EARTHQUAKE RESISTANT DESIGN OF LOW-RISE OPEN GROUND STOREY FRAMED BUILDING

A thesis

Submitted by

SNEHASH PATEL

(210CE2276)

In partial fulfillment of the requirements for

the award of the degree

of

MASTER OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA 769008

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Dr. PRADIP SARKAR

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NATIONAL INSTITUTE OF TECHNOLOGY
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INDIA

This is to certify that the thesis entitled "**EARTHQUAKE RESISTANT DESIGN OF LOW-RISE OPEN GROUND STOREY FRAMED BUILDING**" submitted by **Snehash Patel** in partial fulfillment of the requirement for the award of **Master of Technology** degree in **Civil Engineering** with specialization in **Structural Engineering** to the National Institute of Technology, Rourkela is an authentic record of research work carried out by her under my supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

Project Guide

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ABSTRACT

KEYWORDS: *infill walls*, *diagonal strut*, *open ground storey*, *equivalent static analysis*, *response spectrum analysis*, *pushover analysis*, *low rise building*

Presence of infill walls in the frames alters the behaviour of the building under lateral loads. However, it is common industry practice to ignore the stiffness of infill wall for analysis of framed building. Engineers believe that analysis without considering infill stiffness leads to a conservative design. But this may not be always true, especially for vertically irregular buildings with discontinuous infill walls. Hence, the modelling of infill walls in the seismic analysis of framed buildings is imperative. Indian Standard IS 1893: 2002 allows analysis of open ground storey buildings without considering infill stiffness but with a multiplication factor 2.5 in compensation for the stiffness discontinuity. As per the code the columns and beams of the open ground storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads of bare frames (i.e., without considering the infill stiffness). However, as experienced by the engineers at design offices, the multiplication factor of 2.5 is not realistic for low rise buildings. This calls for an assessment and review of the code recommended multiplication factor for low rise open ground storey buildings. Therefore, the objective of this thesis is defined as to check the applicability of the multiplication factor of 2.5 and to study the effect of infill strength and stiffness in the seismic analysis of low rise open ground storey building.

Infill walls can be modelled in commercial software using two-dimensional area element with appropriate material properties for linear elastic analysis. But this type of modelling may not work for non-linear analysis since the non-linear material properties for a two-dimensional orthotropic element is not very well understood. Seismic evaluation of an

existing reinforced concrete (RC) framed building would invariably require a non-linear analysis. Published literature in this area recommends a linear diagonal strut approach to model infill wall for both linear (Equivalent Static Analysis and Response Spectrum Analysis) and nonlinear analyses (Pushover Analysis and Time History Analysis).

An existing RC framed building (G+3) with open ground storey located in Seismic Zone-V is considered for this study. This building is analyzed for two different cases: (a) considering both infill mass and infill stiffness and (b) considering infill mass but without considering infill stiffness. Two separate models were generated using commercial software SAP2000. Infill weights were modelled through applying static dead load and corresponding masses considered from this dead load for dynamic analyses. Infill stiffness was modelled using a diagonal strut approach. Two different support conditions, namely fixed end support condition and pinned end support condition, are considered to check the effect of support conditions in the multiplication factors. Linear and non-linear analyses were carried out for the models and the results were compared.

The analysis results show that a factor of 2.5 is too high to be multiplied to the beam and column forces of the ground storey of low-rise open ground storey buildings. This study conclude that the problem of open ground storey buildings cannot be identified properly through elastic analysis as the stiffness of open ground storey building and a similar bare-frame building are almost same. Nonlinear analysis reveals that open ground storey building fails through a ground storey mechanism at a comparatively low base shear and displacement and the mode of failure is found to be brittle. Linear and nonlinear analyses show that support condition influences the response considerably and can be an important parameter to decide the force amplification factor.

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ABBREVIATIONS

ABS	maximum ABSolute response
CP	Collapse Prevention
CQC	Complete Quadratic Combination
DCR	Demand to Capacity Ratio
ESA	Equivalent Static Analysis
IO	Immediate Occupancy
IR	Interaction Ratio
IS	Indian Standards
LS	Life Safety
MDOF	Multi Degree Of Freedom
MF	Multiplication Factor
NDA	Nonlinear Dynamic Analysis
OGS	Open Ground Storey
PA	Pushover Analysis
RC	Reinforced Concrete
RSA	Response Spectrum Analysis
SDOF	Single Degree Of Freedom
SRSS	Square Root of Sum of Squares
WI	With Infill
WOI	Without Infill

NOTATIONS

ENGLISH

l	length between centre lines of the column
l'	clear length of infill panel
l_e	eccentricity of equivalent strut
R_c	failure load of infilled frame corresponding to corner crushing
R_s	failure load of infilled frame corresponding to shear crushing
R_i	instantaneous axial load of equivalent diagonal strut
T	fundamental time period of the building (in seconds)
V_B	design base shear
S_a/g	average response acceleration coefficient for rock or soil sites based on appropriate natural periods and damping of the structure
w	width of equivalent strut
w'	width of equivalent strut considering infilled panel aspect ratio
A_h	design horizontal seismic coefficient
W	seismic weight of the building
Z	zone factor
I	importance factor
R	response reduction factor
d	base dimension of the building
f_m'	compressive strength of masonry infills
f_{bs}'	the bond shear strength between the masonry and mortar.
E_s	elastic modulus of the equivalent strut

E_c	elastic modulus of the column in the bounding frame
I_c	moment of inertia of the column
h'	clear height of infill wall
h	height of column between centrelines of beams
t	thickness of infill wall
M_x	moment about x axis
M_y	moment about y axis
$M_{x,c}$	maximum uniaxial bending moment capacity along x axis
$M_{y,c}$	maximum uniaxial bending moment capacity along y axis
P	axial load on the member

GREEK SYMBOLS

λh	relative stiffness parameter for infilled frame
θ	slope of the infill wall diagonal to the horizontal
α, β	Dynamic amplification factor
γ	Coefficient for accidental eccentricity
β_{eq}	Equivalent damping ratio
ω	Frequency
ω_i, ω_j	Circular frequency in i^{th} and j^{th} mode respectively
ω_n	n^{th} mode natural frequency
δ_{max}	Maximum displacement of a building floor
δ_{avg}	Average displacements of two extreme points of a building floor
τ	Modal damping ratio
Λ	Plan aspect ratio of the building (= L_{max}/L_{min})

