estimated using the expressions presented in (Smith 1967; Paulay and Priestley 1992; Saneinejad and Hobbs 1995); ACI-530 (2005); (ASCE-41 2007) and showed that sliding shear govern the failure modes of URM infills. Therefore, sliding shear strength expression given in ASCE-41 (2007) is considered in the Thesis as governing strength of URM infill.

2.5.4 Non-linear Modeling of URM Infills

The shear strength of masonry infill panel is observed to be governing and hence shear strength of URM infill as per ASCE-41 (2007) is considered as a deformation-controlled action and nonlinearity of infills is considered through a generalized force-deformation relationship and acceptance criteria. Fig. 2.7 represents the generalized force-deformation curve according to ASCE-41 (2007), with slight modification in post-peak strength degradation for collapse simulation (Burton and Deierlein 2014). Points B represents yield strength as sliding shear strength, Point C represents ultimate sliding shear strength or upper bound sliding shear strength with a factor of 1.3 times of sliding shear strength at yield point. Point C to D represents strength degradation which important for collapse simulation (Burton and Deierlein 2014). The various limit states (IO, LS, and CP) are specified by ASCE- 41 in terms of the drift ratio (Δ_{eff}/h_{eff}). In the present study, lumped plasticity models of the infills are used, in which axial plastic hinges with strength capacity as described has been assigned at mid-length of the equivalent diagonal struts.

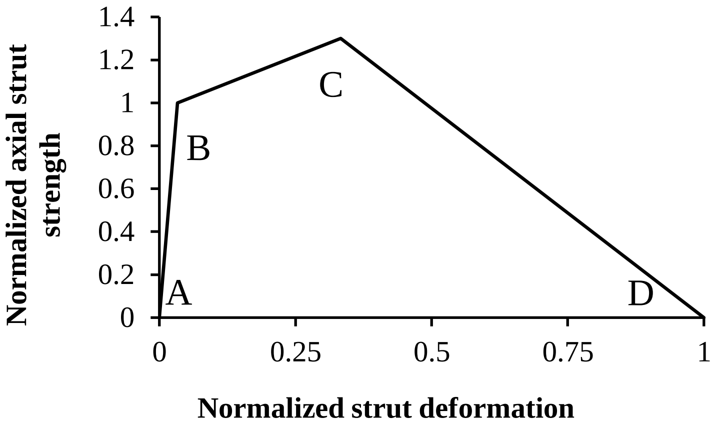


Fig. 2.7 Generalized force-deformation behaviour of masonry infill as per ASCE-41 (2007) with modification in post-peak strength degradation for collapse simulation as per Burton and Deierlein (2014)

2.6 Modeling of Opening in URM Infills

The presence of opening in infills is inherent particularly in residential structures to account for functional requirement like doors and windows. Presence of openings in infills further complicates the interaction of infill and frame which may results in degraded seismic performance in terms of strength, stiffness, ductility causing premature failure of infill-frame composite panel (Demetrios and Karayannis 2007; Dolšek and Fajfar 2008; Demetrios and Christos 2009; Demetrios 2009; Barnaure and Daniel 2015; Martinelli et al. 2015). Effect of opening reduces with increase in opening area and can be neglected if the opening area in infill exceeds 40% of the total area, the composite frame shall be assumed as bare frame instead of infilled frame (Mondal and Jain 2008; Mohammadi and Nikfar 2013; Decanini et al. 2014).

Various methodologies can be found in literature to simulate the effect of openings viz. finite element modeling (Asteris 2003; Liberatore et al. 2020), reduction factors (Mondal and Jain 2008; Decanini et al. 2014), multiple strut arrangement (FEMA-356 2000; Asteris et al. 2012) in infilled frame under lateral loading. Among which use of multi-strut and reduction factors are the two prevalent methods to simulate the effect of openings. Modelling of opening through arrangement of multiple struts as suggested in FEMA356 (2000); ASCE-41 (2013, 2017) is cumbersome limiting its applicability in design of infilled RC frame buildings for practical purposes.

Based on the location of openings, many researchers depending on their experimental and analytical investigations have proposed reduction factors to be applied on equivalent diagonal strut to incorporate the reduction of lateral stiffness and strength of infill due to opening. Table 2.7 represents chronological overview of reduction factor models proposed by various researchers. The available reduction models have been developed either to simulate reduction in strength (Tasnimi and Mohebkhah 2011; Nwofor 2012; Yekrangnia and Asteris 2020), stiffness (Polyakov SV. 1956; Sachanski S. 1960; Imai H. 1989; Mondal and Jain 2008; Asteris et al. 2012; Rahemi 2014; Cetisli 2015; Yekrangnia and Asteris 2020) or strength and stiffness both (Durrani AJ. 1994; Al-Chaar 2002; NZSEE 2006; Mohammadi and Nikfar 2013; Decanini et al. 2014; Mansouri et al. 2014; Chen and Liu 2015). There are very few design standards NZSEE (2006), FEMA-356 (2000); ASCE-41 (2013, 2017) recommend to model the opening in infill panels. FEMA-356 (2000); ASCE-41 (2013, 2017) suggested modelling of opening through arrangement of multiple struts, which is cumbersome limiting its design in practical infilled RC frame buildings, whereas effect of opening in infill panels is considered through a reduction factor by NZSEE (2006). Although, considering complex infill-frame interaction, and its effect on RC frames under lateral load, revised Indian seismic design standard BIS (2016a) prescribed to model the action of infills using single concentric diagonal strut however, however, effect of opening has been completely neglected. Based on the simplicity and applicability to simulate the reduction in both strength and stiffness of infill panel due to presence of opening, efficacy of five reduction factor models from the available literature viz. Al-Chaar (2002), NZSEE (2006), Mohammadi and Nikfar (2013), Decanini et al. (2014), Mansouri et al. (2014) have been evaluated in the present study. The selected five reduction factors models account for reduction in both strength and stiffness of the infill panel and reasonably simple for the application in practical structures.

In order to study the efficacy of the considered reduction factor models which can simulate the response of infilled RC frame with openings, an analytical study has been carried out on ten single bay single storey infilled RC frames with different opening shapes and location having openings ranging from 0% (fully infilled) to 40% of the infill panel area for which experimental results have been reported by Demetrios and Karayannis (2007); Demetrios and Christos (2009).

Table 2.7 Overview of reduction factors for diagonal strut proposed by various researchers

Reference	Empirical equations	Opening type
Polyakov SV. (1956)	$R_k = 1 - (1.155 \frac{h_0}{h_p} + 0.385 \frac{A_0}{A_p})$	Central
Sachanski S. (1960)	$R_k = 1 - (0.4 \frac{l_0}{l_p} + 0.6 \frac{h_0}{h_p})$	Central
Imai H. (1989)	$R_k = Min\ (1 - \frac{l_0}{l_p}; 1 - 10\ (\frac{A_0}{A_p})^{0.5})$	Central
Durrani AJ. (1994)	$R = 1 - (\frac{A_d}{h_p\, l_p})^2$; $A_d = h_p\, l_p - \frac{[(R_p\, Sin2\theta_p) - R_0\, Sin(\theta_p + \theta_0)]^2}{2\, Sin2\theta_p}$; $R_0 = \sqrt{{l_0}^2 + {h_0}^2}$; $R_p = \sqrt{{h_p}^2 + {l_p}^2}$;	Central
Al-Chaar (2002)	$R = 0.6\ (\frac{A_0}{A_p})^2 - 1.6 \left(\frac{A_0}{A_p}\right) + 1$	Central
NZSEE (2006)	$R = 1 - 1.5 \frac{l_0}{l_p}$	Central
Mondal and Jain (2008)	$R_k = 1 - 2.6 \frac{A_0}{A_p}$	Central
Tasnimi and Mohebkhah (2011)	$R_s = 1.49\ (\frac{A_0}{A_p})^2 - 2.238 \left(\frac{A_0}{A_p}\right) + 1$	Central
Nwofor (2012)	$R_s = 0.95\ e^{\frac{A_0}{A_p} 0.03}$	Central
Asteris et al. (2012)	$R_k = 1 - 2\ (\frac{A_0}{A_p})^{0.54} + (\frac{A_0}{A_p})^{1.14}$	Central & Eccentric
Mohammadi and Nikfar (2013)	R_S = -1.085 $(\frac{A_0}{A_p})$ + 1 for $\frac{A_0}{A_p} \leq 0.4$ RC frame R_s = -2.12 $(\frac{A_0}{A_p})$ + 1 for $\frac{A_0}{A_p} \leq 0.25$ steel frame R_k = 1.1859 $(\frac{A_0}{A_p})^2$ -1.6781$\frac{A_0}{A_p}$ + 1 for $\frac{A_0}{A_p} < 0.4$	Central
Rahemi (2014)	$R_k = e^{-\frac{A_0}{A_p} 0.04}$	Central
Decanini et al. (2014)	$R = 0.55e^{-0.035(\frac{A_0}{A_p})} + 0.44e^{-0.025(\frac{l_0}{l_p})}$	Central
Mansouri et al. (2014)	R_k = (1 - 0.31 $\frac{A_0}{A_p}$) (2.78 -1.78 $\frac{d_0}{\sqrt{2h_0 l_0}}$) R_s = (1 – 1.1 $\frac{A_0}{A_p}$) (1.6 - 0.6 $\frac{d_0}{\sqrt{2h_0 l_0}}$) (1 – 0.3 $\frac{x}{l_p}$)	Central & Eccentric
Chen and Liu (2015)	$R = 1 + f\left(\frac{A_0}{A_p}\right) g\left(\frac{x}{L}\right)$ $f\left(\frac{A_0}{A_p}\right) = 2.751\ (\frac{A_0}{A_p})^2 - 3.17 \left(\frac{A_0}{A_p}\right)$ $g\left(\frac{x}{L}\right) = 1 - 1.21 \left(\frac{x}{l_p}\right)$	Central & Eccentric
Cetisli (2015)	$R_k = 1 - 2\ (\frac{A_0}{A_p})^{0.5 k_1 k_2} + (\frac{A_0}{A_p})^{k_1 k_2}$ $K_1 = 1 - 0.4\ (\frac{l_p}{h_p})$, K_2 = 0.2 for corner and 1 for other	Corner
Yekrangnia and Asteris (2020)	$R_s = 1 - [0.45\ \lambda_l h + 0.60] \frac{l_0\, A_0}{l_p\, A_p}$	Central

The analytical study of the considered ten RC infilled frames with openings have been carried out using structural analysis program SAP2000 (2020) under the reported conditions of the test frame and material properties (Demetrios and Karayannis 2007; Demetrios 2009). RC members have been modeled using 3D line elements, and section designer is used for design of members. Infills have been modeled using eccentric single equivalent strut as per ASCE-41 (2007) and openings are simulated using the selected five reduction factor models by Al-Chaar (2002); NZSEE (2006); Mohammadi and Nikfar (2013); Decanini et al. (2014); Mansouri et al. (2014). For nonlinear analysis, concentrated lumped plastic hinges for RC beams and columns were assigned as per ASCE-41 (2017). It can be observed from Table 2.8 that all the considered reduction factor models are in close agreement with the peak strength observed in experiment. Reduction model by Decanini et al. (2014) closely matches with the experimental observations with the least mean deviation of 4.7%. Although, maximum mean deviation of peak strength is quite low, 11% for reduction factor model of Al-Chaar (2002), however, all other considered models by Al-Chaar (2002); NZSEE (2006); Mohammadi and Nikfar (2013); Mansouri et al. (2014) slightly overestimates the peak strength for all the frames with opening.

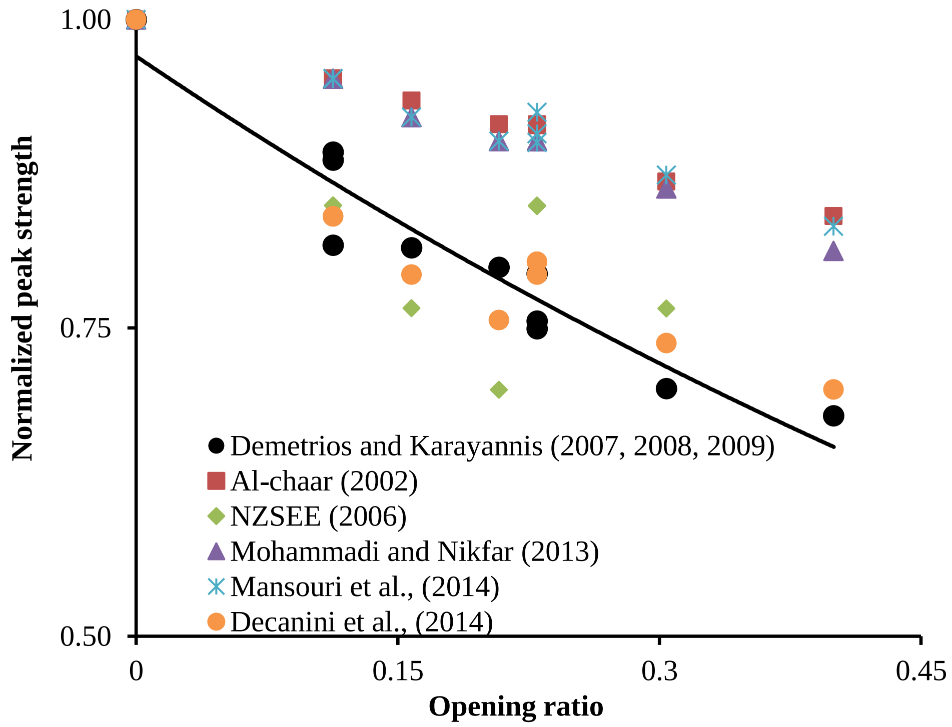


Fig. 2.8 Correlation of peak strength with opening ratio

Fig. 2.8 represents correlation of normalized peak strength (peak strength of infilled frame with openings/peak strength of solid infilled frame) with respect to opening ratio of the infilled RC frame test specimens and estimated peak strength utilizing reduction factor models. A good negative correlation between peak strength and opening ratio can be observed from Fig. 2.8, depicting that lateral load carrying capacity of the composite frame decreases with increasing opening in infills. Reduction factor model proposed by Decanini et al. (2014) has been considered to simulate effect of opening for the further studies in this Thesis.

2.7 Performance-Based Design (PBD)

Performance of structure during seismic event is more related to induced damage in the structure, and Priestley (1993, 2000, 2003) has pointed out that force is a poor indicator of damage. Hence, force cannot be a sole criterion for design. Moreover, in FBD, a flat value of Response Reduction Factor is assumed for a class of buildings is not realistic, because ductility depends on many factors, such as, degree of redundancy, axial force, steel ratio, structural geometry, etc. Therefore, explicit estimation of seismic performance demands a superior design procedure. An alternative design philosophy named “Displacement-Based Design” (DBD) was first introduced by Qi and Moehle (1991) overcoming the limitations of Force-Based Design, which includes translational displacement, rotation, strain, etc. in the basic design criteria to design the structure for a predictable performance. Considerable research effort has been devoted to this topic in the past few decades and different methods have been developed, in which, different deflection parameters are chosen as performance indicators and different techniques are used to proportion the members to achieve the desired performance. Priestley (2000); Priestley et al. (2007) have made significant contribution in developing a practical methodology for Displacement-Based Design. Inter-storey drifts and ductility demand are considered as control parameters for ensuring the desired performance. Authors have specified engineering limit states for different Performance Levels and a draft code on Displacement-Based Design has also been proposed (Priestley et al. 2007). A detailed review of these procedures is beyond the scope of this Thesis, and only the two main approaches are being mentioned here.

Table 2.8 Comparison of experimental and analytically obtained peak strength (*kN*)

Experiments	**Specimen**	**Opening type**	**Open-ing (%)**	**Experimental peak strength** (*kN*)	**Analytical peak strength** (*kN*)				
					Mansouri et al. (2014)	NZSEE (2006)	Decanini et al. (2014)	Al-Chaar (2002)	Mohammadi and Nikfar (2013)
Demetrios and Karayannis (2007); Demetrios and Christos (2009)	Bare	----	-----	44.27	45.28	45.28	45.28	45.28	45.28
	Solid	----	0	81.46	77.53	77.53	77.53	77.53	77.53
	WO2	Central	11.3	66.56	73.82	65.84	65.17	73.83	73.82
	WX1	Eccentric	11.3	72.71	73.82	65.84	65.17	73.83	73.82
	WX2	Eccentric	11.3	72.19	73.82	65.84	65.17	73.83	73.82
	WO3	Central	15.8	66.4	71.39	59.39	61.5	72.45	71.39
	WO4	Central	20.8	65.1	69.89	54.27	58.64	70.96	69.89
	DO2	Central	23	61.5	70.36	65.84	61.5	70.96	69.89
	DX1	Eccentric	23	64.7	61.8	65.84	61.5	70.96	69.89
	DX2	Eccentric	23	61	71.7	65.84	61.5	70.96	69.89
	DO3	Central	30.4	57.1	67.76	59.38	57.2	67.36	66.92
	DO4	Central	40	55.3	64.54	54.29	65.19	65.19	63
	Mean deviation (%)				***10.5***	***6.9***	***4.7***	***11.0***	***9.7***

Another alternative method which became popular in recent times is Performance-Based Design (PBD). PBD methodology for performance evaluation and rehabilitation of existing buildings, documented by ATC-40 (1996); FEMA-273 (1997); FEMA-356 (2000); FEMA-440 (2006); ASCE 41-06 (2007); and ASCE-41 (2017). In PBD, structure is designed to satisfy some target performance objectives in terms of inter-storey drift, inelastic member rotation etc., under a given hazard level. This approach provides building owners and policy makers a framework to decide target performance of objectives based on the importance of the structure. For performance evaluation of designed structure, non-linear analysis is essential and with the development of pushover analysis (ATC-40 1996), performance assessment of structure became very affordable with less computational effort.

2.8 Non-linear Static Analysis Procedure

Performance-based methodology predefines target performance objectives for the structure and in order to achieve the same, estimation of two most important parameters are needed, i.e., capacity of the structure and seismic demand. The capacity of the structure represents its ability to resist earthquake loading and seismic demand is the imposed action on the structure due to earthquake. For accurate estimation of structural response under earthquake loading, Non-linear Dynamic Procedure (NDP) is required, but it requires selection and scaling of appropriate ground motion time histories and large computational effort and resources. On the other hand, Non-linear Static Procedure (NSP) is relatively simple, gives a reasonable insight in to structures non-linear response in terms of overall strength, stiffness, ductility and failure mechanism. Hence, NSP is more preferred from the designer’s point of view because of its simplicity and requires less computational effort.

Non-linear Static Procedure (NSP) or pushover analysis was first introduced by Freeman et al. (1975) in the form of Capacity Spectrum Method and later on it is documented in FEMA-273 (1997); FEMA-356 (2000); FEMA-440 (2006) and ATC-40 (1996). Pushover or capacity curve of a building is the plot between the base shear and roof displacement, under an assumed distribution of lateral vector. The magnitude of the lateral load is increased monotonically, identifying the yield pattern, force demands and weak links in the existing building. It is further documented that this procedure is applicable for buildings having modal mass participation more than 75% along the fundamental mode of vibration in the direction applied lateral load (ATC-40

1996); FEMA-356 (2000). Accuracy of pushover analysis depends on a number of factors including the distribution of lateral load, consideration of higher mode effects (Chopra and Goel 2002), and the procedure used to obtain the performance point. The basic limitation of conventional pushover analysis is its inability to capture higher mode effects and thus making it not applicable to structures dominant by higher modes. To overcome this limitation, Chopra and Goel (2002) proposed concept of Modal Pushover Analysis (MPA), that determines the seismic demand corresponding to first few modes of vibration and combining them with suitable combination rule to get actual seismic demand. Gupta and Kunnath (2000); Kalkan and Kunnath (2006) proposed concept of Adaptive Pushover Analysis (APA) that accounts change in dynamic characteristics of building in each step of loading. However, the simplicity and ease of performing the analysis is lost in MPA and APA procedure, and thereby not becoming as popular as conventional pushover analysis. In this Thesis, pushover analysis is performed with lateral load proportional to fundamental mode of vibration and modal mass participation more than 75% is ensured along the considered directions of the building.

2.9 Inelastic Response Estimation

In the context of performance-based seismic engineering, estimation of inelastic seismic demand of structure is required for design of new structures as well as seismic evaluation and rehabilitation of existing structures. A three-dimensional non-linear time-history analysis can provide best result, however, its associated complexity and cumbersome computation effort, simpler method is required. Such a method is pushover analysis, where inelastic response can be estimated from capacity curve of the structure. Several methods are available to estimate inelastic response of the structure using elastic demand response spectrum, viz. Inelastic Spectrum Estimation (ISE) approach (Veletsos and Newmark 1960; Newmark and Hall 1982; Riddell et al. 1989; Krawinkler and Nassar 1992; Miranda 1993; Vidic et al. 1994; Ordaz and Pérez-Rocha 1998; Miranda 2000; Riddell et al. 2002; Cuesta et al. 2003; Chopra and Chintanapakdee 2004; Ruiz-García and Miranda 2004; Hatzigeorgiou and Beskos 2009), Equivalent Linearization (EL) approach (Rosenbleath and Herrera 1964; Gulkan and Sozen 1974; Iwan 1980; Kowalsky 1994; Grant et al. 2005; Priestley et al. 2007; Pennucci et al. 2011; Xu et al. 2022), Displacement Modification approach (FEMA-356 2000; FEMA-440 2006; ASCE 41-06 2007; ASCE-41 2017), and using

Ground Motion Prediction Equations (GMPEs) for inelastic spectrum (Akkar and Bommer 2007; Rupakhety and Sigbjörnsson 2009; Bozorgnia et al. 2010a; Bozorgnia et al. 2010b).

Inelastic Spectrum approach provides relationships for estimating a 'Response reduction factor for ductility', $R_{\mu d}$ by which the elastic acceleration response spectrum is divided to get the yield acceleration spectrum and the yield displacement spectrum for given ductility, μ_d. Eventually the inelastic displacement is estimated by multiplying the 'Response reduction factor for ductility', $R_{\mu d}$ with the obtained yield displacement. The factor $R_{\mu d}$ depends on factors like ductility, period of vibration (Veletsos and Newmark 1960; Newmark and Hall 1982; Riddell et al. 1989), and site class (Miranda 1993; Chopra and Chintanapakdee 2004; Ruiz-García and Miranda 2004). In Equivalent Linearization approach, inelastic behavior of the structure is represented by an equivalent linear system having an equivalent damping, and equivalent period of vibration. In this approach, the estimated equivalent damping is sensitive to ductility (Rosenbleath and Herrera 1964; Gulkan and Sozen 1974; Iwan 1980; Kowalsky 1994), choice of hysteresis model, and effective period (Kwan and Billington 2003; Grant et al. 2005; Dwairi et al. 2007). FEMA-440 (2006) has presented a comprehensive study on different methods of estimating performance point and has recommended improvements over the original procedures of ATC-40 (1996) and FEMA-273 (1997) known as 'Capacity Spectrum Method' (CSM) and 'Displacement Modification Method' (DMM), respectively. In the 'Displacement Modification Method' the inelastic spectral displacement can be obtained directly from the elastic spectral displacement by multiplying with some prescribed factors. A comparative study of the available methods to estimate inelastic response of structures by Khose and Singh (2012) shows that different methods may yield largely varied results. The study indicates that the DMM is in good agreement with analytically obtained inelastic response of the structure. Accordingly, in the present study the Displacement Modification Method of ASCE-41 (2017) has been preferred. DMM has another advantage over other methods of inelastic response estimation, that it does not require iterations and therefore is more suitable for parametric study. According to DMM the target displacement δ_t at roof level can be obtained as:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{2.2}$$

where, C_0 is modification factor to relate spectral displacement of an equivalent single degree of freedom (SDOF) system to the roof displacement of the building. C_1 is modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 is given as:

$$C_1 = \frac{\mu_{strength} - 1}{a\, T_e^2} \tag{2.3}$$

For effective period greater than 1.0 *sec* the value of C_1 is taken as 1 and for period less than 0.2 *sec* the value of C_1 is considered equal to the value of C_1 at period 0.2 *sec*. T_e is effective fundamental period of the building, the constant a is equal to 130, 90, and 60 for site classes B, C, and D, respectively, and $\mu_{strength}$ is the ratio of elastic strength demand to calculated yield strength, given as:

$$\mu_{strength} = \frac{S_a}{V_y/W}\, C_m \tag{2.4}$$

V_y is the lateral yield strength of the building in the direction under consideration, W is the seismic weight of the building, S_a is the response spectrum acceleration at the effective fundamental period and damping ratio of the building under the considered directions.

C_2 is the modification factor to consider the effect of pinched hysteretic shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response given as:

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{strength} - 1}{T_e}\right)^2 \tag{2.5}$$

2.10 Incremental Dynamic Analysis (IDA)

Significant research in the framework of performance-based earthquake engineering has directed to development of an analytical procedure in which the nonlinear response of structure is evaluated through gradually increasing the intensity of ground motion suits until the structure undergo complete collapse through lateral dynamic instability. This procedure is known as Incremental Dynamic Analysis (IDA), can capture the record-to-record variability in the response of structure. This procedure is first introduced by Vamvatsikos and Cornell (2002) and further developed by (Vamvatsikos and Cornell 2005; Han and Chopra 2006; Vamvatsikos and Cornell 2006; Zarfam and Mofid 2009). Several researchers (Liel et al. 2009; Haselton et al.

2010; Burton and Deierlein 2014; Javanpour and Zarfam 2017; Surana et al. 2018; Song et al. 2023) have carried out IDA for nonlinear response evaluation of structures of over the years. The outcome of the IDA procedure is a curve between the Intensity Measure (IM) (e.g., spectral acceleration (S_a), average spectral acceleration (S_{avg}), Peak Ground Acceleration (PGA), base shear, etc.) and Damage Measure (DM) (spectral displacement, roof displacement, inter-storey drift ratio, etc.). The advantage of IDA is that it addresses both demand and capacity of structures thus enabling thorough understanding of the nature of the structural response as the intensity range of ground motion increases e.g. changes in inter-storey drift, stiffness and strength degradation and their patterns, strength irregularities, considering record to record variability (Vamvatsikos and Cornell 2005).

2.10.1 Selection of Ground Motion Records for IDA

The seismic response of structure is highly sensitive to strong ground motion parameters (e.g., PGA, frequency content, duration), and hence can yield large record-to-record variability. Moreover, these strong ground motion parameters are influenced by magnitude (M_w), source to site distance (R_s), path characteristics and local soil profile thus making it a challenging task to search for the most appropriate intensity measure with smaller dispersion in predicting seismic demand (Maniyar and Khare 2011). The influence of the number of records become important since the standard error of the mean estimate tends to fall with a rate of 1/√N where N is the number of records (Benjamin and Cornell 1970).

Table 2.9 Requirement of the minimum number of ground motion records in some major seismic building design standards

Reference	**Minimum numbers of ground motion requirement**
BIS (2016a)	Appropriate ground motion
Eurocode-8 (2004)	7
NZS-1170.5 (2004)	3
ASCE-7 (2010)	3 and 7 records for using envelope and average response of the scaled selected records, respectively
FEMA-P695 (2009)	28 records for near field and 22 records for far field

Table 2.9 presents an overview of the requirement of minimum number of ground motion records for dynamic analysis, as prescribed in some major seismic building codes (Eurocode-8 2004; NZS-1170.5 2004; FEMA-P695 2009; ASCE-7 2010; BIS 2016a). Based on magnitude and source-site distance, different researchers

have considered different numbers of ground motion records to evaluate the dynamic response of the structures (Vamvatsikos and Cornell 2002; Chopra and Chintanapakdee 2004; Ibarra and Krawinkler 2005; Bandyopadhyay et al. 2023). In the present study, 22 far field ground motion records suit have been considered for IDA as listed in FEMA-P695 (2009). Table 2.10 presents the details of ground motion selected for IDA the present study.

Table 2.10 Details of the ground motion records considered for IDA (PEER 2011)

Sr. No.	Earthquake Name	Year	Recording station	Magnitude (M_w)	Source to site distance (km) (R_s)	PGA (g)
1	Duzce	1999	Bolu, Turkey	7.1	12	0.805
2	Friuli	1976	Tolmezzo, Italy	6.5	15	0.34
3	Hector	1999	Hector, California	7.1	10.4	0.22
4	Imperial Valley	1979	El Centro, California	6.5	12.5	0.367
5	Imperial Valley	1979	Delta, Mexico	6.5	22	0.334
6	Kobe	1995	Morigawachi, Japan	6.9	16.4	0.22
7	Kobe	1995	Shin-Osaka, Japan	6.9	19.1	0.215
8	Kocaeli	1999	Darica, Turkey	7.5	13.6	0.204
9	Kocaeli	1999	Duzce, Turkey	7.5	11.1	0.36
10	Landers	1992	Coolwater, California	7.3	23.6	0.417
11	Landers	1992	Yermo Fire Station, California	7.3	19.7	0.245
12	Loma Prieta	1989	Capitola, California	6.9	17.8	0.511
13	Loma Prieta	1989	Gilroy, California	6.9	12.2	0.526
14	Manjil	1990	Abbar, Iran	7.4		0.485
15	Northridge	1994	Beverly Hills, California	6.7	13.2	0.621
16	Northridge	1994	Canyon County, California	6.7	11.4	0.441
17	San Fernando	1971	Hollywood, California	6.6	22.8	0.225
18	Superstition Hills	1987	El Centro, California	6.5	18.2	0.327
19	Superstition Hills	1987	Poe Road, California	6.5	11.2	0.379
20	Cape Mendocino	1992	Eureka, California	7	17.1	0.149
21	Chi-Chi	1999	Taichung, Taiwan	7.6	10	0.024
22	Chi-Chi	1999	Chiayi, Taiwan	7.6	26	0.177

2.11 Classification of Model Building Types (MBTs)

The composite behaviour and governing failure mechanism of infilled frames not only dependent on the properties of the frame and infill individually, but also on their relative strength and stiffness, degree of infill frame interaction and is highly sensitive to the irregularity of infills configuration. Therefore, it is essential to classify the existing URM infilled RC building stock and develop Model Building Types (MBTs) with prevalent irregular configurations of URM infills for in-depth understanding of their seismic performance and consequent failure mechanism to pave the path for seismic mitigation policies to be undertaken. For reliable assessment of seismic behaviour, the ideal way is to carry out nonlinear analysis of each and every RC building with irregular infills and generate the statistical data to evaluate median and standard deviation to counter various associated uncertainties. Since it is numerically tedious and a time expensive way for dealing with large number of existing building stock, it is not practically feasible. Although, existing Indian buildings have been classified and Model Building Types (MBTs) have been developed by Prasad (2009) for the Indian subcontinent as Adobe and Random Rubble Masonry, Masonry consisting of Rectangular Units, and Framed Structures based on different design levels, floor type and storey height; however, RC buildings with irregular configurations of URM infills are not classified explicitly. To encompass the wide spectrum of irregular infilled Indian RC frame buildings, a scheme has been adapted in the present study. According to the scheme, the infilled RC frame buildings surveyed during a pilot study in Indian cities have been classified based on the prevalent irregular configurations of infills, framing system, design seismic force levels, detailing of reinforcement and height of buildings. Based on a field pilot surveys carried out in urban Indian cities, URM infilled buildings have been classified into 7 categories (WD, OGS, EPGS, EPGSIP1, EPGSIP2, EPGSIP3, POGS) depending on type of prevalent infill irregularity at ground storey which are further sub-divided based on the key parameters influencing seismic behaviour of such buildings i.e., framing system, design seismic force levels, detailing of reinforcement and height of buildings; and a total of 14 different MBTs have been identified. Table 2.11 describes the classification of Model Building Types (MBTs) with irregular configuration of URM infills.

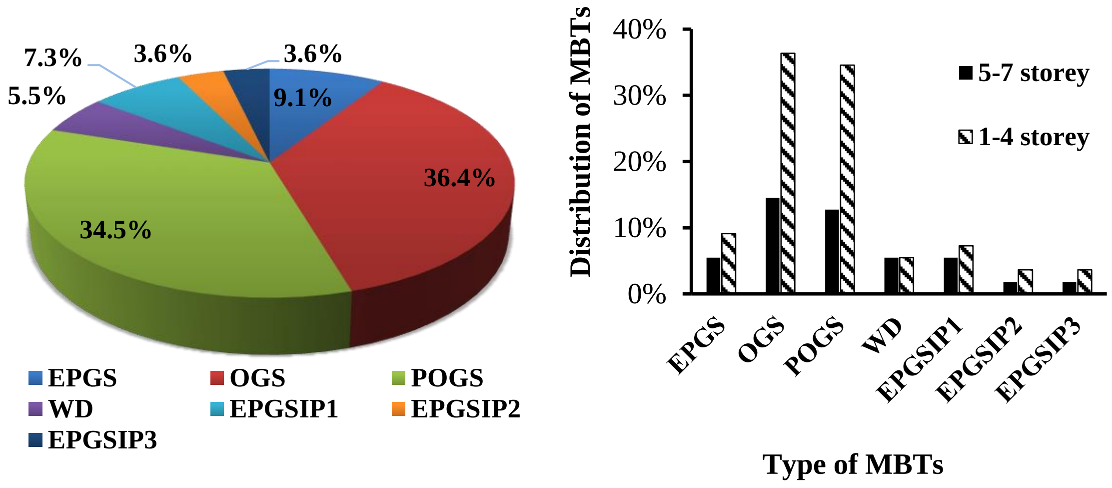
Table 2.11 Classification of Model Building Types (MBTs) with irregular configuration of URM infills

Sr. No.	Model Building Type (MBT)	Framing Type	Design Level	No. of Stories	Description of Infill Irregularity	Structural Configuration of MBT
1	**WD**	RC frames with uniform URM infills including functional openings for doors and windows	Conforming to revised (BIS 2016a, 2016b) and older version (BIS 1993, 2002) of Indian Standards	1-4	Functional openings for **W**indows and **D**oors only	
2				5-7		
3	**OGS**			1 -4	**O**pen **G**round **S**torey including functional openings for doors and windows in upper storey(s)	
4				5 - 7		
5	**EPGS**			1 - 4	Infills only at **E**xternal **P**eriphery of the **G**round **S**torey **without any interior partition** walls including functional openings for doors and windows in upper storey(s)	
6				5 - 7		
7	**EPGSIP**1			1 - 4	Infills at **E**xternal **P**eriphery of the **G**round **S**torey **with I**nterior **P**artition to allocate small commercial stores only including functional openings for doors and windows in upper storey(s)	
8				5 - 7		

Sr. No.	Model Building Type (MBT)	Framing Type	Design Level	No. of Stories	Description of Infill Irregularity	Structural Configuration of MBT
9	**EPGSIP**2			1 - 4	Infills at **E**xternal **P**eriphery of the **G**round **S**torey **with I**nterior **P**artition to allocate small commercial stores only **at front bay** of including functional openings for doors and windows in upper storey(s)	
10				5 - 7		
11	**EPGSIP**3			1 - 4	Infills at **E**xternal **P**eriphery of the **G**round **S**torey with **I**nterior **P**artition **except front bay** to allocate small commercial stores only including functional openings for doors and windows in upper storey(s)	
12				5 - 7		
13	**POGS**			1 - 4	**P**artially **O**pen **G**round **S**torey to allocate space for both commercial stores and vehicle parking including functional openings for doors and windows in upper storey(s)	
14				5 - 7		

Statistical evaluation of the surveyed MBTs in terms percentage of various MBTs and their distribution over mid-rise and high-rise buildings is presented in Fig. 2.9. It is evident from Fig. 2.9 that irregularity of infill configuration is mostly governed by occupational and functional demand that may result in large variation in degree of infill irregularity particularly at the ground floor level. It is further observed that Open Ground Storey (OGS) and Partially Open Ground Storey (POGS) buildings together shares almost 70% of the surveyed MBTs (Fig. 2.9 (a)) as these typologies are mostly preferred in urban areas to serve the purpose of combined parking and commercial spaces.

MBT 'WD' is assigned to the uniformly infilled RC buildings with functional openings due to presence of Windows and Doors only (WD) whereas, RC buildings where infills are placed only at External Periphery of the Ground Storey (EPGS) without any interior partition walls in ground storey including functional openings for doors and windows in upper storey(s) are termed as 'EPGS' MBT. 'EPGS' MBT constitutes almost 10% of the surveyed buildings (Fig. 2.9 (a)).



(a) (b)

Fig. 2.9 (a) Percentage of various MBTs observed in the pilot survey; (b) distribution of various mid-rise (1-4 storey) and high-rise (5-7 storey) identified MBTs

Time period of the building plays a key role in the expected seismic behaviour which is function of building height. Accordingly, identified MBTs are further distributed into mid-rise (1 to 4 storey) and high-rise (5 to 7 storey), presented in Fig. 2.9 (b). It can be noted that the most common infill irregularity viz. OGS (21.82% mid-rise and 14.5% high-rise) and POGS are well distributed both in mid-rise and high-rise buildings. (23.64% mid-rise and 12.7% high-rise). Considering the prevalent MBTs as evident from Fig. 2.9, seismic performance and fragility studies have been performed for WD, OGS, POGS and EPGS MBTs as discussed in the subsequent Chapters (Chapters 3 to 7).

2.12 Summary

A review of available methods for linear and nonlinear modeling of RC frame members has been presented. A literature review has been carried out for modeling of URM infill with and without opening for realistic assessment of infilled RC frame buildings. A simplified macro modeling approach has been identified in order to simulate the in-plane nonlinear response of RC infilled one-bay one-storey experimental specimens. Further, modeling guidelines for URM infills in ASCE-41 (2007) is found to predict the peak strength and failure mechanism of infilled RC frame with reasonable accuracy constructed as per Indian practices, though it under estimates the initial stiffness. However, in absence of proper test data, eccentric single equivalent strut model as per ASCE-41 (2007) can be for assessment of realistic RC infilled buildings. In order to consider the functional openings in practical buildings in the forms of door and window, reduction factor model proposed by Decanini et al. (2014) can be used with sufficient accuracy. To encompass the wide spectrum of irregular infilled Indian RC frame buildings, a field survey has been carried out in Indian cities to develop Model Building Types (MBTs) based prevalent irregular configuration of URM infills in the ground storey.

Chapter 3
Effect of Openings on Seismic Performance of Uniformly Infilled RC Buildings

3.1 Introduction

Functional openings for doors and windows are an integral part of Un-Reinforced Masonry (URM) infills, as they are generally used as external cladding in Reinforced Concrete (RC) frame buildings along with their intended role as partitions in the internal frames. Position and size of these openings are selected based on the functional requirement of the building. It has been experienced from past earthquakes (Jain et al. 2002; Murty et al. 2006), laboratory experiments (Dhanasekhar and Page 1986; Mehrabi et al. 1996; Buonopane and White 1999; Al-Chaar et al. 2002; Cavaleri and Di Trapani 2014; Basha and Kaushik 2016), and analytical studies (Mehrabi and Shing 1997; Chrysostomou et al. 2002; Asteris 2003; El-Dakhakhni et al. 2003; Fardis and Panagiotakos 2007; Asteris et al. 2011; Chrysostomou and Asteris 2012; Asteris et al. 2015b; Kurmi and Haldar 2022) that presence of infills in the RC frames have significant influence on the overall seismic response and collapse mechanism of the infilled frame structure. Infills interact with the adjacent RC frame members, and eventually presence of infill in RC frame increases its lateral strength and stiffness, reduces deformability and fundamental time period leading to alteration of seismic demand as compared to its bare frame counterpart.

In realistic conditions, neither fully infilled frame building exists as it requires openings for doors and windows, nor fully bare frames. A practical infilled frame building comprises of some RC frame bays without opening in infill panel (solid infill), some bays with openings (doors and windows) in infill panel, and some without any infill (bare frame bays). These different types RC frame bay condition arises due to the functional and architectural requirements of the overall buildings. Presence of infills greatly benefits to lateral strength and stiffness of the structure when distributed uniformly over the height of the building (Fardis et al. 1999; Kaushik et al. 2009; Liberatore and Decanini 2011; Favvata et al. 2013; Yuen and Kuang 2015; Haldar et al. 2016; Kurmi and Haldar 2022), and affects the seismic performance if placed in irregular manner in plan and or in elevation (Esteva 1992; Vukazich et al. 2006; DolŠEk and Fajfar 2008b; Haldar et al. 2016). Presence of openings in infills further complicates the interaction of infill and frame which may results in degraded seismic performance in terms of strength, stiffness, ductility causing premature failure of infill-frame composite panel (Demetrios and Karayannis 2007; Dolšek and Fajfar 2008a; Demetrios and Christos 2009; Demetrios 2009; Barnaure and Daniel 2015; Martinelli et al. 2015). Poor performances of infilled frame RC buildings with opening under seismic action has clearly stirred the concern regarding understanding the effect of openings in seismic response of infilled RC frames for realistic simulation of such buildings. Ignoring openings in structural analysis may lead to inaccurate prediction of the lateral stiffness, strength and ductility of the frame, and failure modes of the structure (Demetrios and Karayannis 2007; Demetrios and Christos 2009; Nwofor 2012). Although larger size of openings also exists in real buildings, however, it is been highlighted by many researchers that opening can be neglected if the opening exceeds 40% of the total infill panel area, the composite frame shall be assumed as bare frame instead of infilled frame (Mondal and Jain 2008; Mohammadi and Nikfar 2013; Decanini et al. 2014).

The position and size of the openings have bearing on the overall seismic performance of the RC frame buildings. Several researchers have carried out comparative seismic performance assessment of infilled frames with and without openings, in order to investigate the effect of infill with openings on the overall seismic response of the buildings. Dolšek and Fajfar (2008a) studied the effect of masonry infill on seismic response of 4-storey RC frame, and indicated that full infill configuration significantly increases the strength and stiffness of the composite as compared to partially infilled and bare frame. Barnaure and Daniel (2015) studied the role of central window opening and doors at central and corner in one bay 4-storey infilled RC frame, and concluded that the openings in the infill influence the strength and stiffness particularly infill with central door opening. The window and door openings in the infill lead to the formation of inclined struts with high concentration of stresses near the corners of the openings, lead to failure in the masonry.

Sukrawa (2015) studied the seismic response of 3D RC infilled frame buildings with variable wall openings and indicated that responses of RC frames infilled with walls of opening ratios 20% to 60% are significantly stiffer and stronger as compared to bare frame. Yuen and Kuang (2015) investigated seismic response, and failure mechanisms of infilled RC frame structures with five different infill configurations. The analysis indicated that the degrees of continuity and regularity of the infill panels crucially affect the seismic performance of structures. Infilled frame with doors and windows found to perform worst under the seismic excitation causing extensive damage to bounding RC frame and infill.

Perrone et al. (2017) performed parametric analysis of RC frames designed for gravity loads to investigate the influence of the mechanical and geometrical properties of masonry infills on the whole structural response. The results showed the presence of opening reduces the strength and stiffness of the frame, and increases the elastic period and deformability with respect to fully infilled frame. Ozturkoglu et al. (2017) performed analytical studies of several bare, partially (with openings) and fully infilled RC frame to check the effect of masonry infill walls with openings on nonlinear response of RC frames. They observed that position of partial openings in infill wall significantly affect the lateral stiffness of the frame. In addition, the position and the percentage of the opening are found to be essential parameters reflecting the effect of opening. Choudhury and Kaushik (2018) carried outperformance assessment of open ground storey RC frame with central openings ranging 0% to 100%, and concluded that seismic response of open ground storey buildings remains unaffected by opening in infills. Repapis and Zeris (2019) studied the seismic performance of existing non-conforming Infilled RC buildings, and observed that fully or partially infilled RC frames can perform well, while open ground storey have the worst performance due to the formation of an undesired soft storey mechanism.

Despite significant research efforts, there is still lack of consensus on role of size, shape, and combined effect of opening on seismic performance and consequent fragility of infilled RC frame buildings. Simplified and realistic infill model incorporating openings which can be directly used by practicing design engineers, still needs attention to be paid. Neglecting opening in analytical study of infilled frame building generally leads to overestimating storey strength and stiffness, underestimation of demand drift, and inaccurate estimation of seismic performance and associated fragility. Extensive research efforts over the decades have concluded that under lateral loading, response of infill panel as a compressive diagonal strut get significantly influenced by presence of openings (Durrani AJ. 1994; Demetrios and Karayannis 2007; Demetrios and Christos 2009; Mohammadi and Nikfar 2013; Mansouri et al. 2014). Strength and stiffness of infill panel reduces due to opening, and to account this reduction, many empirical expressions widely popular as reduction factor models as presented in Table 2.7 in Chapter 2. Considering complex infill–frame interaction, and its effect on RC frames under lateral load, the revised Indian seismic design standard BIS (2016a) prescribed to model the action of infills using a single concentric diagonal strut, however, effect of opening has been completely neglected. Moreover, forced based design method adapted by Indian design standard BIS (2016a) does not provide insight into nonlinear force-deformation behaviour of URM infill which is essential to study inelastic behaviour. Although URM infilled RC frame buildings with openings for door and windows are the most common type of building typology observed all over India, modelling of infills considering frame-infill interaction, openings, and nonlinear behaviour in account is a complex, and computationally cumbersome task requiring specialized skill which may not be abundant in design offices. Realistic size of doors and windows in residential buildings have been selected based on the manual of Central Public Works Department (CPWD 2006), Govt. of India. An exhaustive parametric study has been carried out combining different size of doors and windows onsets of mid-rise (4-storey) and high-rise (8-storey) RC buildings in terms of strength, stiffness, ductility, and deformation capacity using nonlinear static analysis. Sets of 26 opening configurations viz. Uniformly Infilled (UI, 0% opening), bare frame (100% opening), and RC frames designed as per Indian standards (BIS 2016a, 2016b) with varying functional openings based on the requirement of doors and windows have been studied.

3.2 Seismic Assessment of Representative Infilled RC Building

To represent the wide spectrum of infilled RC frame buildings in India, the generic building plan have also been selected from the pilot survey conducted. The selection has been based on the statistical evaluation of structural parameters which are expected to affect the seismic response of the buildings such as range of building dimensions (Fig. 3.1 (a)), number of frames in each direction (Fig. 3.1 (b)), time period of vibration (Fig. 3.1 (c)).

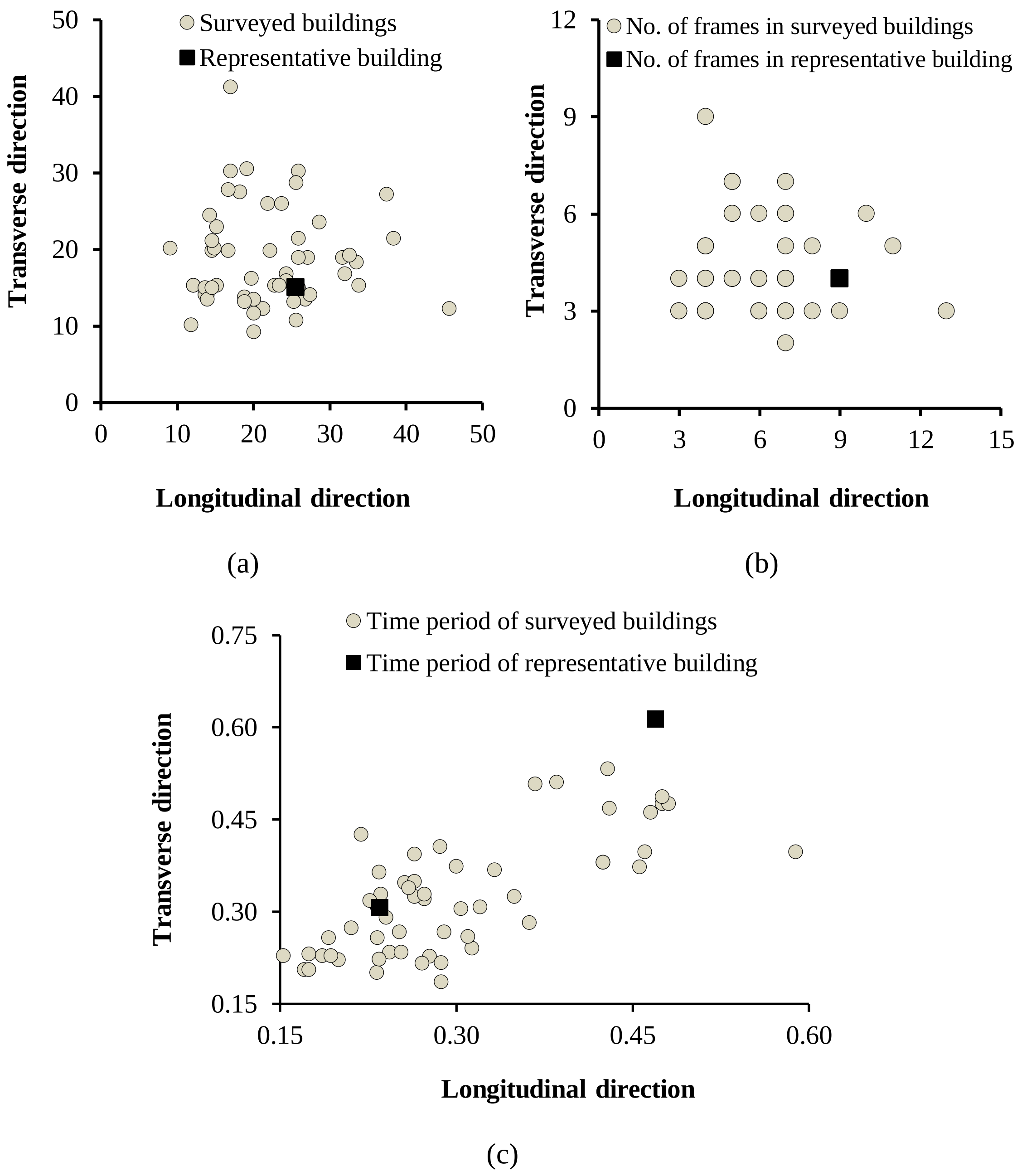
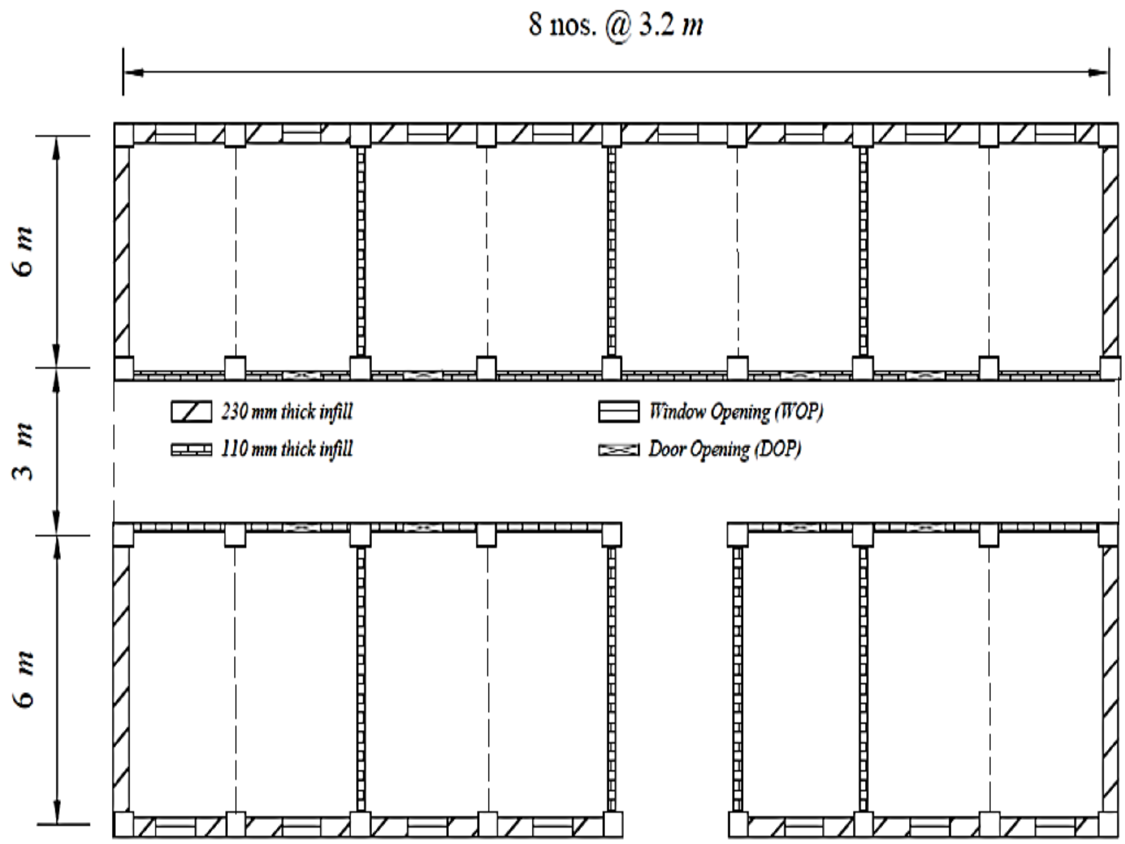
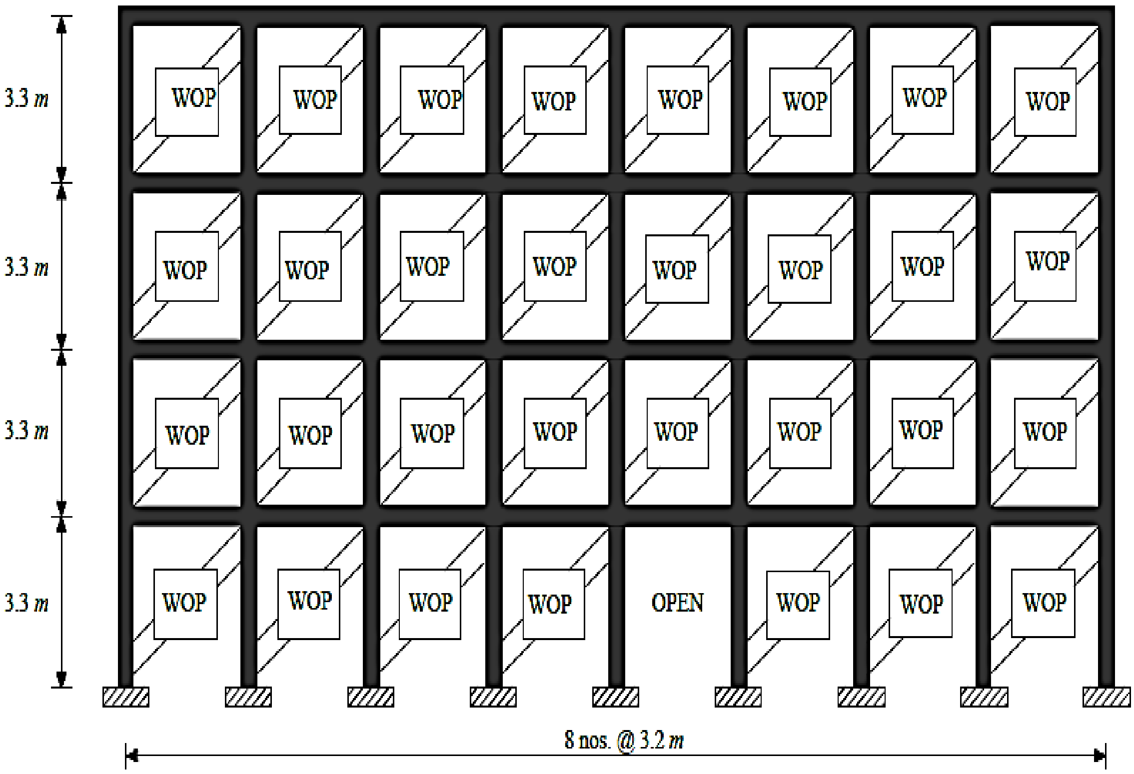


Fig. 3.1 Parameters of surveyed and representative buildings: (a) plan dimensions; (b) number of frames in each direction; (c) time periods of vibration

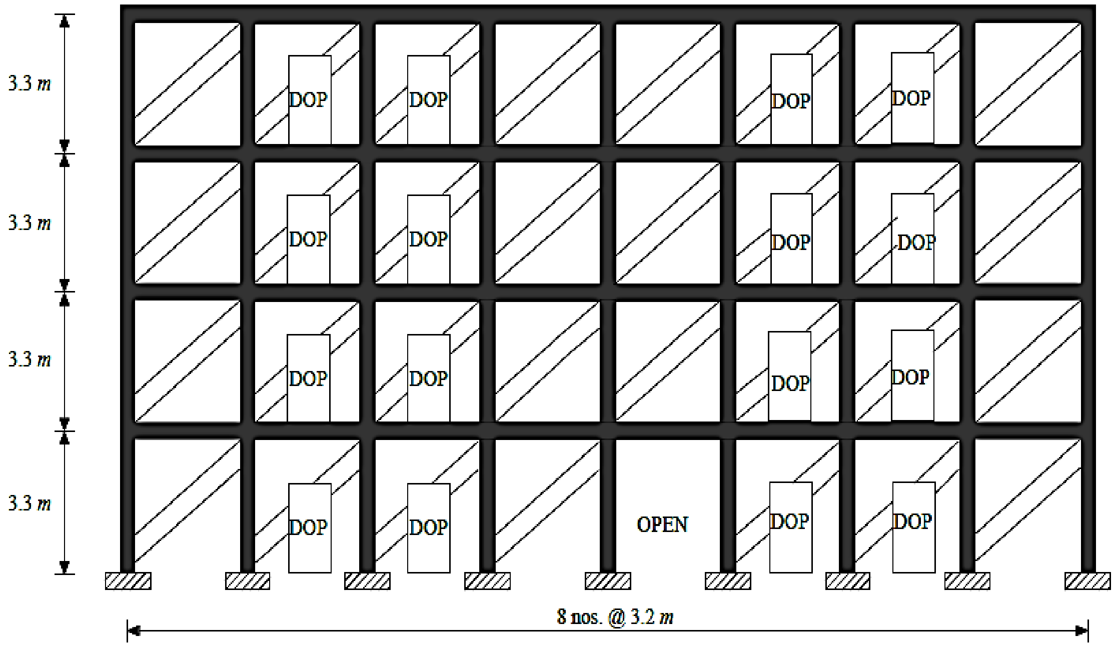
Statistics of the pilot survey indicates that majority of the existing buildings have plan dimensions varying from 15*m* to 35*m*. Accordingly the plan dimensions of the representative buildings have been chosen equal to 15*m* and 25.6*m* in the two directions (Fig. 3.2 (a)) to represent the range of observed dimensions. The parameter of surveyed buildings is fairly comparable to the past pilot survey (DEQ 2009) carried out in the National Capital Region (NCR) of India indicating similarity in building characteristics across the urban areas of India.


8 nos. @ 3.2 m
6 m
3 m
6 m
230 mm thick infill
110 mm thick infill
Window Opening (WOP)
Door Opening (DOP)

(a)


3.3 m
3.3 m
3.3 m
3.3 m
WOP
OPEN
8 nos. @ 3.2 m

(b)



(c)

Fig. 3.2 (a) Plan of the consiered building; (b) External frame elevation with window openings (WOP); and (c) Internal frame elevation with door openings (DOP)

The RC frame buildings with a generic plan and location of Door Openings (DOP) and Windows Openings (WOP) representing wide characteristics of Indian buildings is presented in Fig 3.2 (b) and Fig. 3.2 (c)). All the buildings having generic plan with 4 frames along the longitudinal direction and 9 frames in the transverse direction, respectively. The buildings are assumed to be situated on Type II soil (medium soil) of Indian seismic zone V having effective peak ground acceleration of 0.36*g* as per Indian seismic design standard (BIS 2016a). Present study intends to examine the role of infills being present fully, partially (with openings) and being completely absent (bare frame) on the seismic performance and associated fragility of overall RC buildings as a whole. Therefore, the design base shear is kept same for all the considered buildings having same height with common empirical expression for design period of RC frames with URM infills BIS (2016a). Table 3.1 summarizes the design and modelling parameters for the considered buildings.

To cover the wide spectrum of size and configuration of openings in Indian residential buildings, CPWD manual (CPWD 2006) guidelines for doors and windows in residential buildings have been considered for the parametric study. CPWD (2006) has recommended 15 windows and 6 door sizes by varying the length and height of openings which eventually arrive at variation of window from 6% to 32%, and door opening sizes 20% to 33%.

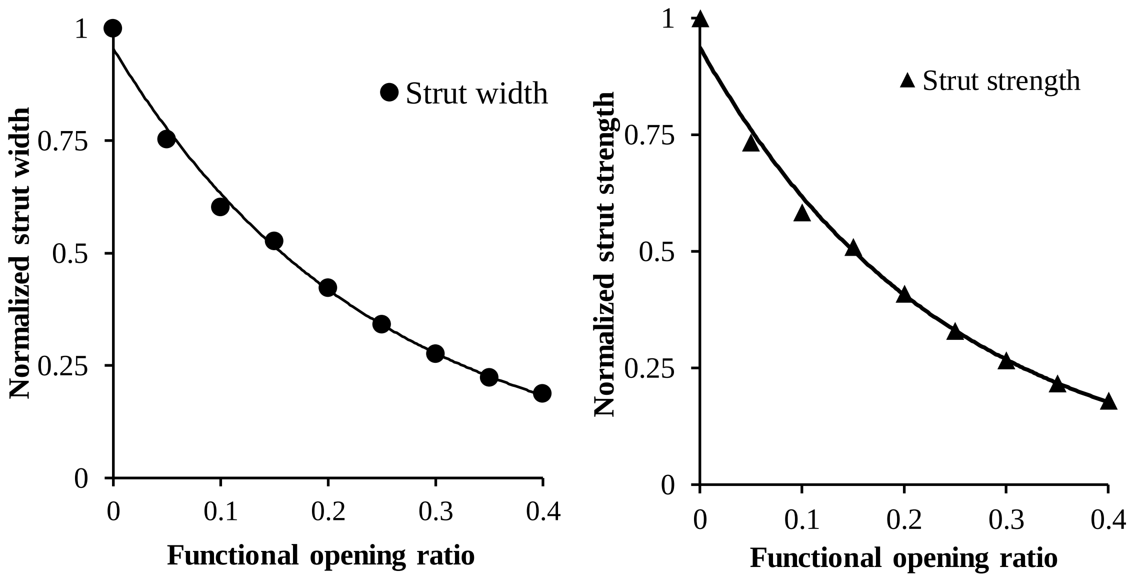
Table 3.1 Design parameters for the considered buildings

General	Design Levels	SMRF bare and infilled frames as per BIS (2016a) and BIS (2016b)
	No. of Stories	Mid-rise (4-storey), high-rise (8-storey)
Material	Concrete and steel	M25 and Fe500
	Compressive strength of infill	$f_c^{/}$= 4.138 MPa
	Modulus of elasticity of infill	550 $f_c^{/}$
Loading	Dead load	Self-weight of members Weight of infill Weight of slab and floor finish Weight of 1*m* high and 230 *mm* thick masonry parapet wall
	Live load	4 kN/m^2 and 3 kN/m^2 on corridor and other floor area
	Design load combination	1.5 (Dead load + Live load) 1.2 (Dead load + Live load ± Earthquake load) 1.2 (Dead load ± Earthquake load) 0.9 Dead load ± 1.5 Earthquake load
Structural modeling	Software used	SAP2000 (2020)
	Structure model	3D space frame model
	Element models	3D line elements for beams and columns Slabs as rigid diaphragm Eccentric strut element for infill
	Plasticity model	Lumped plasticity model ASCE-41 (2017)
	P-delta effect	Considered in linear and nonlinear analyses

In the present study, 24 set of buildings have been evaluated by varying 8 window sizes of opening area ranging from 5% to 40%, and 3 door sizes of opening area ranging from 20% to 33% covering the wide range of window and door openings (Table 3.2) prescribed by CPWD (2006). Reduction factor expression of Decanini et al. (2014) has been considered to simulate effect of opening for the parametric study. The variation of infill modeling properties such as strut width and strength due to window opening ranging from 5% to 40% at the 230*mm* thick exterior infill wall and door opening ranging from 20% to 33% at the 110*mm* thick interior wall is presented in Figs 3.3 and 3.4.

Table 3.2 Description of considered opening combinations for parametric study

Nomenclature of combinations	Opening area (%)		Total Open Area of		Total opening area (m^2)	Total infill panel area (m^2)	Total Opening (%)
	Windows	Doors	Windows	Doors			
UI	0	0	0	0	0		0
W1D1		20	24.24	60.02	99.65		10
W1D2	5	25	24.24	75.03	114.66		12
W1D3		33	24.24	99.03	138.66		14
W2D1		20	48.48	60.02	123.89		13
W2D2	10	25	48.48	75.03	138.89		14
W2D3		33	48.48	99.03	162.90		17
W3D1		20	72.72	60.02	148.13		15
W3D2	15	25	72.72	75.03	163.13		17
W3D3		33	72.72	99.03	187.14		19
W4D1		20	96.96	60.02	172.37		18
W4D2	20	25	96.96	75.03	187.37		19
W4D3		33	96.96	99.03	211.38	984.96	21
W5D1		20	121.20	60.02	196.61		20
W5D2	25	25	121.20	75.03	211.61		21
W5D3		33	121.20	99.03	235.62		24
W6D1		20	145.44	60.02	220.85		22
W6D2	30	25	145.44	75.03	235.85		24
W6D3		33	145.44	99.03	259.86		26
W7D1		20	169.67	60.02	245.09		25
W7D2	35	25	169.67	75.03	260.09		26
W7D3		33	169.67	99.03	284.10		29
W8D1		20	193.91	60.02	269.33		27
W8D2	40	25	193.91	75.03	284.33		29
W8D3		33	193.91	99.03	308.34		31



(a) (b)

Fig. 3.3 Variation of strut properties with windows opening ratio at 230*mm* thick infill (a) Strut width; (b) Strut strength

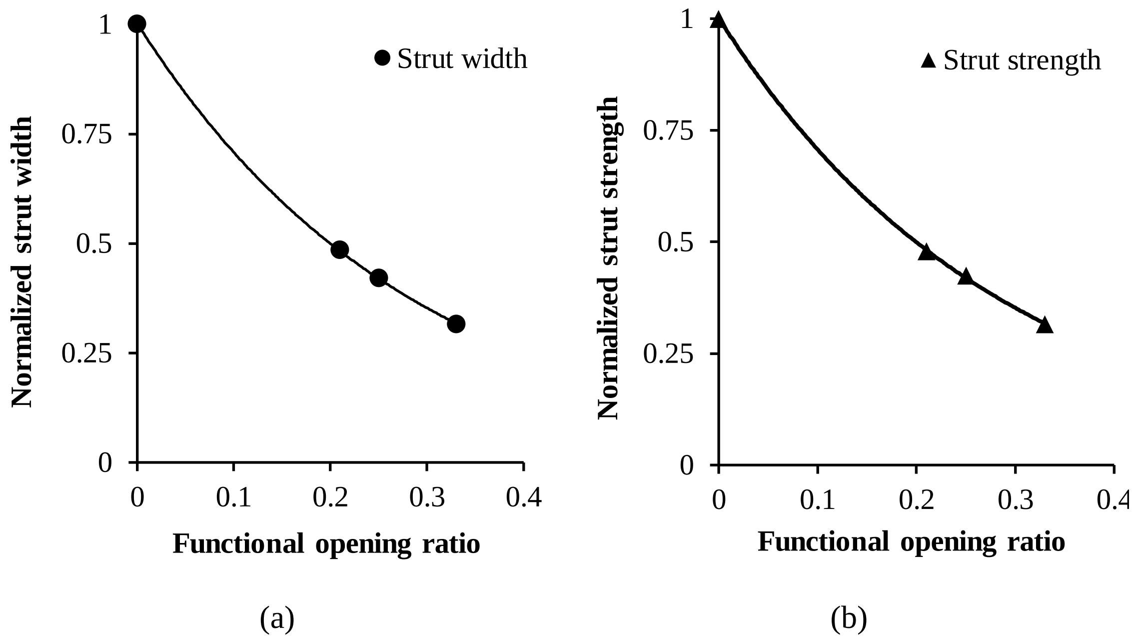


Fig. 3.4 Variation of strut properties with doors opening ratio at 110*mm* thick infill (a) Strut width; (b) Strut strength

It can be observed from Figs. 3.3 and 3.4 that as the opening in infills increases, the width and strength of the strut to model the infills reduces exponentially.

3.3 Effect of Openings on Seismic Performance of RC Buildings

Non-linear static pushover analysis has been carried out to evaluate the effect of openings on the seismic performance in terms of lateral peak strength, stiffness, ductility, deformation capacity of the mid-rise (4-storey) and high-rise (8-storey) infilled RC frame buildings. 75% and above mass participation in the fundamental mode has been observed for all the analytical models of the considered buildings satisfying the non-linear static procedure criteria of FEMA-356 (2000) for non-linear evaluation of the considered buildings. The presence of infills impacts the mass and stiffness of the structure, also alters fundamental time period of buildings (Asteris et al. 2015a; Asteris et al. 2015b). Fig. 3.5 shows the correlation of fundamental time period with opening of the building, and can be observed that fundamental time period increases with increase in opening in infills. The fundamental time period of the building exhibits strong positive correlation with the opening ratio, and similar observations have also been reported in the previous analytical studies (Asteris et al. 2015a; Asteris et al. 2015b).

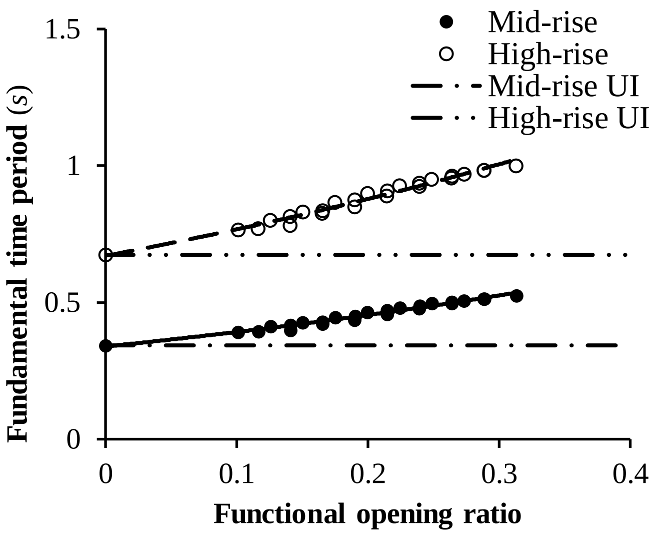
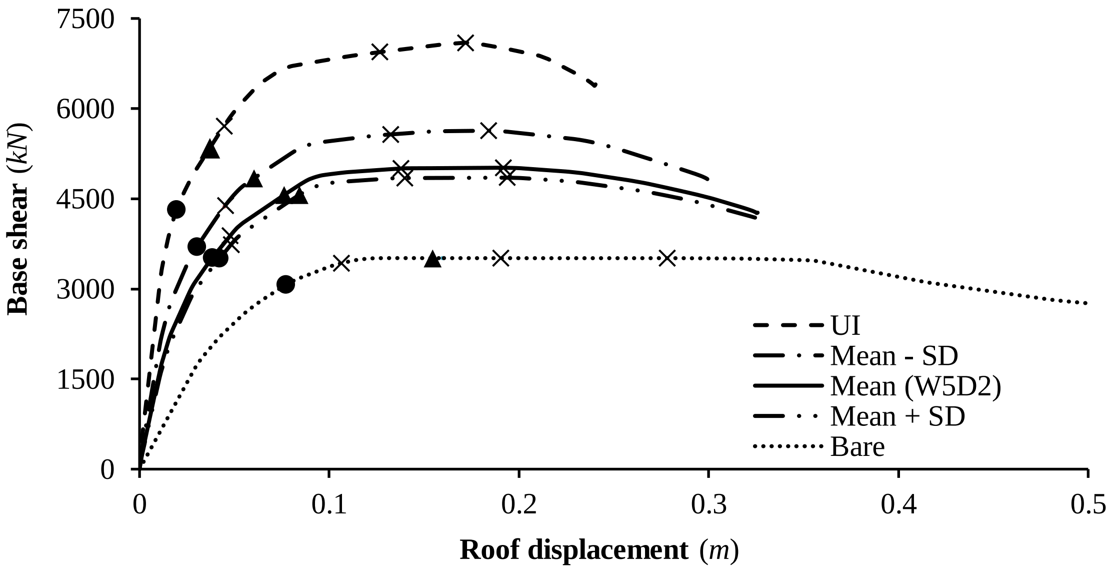
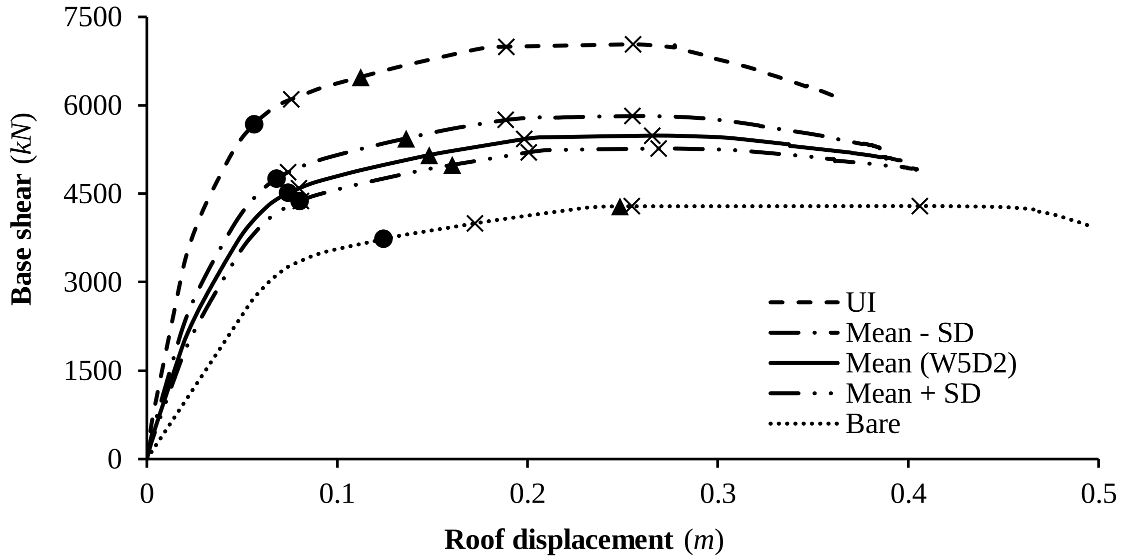


Fig. 3.5 Correlation of fundamental time period with functional opening ratio

Capacity curves of all the 26 considered buildings, including 24 combinations of door-window openings, and two boundary cases of Uniformly Infilled (UI) frame and bare frame as presented in Table 3.2 have not been reproduced in Fig. 3.6; instead capacity curve of the UI, mean opening combination W5D2 having 21% opening of the UI, W5D2 ± Standard Deviation (SD) of opening combination, and bare frame are presented to maintain the clarity of the figure.



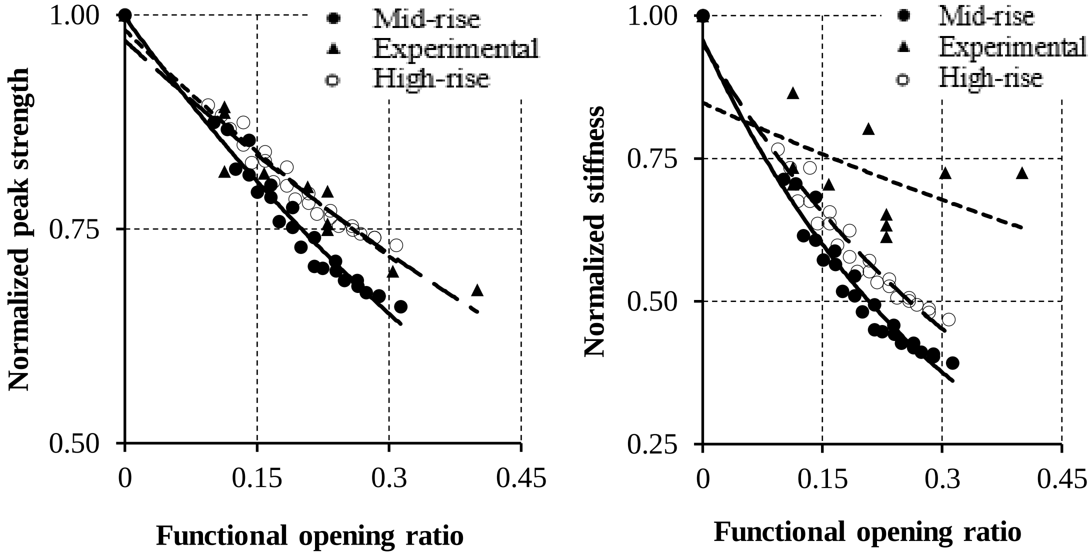
(a)



(b)

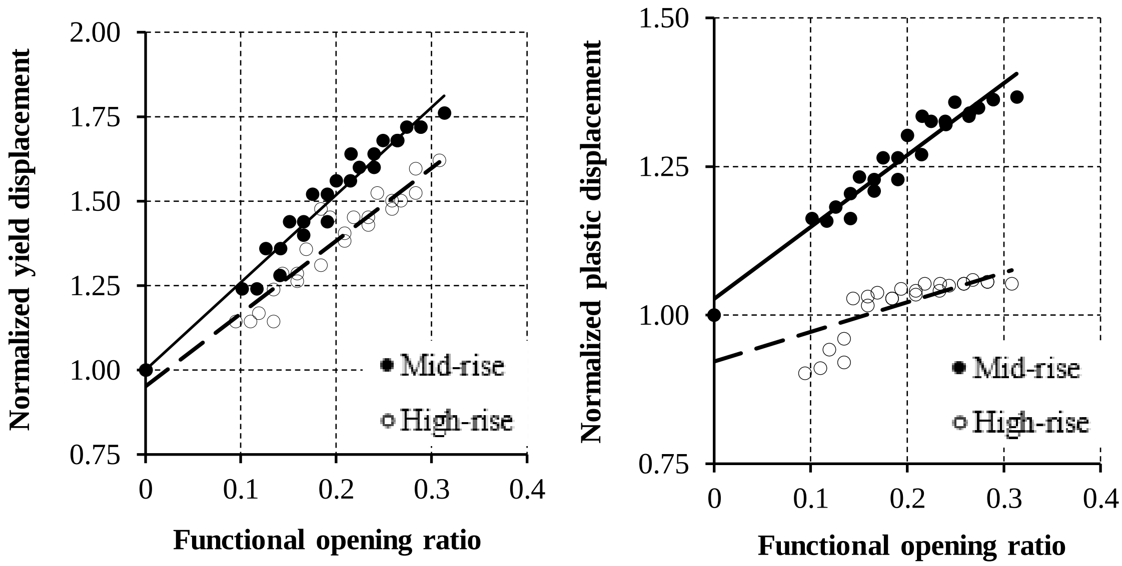
Fig. 3.6 Capacity curves of (a) mid-rise; (b) high-rise buildings. Three crosses (x) in the capacity curves represent Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels, consecutively. Black dot and triangle represent target displacements (ASCE-41 2007) at DBE and MCE hazard levels

It can be observed from Fig. 3.6 that the lateral load carrying capacity, and stiffness of both mid-rise and high-rise buildings decreases while the ultimate displacement of the building increases with increase in opening in infills. It has been further observed that the buildings satisfy IO performance level at Design Basis Earthquake (DBE) hazard (0.18*g*), and LS performance level Maximum Considered Earthquake (MCE) hazard level (0.36*g*) expect mid-rise UI building which is found to satisfy IO performance level at both DBE and MCE hazard level. This superior performance of mid-rise UI building can be attributed to over strength due to adequacy of revised Indian seismic design and detailing standard BIS (2016b) which enforces SCWB criteria, and minimum dimension of column shall not be less than 20 times diameter of largest longitudinal beam rebar. UI buildings exhibits highest strength being stiffest whereas bare frame buildings have lowest strength and stiffness being the most flexible building among all the models considered. In case of mid-rise UI building, maximum strength and stiffness is found to be 99% and 364% higher whereas in case of relatively flexible high-rise UI building, maximum strength and stiffness is found to be 64% and 214% higher than its bare frame counterpart, respectively.



(a) (b)

Fig. 3.7 Correlation of (a) peak strength; (b) stiffness with functional opening ratio



(a) (b)

Fig. 3.8 Correlation of (a) yield; (b) plastic displacement with functional opening ratio

It is further observed that as the total opening in the building increases from 10% to 31%, the lateral load carrying capacity of the mid-rise building sharply decreases from 87% to 66% whereas the increase in opening in high-rise building does not affect drastically, and only reduction of lateral load carrying capacity from 89% to 73% as compared to its UI counterpart can be observed. Similar trend of reduction in load carrying capacity, stiffness (Fig. 3.7) while increase in yield and ultimate displacement (Fig. 3.8) has been witnessed for all the 24 buildings with various opening combinations.

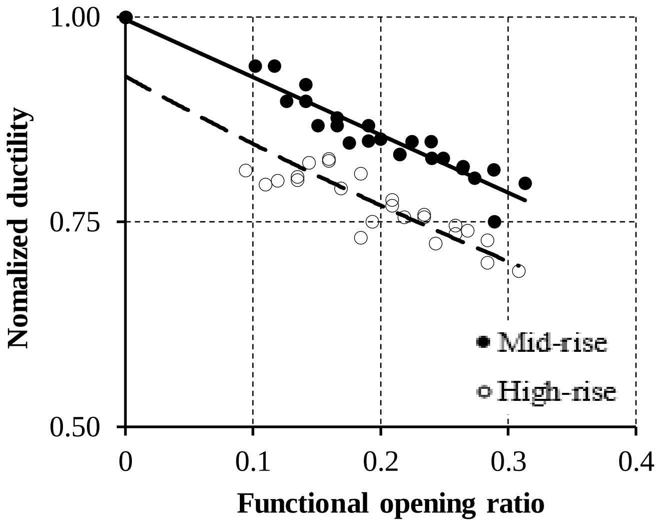


Fig. 3.9 Correlation of ductility capacity with functional opening ratio

Correlation of peak strength, stiffness, yield displacement, plastic displacement, and ductility of the buildings with opening ratio normalized with respective to the similar parameters of UI buildings has been presented through Figs. 3.7-3.9. It can be observed from Fig. 3.7 that the strength and stiffness of the buildings reduces with increase in opening ratio. The relationship of strength and stiffness of the building with opening ratio exhibit a very good negative correlation. Peak strength of considered mid-rise and high-rise buildings for the parametric study show good agreement with the correlation of peak strength observed in experiment of single storey single bay infilled RC frames with openings by Demetrios and Karayannis (2007); Demetrios and Christos (2009). Considering the high sensitivity of stiffness to the structural configuration, shape, size and location of opening in infill panels, limited agreement can be observed in case of stiffness of considered buildings in parametric study with experimentally tested infilled frames. Buildings with openings ranging from 10% to 31% of the total infilled area is found to yield ranging 31*mm* to 44*mm* for mid-rise, and 42*mm* to 68*mm* for high-rise buildings, respectively. It can be observed from Fig. 3.8 that as opening in the building increases, yield and plastic displacement capacity increases relatively. It is further observed in the present study that opening increases the yield displacement of the building with higher factor as compared to ultimate displacement, which eventually reduces the ductility of the buildings as shown in Fig. 3.9. Openings are observed to be more sensitive to yielding of the structure as compared to ultimate deformation.

3.4 Summary

The openings in the form of doors and windows are integral part of infilled frame buildings owing to its functional requirement. Present study evaluates the effect of different infill opening combinations due to presence of doors and windows on the seismic performance of the infilled RC frame buildings. It can be concluded from the parametric study that presence of opening in the infills reduces the lateral strength, stiffness, and ductility of the building. A very good negative correlation is observed for lateral strength, stiffness, and ductility of the building with the opening ratio. Opening in infills further increases the displacement capacity of the buildings as reduction in infills increases flexibility of the building. Openings are observed to be more sensitive to yielding of the structure, increases yield displacement with higher rate as compared to ultimate deformation.

Chapter 4

Seismic Assessment of Open Ground Storey Building with Functional Openings in Upper Storey Infills

4.1 Introduction

Ever increasing popularity of multi-storey Un-Reinforced Masonry (URM) infilled Reinforced Concrete (RC) residential buildings with ground storey kept open for parking/business purposes in urban India and the devastating consequences of their poor performance even in moderate earthquakes, have stirred up the concern regarding in depth understanding of seismic behaviour and collapse mechanism of such buildings. Almost 95% of URM infilled RC frame buildings in the National Capital Region of India was found to have open ground storey (DEQ 2009). Due to absence of URM infills at the open storey, lateral storey strength, and stiffness reduces significantly as compared to its adjacent upper storey (Jain et al. 2002; Kaushik et al. 2009; Choudhury and Kaushik 2018; Mazza et al. 2018; Das et al. 2023). Past earthquakes (Jain et al. 2002; Gattulli et al. 2013; Sharma et al. 2013) caused heavy to severe damage to these open ground storey buildings and even caused structural collapse of these buildings due to formation of soft-storey mechanism.

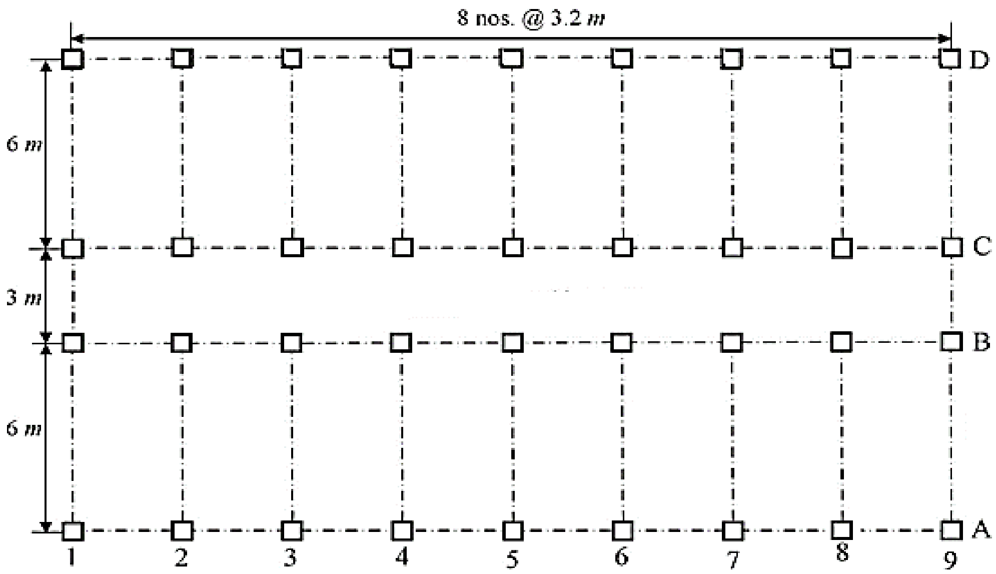
Considering the devastating consequences of poor performance of such building even in moderate earthquakes, significant research efforts have been undertaken in the past, to ensure seismic safety of such buildings by strengthening and stiffening the open storey structural members to meet the seismic performance of these buildings to Uniform Infilled (UI) RC frames (Kaushik et al. 2009; Haldar et al. 2016; Haran Pragalath et al. 2016). In order to avoid severe consequences of poor performance of OGS buildings, International Building Code ICC IBC (2012), New Zealand Code NZSEE (2006), ASCE/SEI 7 (2010) prohibit extremely irregular buildings in seismically active areas. However, to allow the functional requirements of the open storey for all practical purposes, compensation of storey stiffness and strength deficiency is essential at the design stage. Several national standards like Bulgarian seismic code (1987), BIS (2002), and Eurocode-8 (2004) have suggested using multiplication factors to increase the design force in the open storey members. Israel seismic code (SI-413 1995) suggested increasing the design force for the open storey along with adjacent storey members. Haldar et al. (2016) studied the efficacy of design provision for OGS RC frame buildings prescribed in BIS (2002) by comparing the seismic performance of uniformly infilled buildings with its open ground storey buildings counterpart with varying height. They concluded that open ground storey buildings designed as per BIS (2002) can attain the stiffness and strength close to those of the corresponding uniformly infilled frame building, as design of open ground storey members with 2.5 times design base shear led to increase in size of open storey beams and columns. Presence of solid infills enhances lateral strength and stiffness of the structure when distributed uniformly over the height of the building (Fardis et al. 1999; Kaushik et al. 2009; Liberatore and Decanini 2011; Favvata et al. 2013; Yuen and Kuang 2015; Haldar et al. 2016; Kurmi and Haldar 2022a), and degrades seismic performance if placed in irregular manner in plan and or in elevation (Esteva 1992; Vukazich et al. 2006; DolŠEk and Fajfar 2008b; Haldar et al. 2016). Presence of functional openings in infills further complicates the complex interaction of infill and frame which may affect seismic performance in terms of strength, stiffness, ductility and can even cause premature failure of infill-frame composite panel (Demetrios and Karayannis 2007; Dolšek and Fajfar 2008a; Demetrios and Christos 2009; Demetrios 2009; Barnaure and Daniel 2015; Martinelli et al. 2015). Ignoring openings in structural analysis may lead to imprecise prediction of the seismic behaviour of the overall frame, along with failure modes (Demetrios and Karayannis 2007; Demetrios and Christos 2009; Nwofor 2012). Significant research efforts have been made by several researchers (Dolšek and Fajfar 2008a; Barnaure and Daniel 2015; Sukrawa 2015; Yuen and Kuang 2015; Ozturkoglu et al. 2017; Perrone et al. 2017; Repapis and Zeris 2019; Kurmi and Haldar 2022b) in order to investigate the effect of openings on the overall seismic response of uniformly infilled frame buildings. Although past studies revealed that increasing opening in infills significantly affects the seismic performance, however, limited studies can be found on the true seismic response and governing failure mechanism of OGS RC buildings considering the effect of realistic combinations of functional openings due to doors and windows in upper storey infills.

Present study intends to examine the effect of realistic combinations of functional openings in upper storey infills considering different size and shape of doors and windows that typically exists, selected based on the manual of Central Public Works Department (CPWD 2006), Govt. of India, on seismic performance of generic OGS RC frame building. An exhaustive parametric study has been carried out combining different sizes of doors and windows on sets of mid-rise (4-storey) and high-rise (8-

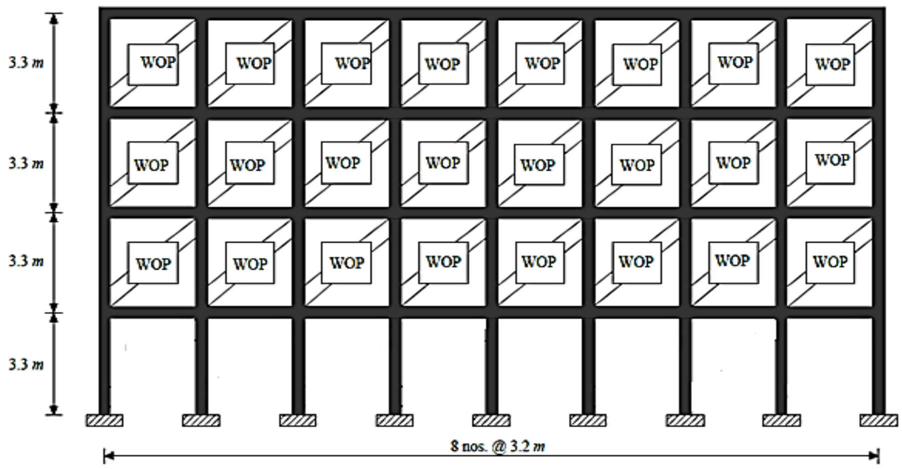
storey) RC buildings with open ground story. Seismic performance assessment OGS buildings are carried out using nonlinear static analysis and compared with and without functional opening in upper storey(s) infills.