λ_i, λ_j	Response quantity in mode i and j respectively
μ	Displacement ductility ratio
ρ_{ij}	Cross-modal coefficient
ρ_s	Volumetric ratio of confining steel
ξ_n	Damping ratio
ϵ_{sm}	Steel strain at maximum tensile stress
ϵ_1, ϵ_2	Extreme fibre strains
φ	Curvature
ϕ_{ik}	Mode shape coefficient at floor i in k mode
$\phi_{n,roof}$	n^{th} mode shape at roof level
φ_u	Ultimate curvature
φ_y	Yield curvature
θ_u	Ultimate rotation
θ_y	Yield rotation

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

Due to increasing population since the past few years car parking space for residential apartments in populated cities is a matter of major concern. Hence the trend has been to utilize the ground storey of the building itself for parking. These types of buildings (Fig. 1.1) having no infill masonry walls in ground storey, but infilled in all upper storeys, are called Open Ground Storey (OGS) buildings. They are also known as 'open first storey building' (when the storey numbering starts with one from the ground storey itself), 'pilotis', or 'stilted buildings'.

Fig. 1.1: Typical example of OGS building

There is significant advantage of these category of buildings functionally but from a seismic performance point of view such buildings are considered to have increased vulnerability. From the past earthquakes it was evident that the major type of failure that occurred in OGS buildings included snapping of lateral ties, crushing of core concrete, buckling of longitudinal reinforcement bars etc. Due to the presence of infill walls in the entire upper storey except for the ground storey makes the upper storeys much stiffer than the open ground storey. Thus, the upper storeys move almost together as a single block, and most of the horizontal displacement of the building occurs in the soft ground storey itself. In other words, this type of buildings sway back and forth like inverted pendulum (Fig. 1.2) during earthquake shaking, and hence the columns in the ground storey columns and beams are heavily stressed. Therefore it is required that the ground storey columns must have sufficient strength and adequate ductility. The vulnerability of this type of building is attributed to the sudden lowering of lateral stiffness and strength in ground storey, compared to upper storeys with infill walls.

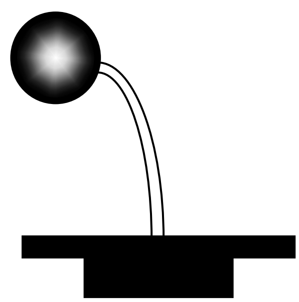


Fig. 1.2: Behaviour of OGS buildings like as inverted pendulum

The OGS framed building behaves differently as compared to a bare framed building (without any infill) or a fully infilled framed building under lateral load. A bare frame is much less stiff than a fully infilled frame; it resists the applied lateral load through frame action and shows well-distributed plastic hinges at failure. When this frame is fully infilled, truss action is introduced. A fully infilled frame shows less inter-storey drift, although it attracts higher base shear (due to increased stiffness). A fully infilled frame yields less force in the frame elements and dissipates greater energy through infill walls. The strength and stiffness of infill walls in infilled frame buildings are ignored in the structural modelling in conventional design practice. The design in such cases will generally be conservative in the case of fully infilled framed building. But things will be different for an OGS framed building. OGS building is slightly stiffer than the bare frame, has larger drift (especially in the ground storey), and fails due to soft storey-mechanism at the ground floor as shown in Fig. 1.3. Therefore, it may be unconservative to ignore strength and stiffness of infill wall while designing OGS buildings.

Fig. 1.3: General mode of failure in OGS buildings

Inclusion of stiffness and strength of infill walls in the OGS building frame decreases the fundamental time period compared to a bare frame and consequently increases the base shear demand and the design forces in the ground storey beams and columns. This increased design forces in the ground storey beams and columns of the OGS buildings are not captured in the conventional bare frame analysis. An appropriate way to analyse the OGS buildings is to model the strength and stiffness of infill walls. Unfortunately, no guidelines are given in IS 1893: 2002 (Part-1) for modelling the infill walls. As an alternative a bare frame analysis is generally used that ignores the strength and stiffness of the infill walls.

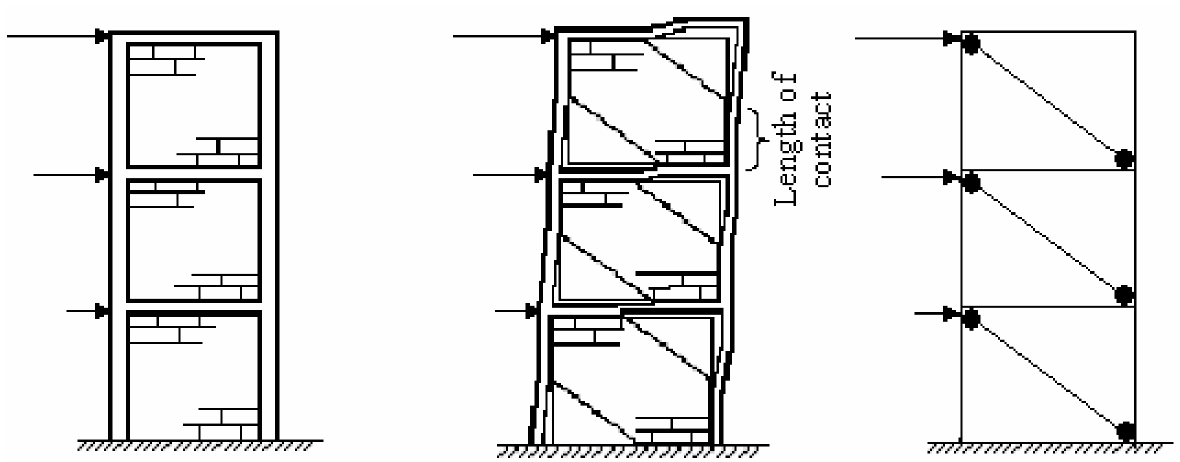
The failure pattern observed in the buildings during the Jabalpur earthquake (1997) showed the vulnerability of OGS buildings. Some reinforced concrete framed building which collapsed partially, had open ground storey on one side for parking, and brick infill walls on the other side. In the aftermath of the Bhuj earthquake, the IS 1893 code was revised in 2002, incorporating new design recommendations to address OGS buildings. Clause 7.10.3(a) states: "The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads of bare frames." The factor 2.5 can be told as a multiplication factor (MF). This multiplication factor (MF) is supposed to be the compensation for the stiffness discontinuity. Other national codes also recommend multiplication factors for this type of buildings. The conservative nature of this empirical recommendation of IS code was first pointed out by Kanitkar and Kanitkar (2001), Subramanian (2004) and Kaushik (2006). Hence the aim of this thesis is to check the applicability of the multiplication factor of 2.5 in the ground storey beams and column when the building is to be designed as open ground storey framed building and to study the effect of infill strength and stiffness in the seismic analysis of low rise open ground storey building.

Non-linear dynamic (NDA) analysis is considered to be the most accurate but at the same time it is most rigorous among all methods. Hence for the present study Equivalent static analysis (ESA), Response spectrum analysis (RSA) and Pushover analysis (PA) is considered for the comparative study. To carry out these analyses a typical building model with two different cases and support conditions are considered.

i) Considering infill strength and stiffness

ii) Without considering infill strength and stiffness

Support condition has a great influence in the global stiffness of the building. Therefore building models were analysed in the present study for two commonly used support conditions: (a) fixed and (b) pinned end support conditions. The hinged end support conditions are considered in case of isolated footing. From literature it is obvious that a hinge is to be provided at column end at the bottom of the foundation. However when it is founded on hard rock, the column end may be modelled as fixed, with the level of fixity at the top of the footing.



(a)Infilled Frame (b) Deformed Frame (c) Equivalent Strut model

Fig. 1.4: Behaviour of Infilled frames (ref. Asokan 2006)

Masonry infill walls are widely used as partitions all over the world. Evidences are that continuous infill masonry walls can reduce the vulnerability of the reinforced concrete structure. Often masonry walls are not considered in the design process because they are supposed to act as non-structural members or elements. Separately the infill walls are stiff and brittle but the frame is relatively flexible and ductile. The composite action of beam-column and infill walls provides additional strength and stiffness. The Fig. 1.4 shows the equivalent diagonal strut model for the infilled frame.

The section of the equivalent pin-jointed strut can be identified by imposing the condition that the initial stiffness of the actual system is equal to the initial stiffness of the braced frame. The equivalent strut method is convenient for modelling the infill walls in the building. The elastic analysis based (Smith and Carter, 1969), the plastic analysis based (Liauw and Kwan, 1983), and the ultimate load based (Saneinejad and Hobbs, 1995) approaches are among them. These approaches aim at calculating the geometric properties and strength of an equivalent strut.

1.2 NEED FOR THE PRESENT STUDY

As experienced by the engineers at design offices the multiplication factor of 2.5 given by IS 1893:2002, for ground storey beams and columns, is not realistic for low rise buildings. This calls for a critical assessment and review of the code recommended multiplication factor. Assessment of the multiplication factor (MF) requires accurate analysis of OGS buildings considering infill stiffness and strength. The presence of infill walls in upper storeys of OGS buildings accounts for the following issues:

- Increases the lateral stiffness of the building frame
- Decreases the natural period of vibration

- Increases the base shear
- Increases the shear forces and bending moments in the ground storey columns.

There is a clear need to assess the design guidelines recommended by the IS code 1893:2002 based on accurate analysis.

1.3 OBJECTIVE

Based on the literature review presented in Chapter 2, the salient objectives of the present study have been identified as follows:

i) To study the effect of infill strength and stiffness in the seismic analysis of OGS buildings.

ii) To check the applicability of the multiplication factor of 2.5 as given in the Indian Standard IS 1893:2002 for design of low rise open ground storey building.

iii) To assess the effect of support condition on the seismic behaviour of OGS buildings.

1.4 SCOPE OF THE STUDY

Open ground storey (OGS) buildings are commonly constructed in populated countries like India since they provide much needed parking space in an urban environment. Failures observed in past earthquakes show that the collapse of such buildings is predominantly due to the formation of soft-storey mechanism in the ground storey columns.

- This study deals with two different types of support conditions commonly used in analysis and design i.e., fixed and pinned end support condition. All other types of support conditions are not considered in this project. Soil-structure interaction is ignored for the present study.
- Number of storey and number of bays in two orthogonal horizontal directions may have a great effect on the lateral load resisting behaviour of OGS buildings. However, the conclusions drawn in the present study are based on a case study of a low-rise building (4 storeys).
- It is assumed in the present study that infill panels are having no window and door openings while modelling the infill walls.
- Point plastic flexural hinges only is considered for modelling the frame elements as the building is designed as per current design codes of practices and it is assumed no shear failure will precede the flexural failure.
- In the present study building models are analyzed only using linear static, dynamic analysis and nonlinear static (pushover) analysis. Although nonlinear dynamic analysis is superior to other analysis procedures, it is kept outside the scope of the present study due to time limitation.

1.5 METHODOLOGY

The methodology worked out to achieve the above-mentioned objectives is as follows:

(i) Review the existing literature and Indian design code provision for designing the OGS building

(ii) Select an existing building model for the case study.

(iii) Model the selected building with and without considering infill strength/ stiffness. Models need to consider two types of end support conditions as mentioned above.

(iv) Linear analysis of the selected building model and a comparative study on the results obtained from the analyses.

(v) Nonlinear analysis of the selected building model and a comparative study on the results obtained from the analyses.

(vi) Observations of results and discussions

1.6 ORGANIZATION OF THESIS

This introductory chapter (Chapter 1) gives a brief introduction to the importance of the seismic evaluation of OGS buildings and the reason why they are adopted by the designers in spite of the fact that they are more vulnerable during earthquake. A literature survey on behaviour of OGS buildings and infill walls during earthquake, have been presented in this chapter. The need, objectives and scope of the proposed research work are identified along with the methodology that is followed to carry out the work.

Chapter 2 presents the description of the selected building and the structural modelling parameters and modelling of infill walls. This chapter also describes the procedures and important parameters to model the nonlinear point plastic hinges.

Results obtained from linear analyses of the building model considering various cases are presented in Chapter 3. This chapter critically evaluate the linear analysis results to compare the building responses with and without considering infill strength/ stiffness

Nonlinear analysis is an important tool to correctly evaluate the seismic performance of a building. Nonlinear static (pushover) analysis of the selected building model is carried out as part of this project and the corresponding results are presented in Chapter 4.

Finally, in Chapter 5, the summary and conclusions are given. The scope for future work is also discussed.