4.2 Seismic Assessment

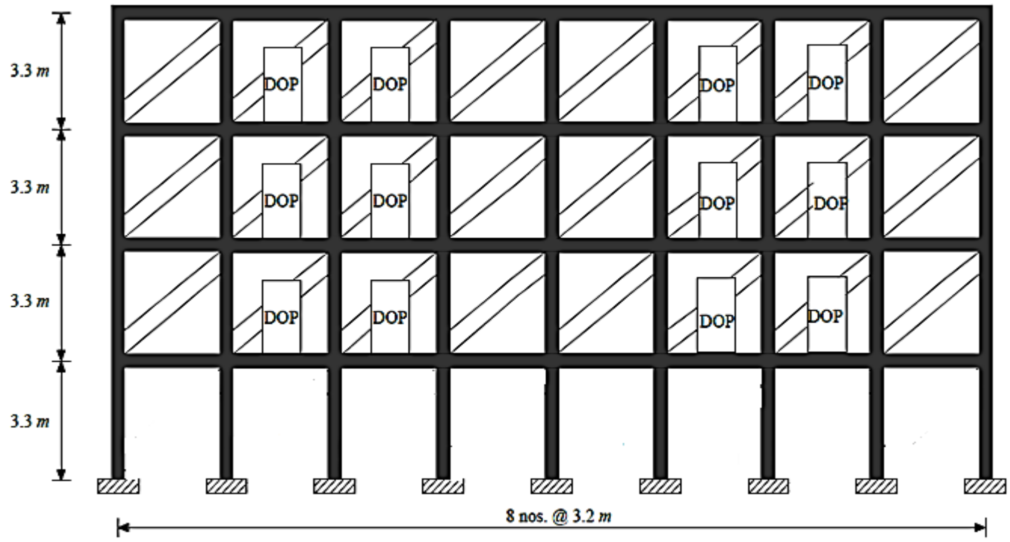
For the present study, a set of mid-rise and high-rise RC frame buildings with a generic plan as discussed in Chapter 3, Section 3.2 (shown in Fig. 4.1) having same structural characteristics (Table 3.1) have been considered. The considered buildings have selected realistic Door Openings (DOP) and Window Openings (WOP) as presented in Table 4.1, representing wide characteristics of multi-storey Indian buildings.



(a) Plan of the considered building



(b) Location of windows (WOP) at external frames



(c) Location of doors (DOP) at internal frame

Fig. 4.1 (a) Plan of the considered buildings; (b) Location of windows (WOP) at external frames; (c) Location of doors (DOP) at internal frames

Indian seismic design standard BIS (2002) prescribe designing the open storey columns and beams for 2.5 times higher design base shear as compared to uniformly infilled frame to remove stiffness irregularity in the open storey, unfortunately, a large stock of OGS RC buildings exist even in high seismic zones (DEQ 2009), where the ground storey is kept open without following the provisions of BIS (2002) for OGS. In order to encompass the large stock of existing OGS buildings in India, two different design levels viz. 'OGS_BIS2002' conforming BIS (2002) and 'OGSW' where the ground storey is kept open without following the provisions of BIS (2002) for OGS buildings have been considered in the present study. The main intend of the present study is to understand the effect of realistic combination of functional openings in upper storey infills due to presence of doors and windows on the overall inelastic behaviour and failure mechanism of open ground storey buildings. Accordingly, reduction factor model proposed by Decanini et al. (2014) has been considered to simulate effect of opening in infills. To cover the wide spectrum of size and configuration of openings in Indian residential buildings, CPWD manual (CPWD 2006) guidelines for doors and windows in residential buildings have been considered for the parametric study. CPWD (2006) has recommended 15 windows and 6 door sizes by varying the length and height of openings which eventually arrive at variation of window from 6% to 32%, and door opening sizes 20% to 33%. In the present study, 4 sets of buildings with 25 combinations of opening have been evaluated by varying 8 window sizes of opening area ranging from 5% to 40%, and 3 door sizes of opening area ranging from 20% to 33% covering the wide range of window and door openings (Table 4.1) prescribed by CPWD (2006).

Table 4.1 Description of considered combinations of door and window openings

Sl. No.	Nomenclature of combinations	Opening area (%)		Total Opening Area (TOA) (m^2)	Total Infill Area (TIA) (m^2)	Total Opening (%) (TOA/TIA)
		Windows	Doors			
1.	OGS-Solid	0	0	0	984.96	0
2.	OGS-W1D1	5	20	99.65		9
3.	OGS-W1D2		25	114.66		12
4.	OGS-W1D3		33	138.66		14
5.	OGS-W2D1	10	20	123.89		13
6.	OGS-W2D2		25	138.89		14
7.	OGS-W2D3		33	162.90		17
8.	OGS-W3D1	15	20	148.13		15
9.	OGS-W3D2		25	163.13		17
10.	OGS-W3D3		33	187.14		19
11.	OGS-W4D1	20	20	172.37		18
12.	OGS-W4D2		25	187.37		19
13.	OGS-W4D3		33	211.38		21
14.	OGS-W5D1	25	20	196.61		20
15.	OGS-W5D2		25	211.61		21
16.	OGS-W5D3		33	235.62		24
17.	OGS-W6D1	30	20	220.85		22
18.	OGS-W6D2		25	235.85		24
19.	OGS-W6D3		33	259.86		26
20.	OGS-W7D1	35	20	245.09		25
21.	OGS-W7D2		25	260.09		26
22.	OGS-W7D3		33	284.10		29
23.	OGS-W8D1	40	20	269.33		27
24.	OGS-W8D2		25	284.33		29
25.	OGS-W8D3		33	308.34		30

4.3 Effect of Functional Openings on Seismic Performance

Non-linear static pushover analyses have been carried out to evaluate the effect of possible realistic combinations of functional openings on the seismic performance parameters in terms of stiffness, peak strength, yield displacement, plastic displacement and ductility capacity of the set of mid-rise (4-storey) and high-rise (8-storey) OGS RC frame buildings with all the 25 combinations of functional openings as presented in Table 4.1. Bi-linearization of capacity curves have been carried out as per ASCE-41 (2007) to estimate yield and ultimate force-displacement parameters with simplicity. Plastic displacement (ultimate displacement minus yield displacement) and ductility capacity (ratio of ultimate to yield displacement) is estimated as per Haldar and Singh (2009). In order to perceive the effect of increasing functional opening in the upper storey infills, all performance parameters have been normalized with respect to performance parameter values obtained for respective OGS building with solid infills at upper storey(s).

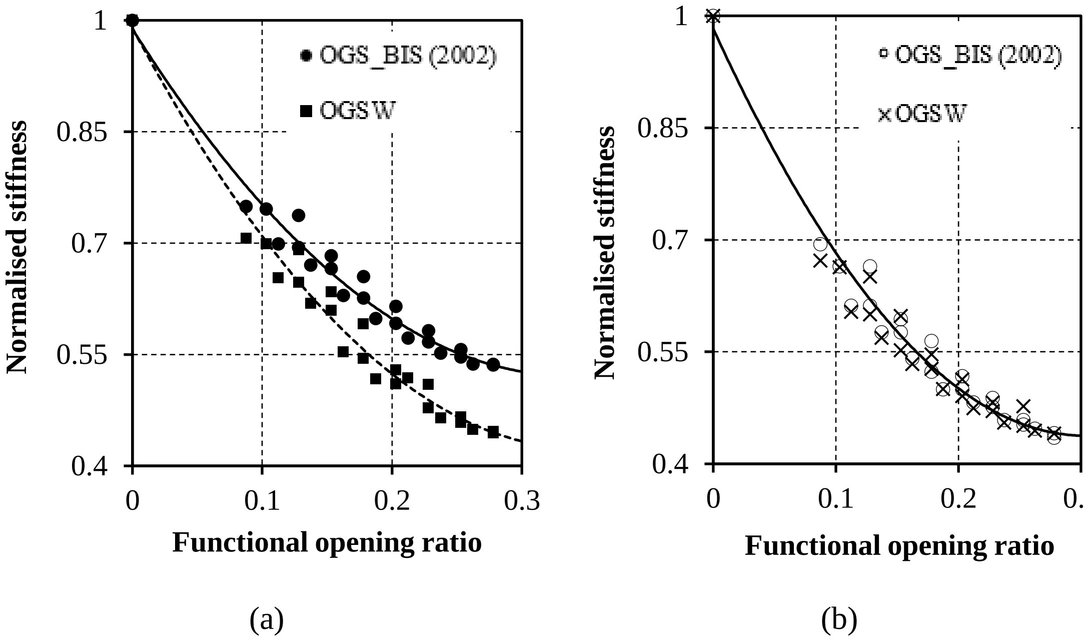
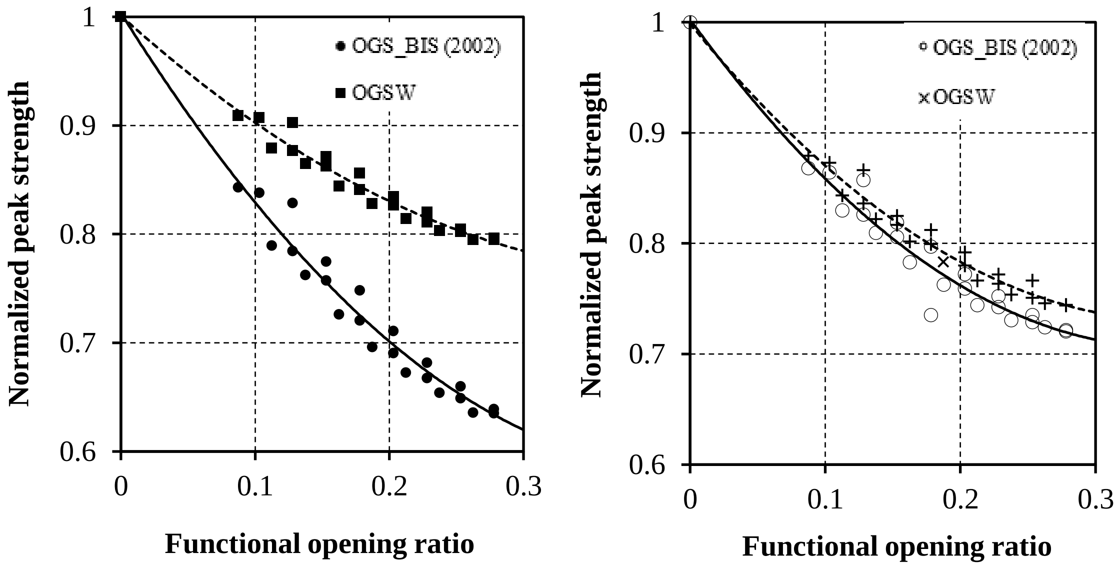


Fig. 4.2 Effect of functional opening on stiffness of (a) mid-rise and (b) high-rise OGS building with and without conforming BIS (2002) OGS design provision

The relationship between stiffness and the fundamental period of a structure is well established, with an evident decrease in the latter as stiffness increases. As illustrated in Fig. 4.2, the sensitivity of building stiffness to the degree of functional opening in upper-story infills is pronounced, showing a robust negative correlation for both mid-rise and high-rise structures. Notably, in OGS buildings featuring approximately 10% functional opening, stiffness experiences a significant drop to around 70% when compared to OGS buildings with solid upper infills. As the percentage of openings further increases from 10% to 30%, the stiffness sees a more substantial reduction to approximately 45% for both mid-rise and high-rise OGS buildings excluding mid-rise OGS buildings designed with BIS (2002) OGS design provisions, which exhibit a reduced stiffness of 55%. The mid-rise OGS buildings conforming to BIS (2002) design standards show a slightly milder reduction in stiffness, attributed to their lower fundamental period resulting in a higher design base shear and, consequently, higher rigidity. This is further supported by their larger member sizes in the open storey, as they were designed for 2.5 times higher design base shear.

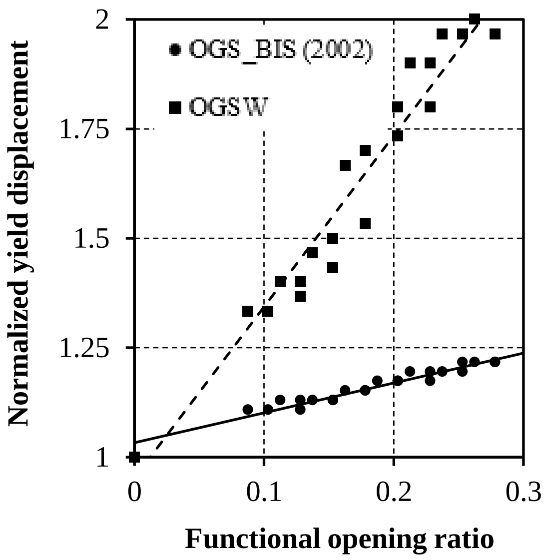
The presence of Un-reinforced Masonry (URM) infills significantly contribute to the lateral strength of RC frame buildings, especially when present in a solid and regular configuration. However, as functional opening increases, there is a notable decrease in the lateral peak strength of the building, indicating a strong negative correlation, as depicted in Fig. 4.3. In the case of mid-rise buildings designed in accordance with BIS (2002) standards, a drastic reduction in lateral strength is observed. This underscores the importance of further exploration and nuanced analysis in understanding the intricate interplay between stiffness, functional opening, and lateral strength in diverse building configurations.



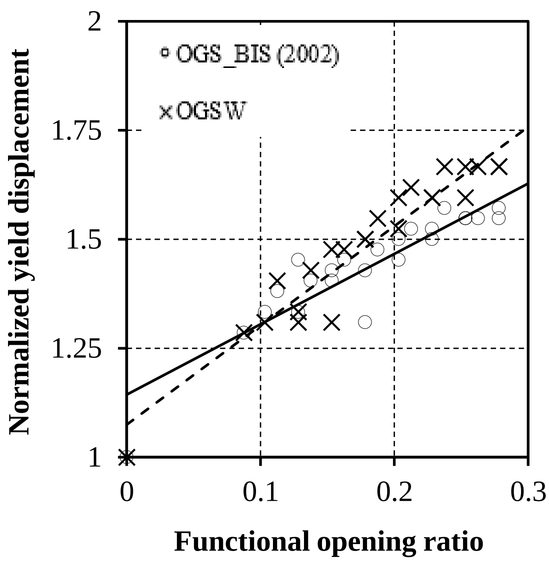
(a) (b)

Fig. 4.3 Effect of functional opening on lateral strength (a) mid-rise and (b) high-rise OGS building with and without conforming BIS (2002) OGS design provision

The lateral strength drops from 84% to 63%, as compared to its solid infill counterpart, when functional opening increases from 9% to 30%, indicating functional opening in upper storey degrades the OGS design adequacy of BIS (2002) and also highly sensitive to functional openings. In case of mid-rise OGS buildings designed without conforming BIS (2002) and relatively flexible high-rise buildings designed with and without conforming BIS (2002), the lateral strength decreased to 80% and 70% respectively with increase in functional opening from 9% to 30%. Fig. 4.4 represents effect of functional opening on yielding of OGS buildings considered for the present study. The yield displacement estimated from the bi-linearized capacity curves are normalized with yield displacement of OGS building with solid upper storey infill and plotted against the functional opening ratio. Presence of stiffer infills in the RC building attracts high lateral loads causing initiation of yielding at lower displacement level. As functional opening in the upper storey infill increases, yielding gets delayed due to enhanced flexibility in the overall buildings.



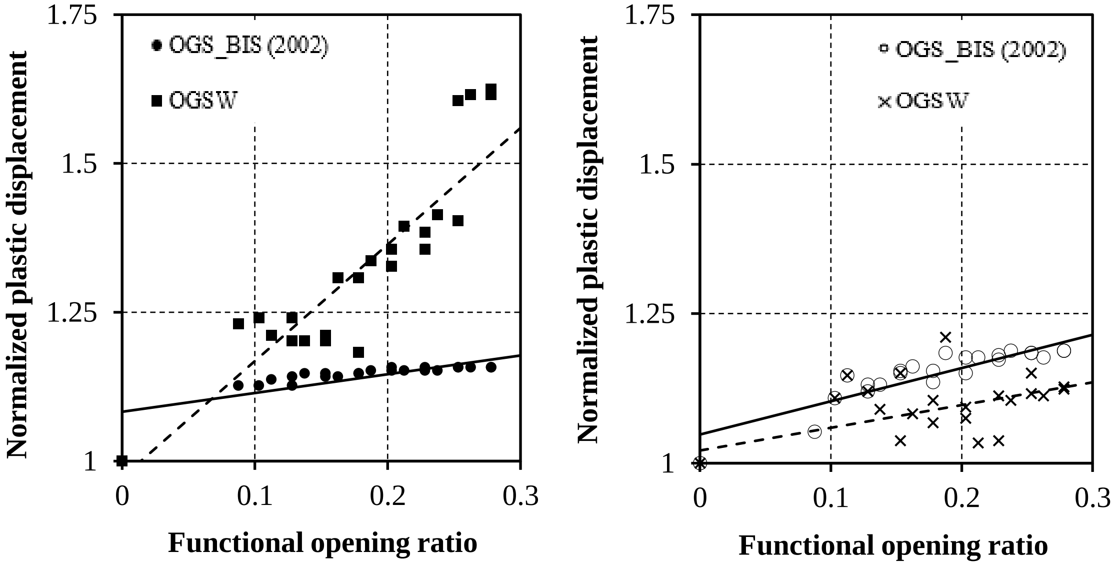
(a)



(b)

Fig. 4.4 Effect of functional opening on (a) yield displacement of (a) mid-rise and (b) high-rise OGS building with and without conforming BIS (2002) OGS design provision

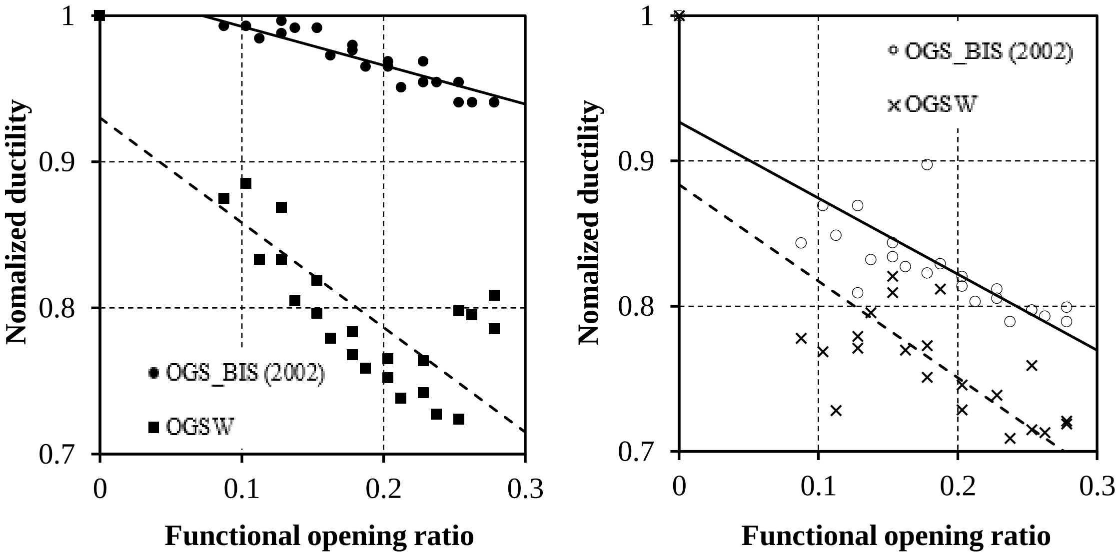
Yield displacement of mid-rise buildings designed without BIS (2002) OGS design requirement is highly sensitive to change in functional opening (9% to 30%), and increased sharply with change of functional opening increment as compared to BIS (2002) conforming counterpart which increased 10% to 20% only. In case of high-rise OGS buildings, designed with and without conforming BIS (2002), the trend of yield displacement is fairly comparable and observed to be increased by 30% to 60%. Fig. 4.5 represents effect of functional opening on plastic displacement capacity of OGS buildings considered for the present study. Plastic displacement capacity of mid-rise buildings designed without BIS (2002) OGS design requirement increased sharply with functional opening (9% to 30%), from 25% to 65% as compared to BIS (2002) conforming counterpart which increased within a narrow range of 10% to 20%. It is interesting to note that both the yield and plastic displacement capacity increases with increase in functional opening in infills, however, yield displacement slope is much higher than the plastic displacement indicating higher sensitivity towards change in opening area.



(a) (b)

Fig. 4.5 Effect of functional opening on plastic displacement capacity of (a) mid-rise and (b) high-rise OGS building with and without conforming BIS (2002) OGS design provision

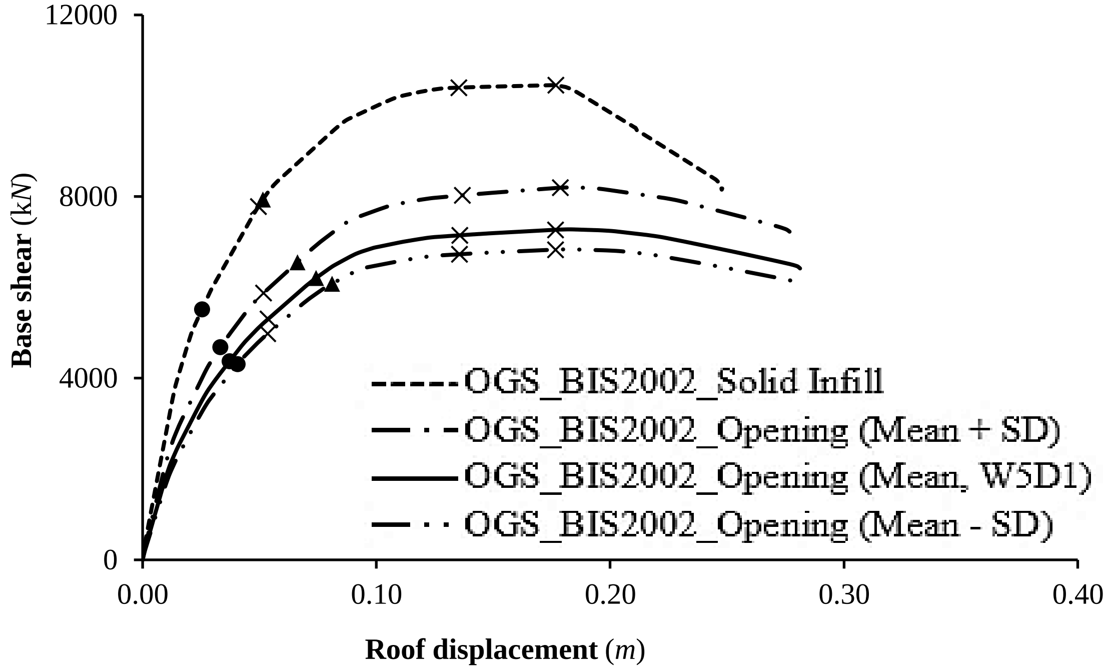
Fig. 4.6 represents response of ductility capacity of the OGS buildings with increase in functional openings in upper storey infills.



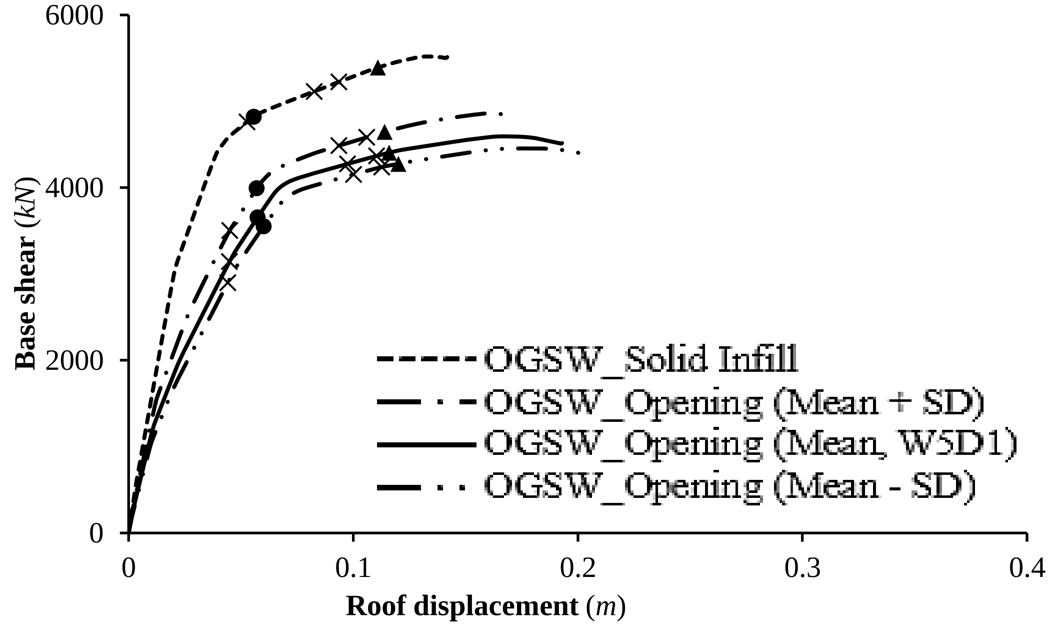
(a) (b)

Fig. 4.6 Effect of functional opening on ductility capacity of (a) mid-rise and (b) high-rise OGS building with and without conforming BIS (2002) OGS design provision

In case of mid-rise OGS buildings conforming to BIS (2002) OGS design, functional opening in upper storey infill is found less influential to its ductility capacity and only 5% decrement in the ductility is observed. In case of OGS mid-rise and high-rise buildings designed without conforming BIS (2002), ductility capacity is observed to be reduced to approx. 70%, when functional opening in upper storey infills reached to 30%. The reduction in ductility can be attributed to the difference between the slope of increment between yield and plastic displacement.



(a)



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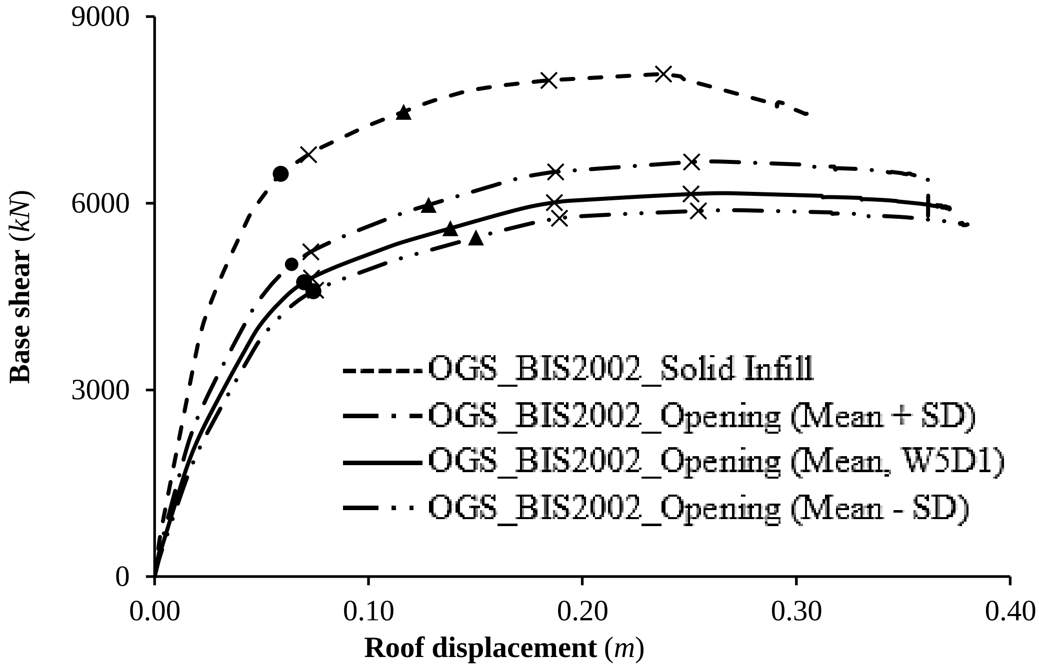

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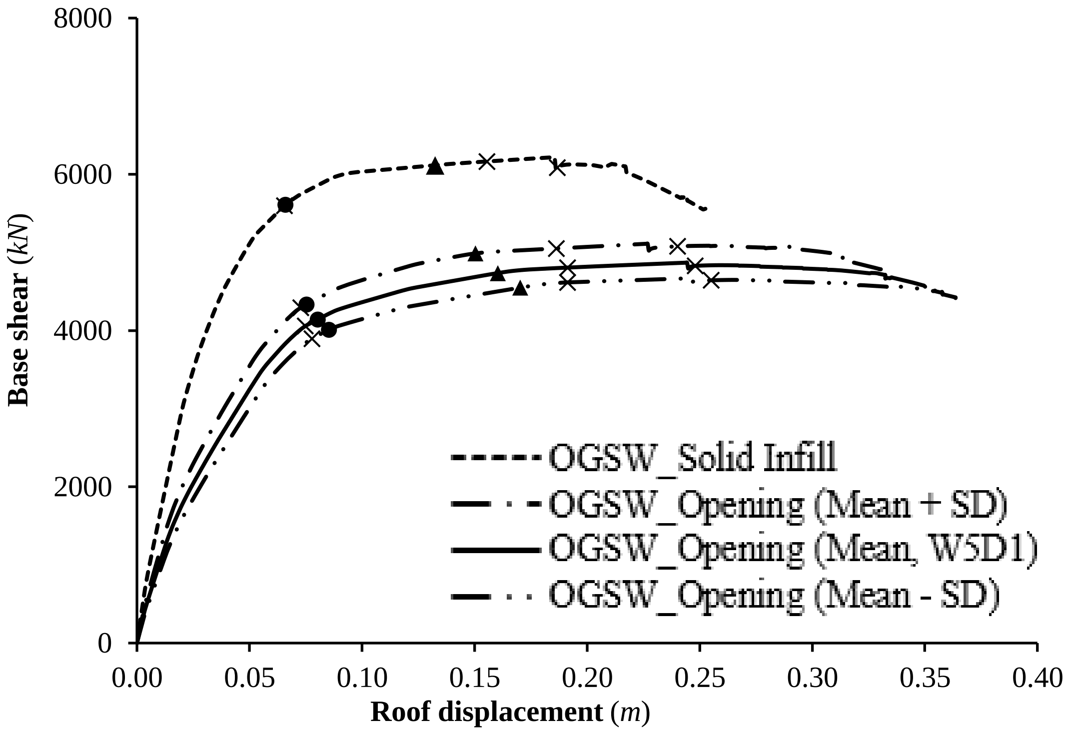


(d)

Fig. 4.7 Capacity curves of (a) mid-rise OGS building as per BIS2002 (b) mid-rise OGS building not conforming BIS2002 OGS design requirement (OGSW) (c) high-rise OGS as per BIS2002 and (d) high-rise OGS not conforming BIS2002 OGS design requirement (OGSW) with and without functional openings. Three crosses (**x**) in the capacity curves represent Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels consecutively. Black dot and triangle represent target displacements (ASCE-41 2017) at DBE and MCE hazard levels

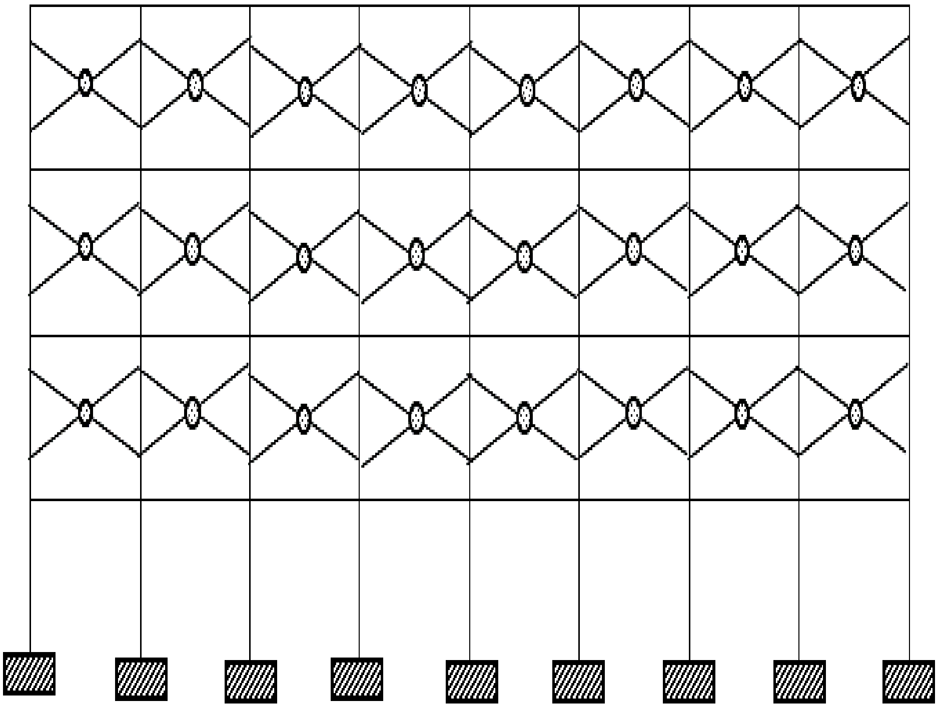
Fig. 4.7 represents the combined effect of each functional opening combination i.e., the 25 probable combinations of door and window opening as presented in Table 4.1 on overall capacity of the OGS buildings with various design levels and heights considered in the present study. Capacity curves have been obtained through nonlinear static analysis of the considered mid-rise and high-rise OGS buildings with and without conforming BIS (2002) design levels. In order to conserve the clarity of the plot, capacity curves of the OGS with solid infills, mean opening combination (W5D1 having 20% opening) and mean ± Standard Deviation (SD) of opening combination have only been presented in Fig. 4.7.

It has been verified that non-linear static analysis procedure is applicable as per FEMA-356 (2000) for predicting inelastic behaviour of the considered buildings with sufficient accuracy as 75% and above mass participation in the fundamental mode has been observed for all the considered buildings. It can be observed from Fig. 4.7 (b), that mid-rise OGS buildings designed without conforming BIS (2002), failure of these buildings occurred at lowest ultimate displacement level as compared to other set of OGS buildings. However, its relatively flexible high-rise counterpart, showed comparable ultimate displacement capacity to its other counterpart before failure. It can be observed from Fig. 4.7, that the lateral load carrying capacity, and stiffness of both mid-rise and high-rise OGS buildings decreased while the ultimate displacement of the building increased with increase in functional opening in infills. OGS buildings having solid URM infills at upper storey designed as per BIS (2002) exhibits highest strength and stiffness among the considered buildings. It is due to combined effect of solid infill and design of open storey members with 2.5 times higher base shear which increased the member sizes, and led to increase in lateral strength and stiffness of the building. OGSW buildings, where ground storey is kept open without following OGS design provisions of BIS (2002), reach complete collapse state at Maximum Considered Earthquake (MCE) hazard level of seismic zone V of BIS (2016) in case of mid-rise buildings, depicting worst seismic performance and Life Safety (LS) performance level for relatively flexible high-rise buildings is observed. Significant overestimation of initial stiffness and peak strength can be observed in mid-rise as well as high-rise OGS buildings when functional openings are neglected in analysis for both the design levels viz. OGS_BIS2002’ (Fig.4.7 (a) and (c)) and OGSW (Fig. 4.7 (b) and (d)).

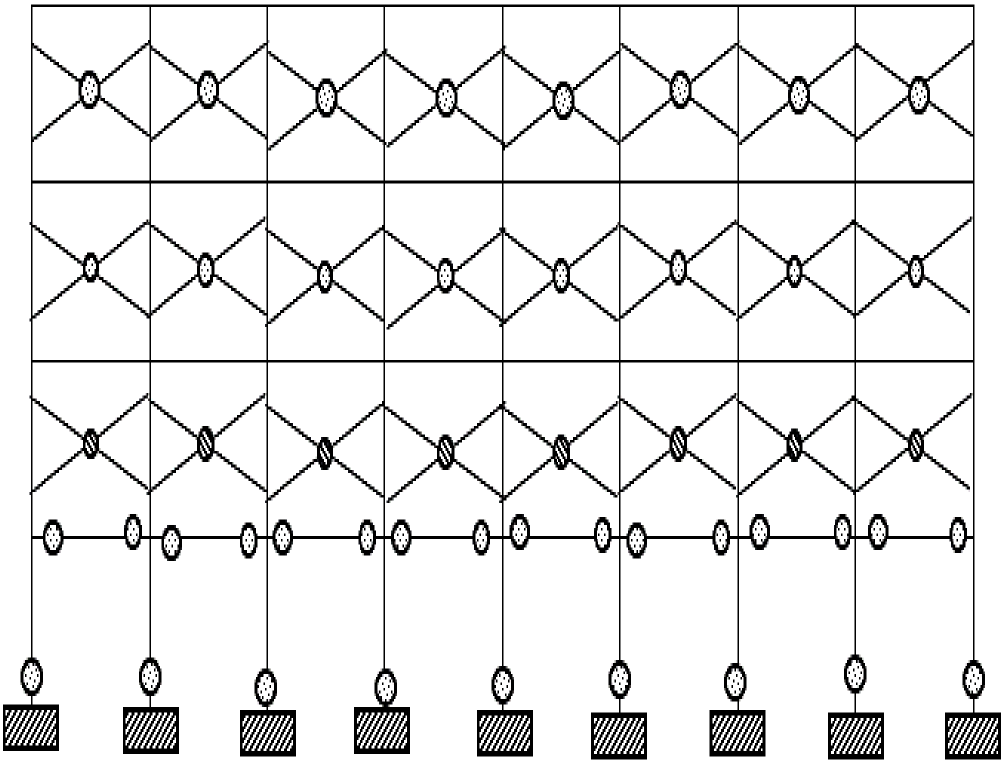
4.4 Effect of Functional Openings on Failure Mechanism of Open Ground Storey RC Buildings

Presence of functional openings also influence the failure mechanism of OGS buildings for all design levels and storey heights (mid-rise and high-rise). It has been observed that yielding first occurs in infills for all the design levels and building heights as these infills attract large lateral forces due to high stiffness even with the presence of functional openings. Further, it has been observed that collapse of open storey beams and columns have been prevented in case of OGS buildings designed with BIS (2002) provisions, both at DBE (Fig. 4.8 (a)) and at collapse hazard levels (Fig. 4.8 (c)) as open storey members were designed with 2.5 times higher design base shear. The global collapse mechanism of OGS buildings designed with BIS (2002) has formed due to failure of infills and beams up to mid-height zone followed by failure of first storey beams and columns as shown in Fig. 4.9 (a) and (c). However, significant amount of strength and stiffness have been attained before collapse of mid-rise buildings as observed from Fig. 4.7 (a).

In case of OGSW buildings, where no open storey members were designed with BIS (2002) OGS provisions, significant number of infill panels along with some open storey beams and columns have yielded even at the DBE hazard level (0.18*g*) for which the structure has been designed (Fig. 4.9 (b) and (d)). Open storey members have reached collapse state at MCE hazard level (0.36*g*) due to excessive inelastic deformation demand at open storey leading to collapse of the whole building (Fig. 4.9 (b) and (d)) which resembles failure pattern of OGS buildings observed in Bhuj earthquake.



(a)



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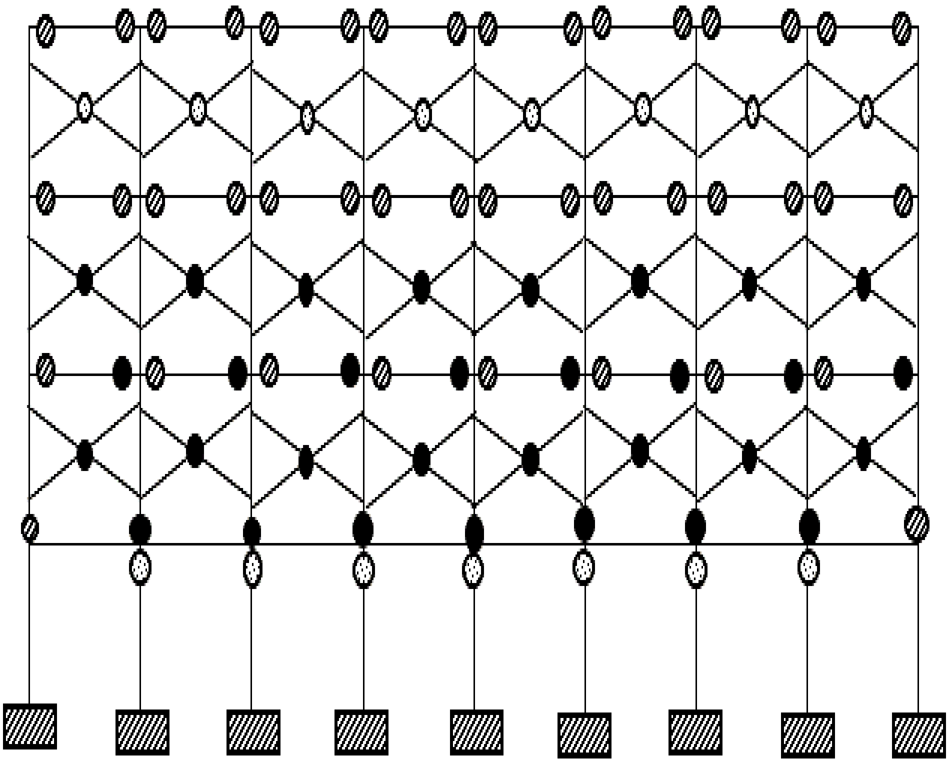

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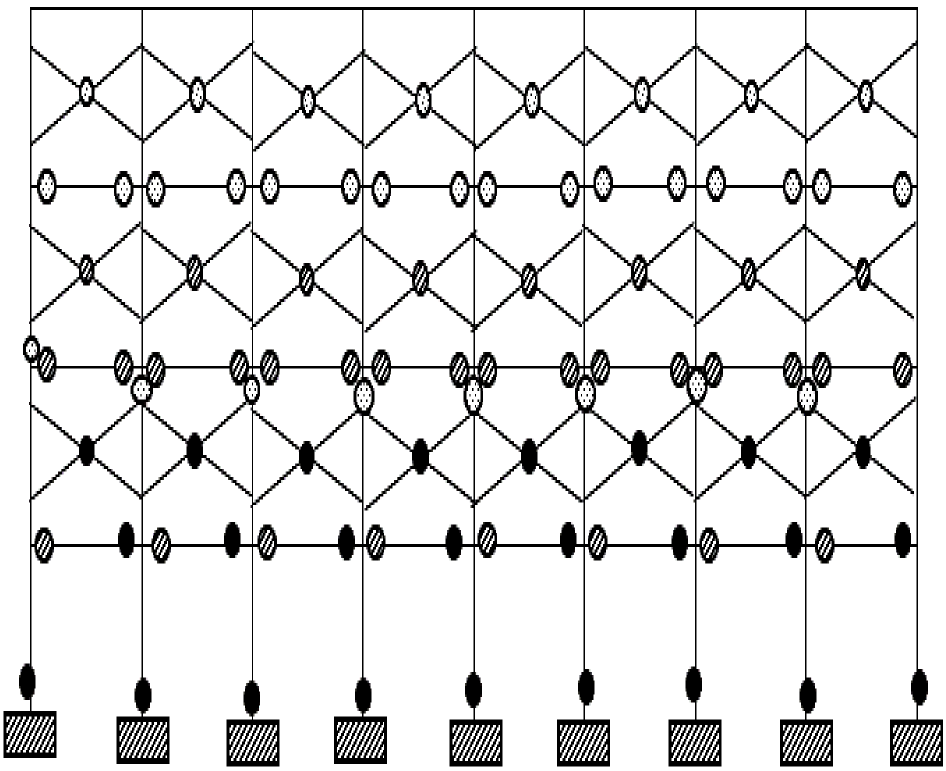
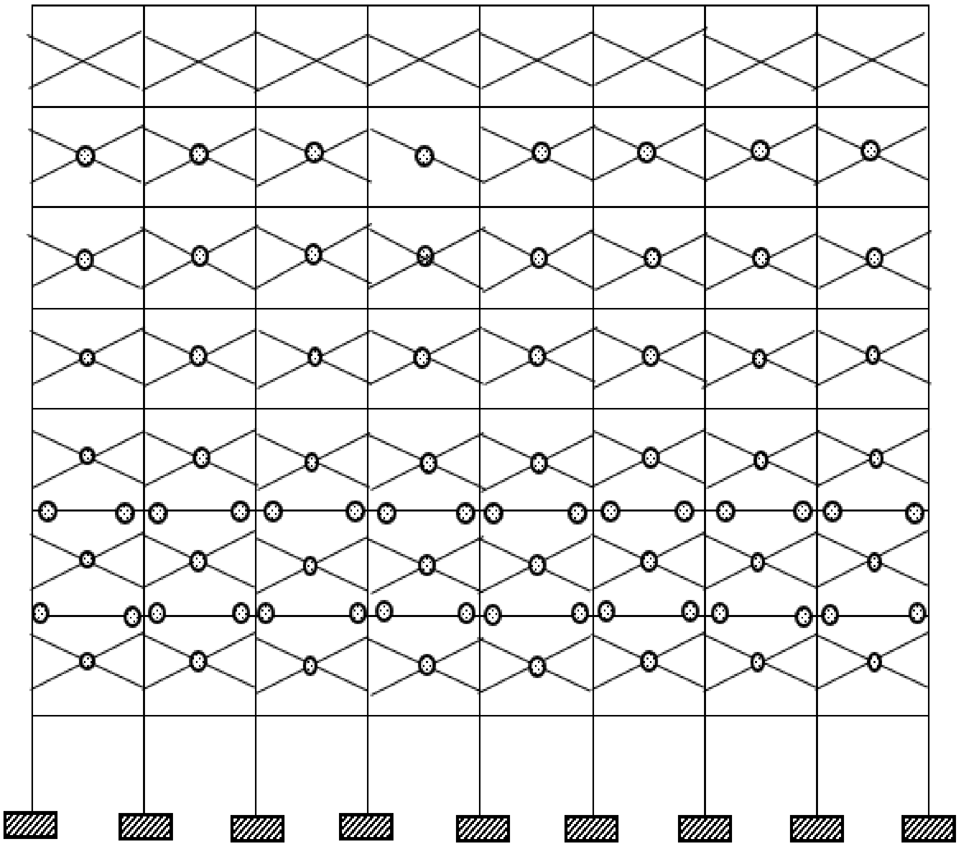