CHAPTER 2

REVIEW OF LITERATURE

2.1 OVERVIEW

A state of the art literature review is carried out as part of the present study. This chapter presents a brief summary of the literature review. The literature review is divided into two parts. The first part deals with the seismic behaviour of the open ground storey buildings whereas the second part of this chapter discusses about the previous work carried out on the linear and nonlinear modelling of infill walls.

2.2 SEISMIC BEHAVIOUR OF OPEN GROUND STOREY BUILDING

Under lateral loading the frame and the infill wall stay intact initially. As the lateral load increases the infill wall get separated from the surrounding frame at the unloaded (tension) corner, but at the compression corners the infill walls are still intact. The length over which the infill wall and the frame are intact is called the length of contact. Load transfer occurs through an imaginary diagonal which acts like a compression strut. Due to this behaviour of infill wall, they can be modelled as an equivalent diagonal strut connecting the two compressive corners diagonally. The stiffness property should be such that the strut is active only when subjected to compression. Thus, under lateral loading only one diagonal will be operational at a time. This concept was first put forward by **Holmes** (1961).

The effect of slip and interface friction between the frame and infill wall was investigated by **Mallick and Severn** (1967) using finite element analysis. The infill panels were simulated by means of linear elastic rectangular finite elements, with two degrees of freedom at each of the four corner nodes. Interface between frame and infill was modelled and contact length was calculated. The slip between frame and infill was taken into account by considering frictional shear forces in the contact region using link element. Each node of this element has two translational degrees of freedom. The element is able to transfer compressive and bond forces, but incapable of resisting tensile forces.

Rao *et. al.* (1982) conducted theoretical and experimental studies on infilled frames with opening strengthened by lintel beams. It was concluded that the lintel over the opening does not have any influence on the lateral stiffness of an infilled frame. **Karisiddappa** (1986) and **Rahman** (1988) examined the effect of openings and their location on the behaviour of single storey RC frames with brick infill walls.

There are many studies on infilled frames under cyclic and dynamic loading condition. **Choubey and Sinha** (1994) investigated the effect of various parameters such as separation of infill wall from frame, plastic deformation, stiffness and energy dissipation of infilled frames under cyclic loading.

The behaviour of RC framed OGS building when subjected to seismic loads was reported by **Arlekar** *et.al* (1997). A four storeyed OGS building was analysed using Equivalent Static Analysis and Response Spectrum Analysis to find the resultant forces and displacements. This paper shows that the behaviour of OGS frame is quite different from that of the bare frame.

The effect of different parameters such as plan aspect ratio, relative stiffness, and number of bays on the behaviour of infilled frame was studied by **Riddington and Smith** (1997).

Scarlet (1997) studied the qualification of seismic forces in OGS buildings. A multiplication factor for base shear for OGS building was proposed. This procedure requires modelling the stiffness of the infill walls in the analysis. The study proposed a multiplication factor ranging from 1.86 to 3.28 as the number of storey increases from six to twenty.

Deodhar and Patel (1998) pointed out that even though the brick masonry in infilled frame are intended to be non-structural, they can have considerable influence on the lateral response of the building.

Davis and Menon (2004) concluded that the presence of masonry infill panels modifies the structural force distribution significantly in an OGS building. The total storey shear force increases as the stiffness of the building increases in the presence of masonry infill at the upper floor of the building. Also, the bending moments in the ground floor columns increase (more than two fold), and the mode of failure is by soft storey mechanism (formation of hinges in ground floor columns).

Das and Murthy (2004) concluded that infill walls, when present in a structure, generally bring down the damage suffered by the RC framed members of a fully infilled frame during earthquake shaking. The columns, beams and infill walls of lower stories are more vulnerable to damage than those in upper stories.

Asokan (2006) studied how the presence of masonry infill walls in the frames of a building changes the lateral stiffness and strength of the structure. This research proposed a plastic hinge model for infill wall to be used in nonlinear performance based analysis of a building and concludes that the ultimate load (UL) approach along with the proposed hinge property provides a better estimate of the inelastic drift of the building.

Hashmi and Madan (2008) conducted non-linear time history and pushover analysis of OGS buildings. The study concludes that the MF prescribed by IS 1893(2002) for such buildings is adequate for preventing collapse.

Sattar and Abbie (2010) in their study concluded that the pushover analysis showed an increase in initial stiffness, strength, and energy dissipation of the infilled frame, compared to the bare frame, despite the wall's brittle failure modes. Likewise, dynamic analysis results indicated that fully-infilled frame has the lowest collapse risk and the bare frames were found to be the most vulnerable to earthquake-induced collapse. The better collapse performance of fully-infilled frames was associated with the larger strength and energy dissipation of the system, associated with the added walls.

There are numerous research efforts found on the seismic behaviour of OGS buildings and on the modelling infill walls for linear and nonlinear analysis. However, no published literature found on the design criterion given in IS 1893:2002 (Part-1) for OGS low rise buildings. This is the primary motivation behind the present study.

2.3 MODELLING OF INFILL WALL

Most of the previous research model infill wall as an equivalent diagonal strut. This section summarises different approaches to model infill was as equivalent struts. Basically there are four approaches to model the equivalent strut found in literature. These approaches are explained below:

2.3.1 Elastic Analysis Approach

The modelling of infill wall as an equivalent diagonal compression member was introduced by Holmes (1961). The thickness of the equivalent diagonal strut was recommended as the thickness of the infill wall itself, and the width recommended as one-third of the diagonal length of infill panel.

The width of the strut using Airy's stress function was found to vary from d/4 to d/11 depending on the panel proportions. Later, a number of tests conducted by Smith (1966) proved that the equivalent strut width (w) is a function of relative stiffness (λh) of the frame and infill wall, strength of equivalent corner crushing mode of failure (R_c) and instantaneous diagonal compression in the infill wall (R_i).

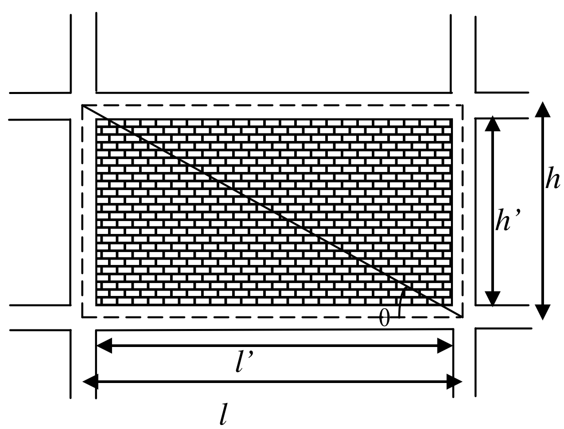


Fig. 2.1: A typical panel of the infilled frame

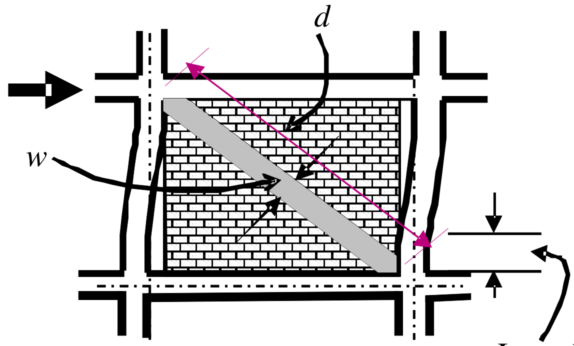


Fig. 2.2: Behaviour of typical panel subjected to lateral load

In 1969, Smith and Carter combined all the previous works (Smith 1962, 1966) and developed an analysis approach based on the equivalent strut concept to predict the width and strength of an infilled frame. This approach of modelling the struts is based on the initial stiffness of the infill wall. Fig 2.1 and 2.2 shows how the infill panels behave when it is designed as equivalent diagonal strut when subjected to lateral load.

Smith and Carter (1969) expressed the parameter, λh, as follows

$$\lambda h = \sqrt[4]{\frac{E_s t \sin 2\theta}{4 E_c I_c h'}} \qquad (2.1)$$

Where,

E_s = elastic modulus of the equivalent strut

E_c = elastic modulus of the column in the bounding frame

I_c = moment of inertia of the column

h' = clear height of infill wall (Fig. 2.1)

h = height of column between centrelines of beams

t = thickness of infill wall

θ = slope of the infill wall diagonal to the horizontal

A relationship between the ratio of axial load in the equivalent strut (R_i) to the capacity of the strut under corner crushing (R_c), and width (w) was derived by Ramesh (2003) from the plot given by Smith and Carter (1969), as given by

$$\frac{w}{w'} = 1.477 + 0.0356h - 0.912(R_i/R_c) \tag{2.2}$$

The parameter w' accounts for the panel aspect ratio. An expression for w'/d is as given:

$$\frac{w'}{d} = \frac{0.43 sin\, 2\theta}{\sqrt{\lambda h}} \tag{2.3}$$

The strength of the equivalent strut is taken as the minimum of the two failure modes, i.e.

(i) Local crushing (R_c) of infills in the corners

(ii) Shear cracking (R_s) along the bed joint of the brickwork.

The failure load corresponding to corner crushing mode was expressed in terms of λh as:

$$\frac{R_c}{f_m'ht} = \frac{\pi}{2\lambda h} sec\theta \tag{2.4}$$

Where f_m' is the compressive strength of the masonry infill wall.

The following relationship was proposed for the diagonal load causing shear cracking failure (R_s) by Govindan *et. al.* (1987), using the curves given by Smith and Carter, 1969

$$\frac{R_s}{f_{bs}'ht} = 1.65(l'/h')^{0.6}\,(\lambda h) - 0.05^{(l'/h')^{0.5}} \tag{2.5}$$

Where f_{bs}' is the bond shear strength between the masonry and mortar

Another equation by Mainstone for the determination of equivalent strut width is

$$\frac{w}{d'} = 0.175(\lambda h)^{-0.4} \tag{2.6}$$

Where d' = is the clear diagonal length of the infill walls. This expression yields a constant strut width, independent of parameters such as axial load on the diagonal strut and infill wall panel aspect ratio.

Paulay and Priestley (1992) suggested that the width of the strut can be taken as $1/4^{th}$ of the diagonal length of the infill panel.

Al-Chaar (2002) proposed an eccentric equivalent strut (Fig.2.3) which was pin connected to the column at a distance l_e from the face of the beam to model the masonry infill wall.

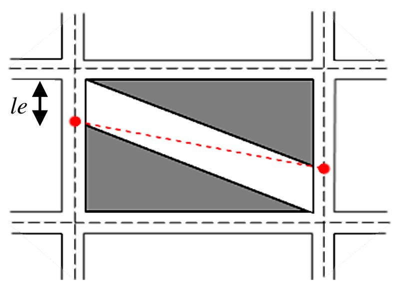


Fig. 2.3: Position of eccentric strut (Al-Chaar, 2002)

Where $l_e = \dfrac{w}{\cos\theta}$ and w is calculated using eq. 2.6

2.3.2 Ultimate Load Approach

Saneinejad and Hobbs (1995) proposed a new model that accounts for the interface stresses and the nonlinear inelastic behaviour of the infill wall. The area of the equivalent strut is calculated from the diagonal load at failure. This approach is based on ultimate strength of the equivalent strut and the strength of the strut is calculated from the three modes of failure: 1) Corner crushing failure at the compressive corners

2) Shear cracking failure along the bedding joints of the brick work

3) Diagonal compression failure of the slender infill wall

The applicability of the two approaches stated above for different types of building analysis was investigated. The calculation of the strut properties by both the approaches was presented through a case study by Asokan (2006) and the justification of using either of the methods was presented. He selected a two bay frame from an existing five storey building which was infilled in the entire four storeys except for the ground floor. The beams and column frames were of same size. The infill wall thickness was 120 mm and he from his study concluded that the EA approach is simple in calculation. A higher strut width gives higher stiffness and hence, higher base shear in a building. Since the EA approach gives higher strut width, it is conservative in estimating the base shear. For estimating the lateral drift of a building, since the UL approach gives lower stiffness of a strut, it is more conservative. To carry out a linear analysis of building by the equivalent static method (static analysis) or the response spectrum method (dynamic analysis), modelling of the infill walls by the simpler EA approach would prove to be adequate. But in a pushover analysis (nonlinear static analysis) of a building, the UL approach would be preferred.

2.3.3 Approach Based on Plastic Analysis

Experimental results (Smith 1962) show that there is a considerable nonlinearity in the infilled frames before their collapse. The nonlinearity arises mainly from cracking and crushing of the infill wall material, confinement of the infill walls in the frames, and formation of plastic hinges in the frame members. In the elastic stage, stress concentration occurs at all four corners. As cracks develop and propagate, the stresses at the tensile corners are relieved while those near the compressive corners are significantly increased. The frame moments increase significantly when the infill wall degrades leading to the formation of plastic hinges and collapse of the structure.