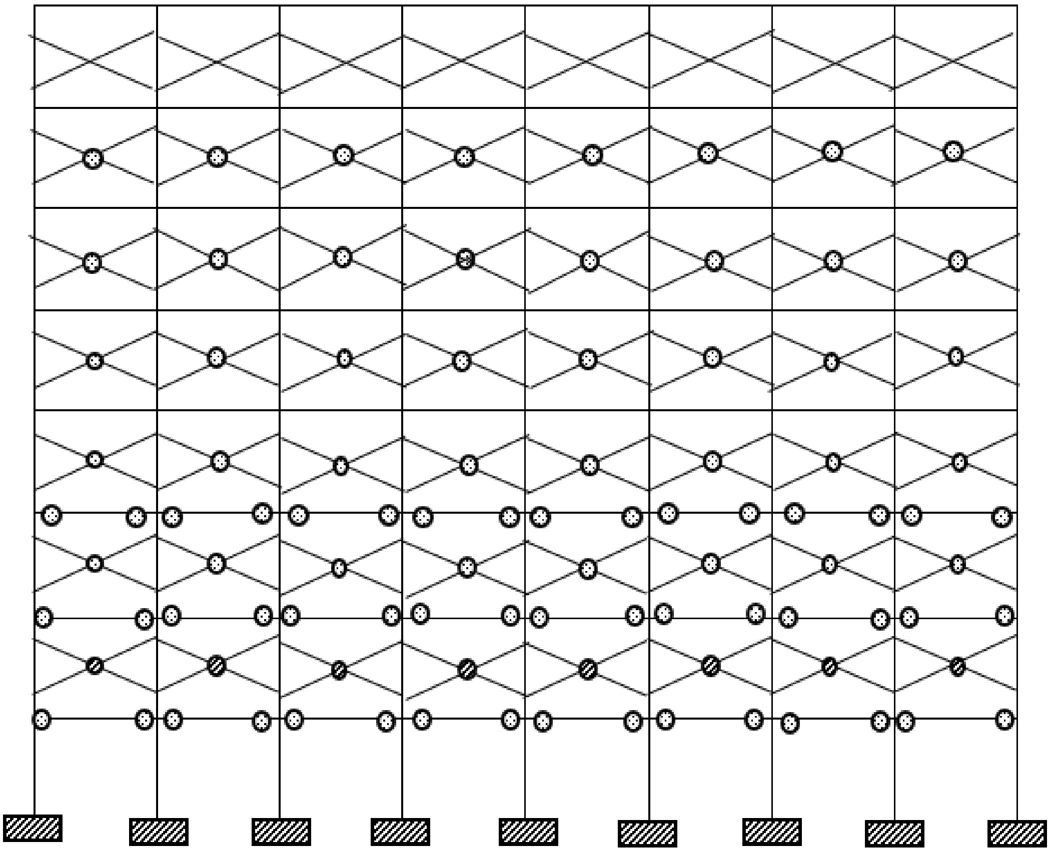
(d)

⊙ Immediate Occupancy (IO) ⊘ Life Safety (LS) ● Failure (ASCE 41-17)

Fig. 4.8 Typical yielding at critical frame of mid-rise buildings at DBE hazard level; (a) OGS_BIS2002; (b) OGSW; at collapse (c) OGS_BIS2002; (d) OGSW



(a)



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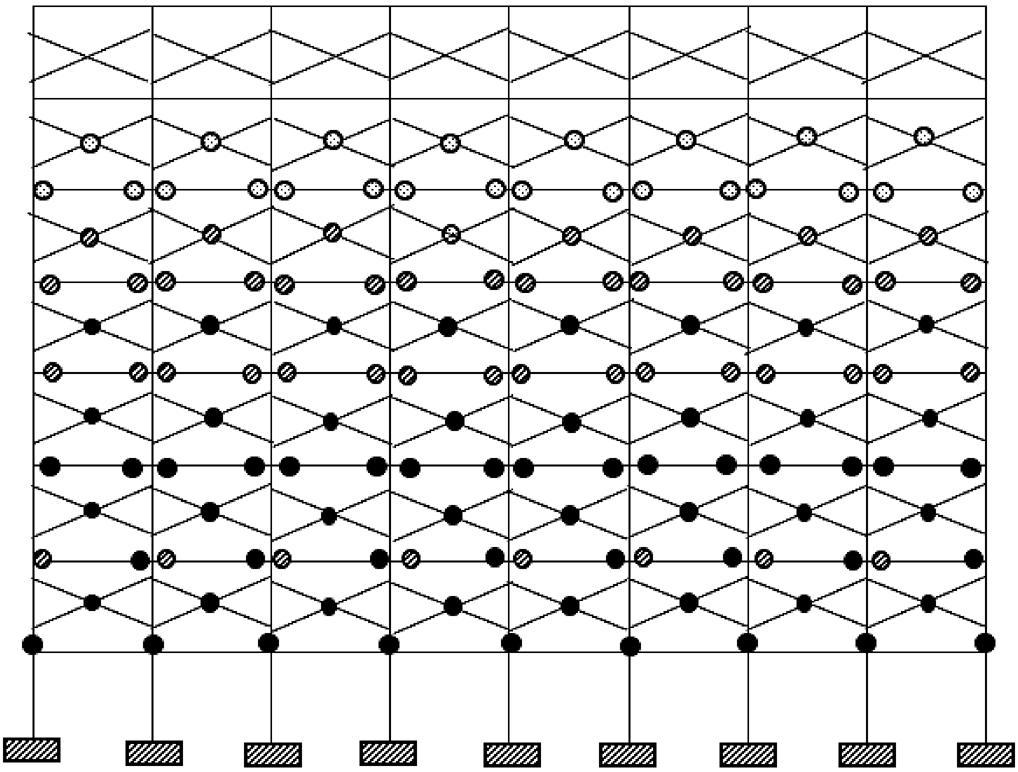

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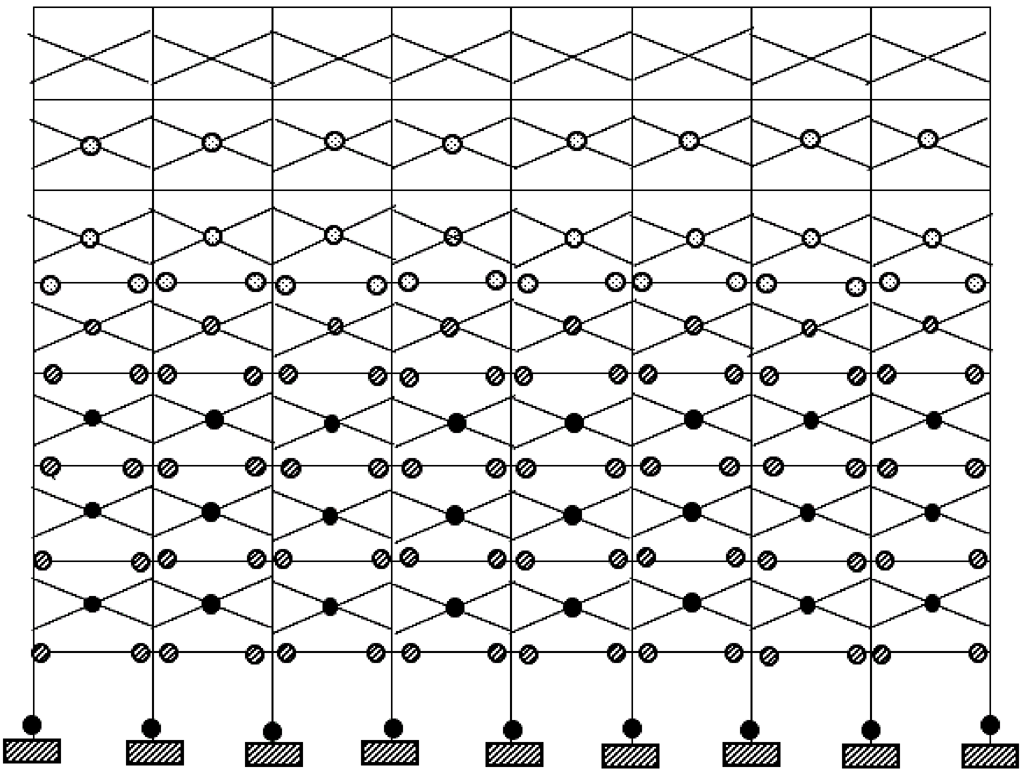


(d)

⊚ Immediate Occupancy (IO) ⊘ Life Safety (LS) ● Failure (ASCE 41-17)

Fig. 4.9 Typical yielding at critical frame of high-rise buildings at DBE hazard level; (a) OGS_BIS2002; (b) OGSW; at collapse (c) OGS_BIS2002; (d) OGSW

4.5 Summary

Open ground storey RC frame buildings have always remained vulnerable to earthquakes, consequently had to endure sever damage to complete collapse during past earthquakes. Although, several studies have been undertaken to remove the strength and stiffness irregularities from the open storey, however, limited studies can be found on the realistic assessment of seismic performance and consequent fragility of representative OGS buildings having realistic combination of functional openings in upper storey infills in the form of doors and windows that are integral part of infilled frame buildings. Present study evaluates the effect of different functional opening combinations in upper storey infills due to presence of doors and windows on the seismic performance of the OGS RC infilled frame buildings with different design levels that are commonly observed in India. It can be concluded from the present study that presence of functional opening in upper storey infills has significant negative impact on the seismic performance of OGS buildings. The lateral strength, stiffness and ductility of OGS building reduces with increase in functional opening in upper storey infills. The lateral stiffness and strength of mid-rise OGS buildings designed as per BIS (2002) provision is reduced to 55% and 65% respectively, when functional opening reached to 30%, and thereby affecting the adequacy of BIS (2002) OGS design provision of designing open storey members with 2.5 times higher design base shear. Functional openings in infills enhanced the flexibility of the OGS building for all the design levels, thus yield and ultimate displacement capacity is found to be increased with increase in opening. However, ductility is reduced as plastic displacement does not increase with the same rate as yield displacement. Ductility of OGS buildings is found to be very sensitive towards increase in functional opening. The reduction in ductility is significant for OGS buildings designed without BIS (2002) OGS provision. It can be further concluded that global collapse of OGS buildings designed without BIS (2002) OGS provision is mainly governed by failure of ground storey members, attributed to excessive inelastic deformation demand in the ground storey. However, collapse of ground storey can be prevented for OGS buildings designed with BIS (2002) provision, both at DBE and MCE hazard levels, and it could attain higher strength, stiffness before global collapse caused by failure of infills and beams up to mid-height zone followed by failure of first storey beams and columns. Functional openings in the form of doors and windows are the integral part of building, and cannot be avoided completely. Functional openings in the form of doors and windows are integral part of a building and therefore cannot be avoided completely. It is evident from the present study that increase in functional openings causes decrease in strength and stiffness thereby significantly affect the seismic performance of OGS buildings. As functional openings cannot be avoided completely due to practical purposes, therefore, selection of optimum level of opening may serve the necessary functional requirement without affecting the seismic performance of OGS buildings drastically. According to the findings of the present study, 15% opening can be considered as optimum percentage of opening in the building where the strength and ductility can be achieved up to 80% of the OGS building as compared to OGS buildings with solid upper storey infills.

Chapter 5
Prescriptive Seismic Design for Open Ground Storey

5.1 Introduction

The prevalence of Reinforced Concrete (RC) frame buildings with Open Ground Storey (OGS) stands as a ubiquitous architectural typology not only in India but also in various regions globally. This design choice, where open space is deliberately maintained at the ground level without infills, serves both aesthetic and architectural purposes while primarily functioning as a parking area for stakeholders. However, this strategic incorporation of an open storey introduces strength and stiffness irregularities, wherein the lateral storey strength and stiffness experience a significant reduction at the open ground storey compared to its adjoining upper storey(s) (Jain et al. 2002; Kaushik et al. 2009; Choudhury and Kaushik 2018; Pavel and Carale 2019; Noorifard et al. 2020; Aggarwal and Saha 2021; Borsaikia et al. 2021). Historical seismic events, as documented (Jain et al. 2002; Gattulli et al. 2013; Sharma et al. 2013; Perrone et al. 2019; Ozturk et al. 2023; Qu et al. 2023) have underscored the vulnerability of open storey members to heavy to severe damage, often leading to structural collapse. This vulnerability is attributed to the manifestation of a soft-storey mechanism during earthquakes, particularly affecting the open storey, while the upper storey typically sustains minimal damage. These observations emphasize the critical need for a comprehensive understanding of the dynamic interplay between architectural choices, structural vulnerabilities, and seismic resilience in RC frame buildings with open ground storey. Further research in this domain is imperative to inform resilient design practices and enhance the seismic performance of this prevalent building typology.

In order to avoid severe consequences of poor performance of OGS buildings, International Building Code ICC IBC (2012), NZSEE (2006), ASCE/SEI 7 (2010) prohibit extremely irregular buildings in seismically active areas. However, to allow the functional advantage of the open storey for all practical purposes, compensation of storey stiffness and strength deficiency is essential at the design stage. Several national standards like Bulgarian seismic code (1987), BIS (2002), and Eurocode-8 (2004) have suggested the use of multiplication factors to increase the design force in the open storey members. Israel seismic code (SI-413 1995) suggests increasing the design force for the open storey along with adjacent storey members. Table 5.1 summarizes an overview of OGS design schemes suggested by various national standards. It can be observed from Table 5.1 that simple design guidelines for open storey suggested in BIS (2002) (i.e. open storey columns and beams to be designed for 2.5 times the normal base shear) have been removed from the revised Indian seismic design standard BIS (2016a), and suitable measures like RC structural wall or bracings at the selected open bays to increase the strength and stiffness of the open storey have been suggested, and left to the intelligence of designer in-charge.

Table 5.1 OGS design methods suggested by various national standards

National standards	Stiffness irregularity criteria	Multiplication Factors (MF) range	Application of multiplication factors
(SI-413 1995)	$\frac{K_i}{K_{i+1}} < 0.7$	2.1 – 3	Open and adjacent storey beams and columns
(Bulgarian seismic code 1987)	$\frac{K_i}{K_{i+1}} < 0.5$	3	Open storey beams and columns
(BIS 2002)	$\frac{K_i}{K_{i+1}} < 0.7$ or $\frac{K_i}{avg(K_{i+1,2,3})} < 0.8$	2.5	Open storey beams and columns
Eurocode-8 (2004)	Drastic reduction of infill in any storey	$1 + \frac{\Delta V_{RW}}{\sum V_{ED}}$ (1.5 – 4.68)	Open storey columns
(NZSEE 2006)	NA	NA	NA
(ASCE/SEI 7 2010)	NA	NA	NA
(ICC IBC 2012)	NA	NA	NA
(BIS 2016a)	$K_i < K_{i+1}$	NA	Use of bracings and RC structural wall

where, k_i and k_{i+1} represents the lateral stiffness of the i^{th} and $i+1$ th storey, respectively. ΔV_{RW} is the strength of infill in the above storey and $\sum V_{ED}$ sum of design lateral force in the storey

Haldar et al. (2016) studied the efficacy of design provision of the open ground storey in BIS (2002) by comparing the seismic performance of Uniformly Infilled (UI) buildings with the designed open ground storey buildings. It has been found that open ground storey buildings designed as per BIS (2002) can attain the stiffness and strength close to those of the corresponding uniformly infilled frame building, as design of open ground storey members with 2.5 times design base shear led to an increase in the size of open storey beams and columns. It has also been reported that open ground storey buildings performed slightly better than the uniformly infilled frame buildings, indicating the adequacy of the BIS (2002) provisions for open ground storey buildings. Considering the susceptibility of large stock of the existing deficient OGS buildings to seismic excitation, significant research efforts have been devoted to develop retrofitting strategies to minimize the strength and stiffness irregularities of soft storey viz. use of bracings (Kaushik et al. 2009; Khan and Rawat 2016; Mashhadiali and Kheyroddin 2018; Shahsahebi et al. 2020; Lal and Remanan 2023) and energy dissipation devices (Sahoo and Rai 2010; Sahoo and Rai 2013; Teruna et al. 2014; Benavent-Climent and Mota-Páez 2017; Mazza et al. 2018; Mashhadiali et al. 2021; Ruiz et al. 2021; Das et al. 2023).

Kaushik et al. (2009) proposed seismic strengthening options for open storey i.e., use of multiplication factor, additional columns, diagonal bracings, and lateral buttress, for a 4-storey 3-bay RC frame. They concluded that multiplication factor of 2.5 used to increase the design force of open storey members enhanced the peak lateral strength of the considered frame, but not the ductility. Further, the use of additional columns at every open bay was found to be most suitable in increasing both the strength and ductility, and the lateral buttress was most suitable to achieve high lateral strength of the overall frame. However, the outcome of this study is suitable for retrofitting the existing buildings but have practical limitations restricting the functional requirement for which OGS is being provided. The use of additional columns and bracings may restrict the functional requirements of the open storey. Moreover, the use of lateral buttress requires a clear lateral dimension of 2*m* to 4*m*, which may not be a feasible solution in densely populated urban areas due to lack of space on the sides owing to building bye-laws.

Haran Pragalath et al. (2016) studied the seismic performance of typical OGS frames (2-bay with varying height) where only open/adjacent storey columns were designed by recommended multiplication factors of various national standards.

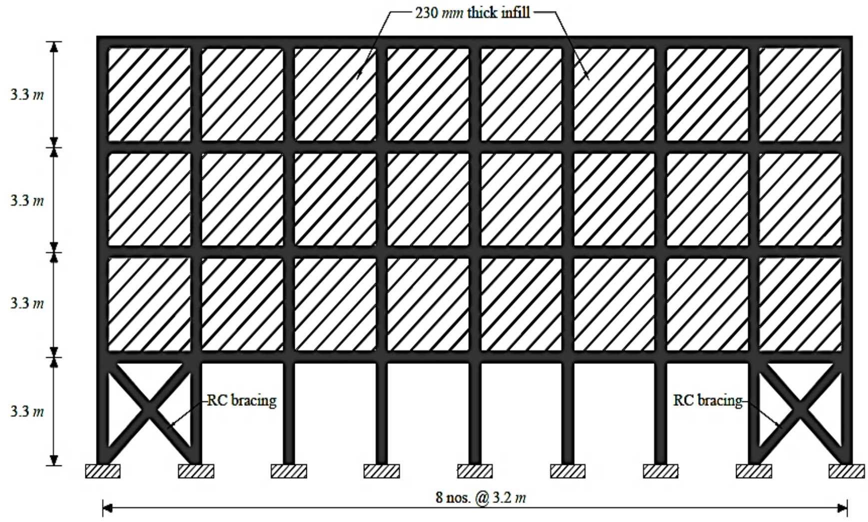
However, the study did not capture the complications of actual three-dimensional structures and their effect on the seismic performance of the overall frame in terms of strength, stiffness and ductility. The outcome of the study indicated that RC frames where both the open ground and adjacent storey columns were designed with a multiplication factor of 3 as suggested by the Israel seismic code resulted in better comparative performance in terms of reliability and cost. Indian standard for seismic design practice is based on the force-based design method like most of the design standards worldwide in which the effect of ductility is considered, indirectly, in the form of a response reduction factor. Therefore, the findings by Haran Pragalath et al. (2016) has limited application in prediction of seismic behaviour in terms of force, displacement and ductility of OGS buildings designed as per force-based design framework followed by most of the seismic design standards of the world.

Despite significant research effort towards retrofitting strategies for existing stock of OGS buildings, design of upcoming OGS buildings by eliminating strength and stiffness irregularity from open storey still remains a challenging task for the structural design practitioners as it requires special skill set, cost, effort and time intensive nonlinear dynamic evaluation. In the search of inexpensive, simple solution for all practical purposes without requiring explicit expertise of non-linear dynamic analysis, an exhaustive comparative study has been carried out in this Thesis, on the effect of OGS design interventions recommended by various national seismic design standards on sets of mid-rise (4-storey) and high-rise (8-storey) infilled RC frame buildings with an open ground storey designed as per relevant Indian standards, in deterministic, as well as probabilistic terms. The effect of relevant design provisions viz. multiplication factor to design for enhanced seismic force in the open storey and use of diagonal RC bracings, on the seismic performance in terms of lateral yield strength, peak strength, effective stiffness, ductility and inelastic deformation capacity of the structure.

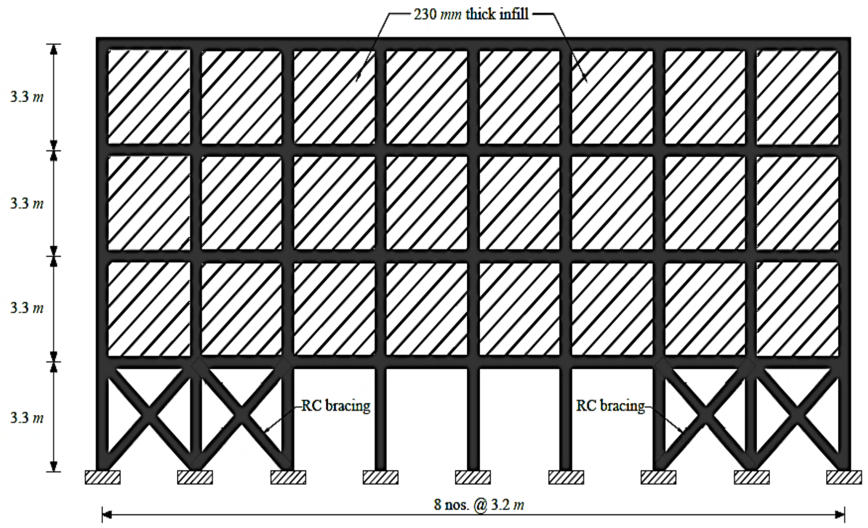
5.2 Efficacy of OGS Design Interventions Recommended by Various National Seismic Design Standards

Two sets of Special Moment Resisting Frame (SMRF) infilled RC buildings with open ground storey viz. mid-rise (4-storey) and high-rise (8-storey) designed (BIS 2016a) and detailed (BIS 2016b) as per revised Indian seismic design standards have been considered for the parametric study. The plan of the considered building is same as considered for parametric study in Chapter 4 (Fig. 4.1).

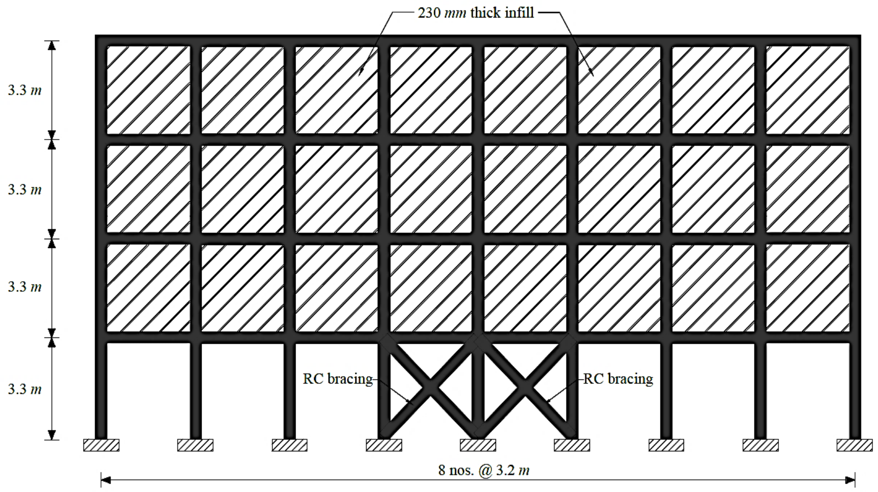
To ensure ductile behaviour of SMRF buildings BIS (2016a) enforces two governing major design criteria: (i) column dimension shall not be less than 20 times the diameter of the larger longitudinal rebar, and (ii) capacity design by maintaining the minimum Strong-Column Weak-Beam (SCWB) ratio of 1.4. Unfortunately, even in high seismic zones, considerably large stock of RC buildings exists in India, where the ground storey is kept open without following any OGS design provisions (DEQ 2009). Therefore, another set of mid-rise and high-rise open ground storey SMRF RC frame buildings have also been considered for this study, neither designed with any multiplication factor nor maintaining SCWB ratio. Further, to eliminate strength and stiffness irregularities of OGS, open storey beams and columns have been designed with multiplication factors suggested by the considered national standards as presented in Table 5.1 and applied to their respective open and adjacent storey members. As discussed in the earlier Section, Indian seismic design standard BIS (2016a) recommends the use of bracings or RC structural wall at the selected open bays instead of any multiplication factor for the design of the open storey beams and columns. However, the locations and design details of bracings or shear walls are not explicitly mentioned by the standard and left to the discretion of the designer in-charge. This may lead to complications as it is not possible to estimate the non-linear response and failure pattern of the structure without performing time expensive cumbersome non-linear analysis requiring special expertise that is generally not abundant in the design offices. In order to assess the efficacy of RC bracings to eliminate stiffness and strength irregularity of the open storey, diagonal RC bracings were provided at all grids along the longitudinal direction as shown in Fig. 5.1, and only at the exterior grids along the transverse direction. In the present study, the placement of diagonal RC bracings was selected considering the utility space of the open storey so that vehicle parking functionality should not get affected. The beam members have been proportioned to have a maximum of 1 to 1.2% steel in each face, and column sizes have been estimated as per Indian ductile design and detailing standard BIS (2016b). In the case of mid-rise and high-rise OGS buildings designed and detailed as per BIS (1993, 2002), columns have been proportioned to have 2 to 4% steel. Description and estimated member sizes of the considered set of buildings are represented in Table 5.2.



(a) Bracings type 1



(b) Bracings type 2



(c) Bracings type 3

Fig. 5.1 Various arrangements of diagonal bracings considered for the parametric study

Table 5.2 Description and estimated member sizes of considered buildings

Buildings	Nomenclature of considered set of buildings	Description of considered set of buildings	Open/adjacent storey member sizes (*mm* X *mm*) Beams: Longitudinal	Beams: Transverse	Columns	Diagonal RC bracings
Mid-rise	**OGSW**	RC infilled frame with OGS without SCWB and MF	250X400	350X500	375X375	
	OOGS	RC infilled frame with OGS with SCWB without MF	250X400	350X500	500X500	--
	OGS_BSC	OGS designed with MF (3) from Bulgarian seismic code (1987)	400X600	400X700	575X575	--
	OGS_ISC	OGS designed with MF (3) from SI 413 (1995)	400X600	400X700	575X575	--
	OGS_BIS	OGS designed with MF (2.5) from BIS (2002)	350X550	350X650	550X550	--
	OGS_EC8	OGS designed with MF (2.17 & 2.23) from EC8 (2004)	250X400	350X500	500X500	--
	OOGS_BR1	OOGS building with diagonal RC bracings of type 1	250X400	350X500	500X500	500X500
	OOGS_BR2	OOGS building with diagonal RC bracings of type 2	250X400	350X500	500X500	500X500
	OOGS_BR3	OOGS building with diagonal RC bracings of type 3	250X400	350X500	500X500	500X500
	UI	RC frame with Uniformly placed infill	250X400	350X500	500X500	--
High-rise	**OGSW**	RC infilled frame with OGS without SCWB and MF	300X500	350X600	475X475	--
	OOGS	RC infilled frame with OGS with SCWB without MF	300X500	350X600	550X550	--
	OGS_BSC	OGS designed with MF (3) from Bulgarian seismic code (1987)	500X700	550X750	700X700	--
	OGS_ISC	OGS designed with MF (3) from SI 413 (1995)	500X700	550X750	700X700	--
	OGS_BIS	OGS designed with MF (2.5) from BIS (2002)	500X700	550X750	650X650	--
	OGS_EC8	OGS designed with MF (1.59 & 1.62) from EC8 (2004)	300X500	350X600	550X550	--
	OOGS_BR1	OOGS building with diagonal RC bracings of type 1	300X500	350X600	550X550	550X550
	OOGS_BR2	OOGS building with diagonal RC bracings of type 2	300X500	350X600	550X550	550X550
	OOGS_BR3	OOGS building with diagonal RC bracings of type 3	300X500	350X600	550X550	550X550
	UI	RC frame with Uniformly placed infill	300X500	350X600	550X550	--

The effect of different type of bracings for open storey strengthening was beyond the scope of the present study. Therefore, referring to the previous study (Kaushik et al. 2009) on various effectiveness of strengthening measure for open storey, RC diagonal bracings have been used with similar section, reinforcement and plastic hinge properties as open storey columns and thus design of RC diagonal bracings at open storey is also easy to apply for practicing engineers with less computational effort. The Presence of OGS mainly leads to stiffness and strength irregularities in the buildings. The storey stiffness is estimated using the fundamental lateral translational mode shape method (Ar et al. 2017), and the storey strength is estimated considering the sum of shear resistance capacity of columns and sliding shear strength of infill. Dynamic properties of the considered sets of buildings are presented in Table 5.3 which shows that modal mass participation is more than 75% in the fundamental mode of the considered direction for all the considered buildings except those having RC bracings.

Table 5.3 Dynamic properties of considered buildings

Buildings	**Building nomenclature**	**Fundamental time period obtained from modal analysis** (*sec*)		**Design time period obtained from BIS (2016a)** (*sec*)		**Modal mass participation** (%)	
		Long.	Trans.	Long.	Trans.	Long.	Trans.
Mid-rise	**OGSW**	0.63	0.69	0.23	0.31	98	97
	OOGS	0.46	0.51			95	93
	OGS_BSC	0.41	0.50			92	89
	OGS_ISC	0.39	0.48			93	89
	OGS_BIS	0.42	0.51			94	90
	OGS_EC8	0.46	0.51			96	94
	OOGS_BR1	0.28	0.34			72	71
	OOGS_BR2	0.26	0.34			70	67
	OOGS_BR3	0.27	0.34			72	70
	UI	0.34	0.41			85	84
High-rise	**OOGS_NOSC**	0.81	1.04	0.47	0.61	92	89
	OOGS	0.76	0.99			89	84
	OGS_BSC	0.65	0.89			80	77
	OGS_ISC	0.61	0.85			77	75
	OGS_BIS	0.66	0.91			82	79
	OGS_EC8	0.76	0.99			87	83
	OOGS_BR1	0.60	0.85			75	73
	OOGS_BR2	0.60	0.85			73	72
	OOGS_BR3	0.61	0.85			74	73
	UI	0.68	0.93			81	80

However, in the present study, response of OGS buildings with RC bracings has also been analyzed using the non-linear static procedure considering insignificant higher modes participation where 90% mass participation has been achieved within a few modes (FEMA-356 2000). In practice, designers tend to make flexible buildings using the natural period from the analytical model, resulting in lower design base shear due to a longer period of vibration (Haldar and Singh 2009) which can be observed from Table 5.3. To safeguard against this error, Indian seismic design standard BIS (2016a) has recommended a capping on the natural period (T_a) used for the base shear calculation. The present study utilizes the empirical expression $T_a = \frac{0.09h}{\sqrt{d}}$ specified in BIS (2016a) to estimate the design natural period for RC frame buildings with Un-reinforced Masonry (URM) infills. Where, T_a is the design natural period of a building in s having the height equal to h in m. D is the base dimension of the building at the plinth level, in m, along the considered direction of the lateral force. The capping is implemented by scaling all the response quantities by a factor equal to $\overline{V_B}/V_B$, where $\overline{V_B}$ is the base shear calculated by using the empirical design period, and V_B is the base shear obtained from period of the analytical model.

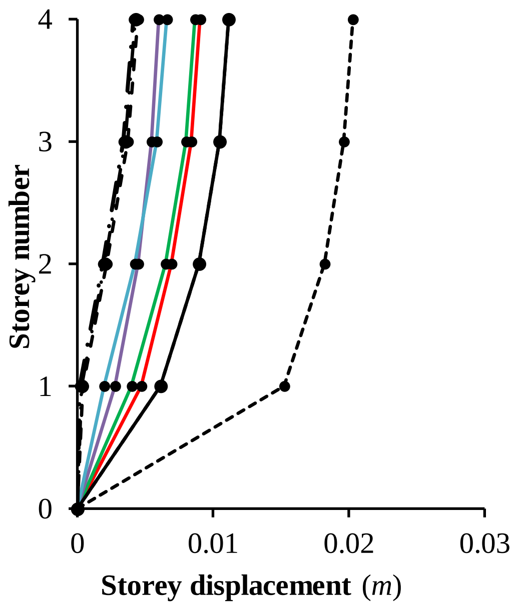
5.3 Evaluation of Stiffness and Strength Irregularities of Considered OGS Buildings

Indian seismic design standard BIS (2016a) defines a building to have soft/weak storey when lateral stiffness/strength of the storey is less than that of the storey above, without any quantitative limits for soft and weak storey, unlike its earlier version BIS (2002) and other national codes (Bulgarian seismic code 1987; SI-413 1995). A closer look at Table 5.1 confirms that many design standards of the world (Bulgarian seismic code 1987; SI-413 1995; BIS 2002) quantify stiffness irregularity using limits for identification of soft storey, which ranges from 50% to 70% stiffness difference between the two consecutive storey(s). In the absence of such limits for quantifying stiffness and strength irregularities in the latest Indian seismic standard BIS (2016a), stiffness and strength irregularity of the considered set of buildings as presented in Table 5.4 have been evaluated as per BIS (2002). Stiffness and strength irregularities were found to be eliminated for all the considered sets of buildings except OOGS_NOSC, where no design provisions for SCWB, and OGS have been adopted. Strength irregularity in mid-rise and high-rise OOGS buildings could be removed in order to satisfy two major design provisions i.e., capacity design and selection of column dimension based on the largest beam longitudinal rebar incorporated in its

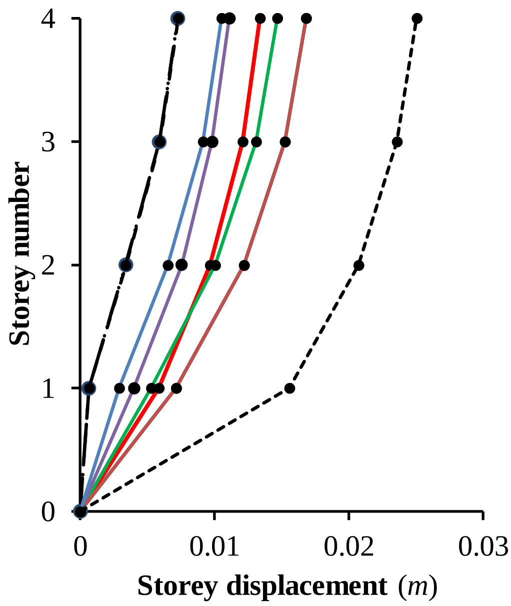
2016 edition (BIS 2016a). The combined effect of these two criteria results in larger column sizes, and hence is found effective in eliminating the storey weakness in case of both the mid-rise and high-rise OOGS buildings.

Table 5.4 Evaluation of storey stiffness and strength irregularity as per BIS (2002) of considered buildings

Buildings	Building nomenclature	Evaluation criteria: Soft storey	Evaluation criteria: Weak storey	$\frac{K_i}{K_{i+1}}$	$\frac{F_i}{F_{i+1}}$	Irregularity type: Soft storey	Irregularity type: weak storey
Mid-rise	OGSW	$\frac{K_i}{K_{i+1}} <$ 0.7	$\frac{F_i}{F_{i+1}} <$ 0.8	0.43	0.75	Yes	Yes
	OOGS			0.63	0.89	Yes	No
	OGS_BSC			1.02	1.03	No	No
	OGS_ISC			0.94	0.92	No	No
	OGS_BIS			0.73	0.92	No	No
	OGS_EC8			0.72	0.89	No	No
	OOGS_BR1			4.7	2.11	No	No
	OOGS_BR2			7.1	2.72	No	No
	OOGS_BR3			5.22	2.11	No	No
	UI			1.34	1.03	No	No
High-rise	OOGS_NOSC			0.62	0.78	Yes	Yes
	OOGS			0.81	0.9	No	No
	OGS_BSC			1.56	1.25	No	No
	OGS_ISC			1.56	1.24	No	No
	OGS_BIS			1.48	1.13	No	No
	OGS_EC8			1.02	0.9	No	No
	OOGS_BR1			5.23	1.8	No	No
	OOGS_BR2			6.38	2.4	No	No
	OOGS_BR3			5.71	1.8	No	No
	UI			2.04	1.02	No	No



(a) Longitudinal direction



(b) Transverse direction

Fig. 5.2 Storey displacement response of mid-rise OGS buildings at design force level

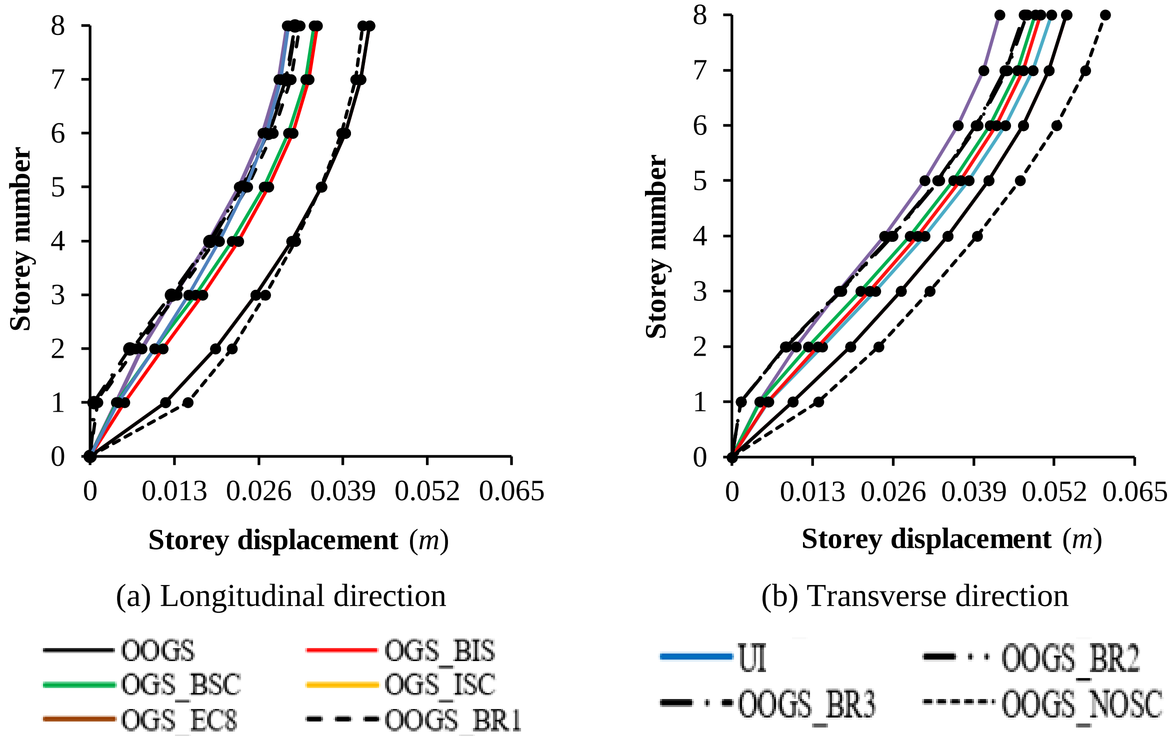
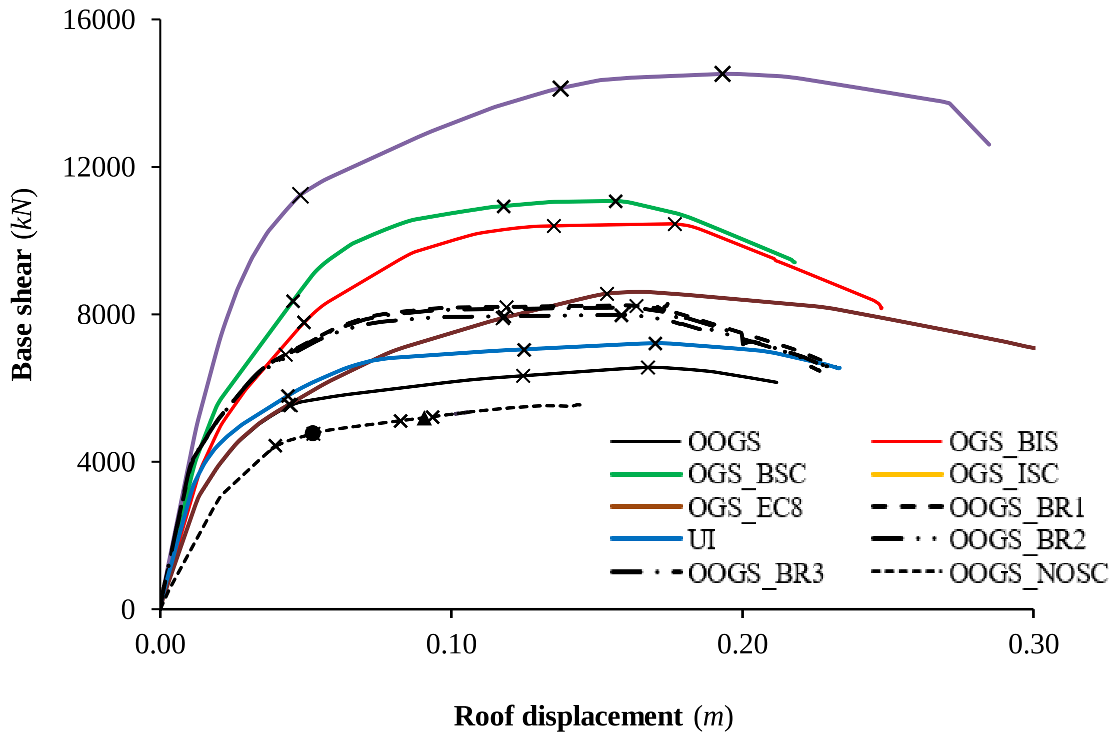


Fig. 5.3 Storey displacement response of high-rise OGS buildings at design force level

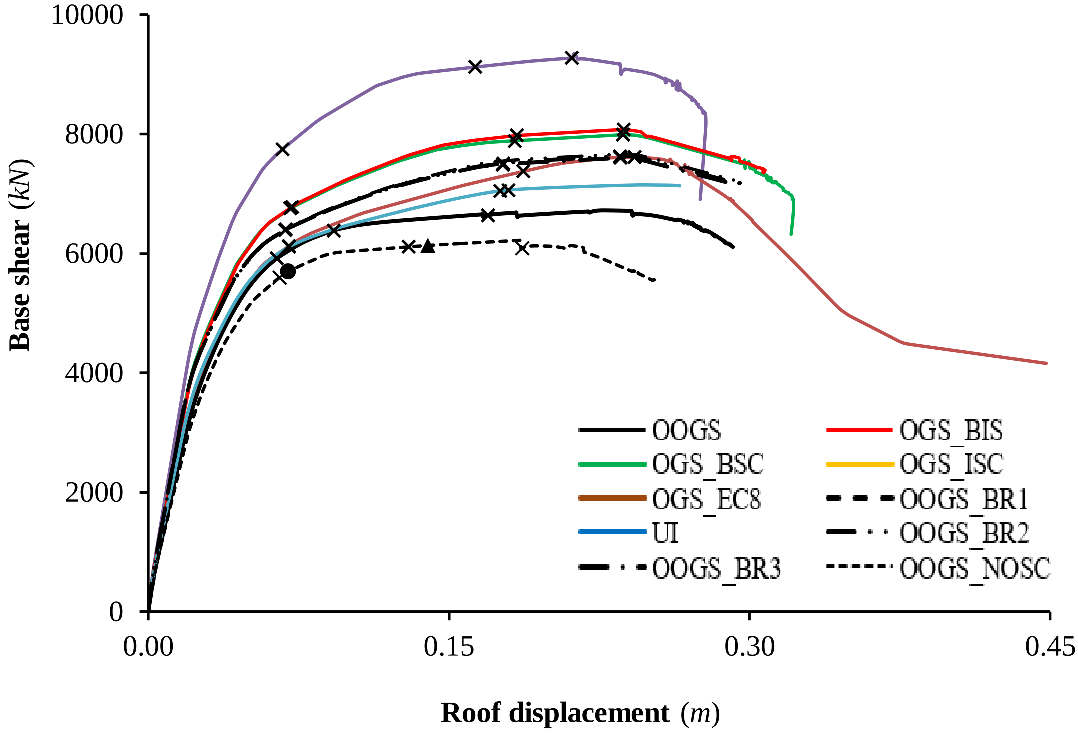
The use of multiplication factors to increase the design force for the open storey members is found effective in eliminating the open storey stiffness and strength irregularity for all the considered mid-rise and high-rise buildings. It is also observed that the use of diagonal RC bracings at the selected open bays have increased the open storey lateral stiffness by 4 to 7 times as compared to the adjacent upper storey for both the mid-rise and high-rise OGS buildings. The lateral strength of the open storey is also increased significantly by 1.8 to 2.7 times due to the presence of diagonal RC bracings. It can be further observed from storey displacement responses extracted from response spectrum analysis as per BIS (2016a), as shown in Figs. 5.2 and 5.3, that the presence of bracings strictly restricts the lateral movement of the open storey, resulting in an abrupt increase in the displacement demand, and eventually stress concentration at the adjacent storey above. On the other hand, a sharp change in displacement, although much lesser than OOGS with RC bracings, can be observed between 2nd and 3rd storey in case of high-rise OGS_ISC building (Fig. 5.3) as in this building, 2nd storey members were also designed with higher seismic forces, resulting in larger member sizes as compared to 3rd storey. Further, noticeable storey stiffness and strength irregularity were also observed between 2nd and 3rd storey(s). The stiffness and strength of 3rd storey are found to be 81% and 73% of 2nd storey, respectively, and making the 3rd storey a weak storey as per BIS (2002).