A plastic theory was developed for integral and non-integral (without shear connectors) infilled frames by Liauw and Kwan (1983). The stress redistribution in the frames towards collapse was taken into account and the friction was neglected for strength reserve for the non-integral infilled frames. The theory was based on the findings from nonlinear finite element analysis and experimental investigation. The local crushing of the infill wall corner is associated with a plastic hinge formation either in the beam or in the column. The following modes of failure were identified.

- Corner crushing mode with failure in columns: This mode of failure is associated with weak columns and strong infill wall. Failure occurs in the columns with subsequent crushing of the infill wall at the compressive corners.
- Corner crushing mode with failure in beams: This mode of failure predominates when beam is relatively weak and the infill wall is strong. Failure occurs in beam after the failure of the infill wall at the compressive corners.
- Diagonal crushing mode: With relatively strong frame and weak infill wall, failure occurs in the infill wall by crushing at the loaded corners with subsequent failure in the joints of the frame.

Based on plastic theory, following are the mathematical expressions were developed (Eq. 2.7) for the above modes of failure.

1. For failure mode 1

$$\frac{H_u}{\sigma_c th} = \sqrt{\frac{2(M_{pj} + M_{pc})}{\sigma_c th^2}} \qquad (2.7a)$$

2. For failure mode 2

$$\frac{H_u}{\sigma_c th} = \frac{1}{tan\theta}\sqrt{\frac{2(M_{pj} + M_{pb})}{\sigma_c th^2}} \quad (2.7b)$$

3. For failure mode 3

$$\frac{H_u}{\sigma_c th} = \frac{4M_{pj}}{\sigma_c th^2} + \frac{1}{6} \quad (2.7c)$$

Where

H_u = lateral load causing the failure

M_{pc} = the plastic moment of resistance of the column

M_{pb} = the plastic moment of resistance of the beam

σ_c = contact stresses in the column

2.3.4 Approach Based on Finite Element Analysis

Finite element analysis was done by many researchers to study the behaviour of the infill wall under lateral load. The different parameters influencing the infill walls under lateral loads were investigated.

A finite element model was developed by Mallick and Severn (1967) to incorporate the effect of slip and interface friction between the frame and infill wall. Riddington and Smith (1977) studied the effect of different parameters such as aspect ratio, relative stiffness parameter, number of bays and beam stiffness. It was found that the bending moments in the frame members were reduced in the presence of the infill wall. Hence, the infilled frame can be modelled as truss elements.

Dhanasekar and Page (1986) developed a finite element program and concluded that the behaviour of a frame not only depends on the relative stiffness of frame and infill wall but also on the properties of masonry, such as shear and tensile bond strengths.

2.4 SUMMARY

This Chapter discusses briefly the previous work done on the area of seismic behaviour of open ground storey RC buildings and modelling of infill walls as equivalent diagonal strut. From these published work it can be concluded that that even though the brick masonry in infilled frame are intended to be non-structural, they can have considerable influence on the lateral response of the building. Multiplication factor to increase the design forces of ground storey columns and beams of OGS buildings is a function of storey numbers. IS 1893:2002 (Part-1) proposal for multiplication factor of 2.5 may not be appropriate for low rise building. The four different approaches namely (a) Elastic analysis approach (b) Ultimate load approach (c) Plastic analysis approach and (d) approach based on Finite element analysis, to model the infill walls is described in detail in this chapter.

CHAPTER 3

STRUCTURAL MODELLING

3.1 OVERVIEW

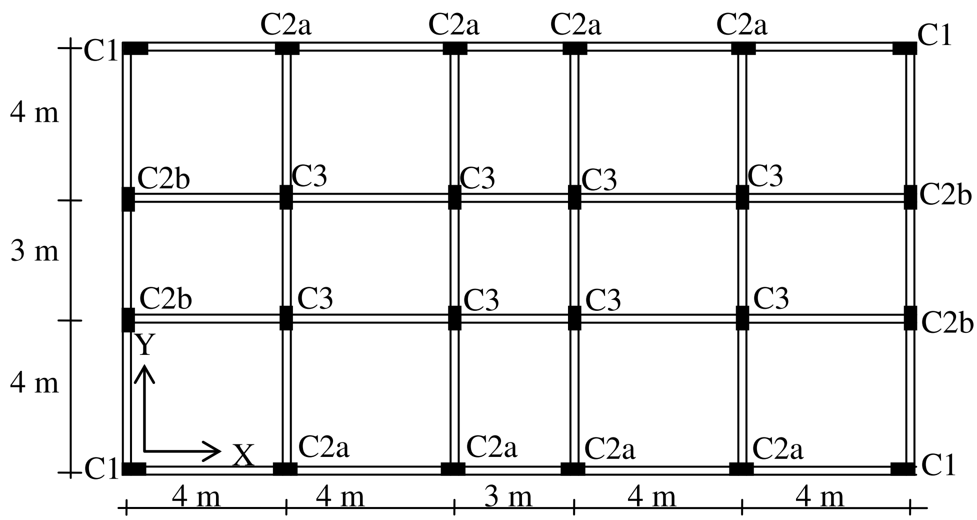
It is very important to develop a computational model on which linear / non-linear, static/ dynamic analysis is performed. The first part of this chapter presents a summary of various parameters defining the computational models, the basic assumptions and the geometry of the selected building considered for this study.

Accurate modelling of the nonlinear properties of various structural elements is very important in nonlinear analysis. In the present study, frame elements were modelled with inelastic flexural hinges using point plastic model. A detailed description on the nonlinear modelling of RC frames is presented in this chapter.