5.4 Seismic Performance of the Considered OGS Buildings

Seismic performance of the considered sets of buildings has been evaluated through capacity curves obtained using non-linear static pushover analyses. Figs. 7 and 8 represent capacity curves of mid-rise and high-rise RC frame buildings, respectively. The three crosses (x) in the capacity curves represent Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels, consecutively as defined by ASCE-41 (2017). Black dot and triangle represent the target displacements (performance points) at DBE and MCE hazard level estimated as per ASCE-41 (2017). To maintain clarity and brevity, capacity curves along longitudinal direction only is shown in Figs. 7 and 8. It has been observed that except the OOGS_NOSC building, all the OGS and UI buildings satisfy IO and LS performance levels at Design Basis Earthquake (DBE) hazard level (0.18*g*), and Maximum Considered Earthquake (MCE) hazard level (0.36*g*), respectively (not marked in Figs. 6 and 7 to maintain clarity).



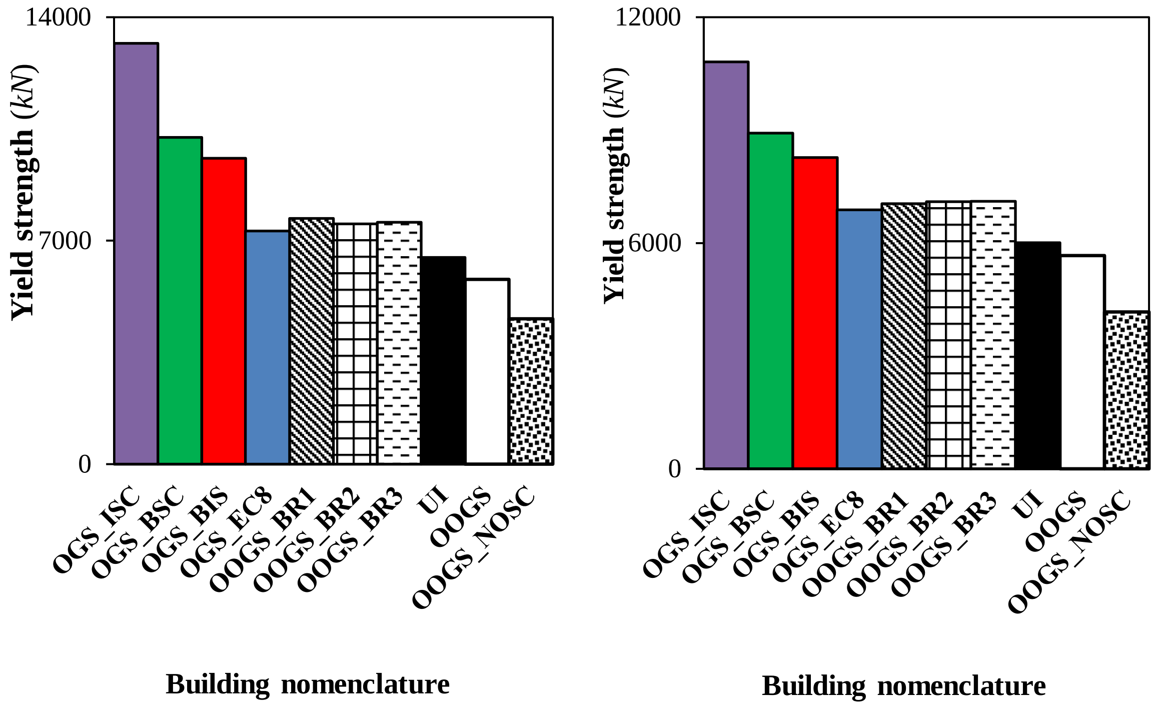
(a)



(b)

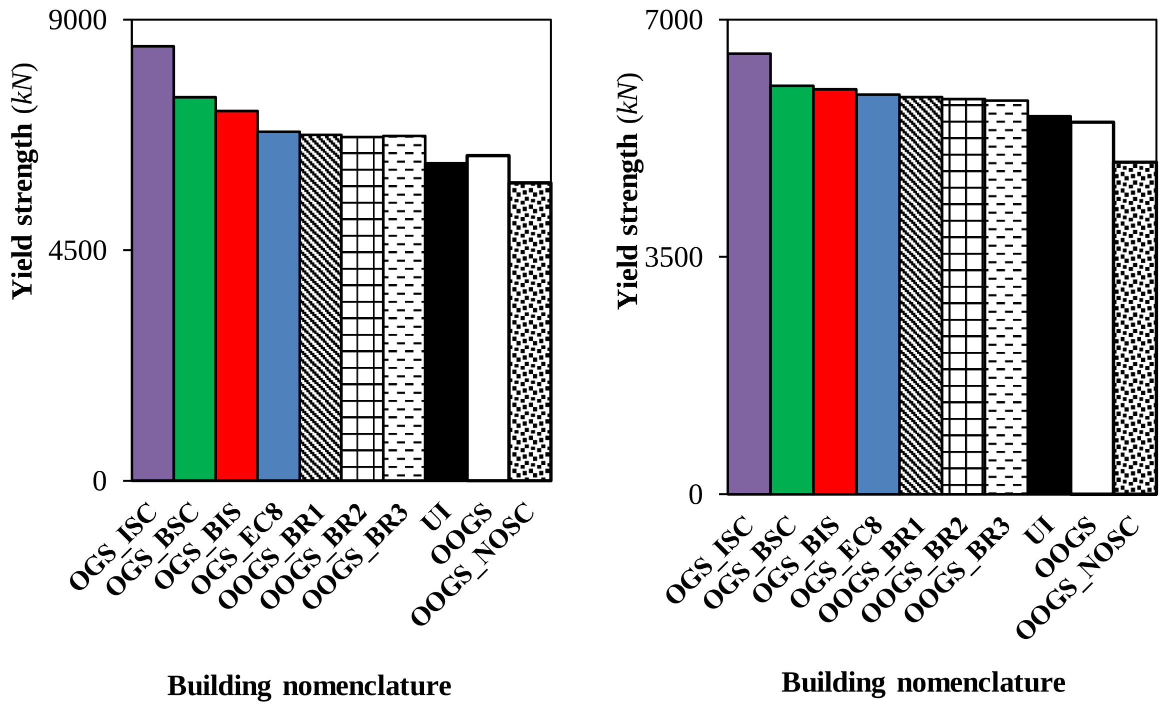
Fig. 5.4 Capacity curves of (a) mid-rise; (b) high-rise buildings. Three crosses (**x**) in the capacity curves represent Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels, consecutively. Black dot and triangle represent target displacements at DBE and MCE level

It has been further observed that OGSW buildings showed LS performance level at DBE level, and suffered near to collapse at MCE hazard level as observed from Figs. 6 and 7, due to strength and stiffness irregularity at the open storey. Seismic performance parameters have been evaluated in terms of strength (yield and peak), stiffness, inelastic displacement, and ductility in terms of Capacity-Demand Ratio (CDR) which are shown in Figs. 5.5 to 5.16.



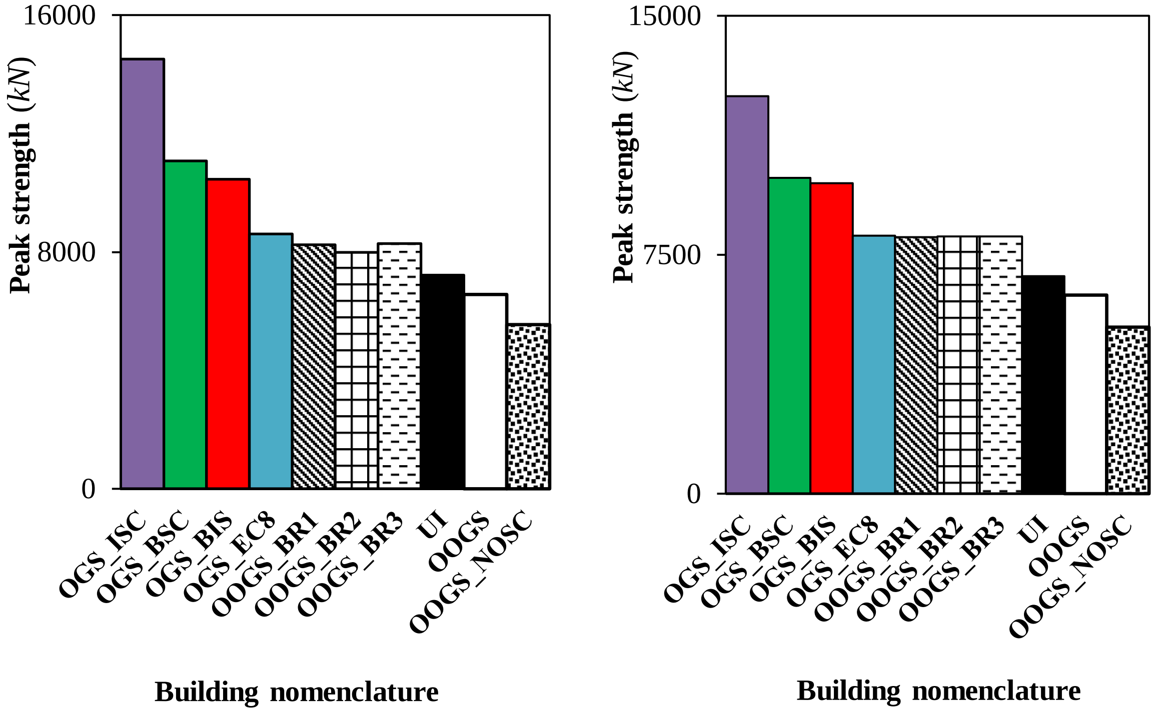
(a) Longitudinal direction (b) Transverse direction

Fig. 5.5 Comparison of yield strength of considered mid-rise OGS buildings



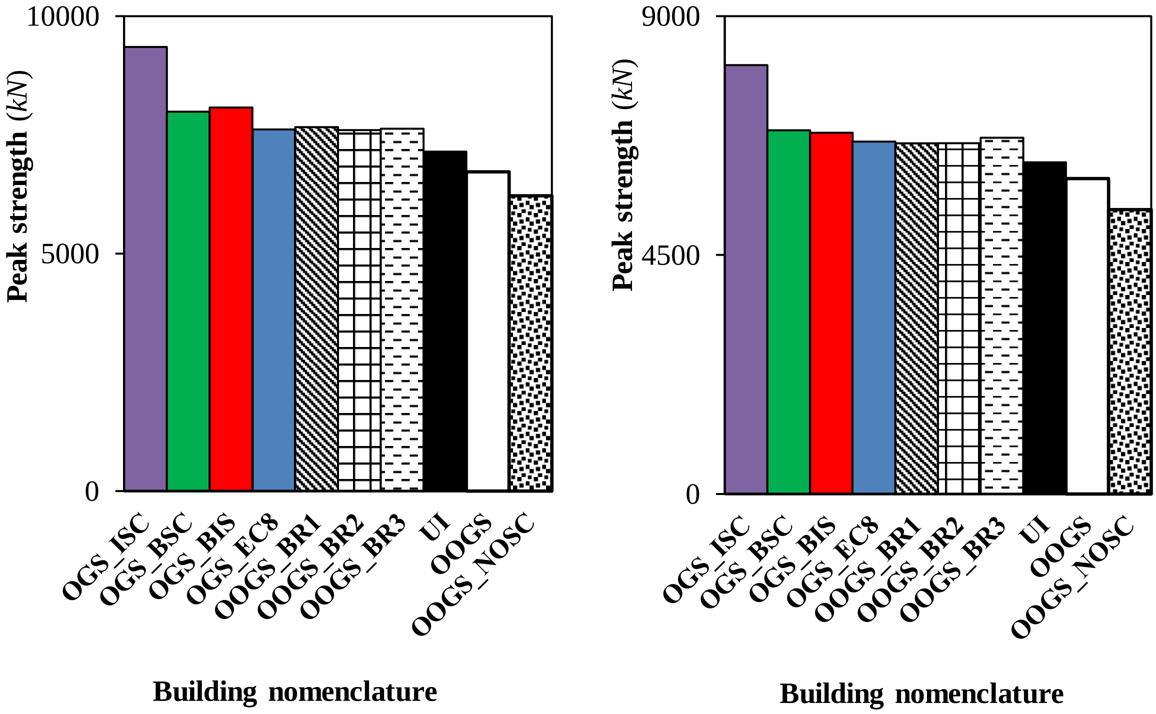
(a) Longitudinal direction (b) Transverse direction

Fig. 5.6 Comparison of yield strength of considered high-rise OGS buildings



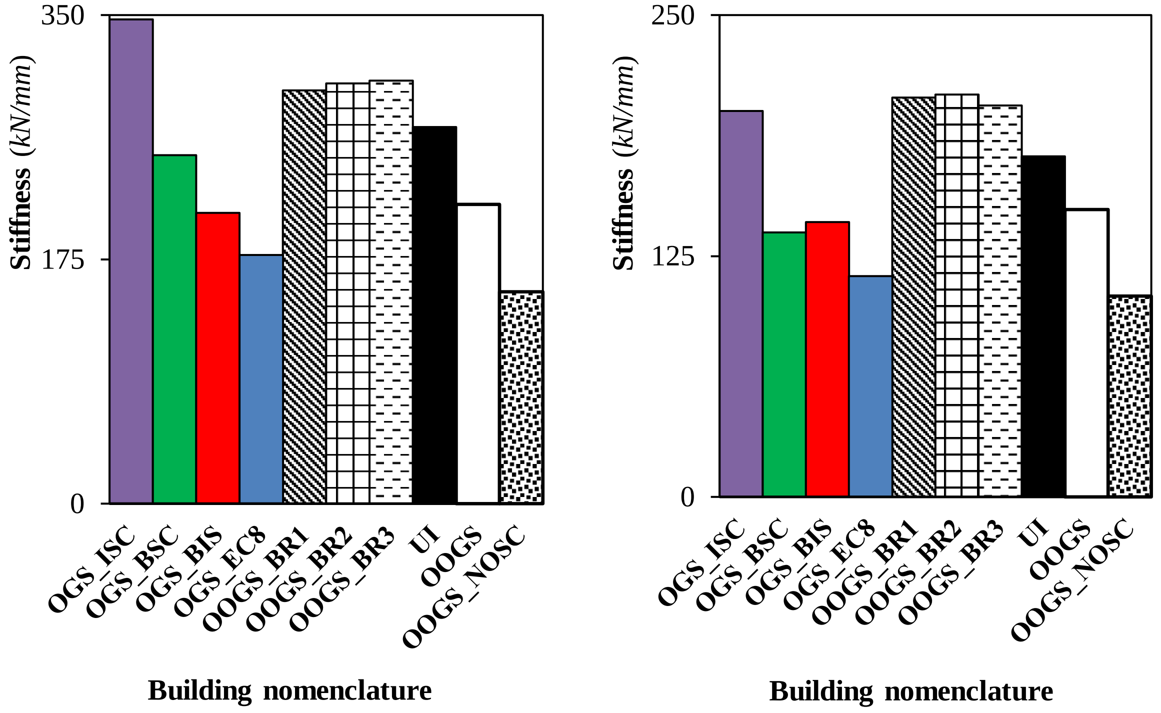
(a) Longitudinal direction (b) Transverse direction

Fig. 5.7 Comparison of peak strength of considered OGS mid-rise buildings



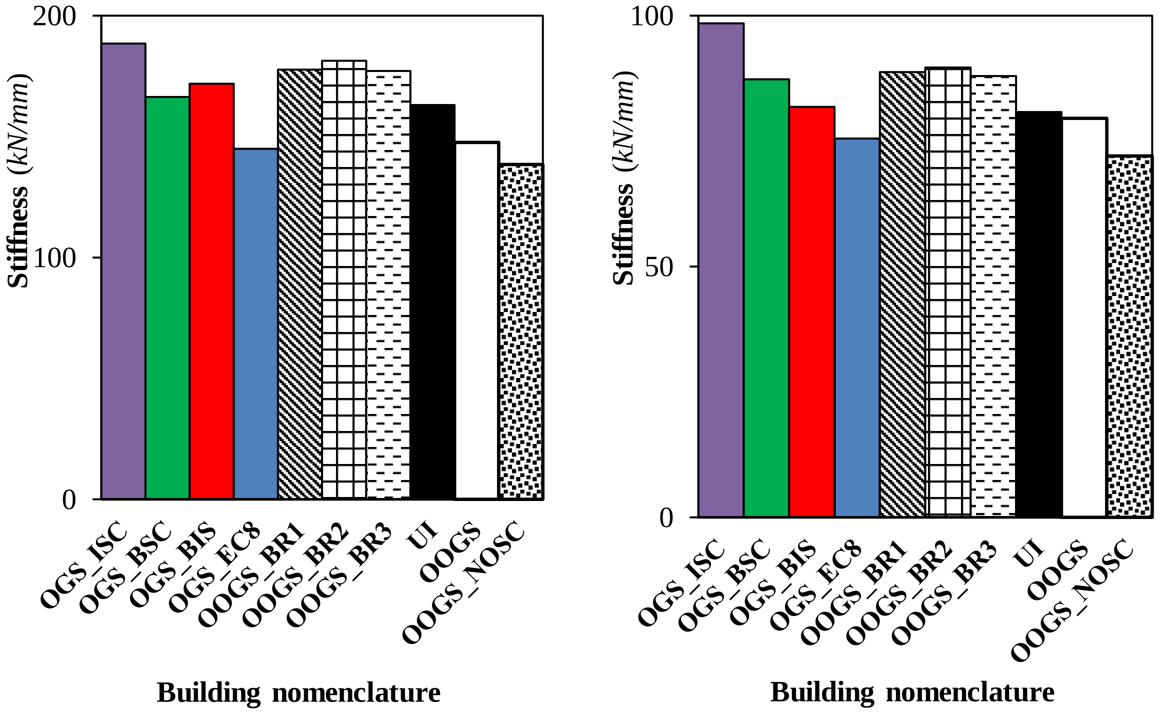
(a) Longitudinal direction (b) Transverse direction

Fig. 5.8 Comparison of peak strength of considered OGS high-rise buildings



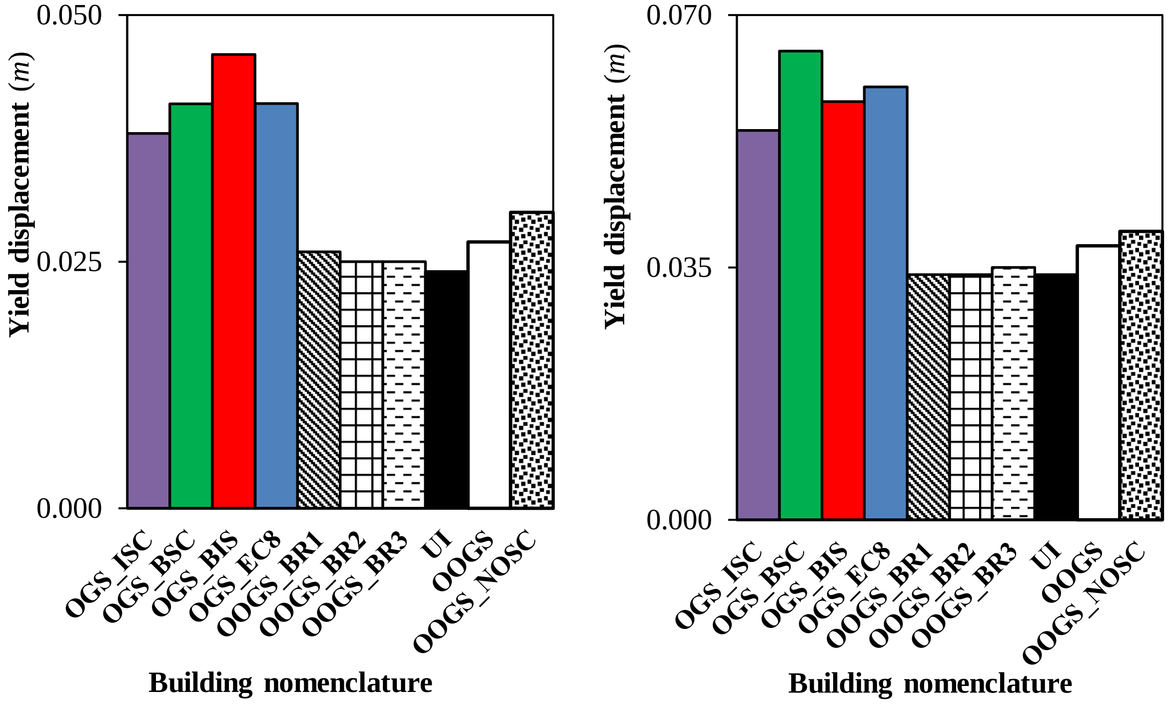
(a) Longitudinal direction (b) Transverse direction

Fig. 5.9 Comparison of stiffness of considered mid-rise OGS buildings



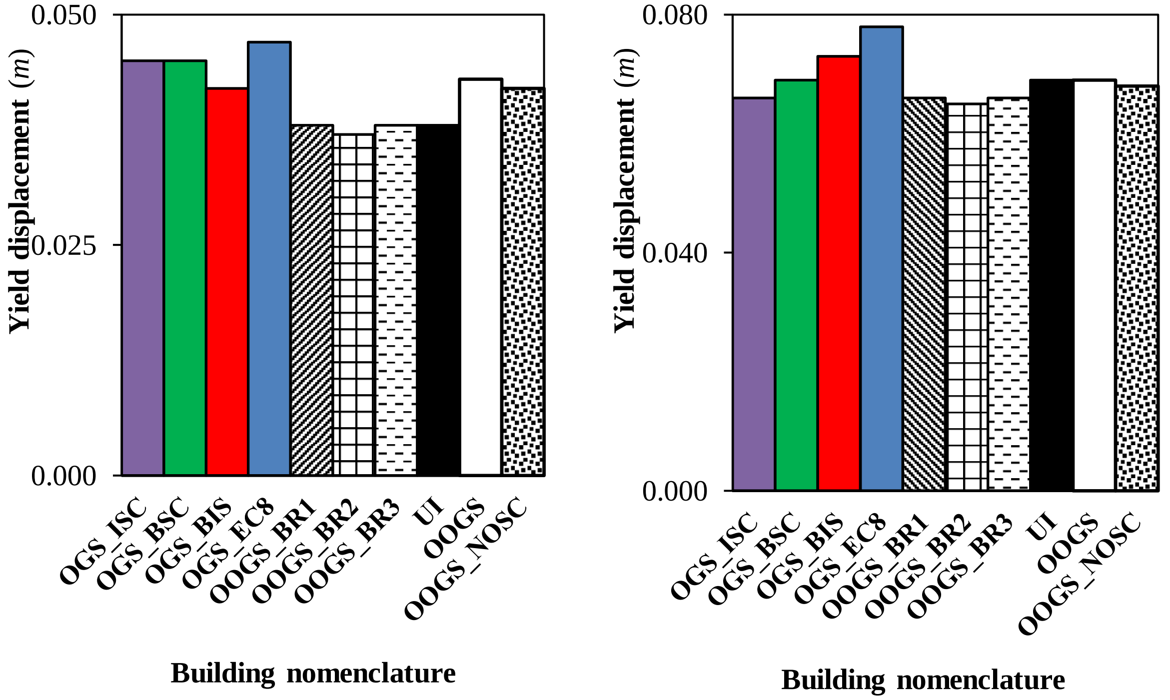
(a) Longitudinal direction (b) Transverse direction

Fig. 5.10 Comparison of stiffness of considered high-rise OGS buildings



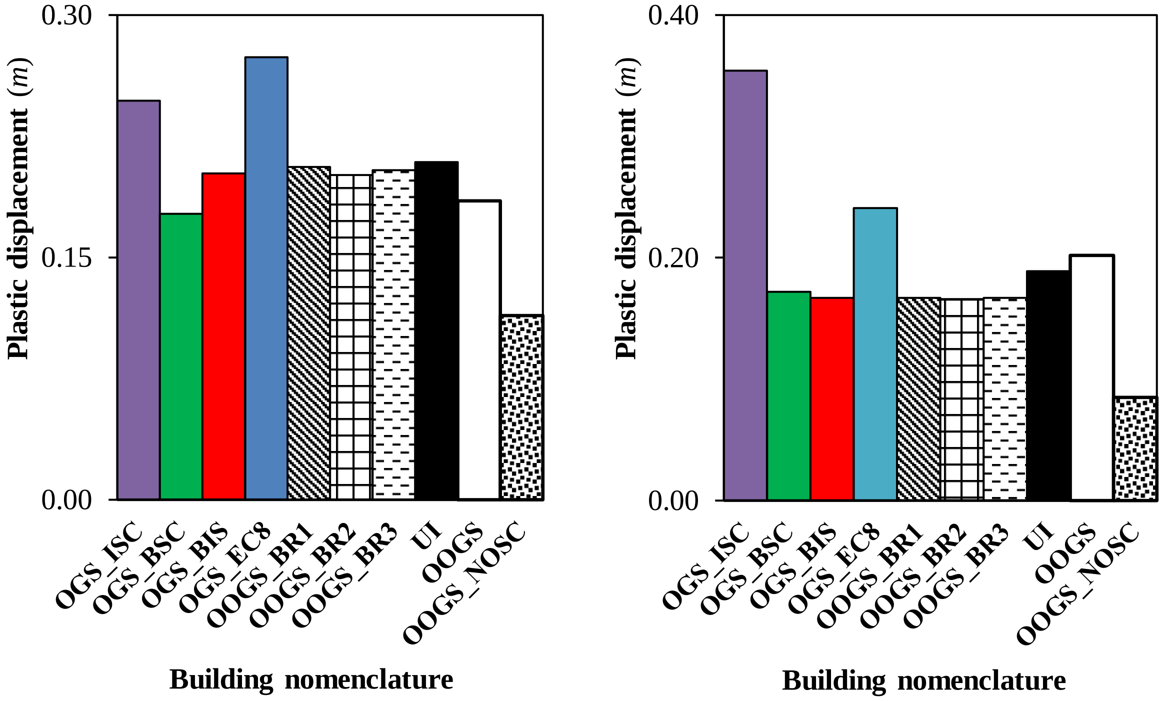
(a) Longitudinal direction (b) Transverse direction

Fig. 5.11 Comparison of yield displacement of considered mid-rise OGS buildings



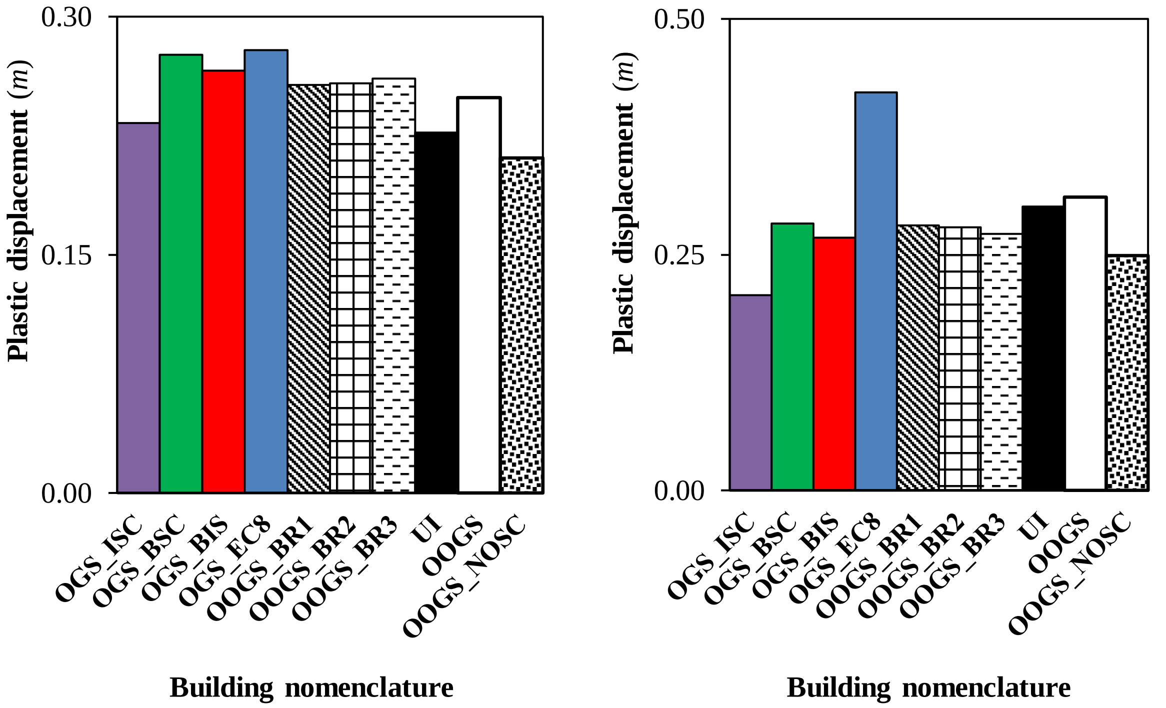
(a) Longitudinal direction (b) Transverse direction

Fig. 5.12 Comparison of yield displacement of considered high-rise OGS buildings



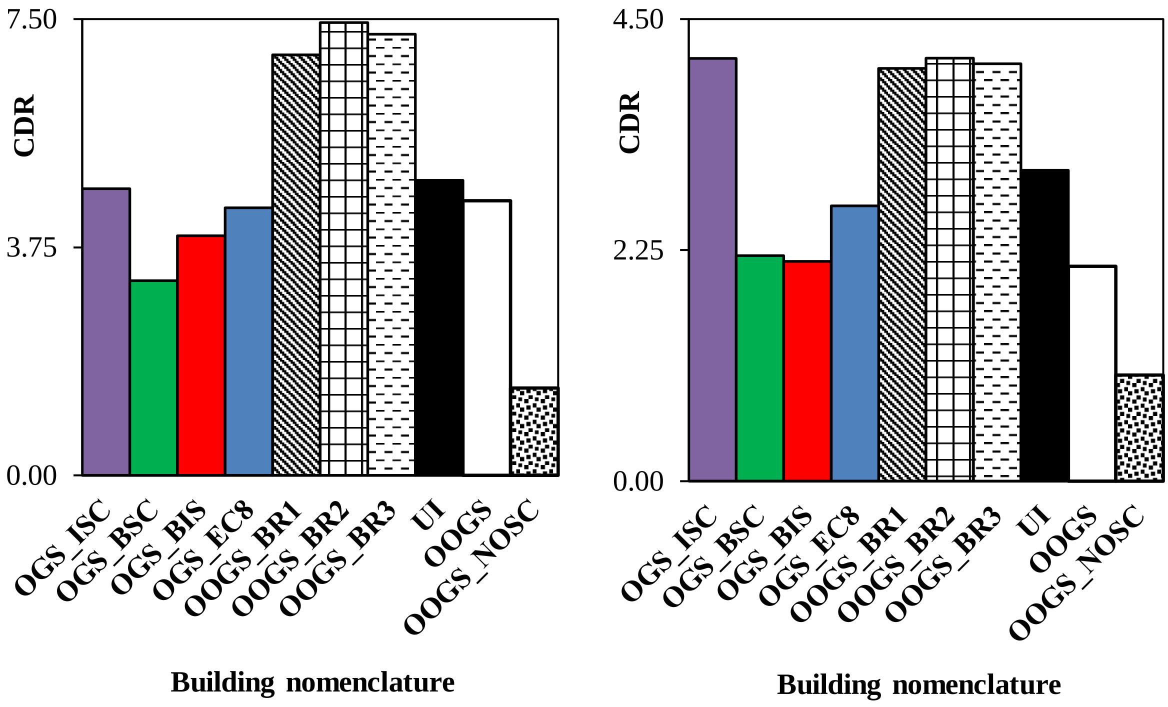
(a) Longitudinal direction (b) Transverse direction

Fig. 5.13 Comparison of plastic displacement of considered mid-rise OGS buildings



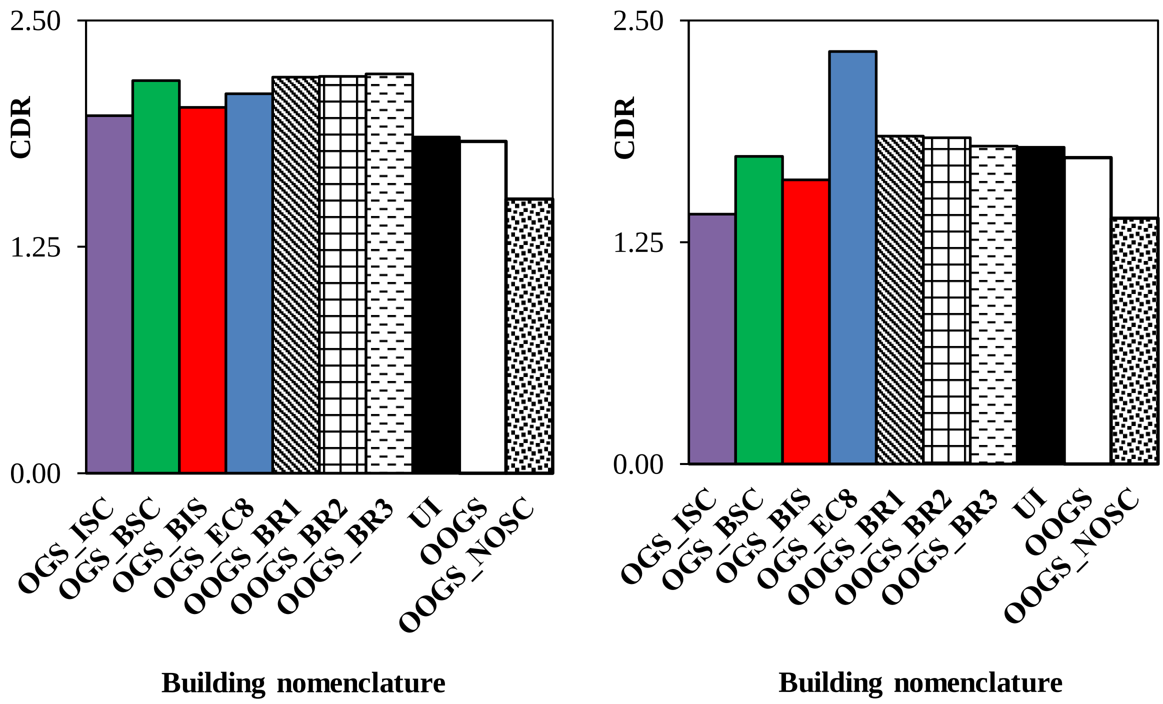
(a) Longitudinal direction (b) Transverse direction

Fig. 5.14 Comparison of plastic displacement of considered high-rise OGS buildings



(a) Longitudinal direction (b) Transverse direction

Fig. 5.15 Comparison of ductility capacity/demand ratio (CDR) of considered mid-rise OGS buildings



(a) Longitudinal direction (b) Transverse direction

Fig. 5.16 Comparison of ductility capacity/demand ratio (CDR) of considered high-rise OGS buildings

5.5 Strength and Stiffness of Considered OGS Buildings

It can be observed from Figs. 5.5 to 5.8 that multiplication factors to increase the design seismic forces for open/adjacent storey members have a significant influence on the lateral (yield and peak) strength capacity of the OGS buildings. OGS_ISC building shows the maximum increase in peak strength (83% to 101% in mid-rise and 29% to 31% in high-rise) as well as increase in stiffness (13% to 29% in mid-rise and 16% to 22% in high-rise) compared to their RC frame building with uniform infills (UI) counterpart as evident from Figs. 5.9 and 5.10. Highest strength and stiffness increase for mid and high-rise OGS_ISC buildings are attributed to the 3 times higher design seismic forces for both the open and adjacent storey members. However, it is to be noted that in case of relatively flexible high-rise buildings, these high design forces make the adjacent upper storey, i.e., 3rd storey, a weak storey as per BIS (2002). Considerable increase in peak strength for OGS_BSC (45% to 53% in mid-rise; 10% to 12% in high-rise) and OGS_BIS (43% to 45% in mid-rise; 9% to 13% in high-rise), which are comparable as in both the cases, only the open storey members were designed for 3 and 2.5 times higher seismic forces, respectively. In case of mid-rise OGS_BIS building, the reduction in stiffness is up to 19% to 23% whereas, stiffness is slightly higher for high-rise OGS_BIS building as compared to UI building due to the increased beam and column sizes in the ground storey. In case of mid-rise OGS_EC8 building, where only open storey columns were designed with 2.17 and 2.23 times higher seismic forces along the longitudinal and transverse direction, respectively, strength increase is about 20% as compared to UI building. In case of mid-rise OGS buildings with bracings, strength is found to be increased by 11% to 18% as compared to UI and are very close to the strength achieved by the OGS_EC8 building. However, in case of relatively flexible high-rise OGS_EC8 and OGS buildings with RC bracings, increment in strength is found to be negligible (6% to 7%) as compared to high-rise UI buildings.

5.6 Ductility, Plastic Deformation Capacity, and Failure Mechanism of Considered Buildings

Priestley (Priestley 1993; Priestley 2000, 2003) and other researchers have pointed out that force is a poor indicator of damage, and there is no clear relationship between strength and damage. Hence, force cannot be the sole criterion for design, and displacement capacity is more fundamental to damage control (Priestley 1993). The inelastic deformation effects are indirectly accounted for using Response Reduction Factor in the traditional Force-Based Design concept adapted by Indian code design practices. The displacement parameters like ductility capacity, ductility demand, plastic deformation have been estimated for all the considered buildings from the bi-linearization of capacity curves as suggested by Haldar and Singh (2009). It can be observed from Figs. 5.15 and 5.16 that OOGS with RC bracings have the highest ductility capacity/demand ratio (CDR) but have the least ultimate displacement among all the designed OGS buildings. The high ductility capacity of OOGS with RC bracings is owing to the lowest yield displacement (Figs. 5.11 and 5.12) due to the presence of very stiff RC bracings at the open storey.

A closer look at Figs. 5.14 and 5.16 revealed that inelastic displacement and ductility capacity of high-rise OGS_BIS and OGS_BSC are better as compared to OGS_ISC. Although, presence of very stiff RC bracings at the open storey has prevented the flexural yielding of open storey columns but being highly stiff and strong caused the failure of relatively flexible adjacent upper storey columns just above bracings due to high concentration of stresses resulting from sudden abrupt change in storey displacement (Figs 5.2 and 5.3), which eventually caused the premature failure (Fig. 5.17) of the structure at lower displacement and force level.

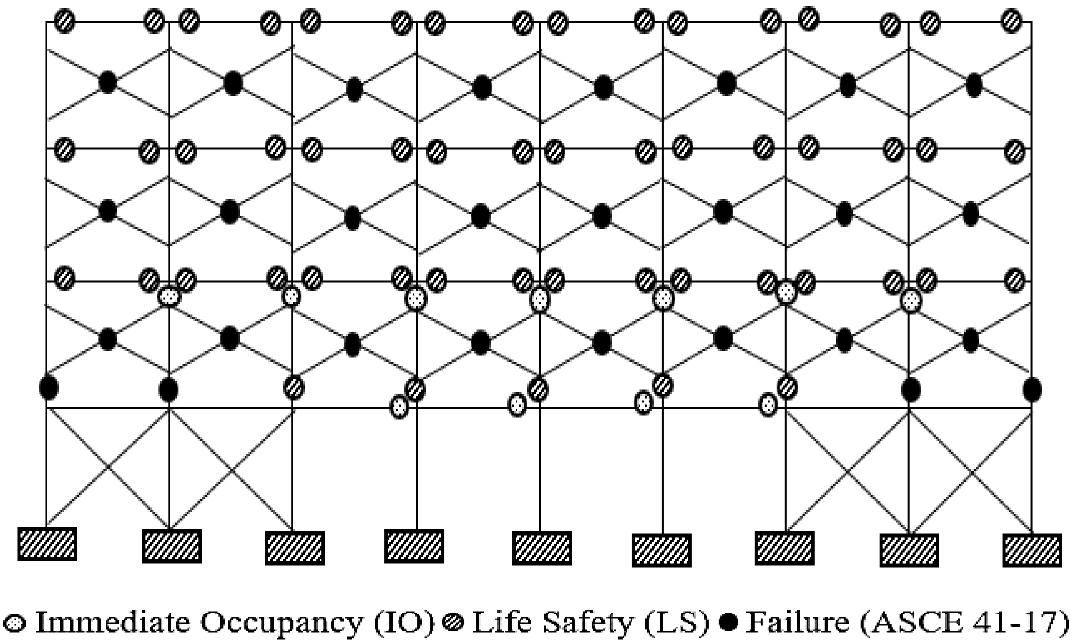


Fig. 5.17 Typical yielding at critical frame of mid-rise OGS building with diagonal RC bracings at DBE hazard level

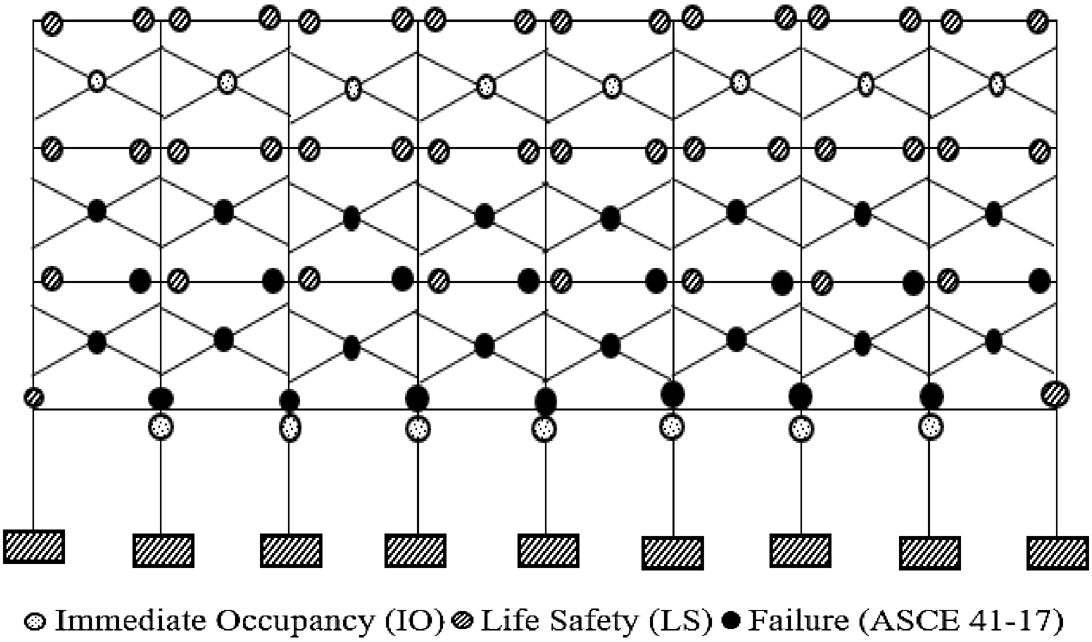
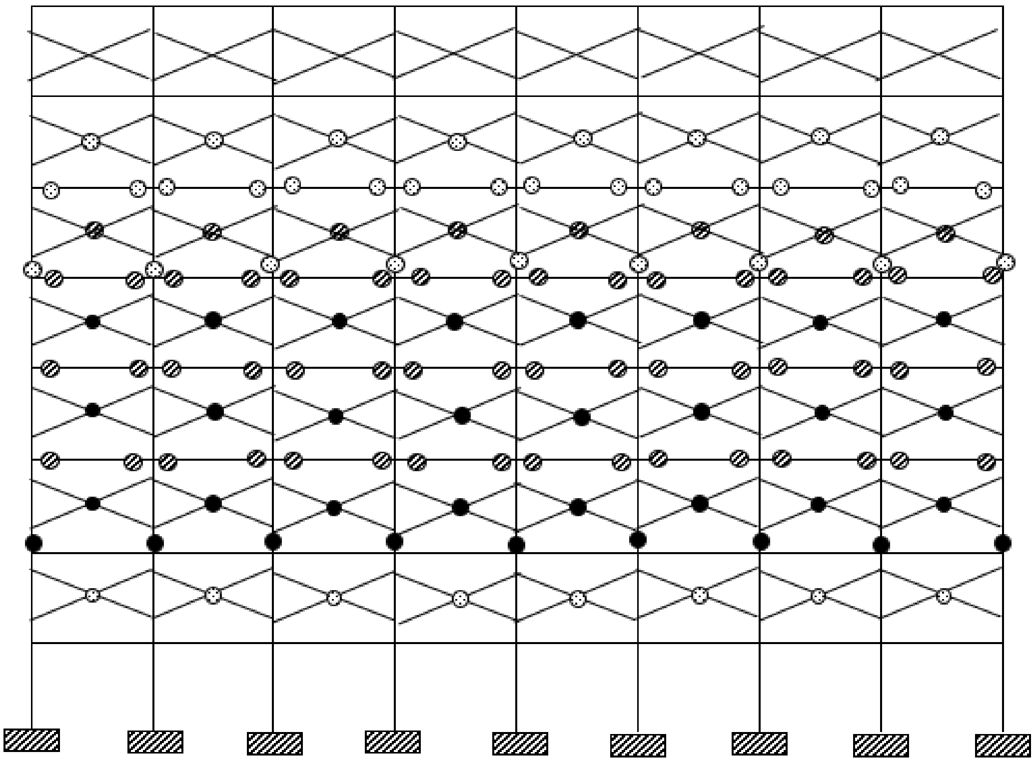


Fig. 5.18 Typical failure mechanism at critical frame of mid-rise OGS_BIS building

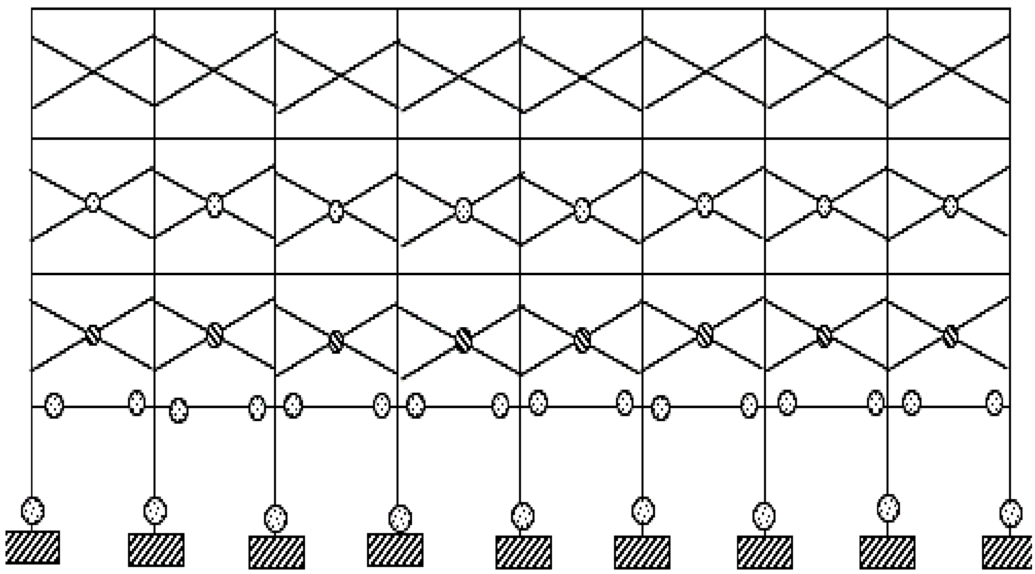
In RC frames with uniformly placed infills, yielding first occur in infills as these attract large forces due to high stiffness. Global failure of these buildings caused by failure of ground storey members and infills up to mid-height of the buildings. Collapse mechanism was formed due to plastic hinges in upper storey beams and columns after failure of infills.

It is important to note that although Immediate Occupancy (IO) level of plastic hinges have been formed, however, failure of open storey columns was avoided in OGS_BIS (Fig. 5.18) and OGS_BSC buildings, as open storey beams and columns were designed for 2.5 and 3 times higher seismic forces, respectively. Although, both mid-rise and high-rise OGS_ISC buildings possess high strength and stiffness but may result in a weak immediate upper storey for high-rise buildings as discussed in Section 5.3, resulting in global collapse of the buildings as shown in Fig. 5.19. At DBE hazard level, no major structural damage was observed in case of all mid-rise and high-rise OGS buildings considered in the study except yielding of infills and a few beams within Immediate Occupancy (IO) damage levels. However, OOGS_NOSC buildings showed the worst seismic performance in terms of strength, stiffness, ductility as well as failure mechanism.



⊚ Immediate Occupancy (IO) ⊘ Life Safety (LS) ● Failure (ASCE 41-17)

Fig. 5.19 Typical failure mechanism of high-rise OGS_ISC building at critical frame



⊚ Immediate Occupancy (IO) ⊘ Life Safety (LS) ● Failure (ASCE 41-17)

Fig. 5.20 Typical yielding at critical frame of mid-rise OOGS_NOSC buildings at DBE hazard level

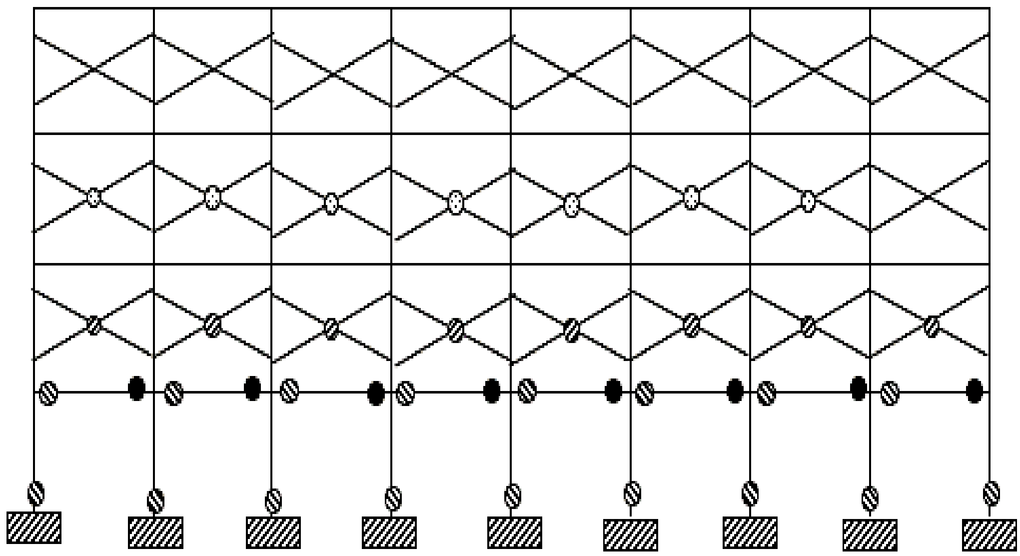


Fig. 5.21 Typical failure mechanism at critical frame of mid-rise OOGS_NOSC building

Even at DBE, many infills along with some open storey beams and columns have yielded in both mid (Fig. 5.20) and high-rise buildings. Open storey members have collapsed at MCE hazard level for both mid and high-rise OOGS_NOSC buildings causing the collapse of the whole building as shown in Fig. 5.21, similar to those observed during the Bhuj earthquake.

5.7 Summary

It has been witnessed in post-earthquake reconnaissance survey that structural collapse of OGS buildings occurs mainly due to the collapse of open storey columns. Therefore, the most appropriate OGS design technique would be to ensure the prevention of collapse of open storey columns before complete failure of other members and infills, and thus governing the utilization of complete inherent strength, stiffness and ductility of the building. Most commonly adapted open storey design interventions among various national standards (Bulgarian seismic code 1987; SI-413 1995; BIS 2002; Eurocode-8 2004; BIS 2016a) are the design of open storey or open including adjacent storey members with higher design seismic forces using multiplication factors or use of diagonal bracings in the open storey for eliminating strength and stiffness irregularities, if any. In the present study efficacy of these open storey design interventions have been evaluated. In order to eliminate the strength and stiffness irregularities, enhancement of seismic design force for open storey/open storey including adjacent storey members have been considered through multiplication factors as recommended by various national design standards or diagonal RC bracings at the selected open bays instead of any multiplication factor. The analytical result indicates that RC bracings being highly stiff and strong, caused undesirable soft storey like displacement demand resulting in stress concentration at the immediate upper storey. Moreover, OGS buildings with RC bracings were found to yield at lower displacement and force level. It can be further concluded that the design of open storey, including adjacent storey with 3 times higher seismic forces as recommended by Israel seismic code, is found predominant in case of mid-rise buildings. Seismic performance of mid-rise buildings increased significantly in terms of strength, stiffness and ductility. However, the same design philosophy may cause formation of immediate weak upper storey in case of relatively flexible high-rise buildings, causing failure of weak storey columns at early stages depicting much lower ductility and plastic deformation capacity. The analytical result further indicates that the use of multiplication factors of 2.5 to 3 for the design of open storey members of OGS buildings is the most effective solution in eliminating the storey stiffness and strength irregularity, and also capable of improving the seismic performance in terms of strength, stiffness, plastic displacement, ductility, and overall failure mechanism. The design of open storey members with higher seismic forces is time inexpensive, simple to apply for all practical purposes without requiring explicit expertise of non-linear dynamic analysis for open ground storey buildings.

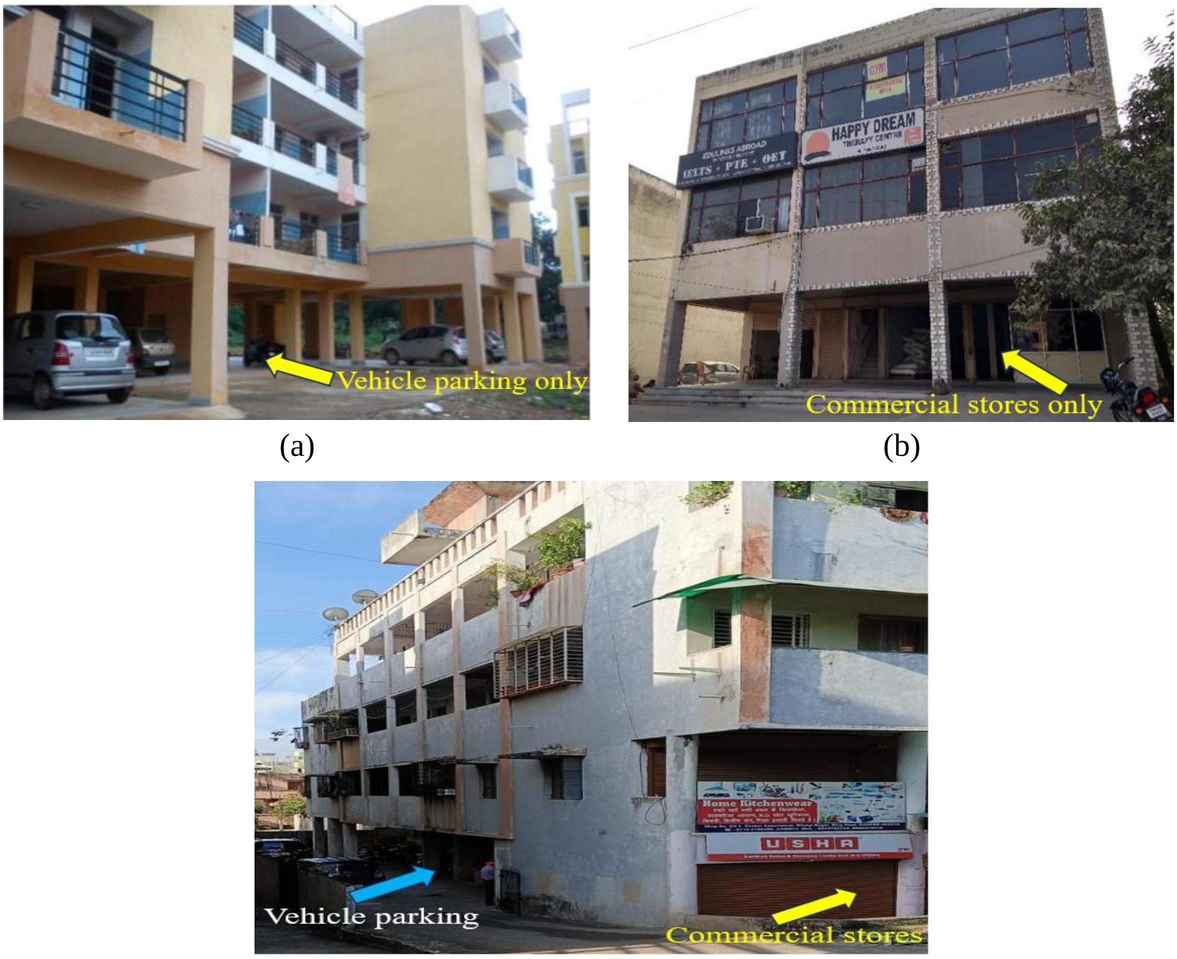
Chapter 6

Seismic Behaviour of Indian RC Buildings with Prevalent Irregular Configurations of URM Infills in Plan

6.1 Introduction

Un-Reinforced Masonry (URM) infills are widely used in Reinforced Concrete (RC) frame buildings across the world. These infills are used as cladding at the exterior periphery walls and as partition in the interior of the building. Experimental investigations (Dhanasekhar M. 1986; Mehrabi et al. 1996; Al-Chaar 2002; Lu 2002; Mohammadi and Nikfar 2013; Cavaleri and Di Trapani 2014; Mansouri et al. 2014; Basha and Kaushik 2016; Teguh 2017) have shown strong interactions between surrounding frame and infills, attributing to various unfavourable modes of failure for infills and frames. Unfortunately, the complex surrounding frame-infill interaction, stiffness and strength contribution of infills to the structural system is ignored in general, during structural analysis and design by the practicing engineers owing to associated modeling complexities, uncertainty in degree of infill-frame interaction and lack of proper guidelines in national design standards (Bulgarian Seismic Code 1987; NBC-201 1995; SI-413 1995; BIS 2002; Eurocode-8 2004) and moreover, the misleading assumption that infill will only provide additional strength and stiffness, which will result in improved performance (Haldar and Singh 2012; Haldar et al. 2012).

The presence of regular solid infill between the frames contribute significantly in terms of lateral strength, stiffness, and energy dissipation capacity of the composite frame system, but reduces the fundamental time period, inelastic deformation capacity thereby altering the failure modes as compared to its bare frame counterpart (Fardis and Panagiotakos 1997; Dolšek and Fajfar 2008a; Uva et al. 2012; Haldar 2013; Borsaikia et al. 2021; Kurmi and Haldar 2022a). The complex infill-frame interaction is intensified when functional openings in infills are introduced due to presence of doors and windows (Demetrios and Karayannis 2007; Kakaletsis and Karayannis 2008; Mondal and Jain 2008; Mohammadi and Nikfar 2013; Yekrangnia and Asteris 2020; Kurmi and Haldar 2022b) causing undesirable seismic behavior leading to extensive damage to RC frame and infills (Ozturkoglu et al. 2017; Repapis and Zeris 2019). The complex behaviour of infilled frames with functional openings under lateral loading gets further complicated when infills are placed irregularly in plan and/or elevation to maximize the usage of available space.



(a) (b)

(c)

Fig. 6.1 Typical RC buildings with irregular placement of infills to serve the purpose of (a) vehicle parking only; (b) commercial stores only; and (c) combination of both vehicle parking and commercial stores at the ground storey while upper storey(s) used for residential purpose

One of the most common vertical irregular configurations of URM infills is Open Ground Storey (OGS) building (Fig. 6.1 (a)) in which infills are completely absent in the ground storey. OGS buildings has always remained vulnerable to earthquakes due to irregular distribution of storey strength and stiffness at the ground storey and performed poorly during seismic events owing to soft-weak storey mechanism formation (DolŠEk and Fajfar 2008b; Haldar et al. 2016; Mazza et al. 2018; Pavel and Carale 2019; Borsaikia et al. 2021; Das et al. 2023; Lal and Remanan 2023) and suffered severe damage to complete collapse in past earthquakes (Jain et al. 2002; Mayorca and Leon 2007; Sharma et al. 2013; Goda et al. 2015).

(a) Gorkha earthquake (Varum et al. 2018) (b) Trapani et al. 2022

Fig. 6.2 Collapse of ground storey on the flexible side due to plan irregular infills

(a) Gorkha earthquake (Timsina et al. 2021) (b) Bhuj Earthquake (Jain et al., 2001)

(c) Turkey earthquake (2023) (d) Italy earthquake (Vona and Attolico, 2016)

Fig. 6.3 Collpase of open ground storey RC building due to vertical irregular infills

A sizeable number of the residential buildings in India are also being used for mixed occupancy, where the ground storey is used for commercial purpose (Fig. 6.1 (b)) or both commercial and parking (Fig. 6.1 (c)); and the upper storey(s) for residential purpose. The increasing urbanization in a country like India with ever decreasing available land for construction compels buildings with irregular configuration of infills not only in elevation in the form of OGS but also irregularity exists in the plan to encompass the occupational and functional demand together. The commercial usage demands for larger free spaces without partitions, and open front and/or sides. However, RC buildings with irregular URM infills in plan alter displacement and ductility demand resulting in high damage indices and worse seismic behavior leading unacceptable collapse mechanism such as excessive torsion under earthquake and consequential premature failure through the flexible side of building as observed in past earthquakes (Figs. 6.2-6.3).

Although significant research effort has been undertaken for vertical infill irregularity resulting OGS buildings, there is very limited study available in the literature on effect of irregular placement of URM infills on the seismic behaviour of infilled frame buildings where infills are placed irregularly in plan to serve commercial purposes along with irregularity arises from functional openings due to presence of doors and windows to serve residential purposes. This Thesis work highlights explicit and combined effect of infill irregularity arising from functional and occupational requirement on the overall behaviour and consequent fragility of infilled frame buildings under seismic excitation. Incremental Dynamic Analysis (IDA) in conjunction with fragility assessment have been employed on a set of RC buildings compliant to Indian seismic design standards BIS (1993, 2002, 2016a, 2016b) with prevalent irregular URM infill configurations identified during the pilot survey in Indian city. Seismic performance in terms of dynamic capacity curves and collapse mechanism of irregular infilled have been examined and compared with the seismic performance of Uniformly Infilled (UI) RC frame buildings.

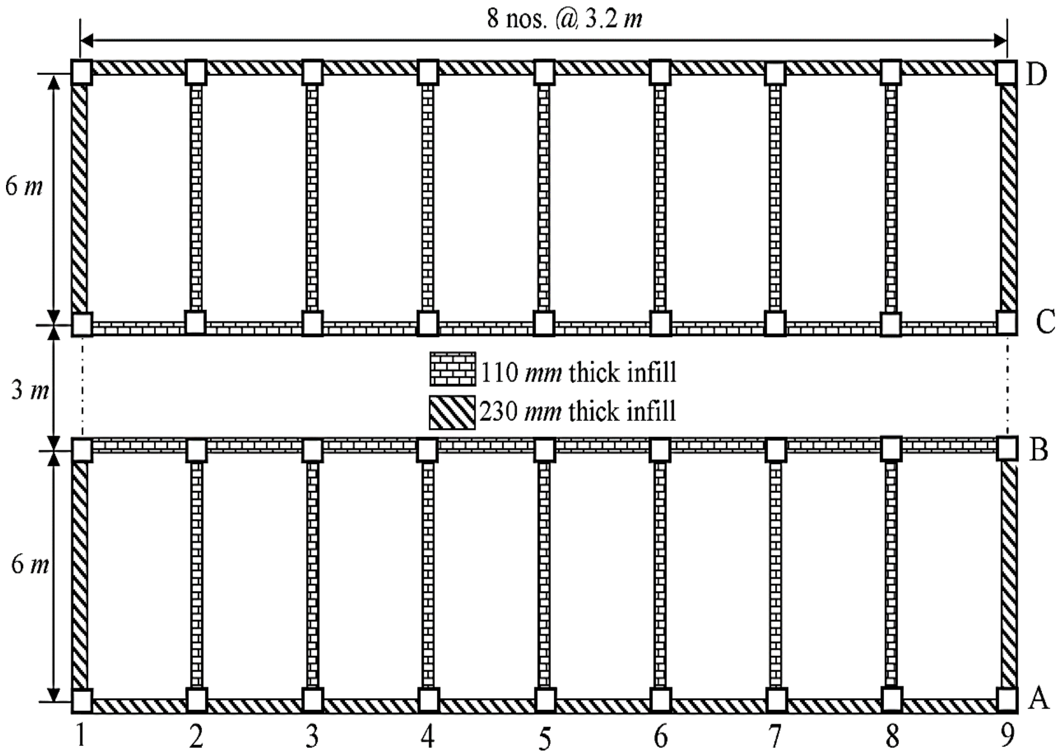
6.2 Seismic Assessment of RC Buildings with Prevalent Irregular Configurations of URM Infills in Plan

As discussed in Chapter 2, EPGS, OGS, and POGS are the most common irregular URM infill configurations found during the pilot survey (Fig. 2.9 (a)), in existing RC buildings which are predominantly present in mid-rise (1-4 storey) buildings (Fig. 2.9

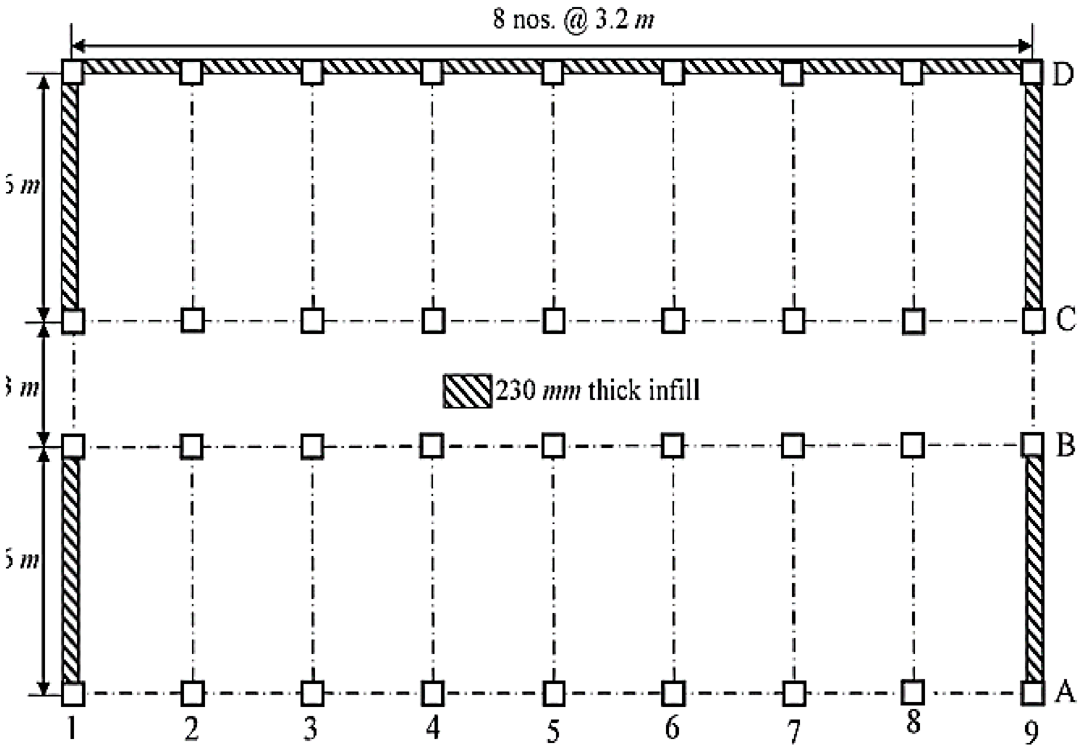
(b)). Accordingly, for the parametric study, a set of mid-rise RC buildings with three different prevalent irregular infill configurations viz. EPGS, OGS, and POGS have been selected along with the ideal case of solid URM infill distributed regularly in plan and elevation of the building (UI). All the buildings have generic plan with 4 frames along the longitudinal direction and 9 frames in the transverse direction. The buildings are assumed to be situated in seismic zone V on Type II soil (medium soil) as per Indian seismic design standard BIS (2016a).

Table 6.1 Description of various buildings considered in the study

Sr. No.	Buildings nomenclature	Design and detailing standards	Relevance of infill placement
1	UI	BIS (2016a) and (2016b)	Ideal case of solid URM infill distributed regularly in plan and elevation of the building
2	OGS_BIS2002	BIS (2002) and BIS (1993) with OGS design provision of BIS (2002)	Entire ground storey is kept open for parking (Fig. 6.1 (a))
3	OGSW	BIS (2002) and BIS (1993) without OGS design provision of BIS (2002)	
4	EPGS_BIS2016	BIS (2016a) and (2016b)	Only external periphery is provided with URM infills and remaining space in ground storey to be utilized for commercial purpose (Fig. 6.1 (b))
5	EPGS_BIS2002	BIS (2002) and BIS (1993)	
6	POGS_BIS2016	BIS (2016a) and (2016b)	Front portion of building is arranged for commercial purpose while back portion is kept open for parking (Fig. 6.1 (c))
7	POGS_BIS2002	BIS (2002) and BIS (1993)	


8 nos. @ 3.2 m
6 m
3 m
6 m
110 mm thick infill
230 mm thick infill
D
C
B
A
1
2
3
4
5
6
7
8
9

(a)


8 nos. @ 3.2 m
230 mm thick infill
D
C
B
A
1
2
3
4
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8
9

(b)

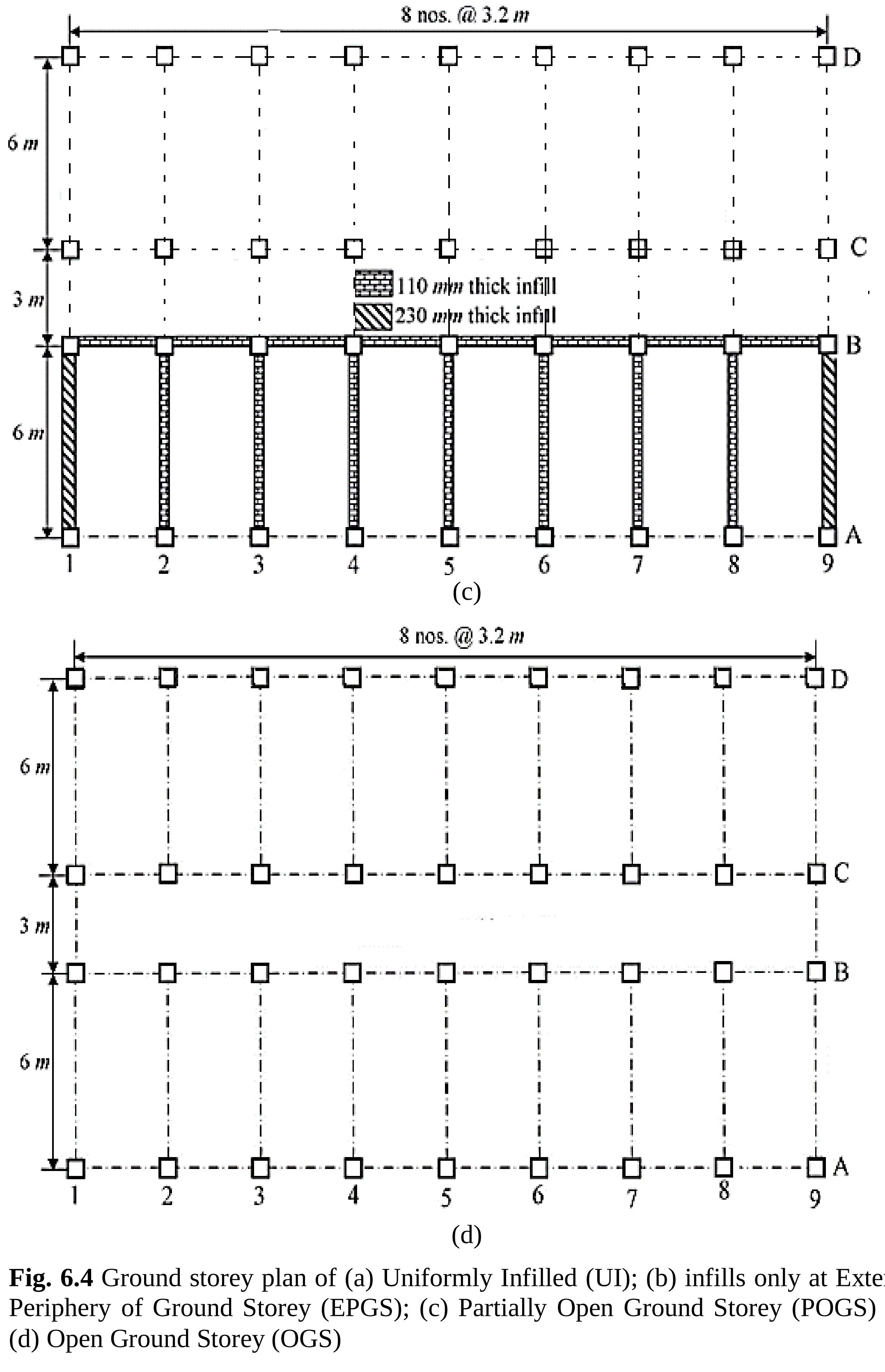
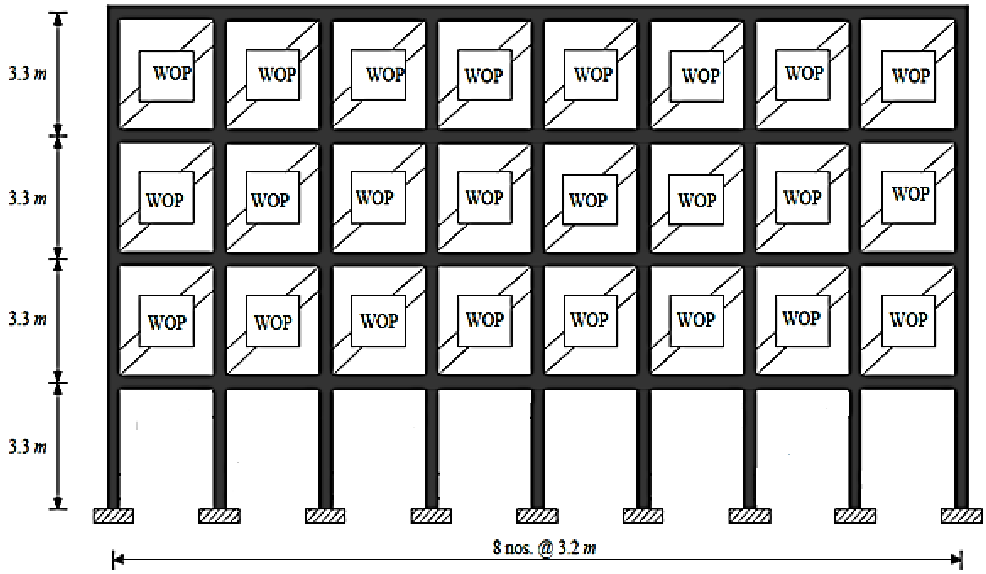
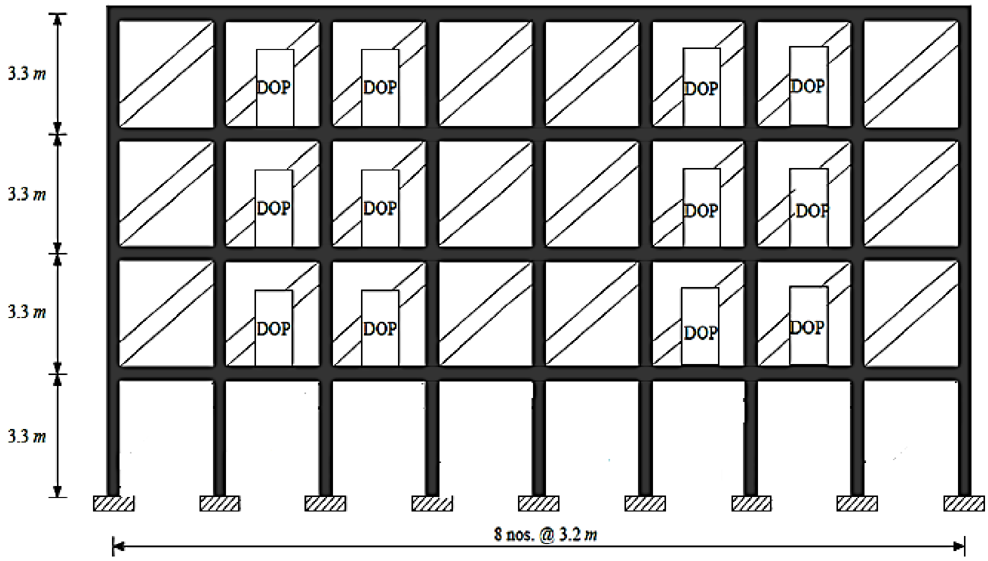


Fig. 6.4 Ground storey plan of (a) Uniformly Infilled (UI); (b) infills only at External Periphery of Ground Storey (EPGS); (c) Partially Open Ground Storey (POGS) and (d) Open Ground Storey (OGS)

Although, to ensure ductile behaviour of SMRF buildings BIS (2016a) enforces two governing major design criteria: (1) column dimension shall not be less than 20 times the diameter of the larger longitudinal rebar, and (2) capacity design by maintaining the minimum Strong-Column Weak-Beam (SCWB) ratio of 1.4, however, a large stock of URM infilled RC buildings exist even in high seismic zones (DEQ 2009b), designed as per its older counterpart BIS (2002) and BIS (1993). Therefore another set of mid-rise RC buildings designed and detailed as per BIS (2002) and BIS (1993) have also been considered in this study. Different design levels of RC buildings with prevalent irregular infills considered in this present study is summarized in Table 6.1 while the generic building plan at ground level and location of doors and windows are presented in Fig. 6.4 and Fig. 6.5, respectively.



(a)



(b)

Fig. 6.5 Location of windows (WOP) and doors (DOP) (a) external frame; (b) internal frame

To cover the wide spectrum of size and configuration of infill openings in Indian residential buildings, CPWD (2006) guidelines for doors and windows have been considered for the parametric study. CPWD (2006) has recommended 15 windows and 6 door sizes by varying the length and height of openings which eventually arrive at variation of window from 6% to 32%, and door opening sizes 20% to 33% of the solid infill area. In the present study, all window openings and doors are kept with 25% opening of solid infill area representing a generic mean opening combinations as shown in previous study of Kurmi and Haldar (2022b).

6.3 Seismic Performance of Considered Buildings

Non-linear static pushover analysis has been performed to study the effect of irregular placement of infills on the seismic performance of infilled RC frame buildings. Dynamic properties of all the considered building presented in Table 6.2 shows that mass participation in the fundamental mode of all considered buildings except UI and OGS buildings is less than 75% and significant higher mode contributions has been observed. Therefore, Incremental Dynamic Analysis (IDA) has also been performed for all the buildings.

Table 6.2 Dynamic properties of considered buildings

Sr. No.	Buildings nomenclature	Design and detailing standards	Modal time period (*s*)		Modal mass participation (%)	
			Longitudinal	Transverse	Longitudinal	Transverse
1	UI	BIS (2016a) and (2016b)	0.34	0.41	85	84
2	OGS_BIS2002	BIS (2002) and BIS (1993)	0.48	0.53	85	85
3	OGSW	Designed as per BIS (2002) and BIS (1993) without OGS provision of BIS (2002)	0.69	0.72	96	79
4	EPGS_BIS2016	BIS (2016a) and (2016b)	0.5	0.5	51	56
5	EPGS_BIS2002	BIS (2002) and BIS (1993)	0.56	0.58	58	72
6	POGS_BIS2016	BIS (2016a) and (2016b)	0.49	0.5	76	77
7	POGS_BIS2002	BIS (2002) and BIS (1993)	0.53	0.57	53	87

IDA (Vamvatsikos and Cornell 2002) has been performed using 22 far-field ground-motion records (Fig. 6.6) listed in FEMA-P695 (2009), applied in the two orthogonal building directions simultaneously.

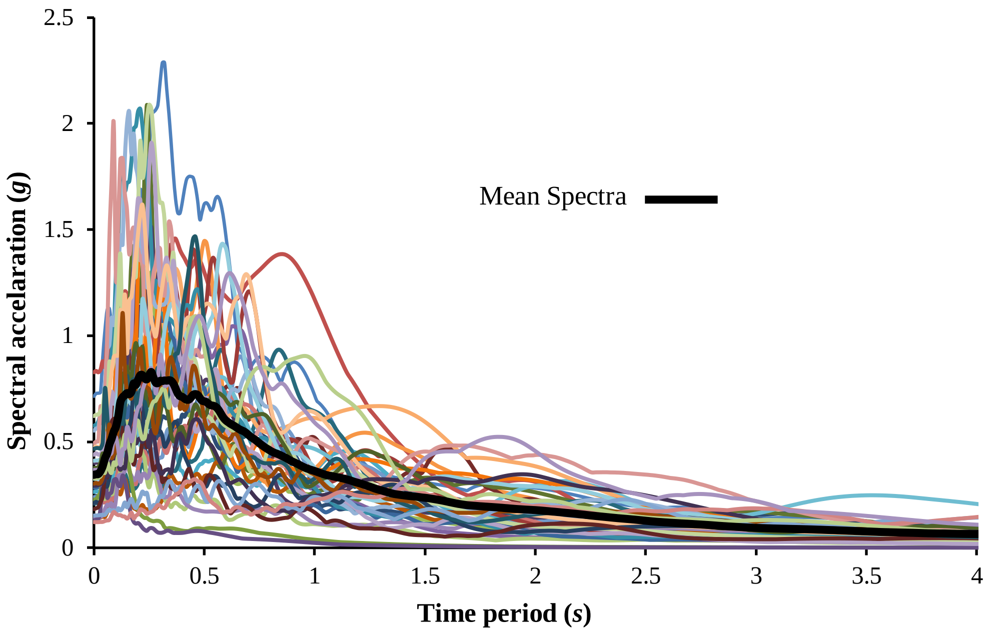
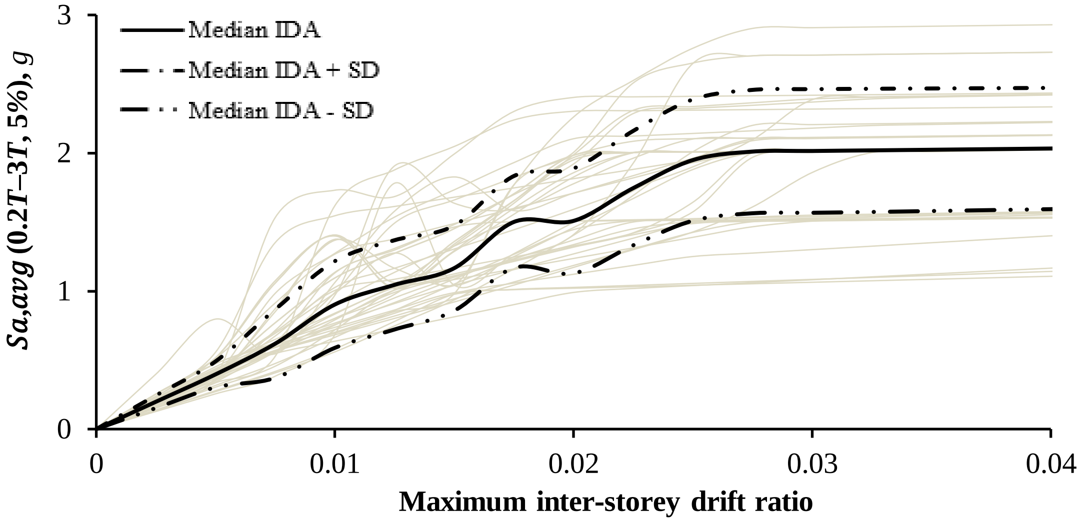
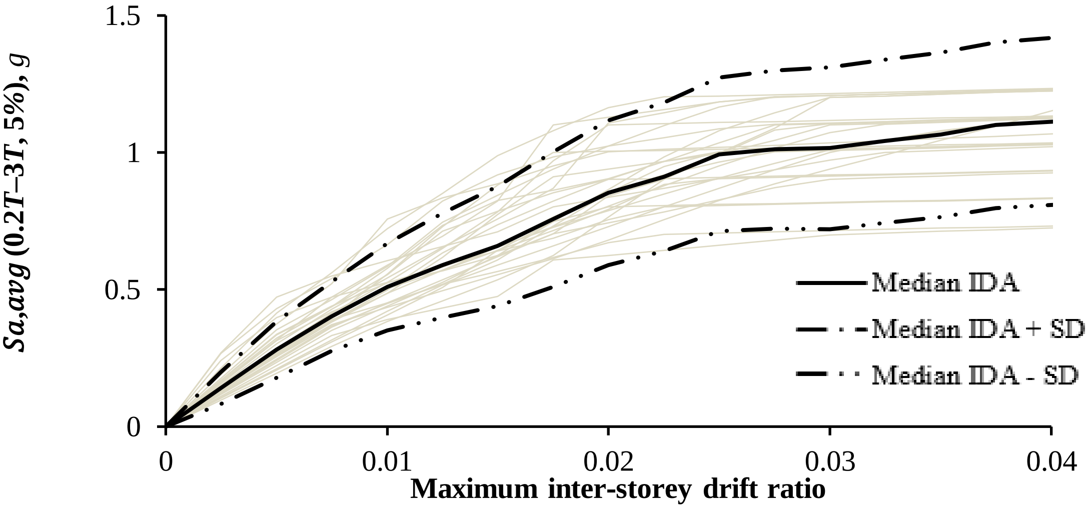


Fig. 6.6 Spectral and mean spectral acceleration of 22 ground motions considered

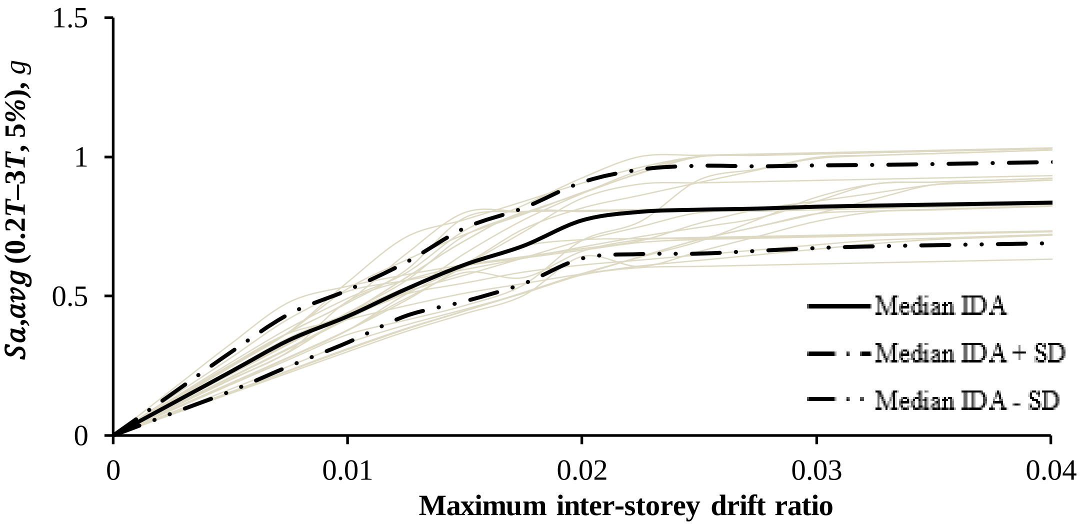
In the present study, all ground motions are scaled to match the average spectral acceleration ($S_{a,avg}$) taking the geometric mean of the spectral accelerations between the time periods 0.2T to 3T, where T is the arithmetic mean of the periods in the fundamental translational modes along the two orthogonal directions of the considered building. $S_{a,avg}$ (0.2T–3T, 5%) has been used as the scaling parameter as it is capable of capturing spectral shape effects by considering the spectral ordinates at elongated periods due to higher modes effect (Eads et al. 2015). Each ground-motion has been scaled in amplitude, until it causes structural collapse evidenced by lateral dynamic instability. Rayleigh damping of 5% has been assigned at periods corresponding to the lowest mode, and the mode resulting in a total of 95% cumulative mass participation to model the damping effects. Fig. 6.7 represents dynamic capacity curves generated through IDA for considered mid-rise buildings. The dynamic capacity curves are presented in terms of Intensity Measure (IM) i.e., $S_{a,avg}$ (0.2T–3T, 5%) versus Damage Measure (DM) i.e., maximum inter-storey drift ratio for ground-motion record suits.