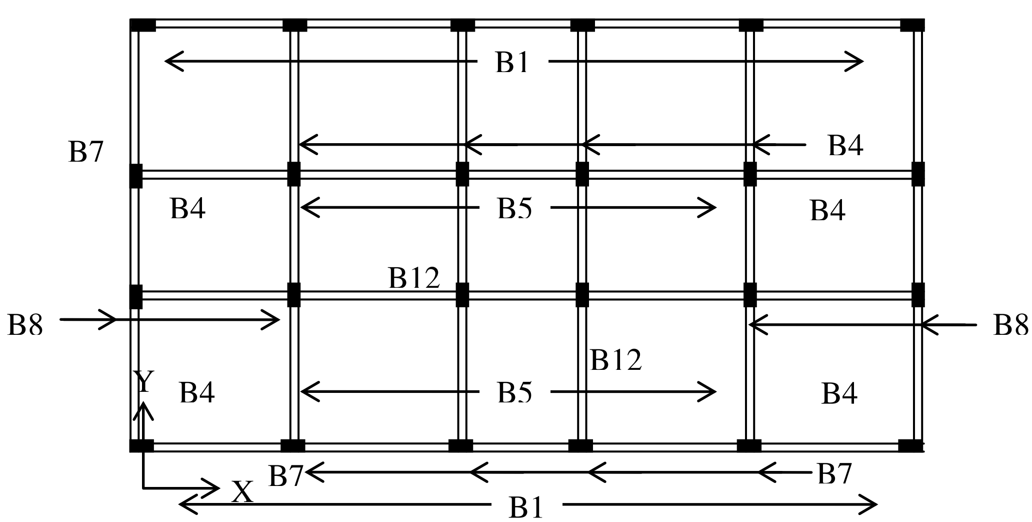
Infill walls are modelled as equivalent diagonal strut elements. The last part of the chapter deals with the computational model of the equivalent strut including modelling nonlinearity.

3.2 BUILDING DESCRIPTION

An existing OGS framed building located at Guwahati, India (Seismic Zone V) is selected for the present study. The building is fairly symmetric in plan and in elevation. This building is a G+3 storey building (12m high) and is made of Reinforced Concrete (RC) Ordinary Moment Resisting Frames (OMRF). The concrete slab is 150mm thick at each floor level. The brick wall thicknesses are 230 mm for external walls and 120 mm for internal walls. Imposed load is taken as 2 kN/ m^2 for all floors. Fig. 3.1 presents typical floor plans showing different column and beam locations. The cross sections of the structural members (columns and beams 300 mm×600 mm) are equal in all frames and all stories. Storey masses to 295 and 237 tonnes in the bottom storyes and at the roof level, respectively. The design base shear was equal to 0.15 times the total weight.



(a) Column Locations



(b) Beam Locations

Fig. 3.1: Typical floor plan of the selected building

The amount of longitudinal reinforcement in the columns and beams is given in Table 3.1. Although the columns have equal reinforcement in all storey levels beam reinforcement in floor and roof are different. Refer Fig. 3.1 (a) and (b) for column and beam identification (ID).

Table 3.1: Longitudinal reinforcement details of frame sections

Column ID	Longitudinal Reinforcement	Beam ID	Top steel	Bottom steel
C1	12Y16	B1	4Y16	3Y16
C2(a)	8Y20	B4	3Y16	2Y16
C2(b)	8Y20	B5	2Y16, 1Y12	2Y16
C3	8Y16	B7	3Y16	3Y16
		B8	3Y16	3Y16
		B12	3Y16	2Y16, 1Y12
		Roof Beams	2Y16	2Y16

3.3 STRUCTURAL MODELLING

Modelling a building involves the modelling and assemblage of its various load-carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modelling of the material properties and structural elements used in the present study is discussed below.

3.3.1 Material Properties

M-20 grade of concrete and Fe-415 grade of reinforcing steel are used for all the frame models used in this study. Elastic material properties of these materials are taken as per Indian Standard IS 456: 2000. The short-term modulus of elasticity (E_c) of concrete is taken as:

$$E_c = 5000\sqrt{f_{ck}}\,MPa \tag{3.1}$$

f_{ck} is the characteristic compressive strength of concrete cube in MPa at 28-day (20 MPa in this case). For the steel rebar, yield stress (f_y) and modulus of elasticity (E_s) is taken as per IS 456:2000. The material chosen for the infill walls was masonry whose compressive strength (f_m') from the literature was found out to be 1.5 MPa and the modulus of elasticity was stated as:

E_m = 350 to 800 MPa for table moulded brick

= 2500 to 5000 MPa for wire cut brick

According to FEMA 356:2000 elasticity of modulus of brick is taken as E_m = 750 f_m'. For the present study the modulus of elasticity of the masonry is taken as given in literature by Asokan (2006).

3.3.2 Structural Elements

Beams and columns are modelled by 3D frame elements. The beam-column joints are modelled by giving end-offsets to the frame elements, to obtain the bending moments and forces at the beam and column faces. The beam-column joints are assumed to be rigid.

Beams and columns in the present study were modelled as frame elements with the centrelines joined at nodes using commercial software SAP2000NL. The rigid beam-column joints were modelled by using end offsets at the joints (Fig. 3.2). The floor slabs were assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was distributed as triangular and trapezoidal load to the surrounding beams.

The structural effect of slabs due to their in-plane stiffness is taken into account by assigning 'diaphragm' action at each floor level. The mass/weight contribution of slab is modelled separately on the supporting beams.

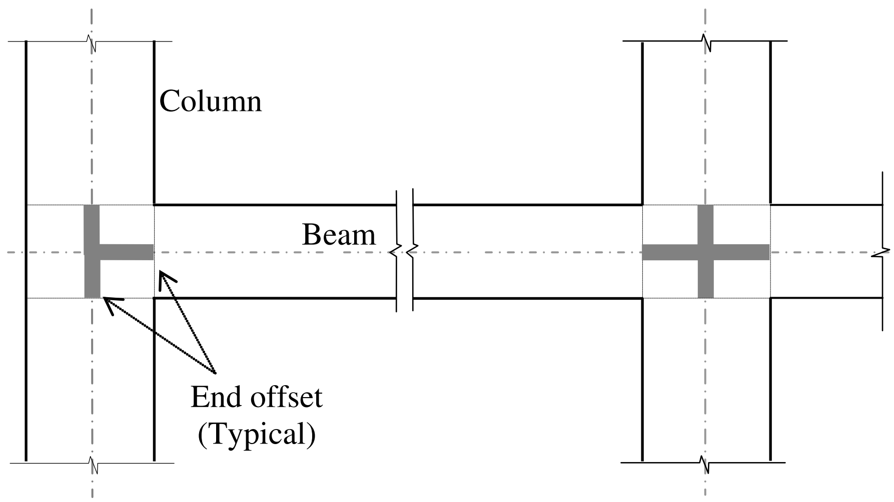


Fig. 3.2: Use of end offsets at beam-column joint

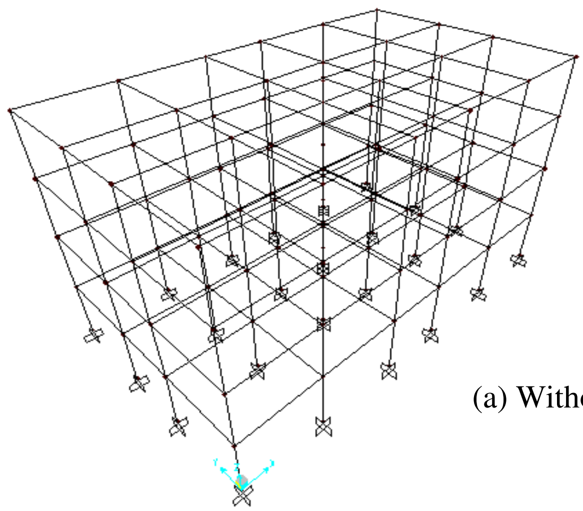
3.3.3 Modelling of Column Ends at the Foundation