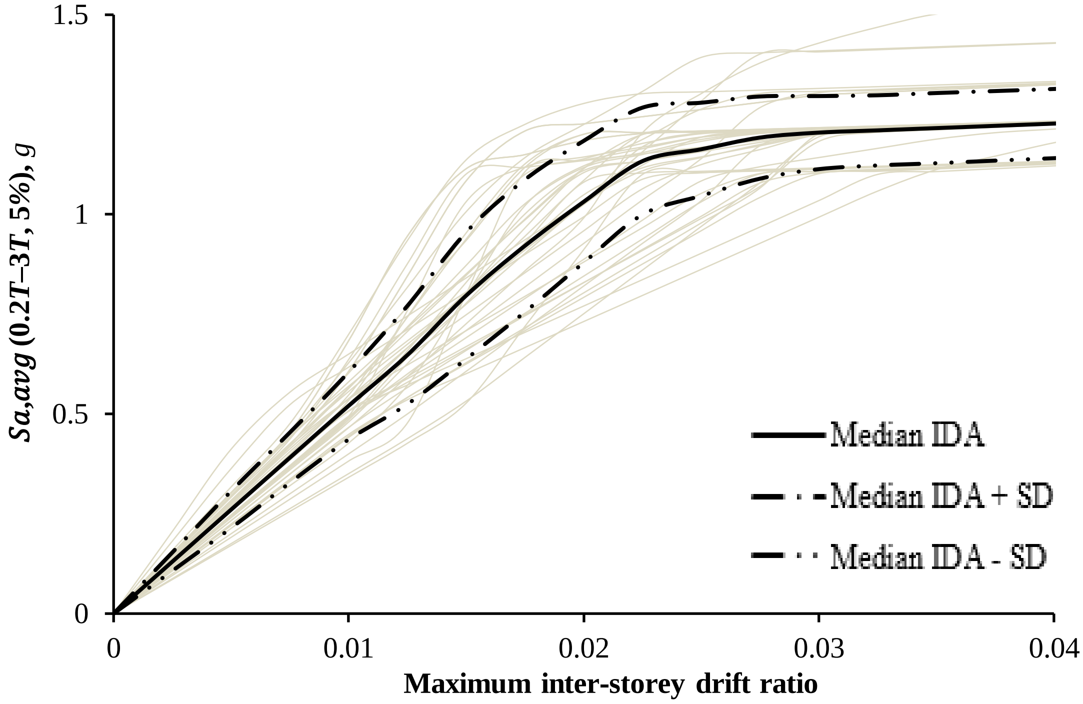
(a) UI



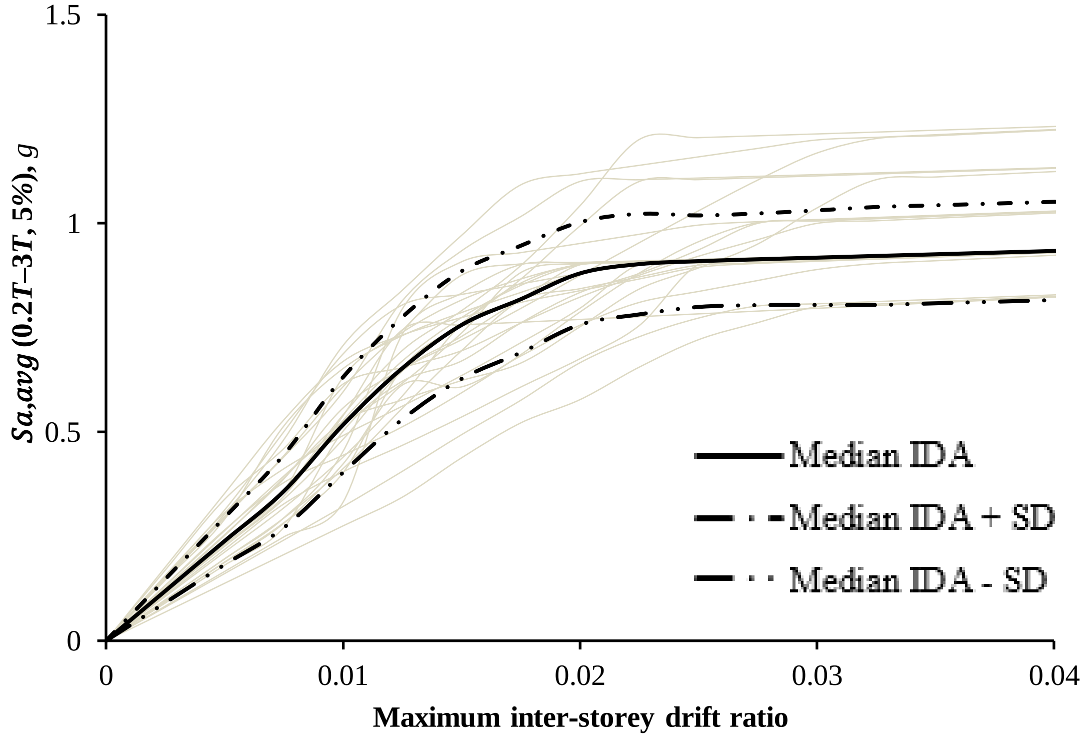
(b) OGS_BIS2002



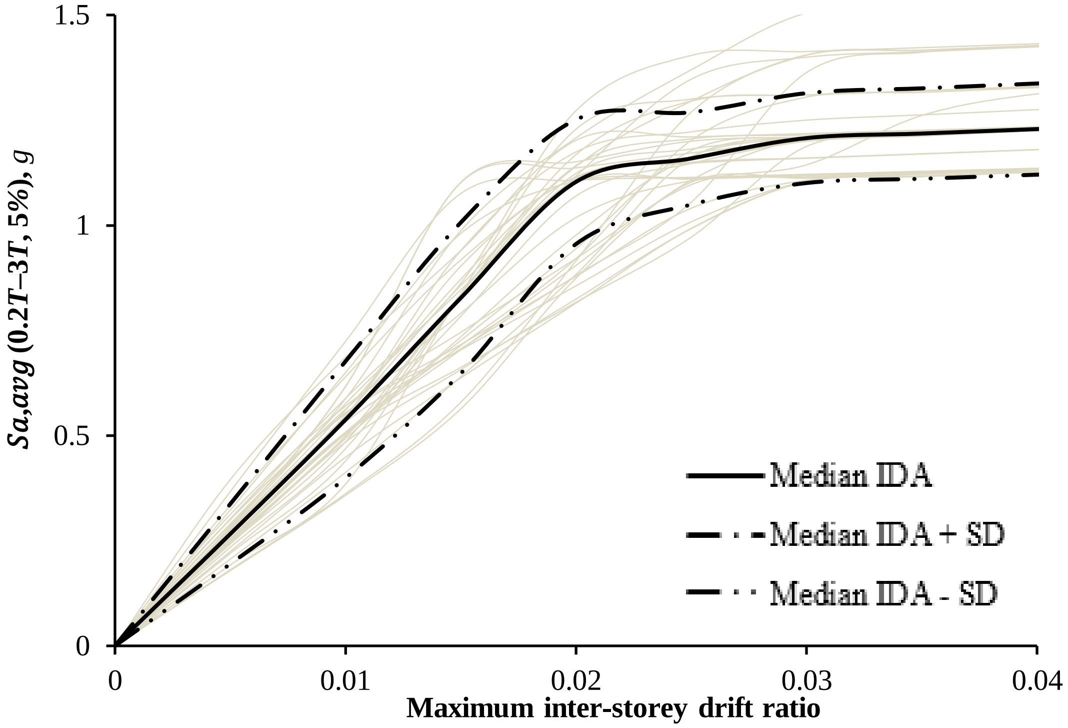
(c) OGSW



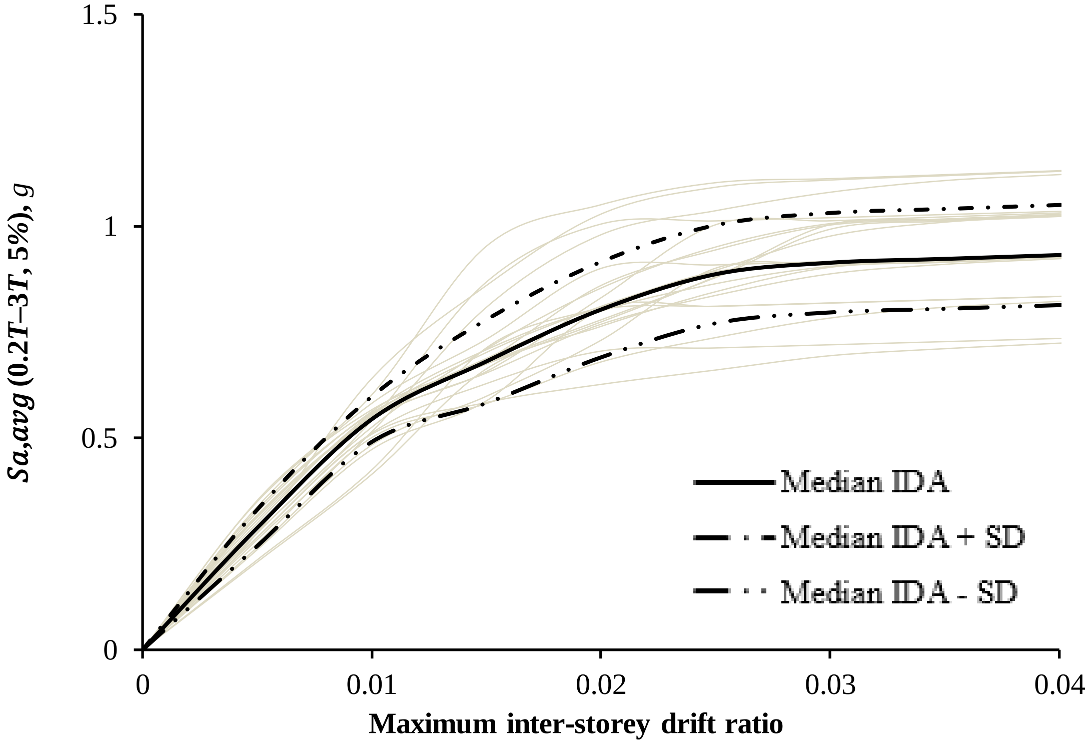
(d) EPGS_BIS2016



(e) EPGS_BIS2002



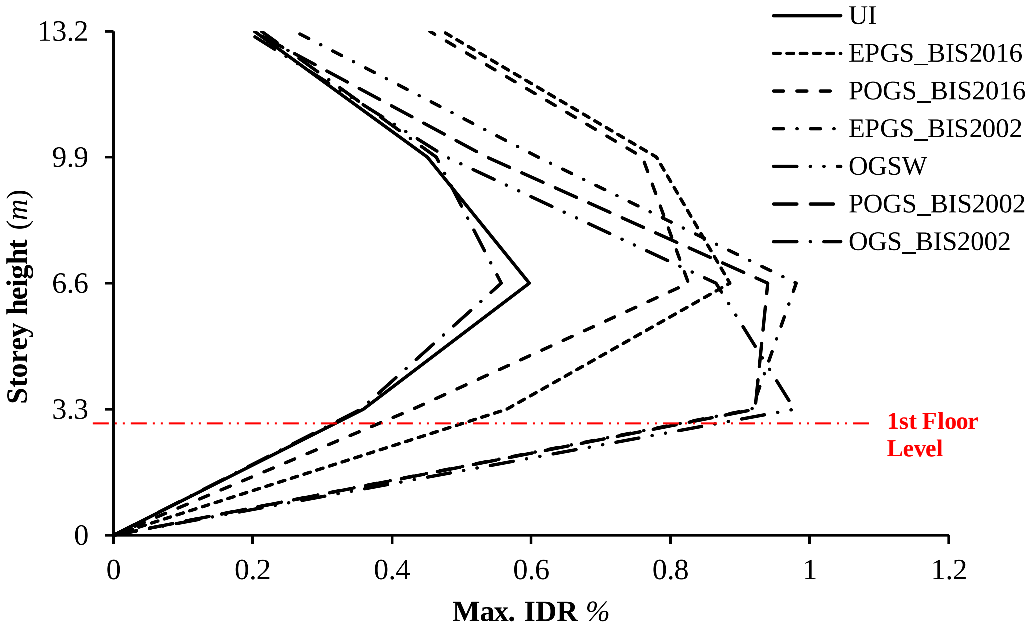
(f) POGS_BIS2016



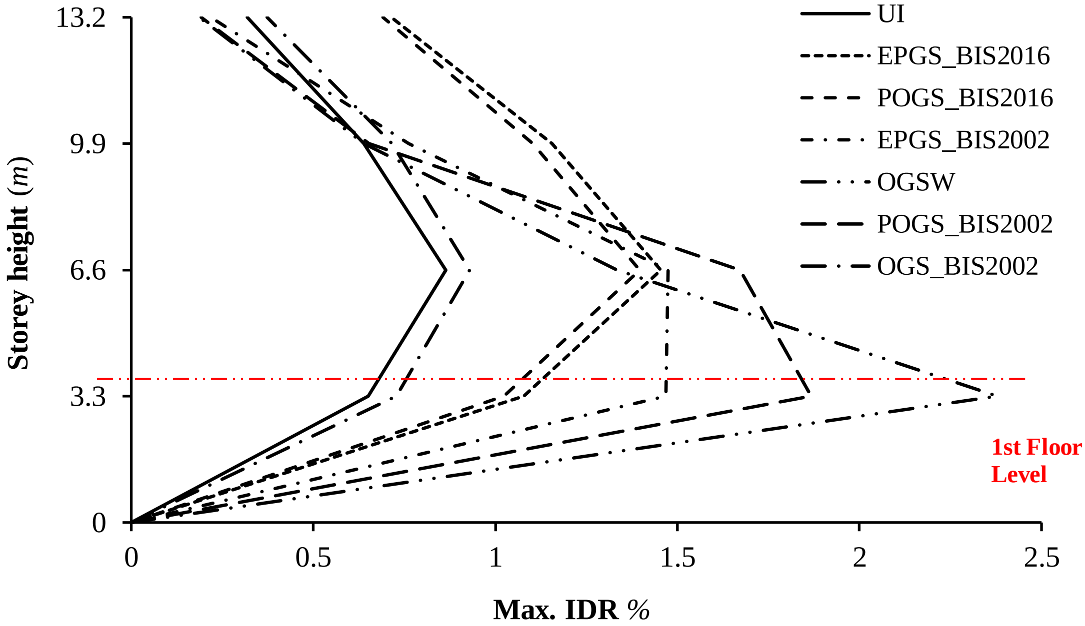
(g) POGS_BIS2002

Fig. 6.7 Dynamic capacity curves of the considered buildings

Fig. 6.7 also presents median and median ± Standard Deviation (SD) of the IDA curves. The maximum Inter-storey Drift Ratio (IDR) plotted along the storey height of the considered buildings in order to evaluate the displacement demand of the considered buildings presented in Fig. 6.8.



(a) DBE hazard level



(b) MCE hazard level

Fig. 6.8 Variation of Inter-storey Drift Ratio (IDR) along the storey height

A closer look at Figs. 6.7 and 6.8 reveals that UI building experience gradual increase of drift demand helping it to attain highest median collapse capacity before failure which is attributed to beneficial effect of regular placement of infills along the plan and elevation of the building. Whereas OGSW building which is not designed for

OGS design provision of BIS (2002) shows the lowest capacity and experience the maximum IDR concertation at the first storey level. However, for OGS_BIS2002 building, IDR along the height of the building is gradual and comparable to the UI building shows the adequacy of OGS design provision of BIS (2002).

The median collapse capacity of EPGS and POGS buildings (Fig. 6.7) enhances by 31.5% and 26.3% respectively, when designed with the latest seismic standards BIS (2016a, 2016b) in comparison to its older counterpart BIS (1993, 2002) is observed to be reduced as compared to UI building. It can be observed that higher IDR demand is mainly concentrated at the first storey level due to irregular configuration of infills at the ground storey. The drift concentration is abrupt and higher for buildings designed with older Indian seismic standards (BIS 1993, 2002) and collapsed at lower median capacity as compared to its revised counterpart (BIS 2016a, 2016b) reflects the adequacy of design provisions of revised Indian seismic standards. The IDR concentration at first storey level (Fig. 6.8) further indicates that ground storey members experience higher damage and may cause global collapse affecting the overall stability of the buildings.

6.4 Collapse Mechanism of the Considered Buildings

The damage pattern of the representative buildings at the onset of instability in the structure is shown through Figs. 6.9 - 6.12. It has been observed that for all the buildings, yielding first occur in infills as these attract large forces owing to high in-plane stiffness. The global failure of UI building is caused by collapse of ground storey members and infills up to mid-height of the building. In case of OGS building designed with BIS (2002) provision, the global collapse mechanism occurred due failure of first storey beams and columns (Fig. 6.9). Yielding of the columns is observed in ground storey; however, prevention of their failure shows the adequacy of OGS design provision of BIS (2002). OGS building designed as per BIS (2002) provision performed better than its counterpart OGSW, in which the collapse mechanism is formed due to failure of ground storey columns and beams at lower force level (Fig. 6.10). Similar failure mechanism of OGS building was also observed in Bhuj earthquake (Jain et al. 2002).

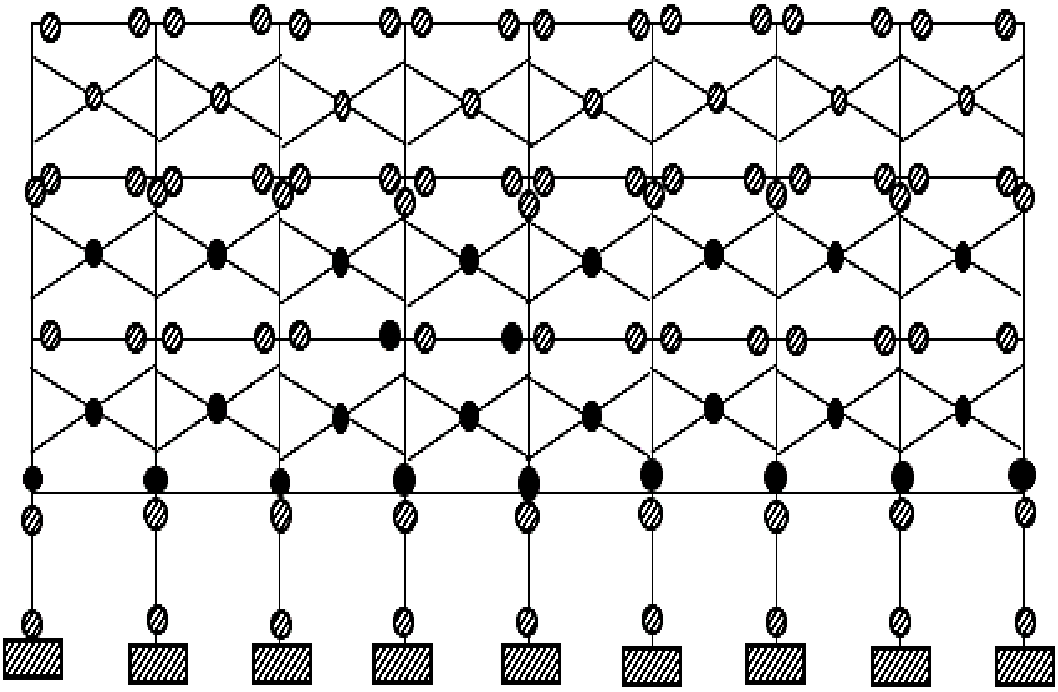


Fig. 6.9 Collapse mechanism at the critical frame of OGS_BIS2002 buildings

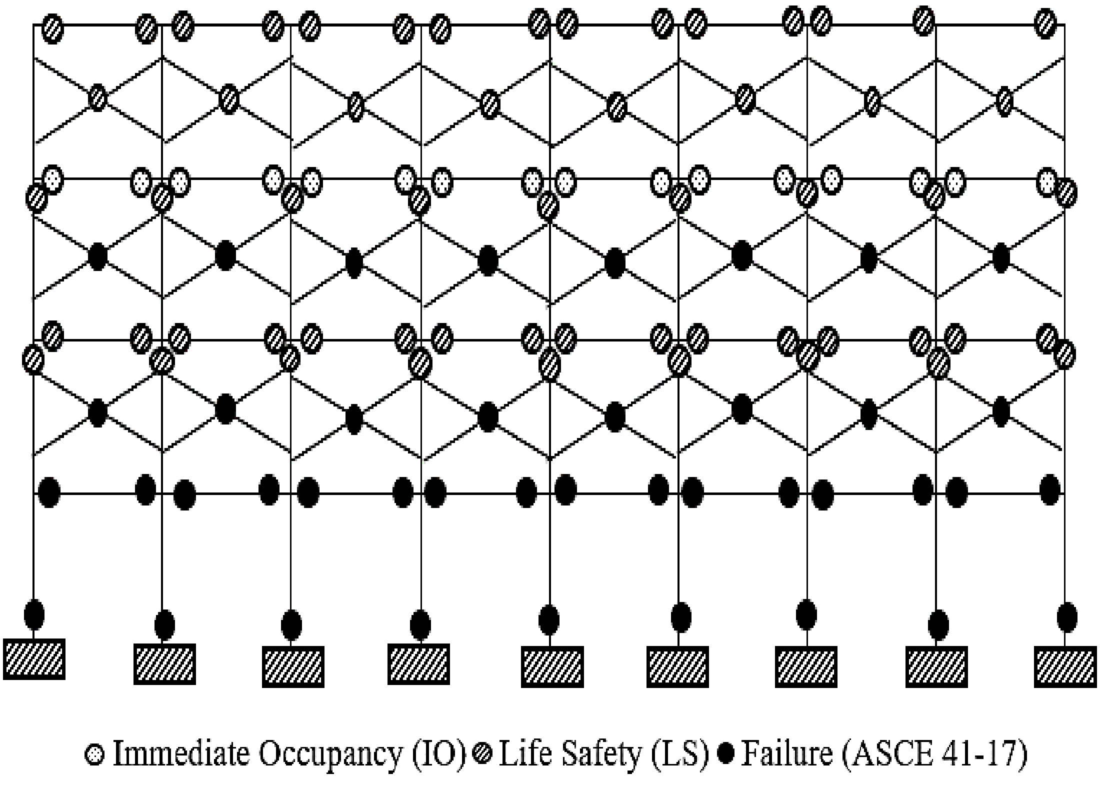


Fig. 6.10 Collapse mechanism at the critical frame of OGSW building

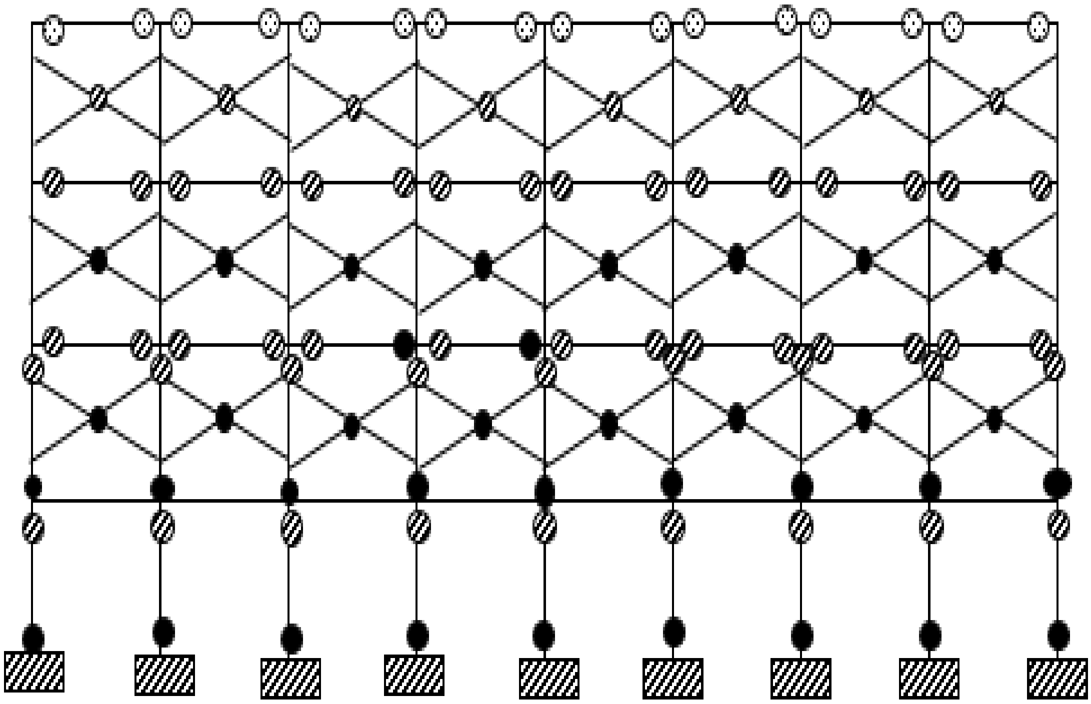
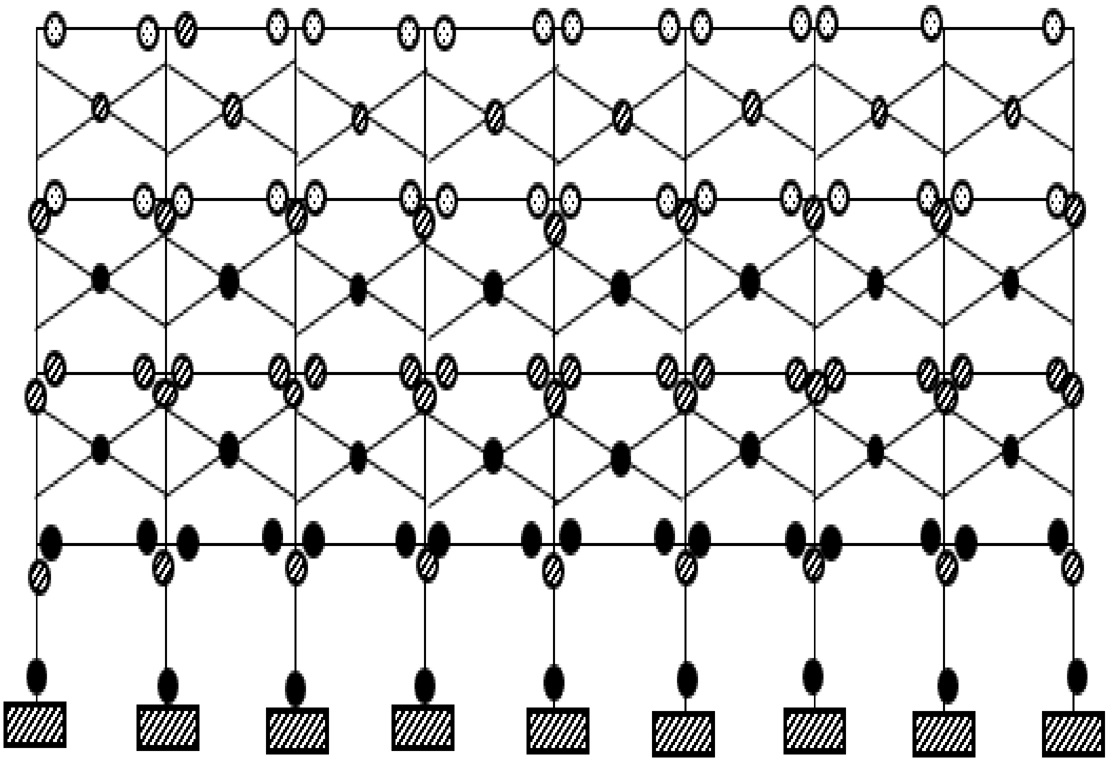


Fig. 6.11 Collapse mechanism at the critical frame of POGS building



Immediate Occupancy (IO) Life Safety (LS) Failure (ASCE 41-17)

Fig. 6.12 Collapse mechanism at the critical frame of EPGS buildings

In case of POGS buildings designed with latest BIS (2016a, 2016b) and older Indian seismic standards BIS (1993, 2002), global failure occurred due to collapse of ground storey columns in the flexible side of the building (Fig. 6.11), attributed to strength and stiffness irregularity resulting from absence of infills. Collapse of EPGS buildings occurred in the internal frames, due to irregular infills causing irregular strength and stiffness as compared to the external infilled peripheral frames (Fig. 6.12) which is in agreement with the collapse pattern observed in past earthquakes (Jain et al. 2002; Mayorca and Leon 2007; Sharma et al. 2013; Goda et al. 2015) and as shown in Fig. 6.2.

6.5 Summary

To accommodate modern occupational and functional necessity buildings are being used for mixed occupancy, where the ground storey is used for commercial purpose or both commercial and parking and the upper storey(s) for residential use. The mixed usage requires larger free space resulting in irregular infills in the ground storey. Placement of Infills and degree of irregularity of infill configuration play vital role on the seismic response and associated collapse probability of uniformly infilled RC frame buildings. In the present study, seismic behaviour of Indian RC buildings with prevalent irregular configurations of URM infills have been studied. The open ground storey building (OGSW) designed without BIS (2002) OGS design provision is found to be most vulnerable to earthquakes among the considered set of buildings attributed to abrupt and excessive concentrated displacement demand in the ground storey. The adequacy of BIS (2002) provisions for designing open storey members with 2.5 times design base shear can be observed in terms of median collapse capacity as it closely follows its UI counterpart. Irregular infilled RC buildings designed with revised Indian standards is found to perform better than its older counterpart. It can be further concluded that EPGS MBT designed with older Indian seismic standards (BIS 1993, 2002) are comparatively higher vulnerable than the POGS MBT. Approximately 50% reduction of median collapse capacity can be observed in buildings with irregular configuration of infills in both plan and elevation as compared to its ideal counterpart uniformly infilled building.

Chapter 7

Seismic Fragility of Indian RC Building with Irregular Infills

7.1 Introduction

Seismic vulnerability (or fragility) assessment of existing building stock is one of the vital tasks in seismic risk assessment and several methodologies are available in literature based on empirical, analytical or hybrid approaches. India has suffered several damaging earthquakes in past, but unfortunately adequate and systematic damage data for development of empirical vulnerability functions is not available (Haldar 2013). Therefore, analytical vulnerability analysis is one of the available alternatives. Attempt has been made for development of fragility functions for Indian buildings (Haldar 2008; Prasad 2009; Haldar 2013; Singh et al. 2013). As discussed in the previous Chapters that RC frame buildings with URM infills are one of the most common building typologies in India and irregular configuration of infills along with functional openings are very much prevalent in the urban areas to meet the modern occupational and functional demand. The present study is attempted to develop the fragility functions for Indian RC frame buildings with prevalent irregular configuration of infills along with functional openings, which can be used in seismic risk assessment studies for Indian urban areas. Analytical procedure of fragility assessment is used while applying the capacity curve parameters of the representative building. The most accurate way to carry fragility assessment is performing non-linear analysis of large sample of buildings of the selected class and generate the statistical data to account for the variability in different parameters (Ghosh and Chakraborty 2017). Such exercise is numerically tedious and time expensive, therefore, to incorporate wide range of Indian URM infilled RC buildings, a representative building has been selected based on a pilot survey carried out in Indian cities as presented in Chapter 2. In this Chapter, an overview of the existing analytical methods for vulnerability assessment has been presented and the capacity spectra developed in Chapters 3 to 5 have been used to construct fragility functions for the RC frame buildings, with prevalent irregular configuration of URM infills.

7.2 Seismic Fragility Assessment

Seismic vulnerability (or fragility) of a structure is defined as its susceptibility to experience damage due to ground shaking of a given intensity. It is expressed as a relationship between the ground motion severity (i.e., intensity, PGA, spectral acceleration or spectral displacement) and structural damage (expressed in terms of damage grades, maximum inter-storey drift ratio). The vulnerability of a structure is usually expressed through fragility curves and Damage Probability Matrices (DPMs). Both methods describe the conditional probability of exceeding different levels of damage at given levels of ground motion intensity. Fragility curves express the data in a graphical format as continuous curve, whereas DPMs express it numerically in terms of discrete values. For the development of vulnerability relations, several approaches are available in literature, which can be classified in three groups namely (i) empirical methods (Whitman et al. 1973; Spence et al. 1992; Hassan and Sozen 1997; Rossetto and Elnashai 2003; Yakut 2004) (ii) analytical methods (Singhal and Kiremidjian 1996; Masi 2003; Rossetto and Elnashai 2005; Liel et al. 2009) and (iii) hybrid methods (Kappos et al. 1995; Barbat et al. 1996; Kappos et al. 1998; FEMA 1999, 2003). The most realistic approach for vulnerability assessment is empirical method which relies on real post-earthquake damage scenarios. These methods are based on intensity scales as a measure of ground motion severity. However, its applicability is very much limited due to lack of reliable damage data for various building typologies subjected to different earthquake intensities. Owing to scarcity of empirical post-earthquake damage data and prohibitively high cost of experimental tests, analytical methods have become more popular for vulnerability assessment. This method includes Capacity Spectrum Method (ATC-40 1996; FEMA-440 2005), Collapse-Based Method (D'Ayala and Speranza 2003), Displacement-Based Method (Silva et al. 2014), Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002), Multi-Strip Analysis (Bandyopadhyay et al. 2023). The main disadvantage of analytical methods is that these requires high computational effort and are time consuming, and therefore not suitable for a large area or country with widely varying construction practices. Moreover, it is a very challenging task to simulate real behavior of structure under earthquake shaking with available technique of analytical modeling and may yield different result if not handled properly. Hybrid approach of vulnerability assessment is the combination of the available empirical data with the results of numerical analysis and thus bridges the gap between lack of empirical data and uncertainty of analytical estimation. The main difficulty of hybrid methods is to calibrate analytical results based on the observed data, because the two sets of data have two different sources of uncertainty, and therefore cannot be compared directly.

India has suffered several devastating earthquakes (1897 Great Assam earthquake, 1991 Uttarkashi earthquake, 1993 Killari earthquake, 1997 Jabalpur earthquake, 1999 Chamoli earthquake, 2001 Bhuj earthquake, and 2005 Kashmir earthquake, etc.) in the past, unfortunately, very few systematic post-earthquake damage surveys have been conducted in India and the available data is highly inadequate and is not in a format suitable for development and calibration of reliable vulnerability estimates. Based on the available information for Indian earthquakes, Prasad (2009) has proposed intensity based DPMs for Indian buildings. In the absence of adequate empirical data, these need to be supported and supplemented by extensive analytical studies for different Indian building types. The analytical approach of fragility assessment requires definition of various damage states and their corresponding threshold parameters. The definition of different damage states also varies significantly in the literature. These damage states are expressed in terms of demand parameters (e.g., peak ground acceleration, spectral acceleration, spectral displacement, inter-storey drift ratio, etc.) based on the aim of response evaluation. In the present study, fragility estimation has been carried using HAZUS (2003) methodology and IDA procedures. The capacity curve parameters extracted for the MBTs discussed in Chapters 3 to 5 is utilized to develop fragility functions in conjunction with HAZUS methodology, as these MBTs satisfy the criteria for non-linear static procedures, whereas collapse fragility estimation of MBTs discussed in Chapter 7 is carried out using IDA procedures.

In HAZUS (2003) methodology, building damage functions are presented in the form of lognormal fragility curves that relate the probability of being in, or exceeding, a building damage state for a given demand parameter (e.g., spectral displacement). The fragility curves are lognormal distributions representing the probability of exceeding a given damage state, which is expressed as:

$$P[ds/S_d] = \emptyset[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\overline{S_{d,ds}}}\right)] \tag{7.1}$$

Here, $\overline{S_{d,ds}}$ is the median spectral displacement for the damage state ds, $\emptyset$ is the standard normal cumulative distribution function, β_{ds} is the standard deviation of the natural logarithm of the spectral displacement threshold for the damage state, ds, representing the combined uncertainties in the capacity curve, damage levels, modelling errors, and seismic hazard.

$$\beta_{ds} = \left\{\left(CONV\left[\beta_C, \beta_D, \overline{\beta_{d,ds}}\right]\right)^2 + \left(\beta_{M,ds}\right)^2\right\}^{1/2} \tag{7.2}$$

Here, β_C is the lognormal standard deviation parameter representing variability in the capacity properties of the building, β_D represents the variability in the demand spectrum due to spatial variability of the ground motion, and $\beta_{M(ds)}$ represents the uncertainty in the estimation of damage state threshold.

In case of fragility developed from IDA, the structural response, from elastic to global dynamic instability is captured under a suit of ground motion records and collapse is defined as damage state when a slight increase in Intensity Measure (IM) cause large increase in Damage Measure (DM). The probability of reaching or exceeding damage state (ds) ('collapse' for IDA) for a given median estimate of IM is expressed as:

$$P[C/IM] = \emptyset[\frac{1}{\beta_T}\ln\left(\frac{IM}{\overline{IM}}\right)] \tag{7.3}$$

Here, $\overline{IM}$ is median IM for collapse damage state, $\emptyset$ is the standard normal cumulative distribution function, β_T is the log-normal standard deviation of the IM for collapse damage state, describing total variability that takes in to account the record-to-record variability (β_{RTR}) and modeling variability (β_m). β_{RTR} is computed directly from IDA results and β_m is the function of prevalent construction practices, material of construction, design and detailing provisions, robustness and completeness of the analytical model used for collapse simulation (FEMA-P695 2009). In absence of reliable estimates of modeling variability for buildings complaint to Indian standards, modeling variability from previous studies have been adopted. Both β_{RTR} and β_m is combined with Square Root of Sum of Squares (SRSS) method, in order to estimate the total variability (β_T).

7.3 Definition of Damage States

Development of fragility functions (fragility curves/DPMs) requires to identify the level of severity of damage (damage states), expressed in terms of demand parameters and their corresponding threshold limit. HAZUS (2003) provides fragility functions for four damage states, and the probability of fifth damage state (i.e., "Collapse"), is specified as a fraction of complete damage which varies for different MBTs. Performance Levels of individual member is used for defining the damage state threshold based on two specified criteria. Barbat et al. (2006) classified four damage

States based on yield (S_{dy}) and ultimate (S_{du}) spectral displacement parameter, and the damage grades are similar to HAZUS (2003) reported in Table 7.1. Kappos et al. (2006) classified five types of damage grade based on yield and ultimate spectral displacement parameters of the buildings reported in Table 7.2.

Table 7.1 Damage state definition as per HAZUS (2003) and Barbat et al. (2006)

Damage Grade	Damage State	HAZUS						Barbat et al. (2006)
		Criteria No. 1			Criteria No. 2			
		Fraction	Limit	Factor	Fraction	Limit	Factor	
DS1	Slight	> 0%	C	1.0	50%	B	1.0	$0.7S_{dy}$
DS2	Moderate	> 5%	C	1.0	50%	B	1.5	S_{dy}
DS3	Extensive	> 25%	C	1.0	50%	B	4.5	$S_{dy}+0.25(S_{du}-S_{dy})$
DS4	Complete	> 50%	E	1.0-1.5	50%	B	12	S_{du}

Table 7.2 Damage state definition as per Kappos et al. (2006)

Damage Grade	Damage State	Spectral Displacement
DS1	Slight	$0.7\,S_{dy} < S_d < S_{dy}$
DS2	Moderate	$S_{dy} < S_d < 2\,S_{dy}$
DS3	Substantial to heavy	$0.7\,S_{dy} < S_d < S_{dy}$
DS4	Heavy to very heavy	$0.7\,S_{du} < S_d < S_{du}$
DS5	Collapse	$> S_{du}$

For simplicity, damage state definition proposed by Barbat et al. (2006) is considered for the present study, which based on yield and ultimate spectral displacment of the structure. In case of IDA, a single damage state ‘Collapse’ has been used, which represents instability of the structure due to slight increase in IM with large increase in DM. For the present study, IM is represented by average spectral acceleration ($S_{a,avg}$) and DM by maximum inter-storey drift ratio.

7.4 Selection of Variability Parameters

Selection of variability parameters is a crucial task for fragility analysis, generally a complex process requiring large amount of statistical data (Meslem and D’Ayala 2013). Estimation of suitable variability parameters is a challenging task in India due to unavailability of highly variable material properties of masonry, significant variation in construction practices, and lack of ground motion records (Haldar and Singh 2009; Choudhury and Kaushik 2018). Although India has suffered several major earthquakes in the past, unfortunately, such systematic data is lacking for Indian conditions. Equivalence established between HAZUS (2003) and Indian construction practices of infilled RC buildings, Haldar (2013) has arrived at the variability parameters for Indian RC buildings with URM infills which has been considered for the present study. Variability parameters for the corresponding classes of considered buildings are presented in Table 7.3.

Table 7.3 Selection of variability parameters

SMRF buildings designed and detailed as per BIS (2002), BIS (2016a, 2016b), BIS (1993)							
Damage grade	**Damage states**	**Spectral displacement**	**Post-yield degradation**	**Damage state variability ($\beta_{M(ds)}$)**	**Capacity curve variability (β_c)**	**Total variability (β_{ds})**	
						Mid-rise	**High-rise**
DS1	Slight	$0.7\ S_{dy}$	Minor Degradation (0.9)	Moderate (0.4)	Moderate (0.3)	0.75	0.7
DS2	Moderate	S_{dy}					
DS3	Extensive	$S_{dy} + 0.25(S_{du} - S_{dy})$	Major Degradation (0.5)			0.85	0.8
DS4	Heavy	S_{du}					
SMRF OGS buildings not designed as per OGS design provision of BIS (2002)							
Damage grade	**Damage states**	**Spectral displacement**	**Post-yield Degradation**	**Damage state variability ($\beta_{M(ds)}$)**	**Capacity curve variability (β_c)**	**Total variability (β_{ds})**	
						Mid-rise	**High-rise**
DS1	Slight	$0.7\ S_{dy}$	Minor Degradation (0.9)	Moderate (0.4)	Moderate (0.3)	0.75	0.7
DS2	Moderate	S_{dy}	Major Degradation (0.5)			0.85	0.8
DS3	Extensive	$S_{dy} + 0.25(S_{du} - S_{dy})$				0.85	0.8
DS4	Heavy	S_{du}	Extreme Degradation (0.1)			1	1

In the present study, variability parameters suggested by Haldar (2013) has been considered. SMRF buildings designed and detailed as per BIS (1993, 2002, 2016a, 2016b) are expected to experience minor degradation immediately after yielding, consequently variability of 0.9 corresponding to minor post-yield degradation have been assigned to lower damage grades. However, presence of URM infills result in rapid post-yield degradation of these buildings, and therefore variability of 0.5 corresponding to major degradation have been assigned to higher damage grades as shown in Table 7.3. In case of SMRF OGS buildings which are not designed as per OGS provision of BIS (2002), extreme degradation with variability of 0.1 is considered at heavy damage state.

7.5 Capacity Curve Parameters

To obtain the capacity curve parameters, the capacity curves/ pushover curves of the buildings for the selected design levels (mentioned in previous Chapters) have been idealized as bilinear capacity spectra, as shown in Fig. 7.1, using the ASCE-41 (2007) guidelines. The capacity curves (base shear vs. roof displacement) obtained in the previous Chapters (Chapter 3-5) are first transformed into capacity spectra (S_a vs. S_d) and then bi-linearized. The transformation from capacity curves to capacity spectra is performed using the following relationships:

$$S_a(g) = \frac{V_B}{W\alpha_m} \tag{7.4}$$

$$S_d = \frac{\Delta_{roof}}{\Gamma\phi_{roof}} \tag{7.5}$$

where,

$$\alpha_m = \frac{\sum (W_i\phi_i)^2}{\sum W_i \sum W_i {\phi_i}^2} \tag{7.6}$$

$$\Gamma = \frac{\sum W_i\phi_i}{\Sigma W_i {\phi_i}^2} \tag{7.7}$$

V_B is the base shear representing the building lateral load resistance, W is the total weight of building, W_i is the lumped storey weight at i^{th} floor level, Δ_{roof} is the roof displacement, ϕ_i is the modal shape coefficient for i^{th} floor, α_m is the modal mass coefficient (or fraction of the buildings weight effective in the pushover mode), and Γ is the modal participation factor for the pushover mode. It is to be noted that the yield spectral displacement (S_{dy}) on the bi-linearized capacity curve shown in Fig. 7.1, does not represent the point where the first member of the building has reached yield point, but the point where a sizable number of members has yielded, resulting in significant reduction in structural stiffness. Similarly, ultimate spectral displacement (S_{du}) represents the point where either the building becomes unstable due to formation of a failure mechanism, or the strength of the building degrades below 80% of the peak strength of the building.

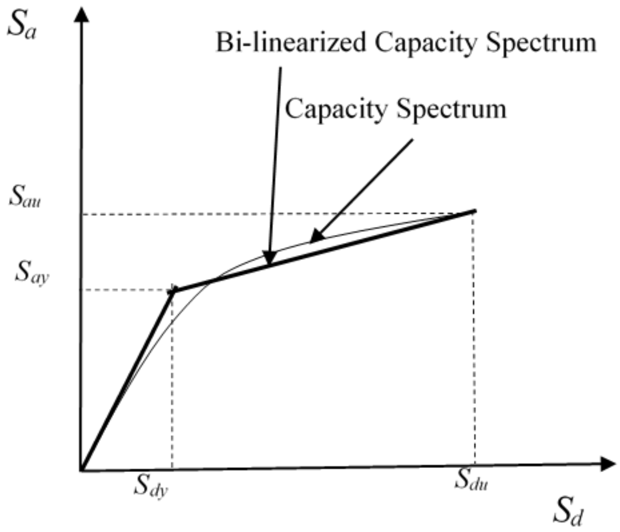


Fig. 7.1 Bi-linearization of capacity curve as per ASCE-41 (2007)

7.6 Effect of Functional Opening in Seismic Fragility of Uniformly Infilled Building

Indian seismic standards (BIS 2002, 2016a) like many other national standards follow force-based design methodology, which does not provide complete insight into the expected performance and associated seismic fragility of the designed buildings. Seismic performance obtained from deterministic analysis has been evaluated in probabilistic framework through fragility analysis to get the insight of expected damage under defined hazard level. To assess the associated seismic risk in a probabilistic framework, seismic fragility analysis has also been carried out for all the sets of buildings considered for seismic performance evaluation using HAZUS (2003) methodology. The capacity curve parameters obtained from non-linear static pushover analysis for the considered buildings in Chapter 3 is presented in Table 7.4. Figs. 7.2 and 7.3 represents seismic fragility curves of mid-rise and high-rise UI, mean opening combination W5D2 with 21% opening of the UI, W5D2 $\pm$ Standard Deviation (SD) of opening combination, and bare frame. The fragility curves for UI frame buildings show highest probability of damage due to lower yield displacement as a sizable number of infills yield at early stage, and bare frame being the most flexible building encounters lowest probability of damage at any given spectral displacement. However, it should be noted that fragility curves compare the probability of damage for a given spectral displacement. Hence a direct comparison of damage for any given hazard level for buildings having different dynamic characteristics is not possible from fragility curves alone.

Table 7.4 Capacity curve parameters of the considered buildings

S. No.	Building nomenclature	No. of Storey	Capacity Spectrum Parameters			
			Yield Point		Ultimate Point	
			S_{dy} (mm)	S_{ay} (g)	S_{du} (mm)	S_{au} (g)
1	**UI**	4	19	0.357	186	0.383
2		8	33	0.175	289	0.181
3	**Mean (W5D2) - SD**	4	27	0.294	229	0.299
4		8	43	0.145	305	0.152
5	**Mean (W5D2)**	4	31	0.263	248	0.267
6		8	47	0.136	312	0.144
7	**Mean (W5D2) + SD**	4	32	0.256	251	0.257
8		8	49	0.135	313	0.136
9	**Bare frame**	4	44	0.182	402	0.182
10		8	62	0.109	372	0.112

Therefore, discrete damage probabilities have been expressed in terms of Peak Ground Acceleration (PGA) for direct comparison purpose as reported in Table 7.5. It can be observed from Table 7.5 that mid-rise buildings with openings may suffer higher damages at all the damage grades as compared to UI buildings at both DBE and MCE hazard levels of seismic zone IV and V. However, in case of high-rise buildings with openings, damage probabilities at lower damage grades (DS1 and DS2) are equivalent to UI at both DBE and MCE hazard levels, but much higher damages can be expected at higher damage grade (DS3). It is further observed that being highly flexible, bare frame buildings show significantly higher damage probabilities from moderate to complete damage states as compared to its infilled counterparts. High-rise buildings are found to be more prone to seismic damages as compared to its mid-rise counterpart. Although all the considered buildings were designed for seismic zone V of BIS (2016a), however, damages up to moderate state can be expected with probability of 10% to 40% even at MCE level of shaking in the seismic zone IV of BIS (2016a).

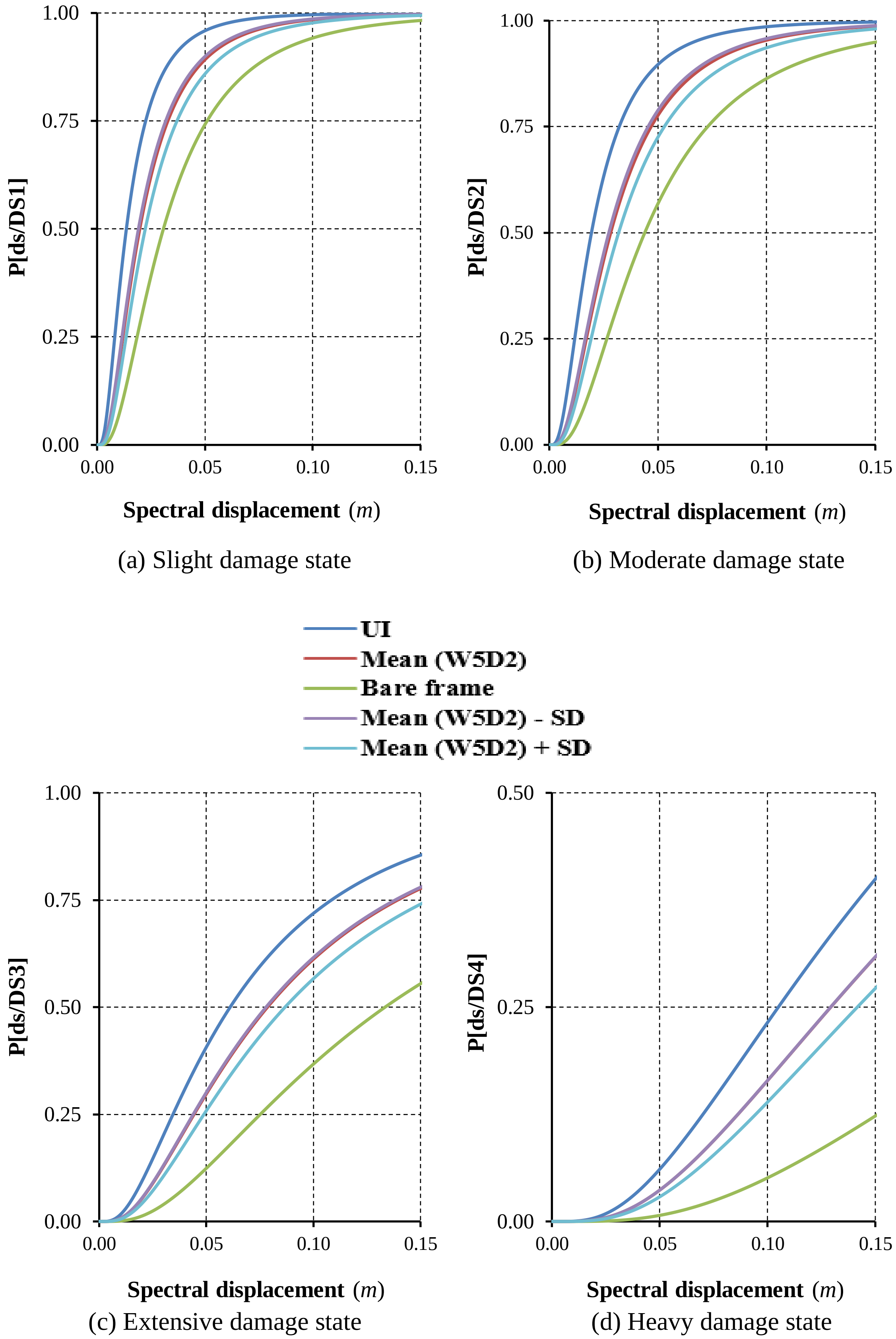
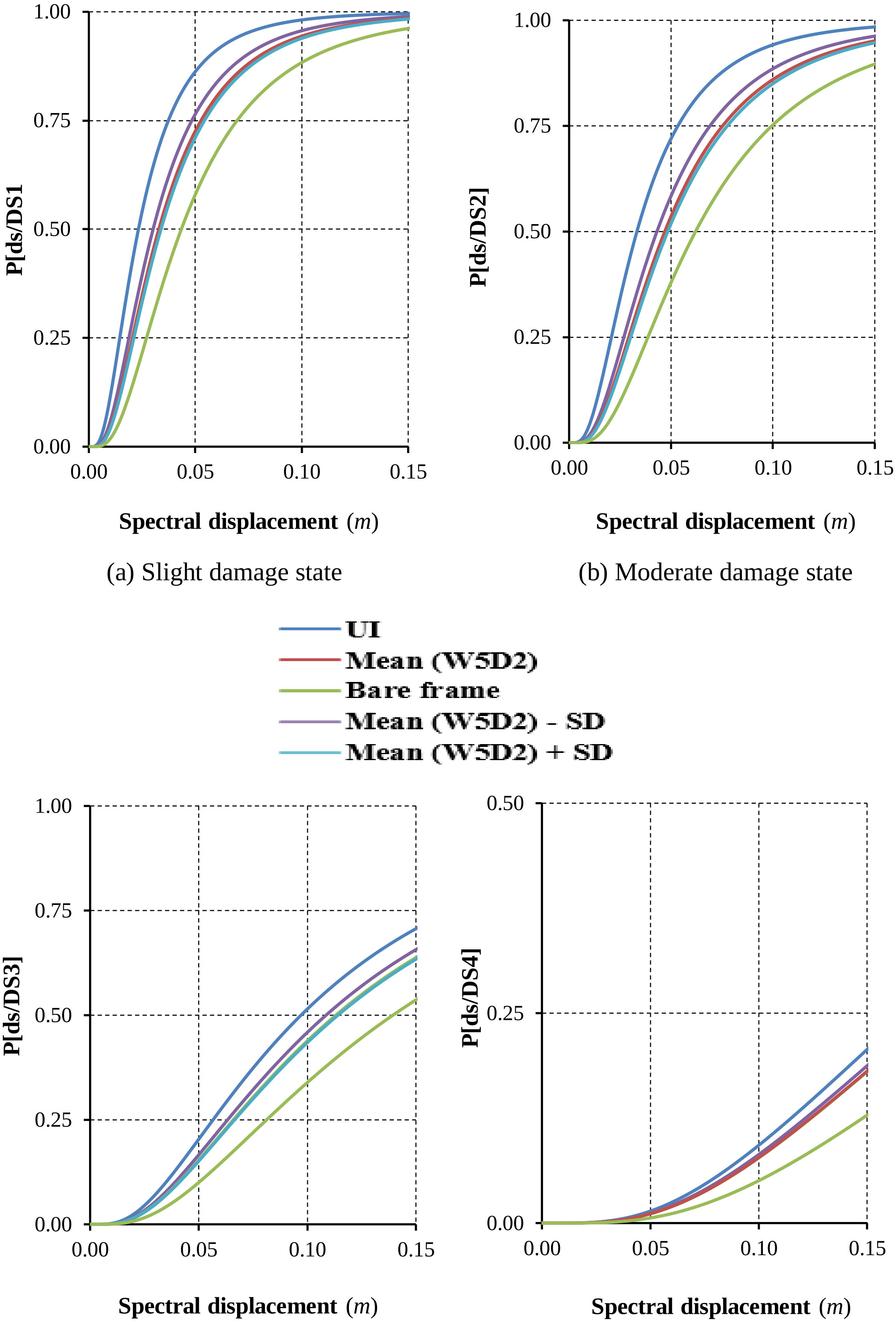


Fig. 7.2 Effect of functional opening on fragility of mid-rise buildings



(a) Slight damage state

(b) Moderate damage state

(c) Extensive damage state

(d) Heavy damage state

Fig. 7.3 Fragility curves of high-rise buildings

Table 7.5 Damage Probability Matrix (DPMs) (%) of mid-rise infilled RC buildings with functional openings

Building level	**Building nomenclature**	**Zone IV**								**Zone V**							
		DBE (0.12*g*)				**MCE (0.24*g*)**				**DBE (0.18*g*)**				**MCE (0.36*g*)**			
		DS1	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**
Mid-rise	**UI**	2	1	0	0	10	8	1	0	6	3	0	0	16	18	2	0
	Mean - SD	3	1	0	0	12	10	1	0	8	5	0	0	17	21	3	0
	Mean (W5D2)	5	3	0	0	14	14	2	0	10	7	1	0	18	26	5	0
	Mean + SD	5	2	0	0	14	14	2	0	10	8	1	0	18	27	5	0
	Bare frame	17	20	2	0	17	44	12	1	19	35	6	0	12	51	22	2
High-rise	**UI**	12	9	1	0	20	33	5	0	18	21	2	0	18	47	13	1
	Mean - SD	15	13	2	0	20	36	11	1	20	26	6	0	15	46	22	2
	Mean (W5D2)	16	16	3	0	19	38	14	1	20	29	8	0	14	44	26	4
	Mean + SD	17	18	4	0	18	39	17	2	20	31	10	1	12	43	30	5
	Bare frame	19	36	16	2	8	38	38	10	13	41	29	5	3	28	45	22

7.7 Effect of Prescriptive Design Guidelines of OGS on Seismic Fragility of Uniformly Infilled RC Building

The non-linear static pushover analysis performed for considered OGS buildings designed with various OGS design interventions discussed in Chapter 5, and subsequently their capacity curve parameters obtained through bi-linearization are reported in Table 7.6. It can be observed from Table 7.6 that mid-rise OGS buildings with bracings yield at the lowest displacement levels among the considered OGS buildings and very much comparable to the UI building.

Table 7.6 Capacity curve parameters of the OGS buildings designed with OGS design interventions

S. No.	Building nomenclature	No. of Storey	Capacity Spectrum Parameters			
			Yield Point		Ultimate Point	
			S_{dy} (*mm*)	S_{ay} (*g*)	S_{du} (*mm*)	S_{au} (*g*)
1	**OGS_ISC**	4	29	0.539	219	0.567
2		8	31	0.222	192	0.234
3	**OGS_BSC**	4	32	0.531	168	0.546
4		8	34	0.198	243	0.198
5	**OGS_BIS**	4	37	0.498	202	0.503
6		8	32	0.188	233	0.204
7	**OGS_EC8**	4	35	0.382	265	0.421
8		8	38	0.165	259	0.169
9	**OGS_BR1**	4	20	0.512	176	0.512
10		8	30	0.192	228	0.214
11	**OGS_BR2**	4	19	0.5	173	0.5
12		8	29	0.191	229	0.212
13	**OGS_BR3**	4	19	0.504	175	0.513
14		8	29	0.191	232	0.213
15	**UI**	4	19	0.357	186	0.383
16		8	33	0.175	289	0.181
17	**OOGS**	4	23	0.303	178	0.336
18		8	34	0.159	230	0.163
19	**OGSW**	4	27	0.239	127	0.291
20		8	33	0.155	200	0.160

It is evident from Fig. 7.4 that at the spectral displacement of 25*mm*, higher damage probability (32% moderate, and 15% extensive) can be observed in case of mid-rise OGS buildings with bracings and UI (35% moderate, and 14% extensive) due to yielding of infills and structural members at lower displacement level (Fig. 15). However, at the same spectral displacement, the highest probability of complete damage (8%) was observed for mid-rise OGSW building.

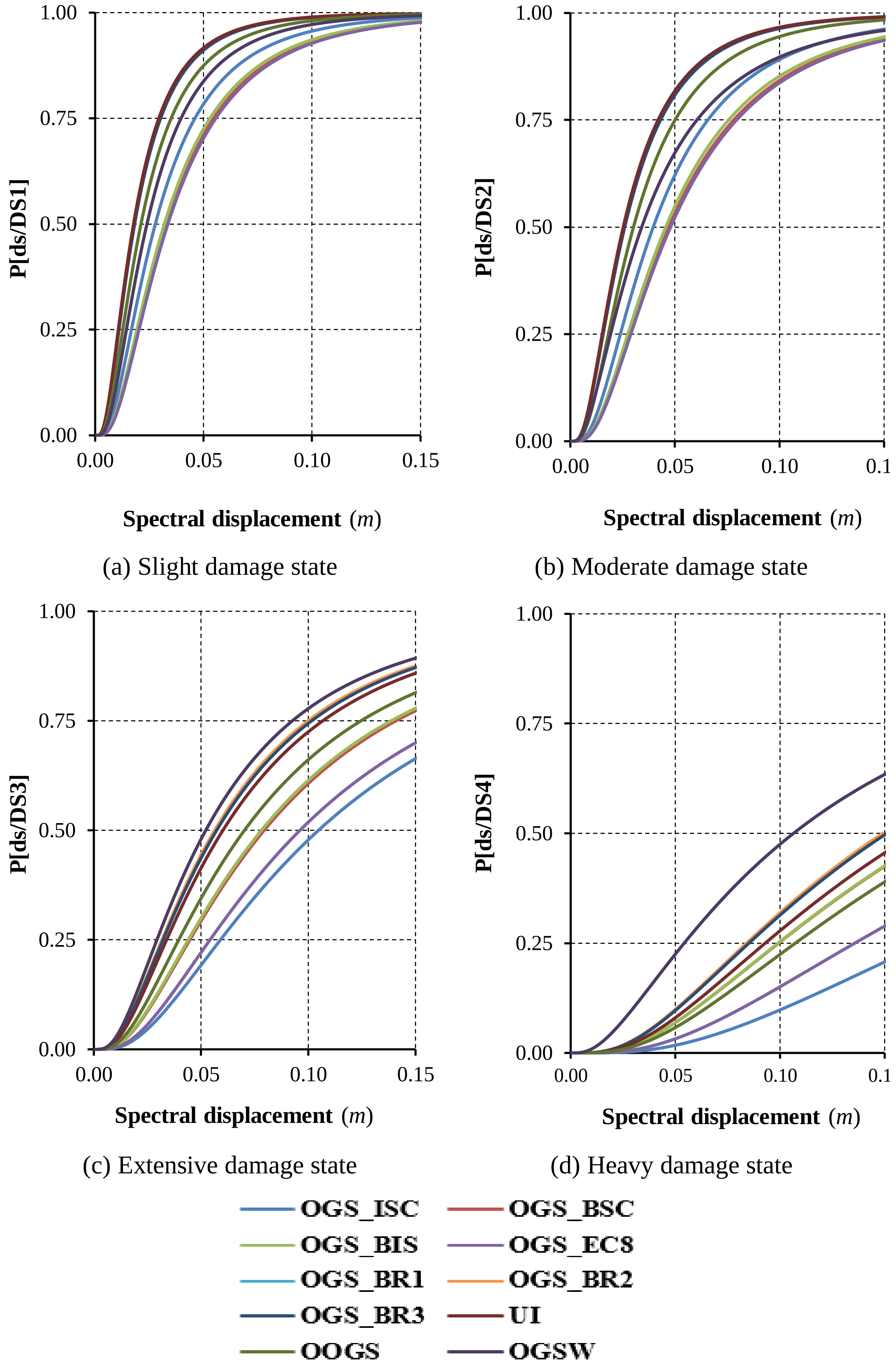
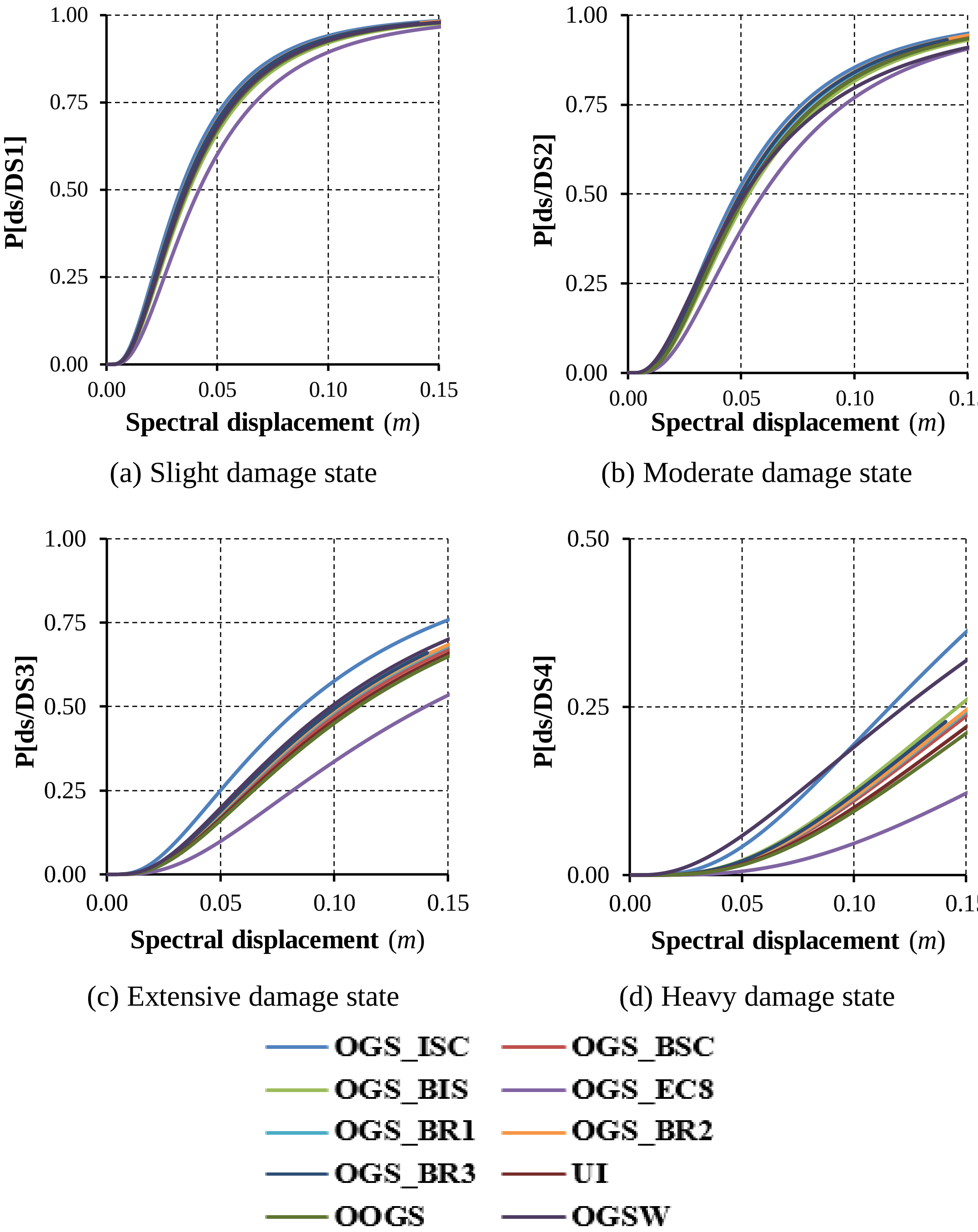


Fig. 7.4 Fragility curves of mid-rise buildings with OGS design interventions



(a) Slight damage state

(b) Moderate damage state

(c) Extensive damage state

(d) Heavy damage state

Fig. 7.5 Comparison of fragility curves of high-rise buildings with various OGS design interventions prescribed by different national seismic design standards

In case of high-rise buildings (Fig. 7.5), very little dispersion can be observed among fragility curves for slight and moderate damage states due to the close range of yield displacements of the considered buildings. At extensive damage state, OGS_ISC building shows the highest damage probability because the design of open and adjacent storey with higher seismic forces may cause formation of weak immediate upper storey, as discussed in Chapter 5 (Sections 5.3 and 5.6) causing accumulation of higher grade damages at lower displacement.

Table 7.7 Damage Probability Matrix (DPMs) (%) of the considered buildings with OGS design interventions

Building level	**Building nomenclature**	**Zone IV**								**Zone V**							
		DBE (0.12*g*)				**MCE (0.24*g*)**				**DBE (0.18*g*)**				**MCE (0.36*g*)**			
		DS1	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**
Mid-rise	**UI**	2	1	0	0	10	8	1	0	6	3	0	0	16	18	2	0
	OGS_ISC	2	1	0	0	11	9	1	0	6	3	0	0	15	16	3	0
	OGS_BSC	1	0	0	0	9	5	2	0	5	2	1	0	14	10	5	0
	OGS_BIS	1	0	0	0	9	5	2	0	4	2	1	0	14	10	5	0
	OGS_EC8	2	1	0	0	12	9	2	0	6	3	1	0	16	16	4	0
	OGS_BR1	1	1	0	1	6	3	0	0	3	1	0	0	11	9	1	0
	OGS_BR2	1	0	0	1	4	2	0	0	2	1	0	0	10	8	1	0
	OGS_BR3	1	0	0	1	3	1	0	0	3	1	0	0	10	8	1	0
	OOGS	8	5	1	0	18	23	6	0	12	9	2	0	19	29	8	1
	OGSW	10	11	4	2	18	23	16	9	15	18	8	4	18	25	22	14
High-rise	**UI**	12	9	1	0	20	33	5	0	18	21	2	0	18	47	13	1
	OGS_ISC	14	11	4	0	19	31	18	4	18	16	6	1	18	33	21	5
	OGS_BSC	14	12	3	0	18	36	17	2	18	19	6	0	17	39	20	3
	OGS_BIS	16	13	4	0	18	37	19	3	19	20	6	0	17	39	22	4
	OGS_EC8	18	19	4	0	16	43	20	2	20	25	6	0	15	44	23	3
	OGS_BR1	14	11	3	0	19	36	15	2	18	18	5	0	18	39	18	2
	OGS_BR2	13	10	2	0	19	37	16	2	18	19	5	0	17	40	18	2
	OGS_BR3	14	12	3	0	19	36	15	2	18	18	5	0	18	39	18	2
	OOGS	19	22	6	0	14	42	27	4	20	28	9	1	13	42	29	5
	OGSW	18	26	8	3	15	35	27	13	19	30	12	4	14	34	29	15

Discrete Damage Probabilities Matrix (DPMs) have been expressed in the form of Peak Ground Acceleration (PGA), for direct comparison, reported in Table 7.7. It can be observed from Table 7.7 that both at DBE and MCE levels of hazard, OGSW buildings (mid and high-rise) show the highest damage probability for all the damage grades followed by OOGS buildings. The high-rise buildings for all design levels are susceptible to greater damage as compared to their mid-rise counterpart. However, SMRF OGS for all the design interventions show comparable damage probabilities with UI buildings.

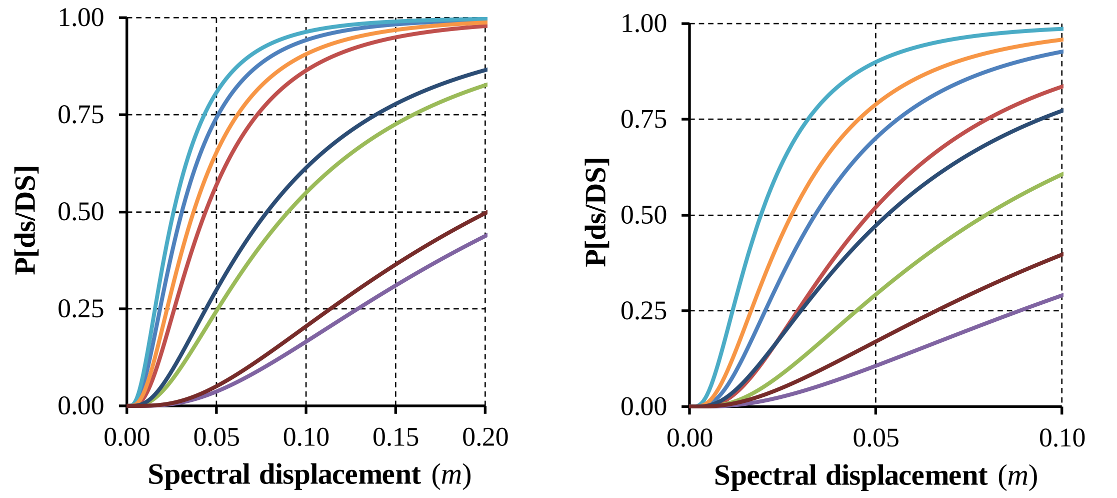
7.7.1 Effect of Functional Opening in Upper Storey Infills on the Seismic Fragility of OGS Buildings

The capacity curve parameters presented in Table 7.8 are developed from the non-linear static analysis of SMRF OGS buildings with functional openings in upper storey infills designed with and without OGS design provision of BIS (2002) as carried out in Chapter 4.

Table 7.8 Capacity curve parameters of the OGS buildings with functional openings in upper storey infills

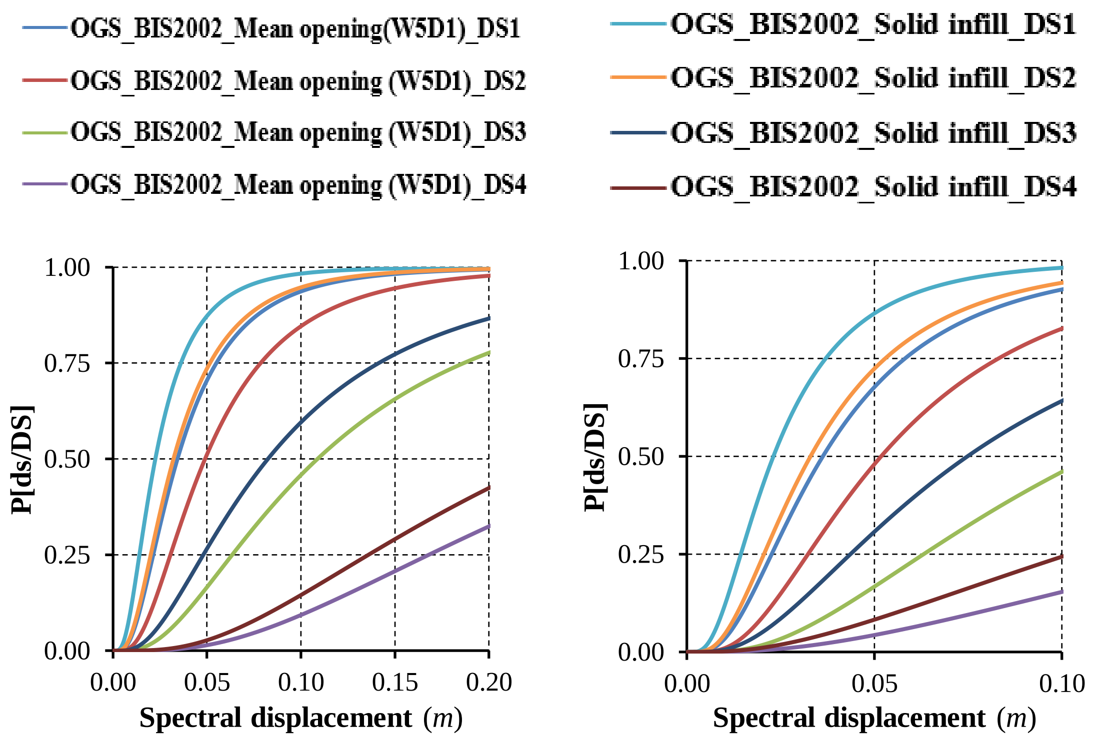
S. No.	Building nomenclature	No. of Storey	Capacity Spectrum Parameters			
			Yield Point		Ultimate Point	
			S_{dy} (*mm*)	S_{ay} (*g*)	S_{du} (*mm*)	S_{au} (*g*)
1	**OGS_BIS2002_Solid Infill**	4	37	0.498	202	0.503
2		8	32	0.188	233	0.204
3	**OGS_BIS2002_Opening (Mean, W5D1) + SD**	4	52	0.391	277	0.409
4		8	58	0.156	361	0.171
5	**OGS_BIS2002_Opening (Mean, W5D1)**	4	54	0.351	281	0.363
6		8	62	0.144	372	0.158
7	**OGS_BIS2002_Opening (Mean, W5D1) - SD**	4	45	0.331	230	0.342
8		8	65	0.138	380	0.151
9	**OGS_OGSW_Solid Infill**	4	27	0.239	127	0.291
10		8	33	0.155	200	0.160
11	**OGS_OGSW_Opening (Mean, W5D1) + SD**	4	42	0.214	168	0.255
12		8	52	0.124	274	0.136
13	**OGS_OGSW_Opening (Mean, W5D1)**	4	53	0.218	193	0.240
14		8	65	0.122	350	0.127
15	**OGS_OGSW_Opening (Mean, W5D1) - SD**	4	59	0.215	205	0.233
16		8	70	0.118	364	0.122

As discussed in earlier section (Section 7.6), similar pattern in capacity spectrum parameters can be observed, where yield and ultimate displacement level increases with functional opening ratio, but yielding and failure occurs at lower force level.



(a) (b)

Fig. 7.6 Seismic fragility curves of mid-rise buildings designed with and without conforming BIS (2002) OGS design requirement with upper storey infills being solid and mean opening combination (W5D1), (a) BIS_2002 and (b) OGSW



(a) (b)

Fig. 7.7 Seismic fragility curves of high-rise buildings designed with and without conforming BIS (2002) OGS design requirement with upper storey infills being solid and mean opening combination (W5D1), (a) BIS_2002 and (b) OGSW

Figs. 7.6 and 7.7 represent seismic fragility curves of mid-rise buildings designed with and without conforming BIS (2002) OGS design requirement with upper storey infills being solid and mean opening combination (W5D1). The fragility curves for OGS with solid infills show highest probability of damage at any given spectral displacement due to lower yield displacement as sizable number of infills yield at early stages. However, fragility curves comparing the probability of damage for a given spectral displacement cannot be used for comparison of damage for any given hazard level for buildings having different dynamic characteristics. Therefore, discrete damage probabilities have been expressed in terms of Peak Ground Acceleration (PGA) for direct comparison purpose as reported in Table 7.9. It can be observed from Table 7.9 that mid-rise OGS buildings designed as per BIS (2002) provisions with and without functional openings in upper storey infills show almost equivalent damage probabilities with deviation of $\pm$ 2% at both DBE and MCE hazard levels, and its high-rise counterpart attract higher damages at Slight damage state (DS1) to Extensive damage state (DS3). In case of mid-rise OGS buildings designed without any BIS (2002) OGS design provisions, OGS with solid infills at upper storey levels may undergo slightly higher damages due to high stiffening effect of solid infills. However, effect of OGS design provision is prominent for both mid and high-rise buildings, as it can be noted that more than 10% OGSW buildings are expected to suffer Extensive damage (DS3) to Heavy damage (DS4) at MCE hazard level of seismic zone V. It can be further observed that OGSW buildings can undergo noticeable amount of damages (DS1 and DS2) even at DBE and MCE hazard levels of seismic IV.

Table 7.9 Damage Probability Matrix (DPMs) (%) of OGS buildings designed with and without conforming BIS (2002) OGS design provision with openings upper storey infills

Building level	**Building nomenclature**	**Zone IV**								**Zone V**							
		DBE (0.12*g*)				**MCE (0.24*g*)**				**DBE (0.18*g*)**				**MCE (0.36*g*)**			
		DS1	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**	**DS1**	**DS2**	**DS3**	**DS4**
Mid-rise	**OGS_BIS2002_Solid Infill**	1	0	0	0	9	5	2	0	4	2	1	0	14	10	5	0
	OGS_BIS2002_Opening (Mean, W5D1) + SD	2	1	0	0	9	5	1	0	4	2	0	0	14	11	3	0
	OGS_BIS2002_Opening (Mean, W5D1)	1	0	0	0	8	5	1	0	5	2	1	0	14	12	4	0
	OGS_BIS2002_Opening (Mean, W5D1) - SD	2	1	0	0	10	6	2	0	5	3	1	0	15	13	5	0
	OGS_OGSW_Solid Infill	10	11	4	2	18	23	16	9	15	18	8	4	18	25	22	14
	OGS_OGSW_Opening (Mean, W5D1) + SD	10	5	2	1	18	17	10	4	15	11	6	2	18	25	18	9
	OGS_OGSW_Opening (Mean, W5D1)	7	3	1	1	17	12	8	3	13	7	4	2	19	20	15	8
	OGS_OGSW_Opening (Mean, W5D1) - SD	7	3	1	1	16	11	7	3	12	7	4	2	19	19	14	8
High-rise	**OGS_BIS2002_Solid Infill**	16	13	4	0	18	37	19	3	19	20	6	0	17	39	22	4
	OGS_BIS2002_Opening (Mean, W5D1) + SD	12	8	1	0	20	29	9	1	18	19	5	0	18	40	19	2
	OGS_BIS2002_Opening (Mean, W5D1)	14	11	2	0	20	31	12	1	19	22	6	0	16	40	23	3
	OGS_BIS2002_Opening (Mean, W5D1) - SD	13	9	2	0	20	30	11	1	19	20	6	0	17	39	22	3
	OGS_OGSW_Solid Infill	18	26	8	3	15	35	27	13	19	30	12	4	14	34	29	15
	OGS_OGSW_Opening (Mean, W5D1) + SD	15	19	5	2	19	31	16	6	19	28	11	4	15	32	28	15
	OGS_OGSW_Opening (Mean, W5D1)	17	15	4	1	20	31	12	4	20	27	9	3	13	38	27	12
	OGS_OGSW_Opening (Mean, W5D1) - SD	15	19	4	1	19	31	13	5	19	29	10	4	15	34	28	13

7.8 Effect of Irregular Infills in Plan on Seismic Fragility of RC Building

Seismic fragility is a parameter that describes the probability of achieving and exceeding a particular 'damage state' for a given intensity measure (IM). Based on the median collapse capacities obtained from Incremental Dynamic Analysis (IDA) presented in Chapter 6, fragility curves are generated (Haselton et al. 2010) by determining the probability of collapse of the representative buildings. The variability in collapse capacity has two components: (i) record-to-record variability (β_{RTR}) which is obtained directly by post-processing IDA results and (ii) modelling variability (β_m) considered as 0.5 based on the previous collapse fragility studies (Haldar 2008; Haldar 2013). Total variability β is obtained by combining β_{RTR} and β_m using Square-Root-of-Sum-of-Squares (SRSS) method.

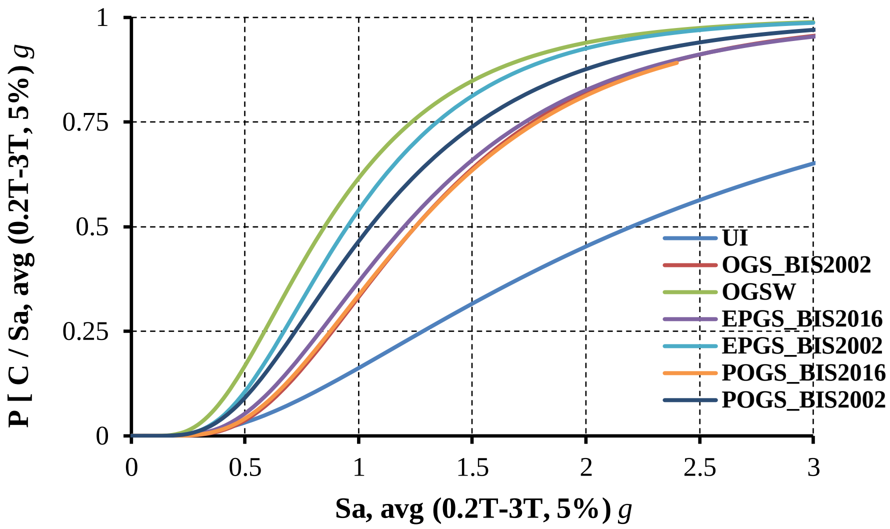


Fig. 7.8 Collapse fragility curves for buildings with irregular infills in plan

The fragility curves of the considered buildings plotted in terms of average spectral acceleration demand (Fig. 7.8) cannot be compared directly as the spectral acceleration of the considered buildings is dependent on the time period of the buildings which are quite different as discussed in Chapter 6. Therefore, comparison of discrete damage probabilities for different values of Effective Peak Ground Acceleration (EPGA), conventionally used in the design codes as zone factor, can provide a clearer picture of the relative damageability of different considered buildings with varying infill configurations and design levels. Table 7.10 represents fragility curve parameters of the considered buildings obtained through IDA, median collapse capacity, and collapse probability conditioned on the occurrence of DBE and MCE hazard levels of seismic zone V of Indian seismic design standards (BIS 2002, 2016a). UI building have least collapse probability for DBE and MCE hazard levels which is in agreement with the higher median collapse capacity observed in Chapter 6 (Fig. 6.7). OGSW building which is not designed with open ground storey provisions of BIS (2002) is most vulnerable among all the considered buildings for any given $S_{a,avg}$ whereas, the damage probability significantly lowered when the open storey members are designed with guidelines of BIS (2002).

Table 7.10 Variability, median collapse capacity and collapse probability of the considered buildings

Buildings nomenclature	$\boldsymbol{\beta}$	**Median collapse capacity** $S_{a,avg}$ $(0.2T–3T, 5\%)$ g	**Collapse probability** (%)	
			DBE (0.18g)	**MCE (0.36g)**
UI	0.66	2.05	2.8	14.1
OGS_BIS2002	0.59	1.15	2.5	26.1
OGSW	0.52	0.85	6	38.2
EPGS_BIS2016	0.51	1.25	3.8	26.3
EPGS_BIS2002	0.51	0.95	7.6	45.8
POGS_BIS2016	0.51	1.2	3.5	26.7
POGS_BIS2002	0.51	0.95	6.9	39.8

The capacity as well as collapse probability of EPGS and POGS buildings are comparable, however, the collapse probability reduces significantly when designed with latest BIS (2016a, 2016b) as compared to older seismic standards BIS (1993, 2002).

7.9 Summary

In this Chapter, seismic fragility in terms of fragility curves and DPMs have been studied for RC buildings with prevalent configuration of URM infills along with functional openings, using HAZUS and IDA procedures. It has been observed that mid-rise buildings with openings may suffer higher damages at all the damage grades as compared to UI buildings at both DBE and MCE hazard levels. However, in case of high-rise buildings with openings, damage probabilities at lower damage grades (DS1 and DS2) are equivalent to UI at both DBE and MCE hazard levels, but much higher damages can be expected at higher damage grades. It is further observed that being highly flexible, bare frame buildings show significantly higher damage probabilities from moderate to heavy damage grades as compared to its infilled counterparts. However, RC bare frames do not represent realistic building and considered as an extreme boundary case of opening. High-rise buildings with increasing openings are found to be more prone to seismic damages as compared to its mid-rise counterpart.

In case of open ground storey building which are not designed with any OGS design interventions showed the highest damage probability for all the damage grades. However, SMRF OGS buildings designed with OGS design interventions showed comparable damage probabilities with UI buildings. No significant effect of functional opening in upper storey infills is observed on the estimated seismic damage probabilities of OGS buildings designed with and without OGS design provision of BIS (2002).

It is further observed through IDA procedure that irregular infilled RC buildings designed with revised Indian standards is found to perform better than its older counterpart owing to the adequacy of revised provisions of BIS (2016a). It can be further concluded that EPGS MBT designed with older Indian seismic standards (BIS 1993, 2002) have comparatively higher vulnerability than the POGS MBT.

Chapter 8
Conclusions and Recommendations for Future Work

In this Thesis, an attempt has been made to develop a reliable, cost-effective methodology for seismic performance assessment of practical RC buildings compliant to Indian standards and construction practices. The macro-model capable of simulating the effect of infills on the seismic behaviour of RC frames efficiently has been identified. Further, a simplified and realistic macro-model has been identified for simulating the effect of functional openings due to doors and windows which can be directly used by practicing design engineers for realistic prediction of seismic response. Using the identified modeling guidelines for URM infills, with functional openings, its effect on the seismic performance and consequent fragility of infilled RC frame buildings has been studied. The Thesis work also address the concern of irregular placement of infills in the ground storey of URM infilled RC buildings due to mixed occupancy as a result to accommodate both residential and commercial purposes together in Indian urban areas. An inexpensive, simple solution is prescribed in this Thesis to design Open Ground Storey (OGS) buildings by eliminating strength and stiffness irregularity from the open storey for all practical purposes without requiring explicit expertise of non-linear dynamic analysis, which is a challenging task for the structural design practitioners as it requires special skill set, cost, effort and time intensive non-linear dynamic evaluation. Particular focus of this Thesis is on evaluation of capacity parameters and fragility functions considering all possible failure modes of infill panels and surrounding frame members of existing RC frame buildings with prevalent irregular configurations of URM infills in Indian urban areas. Possible shear failure of structural members is also considered in the present study. The fragility parameters for Indian RC frame building with irregular configurations of URM infills derived in this Thesis have been incorporated in the spreadsheet-based open-source seismic risk evaluation software tool ‘SeisVARA’(Haldar et al. 2013).

8.1 Major Conclusions of the Thesis

The major conclusions of this Thesis are as following:

- A simplified macro modeling approach has been developed in order to simulate the in-plane nonlinear response of infilled RC frame (weak-infill-ductile frame). Modeling guidelines by ASCE-41 (2007) for URM infills is found to predict the peak strength and failure mechanism of infilled RC frame constructed as per Indian practices, though it under estimates the initial stiffness.

- Reduction model proposed by Decanini et al. (2014) for simulation of opening in infills closely matches with the experimental observations with the least mean deviation of 4.7% and can be used for modeling of realistic opening in infilled RC buildings.

- To encompass the wide spectrum of irregular infilled Indian RC frame buildings, a pilot survey is carried out in Indian cities based on the prevalent irregular configurations of infills and key parameters such as framing system, design seismic force levels, detailing of reinforcement and height of buildings. URM infilled buildings have been classified into 7 categories (WD, OGS, EPGS, EPGSIP1, EPGSIP2, EPGSIP3, and POGS) depending on type of prevalent infill irregularity at ground storey which are further sub-divided based on the key parameters influencing seismic behaviour of such buildings and a total of 14 different Model Building Types (MBTs) have been identified.

- From the statistical evaluation of the surveyed MBTs in terms percentage of various MBTs and their distribution over mid-rise and high-rise buildings, it is observed that mid-rise Open Ground Storey (OGS) and Partially Open Ground Storey (POGS) buildings together shares almost 70% of the surveyed MBTs followed by External Periphery of the Ground Storey without any interior partition walls (EPGS), as these typologies are mostly preferred in urban areas to serve the purpose of combined parking and commercial spaces.

- To cover the wide spectrum of size and configuration of openings in Indian residential buildings, CPWD manual (CPWD, 2006) guidelines for doors and windows in residential buildings have been considered for the parametric study. CPWD (2006) has recommended 15 windows and 6 door sizes by varying the length and height of openings which eventually arrive at variation of window from 6% to 32%, and door opening sizes 20% to 33% of the solid infill panel area.

- Presence of functional opening due to doors and windows in infills reduces the lateral strength, stiffness, and ductility of the building. Strong negative correlation is observed for lateral strength, stiffness, and ductility of the building with the increase of functional opening ratio. Opening in infills further increases the displacement capacity of the buildings as reduction in infills enhances flexibility of the building.

- Mid-rise buildings with openings may suffer higher damages at all the damage grades as compared to UI buildings at both DBE and MCE hazard levels. However, in case of high-rise buildings with openings, damage probabilities at lower damage grades are equivalent to UI at both DBE and MCE hazard levels, but considerable higher damages can be expected at higher damage grades.

- Presence of functional opening in upper storey infills has significant negative impact on the seismic performance of OGS buildings. The lateral stiffness and strength of mid-rise OGS buildings designed as per BIS (2002) provision is reduced to 55% and 65% respectively, when functional opening reached to 30%, and thereby affecting the adequacy of BIS (2002) OGS design provision.

- Functional openings in infills enhanced the flexibility of the OGS building however; ductility is reduced as plastic displacement does not increase with the same rate as that of yield displacement. The reduction in ductility is significant for OGS buildings designed without BIS (2002) OGS provision.

- Functional openings in the form of doors and windows are integral part of a building and therefore cannot be avoided completely. It is evident form the present study that increase in functional openings causes decrease in strength and stiffness thereby significantly affect the seismic performance of OGS buildings. As functional openings cannot be avoided completely due to practical purposes, therefore, selection of optimum level of opening may serve the necessary functional requirement without affecting the seismic performance of OGS buildings drastically. According to the findings of the present study, 15% opening can be considered as optimum percentage of opening in the building where the strength and ductility can be achieved up to

80% of the OGS building as compared to OGS buildings with solid upper storey infills.

- No significant effect of functional opening in upper storey infills is observed on the estimated seismic damage probabilities OGS buildings designed with and without OGS design provision of BIS (2002).

- Global collapse of OGS buildings designed without BIS (2002) OGS provision is mainly governed by failure of ground storey members, attributed to excessive inelastic deformation demand in the ground storey. However, collapse of ground storey can be prevented for OGS buildings designed with BIS (2002) provision, both at DBE and MCE hazard levels of Zone V of Indian seismic design standards.

- Adequacy of BIS (2002) provisions for designing open storey members with 2.5 times design base shear can be observed in terms of seismic performance and damage probability as it closely follows its UI counterpart.

- Design of open storey members with higher seismic forces is time inexpensive, simple to apply for all practical purposes without requiring explicit expertise of non-linear dynamic analysis for open ground storey buildings.

- Use of RC bracings for elimination of OGS strength and stiffness irregularity, suggested in Indian seismic design standard (BIS 2016), may cause undesirable soft storey like displacement demand resulting in stress concentration at the immediate upper storey owing to high stiffness and strength of RC bracings.

- Design of open storey, including adjacent storey with 3 times higher seismic forces as recommended by Israel seismic code, is found predominant in case of mid-rise buildings. However, the same design philosophy may cause formation of immediate weak upper storey in case of relatively flexible high-rise buildings, causing failure of weak storey columns at early stages.

- OGS buildings which are not designed with any OGS design interventions showed the highest damage probability for all the damage grades. However, SMRF OGS buildings designed with OGS design interventions showed comparable damage probabilities with UI buildings.

- Irregular configuration of infills in RC buildings, increases the collapse risk, reduces median collapse capacity resulting global collapse due to failure of structural members in the vicinity of irregularities at ground storey of the building.

- Irregular infilled RC buildings designed with revised Indian standards are found to perform better than its older counterpart. EPGS buildings designed with older Indian seismic standards (BIS 1993, 2002) poses higher vulnerability than the POGS buildings.

- Approximately 50% reduction of median collapse capacity can be observed in buildings with irregular configuration of infills in both plan and elevation as compared to its ideal counterpart uniformly infilled building.

8.2 Recommendations for Future Work

The present study is based on analytical simulation of the seismic behaviour, which needs to be validated by experimental results. Therefore, large scale tests of bare and infilled RC frames with regular and irregular infills in plan and/or elevation are required to be undertaken.

In case of URM infills, both in-plane and out-of-plane actions are important. Only in-plane actions have been considered in the present study, however, presence of larger opening in infills may initiate out of plane failure, and thereby combined effect of in-plane and out-of-plane response of infilled frame need to be investigated.

Variabilities in different input parameters for fragility analysis have been considered from literature. However, these variabilities in Indian constructions need to be evaluated using extensive field studies.

The present study mainly focused on SMRF buildings compliant to Indian seismic design standards. Due to lack of proper enforcement, large stock of non-seismically designed RC buildings with irregular infill configuration exists which need to be studied further.

The present study considered the prevalent irregular infill configuration at ground storey only, which needs to be extended for other variation of irregular infill in upper storey for seismic response studies.

Present study is limited to residential and office buildings in flat terrain only. This can be extended to hill buildings, commercial buildings having large span floor systems and shear wall cores.

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