The selected building is supported on a raft foundation. Therefore, the column ends are modelled as fixed at the top of the raft and analysed. To study how the response of the building changes with the support conditions, the same building model also analysed by providing a hinge in place of fixity.

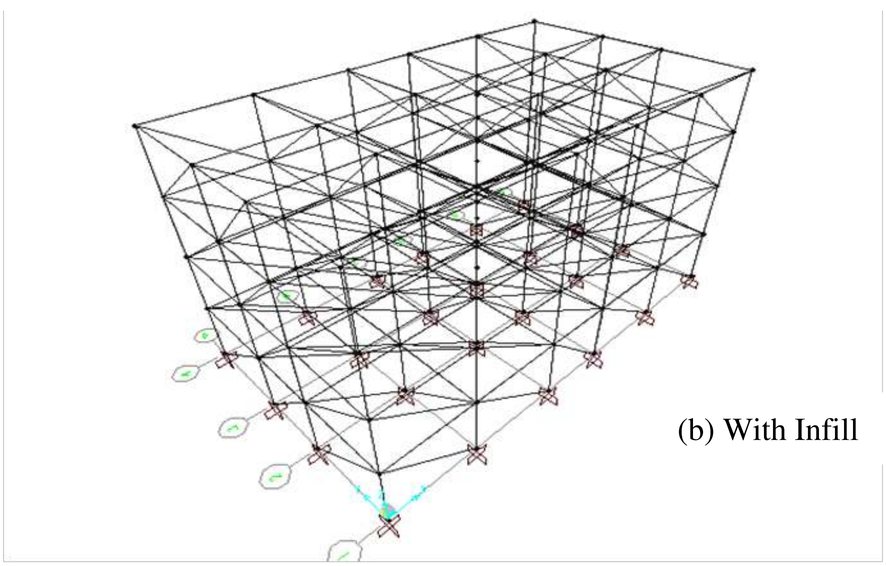
3.3.4 Modelling Infill Walls

Infill walls are two dimensional elements that can be modelled with orthotropic plate element for linear analysis of buildings with infill wall. But the nonlinear modelling of a two dimensional plate element is not understood well. Therefore infill wall has to be modelled with a one-dimensional line element for nonlinear analysis of the buildings. Same building model with infill walls modelled as one-dimensional line element is used in the present study for both linear and nonlinear analyses. Infill walls are modelled here as equivalent diagonal strut elements. Section 3.5 explains the modelling of infill was as diagonal strut in detail.

Fig. 3.3 presents a three-dimensional computer model of building without and with considering infill stiffness.



(a) Without Infill



(b) With Infill

Fig. 3.3: 3D Computer model of building without and with considering infill stiffness respectively.

3.4 MODELLING OF FLEXURAL PLASTIC HINGES

In the implementation of pushover analysis, the model must account for the nonlinear behaviour of the structural elements. In the present study, a point-plasticity approach is considered for modelling nonlinearity, wherein the plastic hinge is assumed to be concentrated at a specific point in the frame member under consideration. Beam and column elements in this study were modelled with flexure (M3 for beams and P-M2-M3 for columns) hinges at possible plastic regions under lateral load (i.e., both ends of the beams and columns). Refer Fig. 3.4 for the local axis system considered. Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load. In the present study the plastic hinge properties are calculated by SAP 2000. The analytical procedure used to model the flexural plastic hinges are explained below.

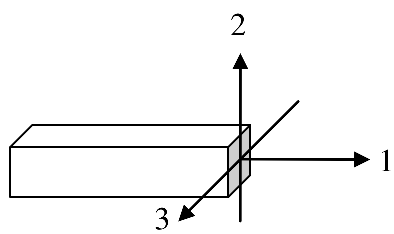


Fig. 3.4: The coordinate system used to define the flexural and shear hinges

Flexural hinges in this study are defined by moment-rotation curves calculated based on the cross-section and reinforcement details at the possible hinge locations. For calculating hinge properties it is required to carry out moment–curvature analysis of each element. Constitutive relations for concrete and reinforcing steel, plastic hinge length in structural element are required for this purpose. The flexural hinges in beams are modelled with uncoupled moment (M3) hinges whereas for column elements the flexural hinges are modelled with coupled P-M2-M3 properties that include the interaction of axial force and bi-axial bending moments at the hinge location. Although the axial force interaction is considered for column flexural hinges the rotation values were considered only for axial force associated with gravity load.

3.4.1 Stress-Strain Characteristics for Concrete

The stress-strain curve of concrete in compression forms the basis for analysis of any reinforced concrete section. The characteristic and design stress-strain curves specified in most of design codes (IS 456: 2000, BS 8110) do not truly reflect the actual stress-strain behaviour in the post-peak region, as (for convenience in calculations) it assumes a constant stress in this region (strains between 0.002 and 0.0035). In reality, as evidenced by experimental testing, the post-peak behaviour is characterised by a descending branch, which is attributed to 'softening' and micro-cracking in the concrete. Also, models as per these codes do not account for strength enhancement and ductility due to confinement. However, the stress-strain relation specified in ACI 318M-02 consider some of the important features from actual behaviour. A previous study (Chugh, 2004) on stress-strain relation of reinforced concrete section concludes that the model proposed by Panagiotakos and Fardis (2001) represents the actual behaviour best for normal-strength concrete. Accordingly, this model has been selected in the present study for calculating the hinge properties. This model is a modified version of Mander's model (Mander*et. al.*,

1988) where a single equation can generate the stress f_c corresponding to any given strainε_c:

$$f_c = \frac{f_{cc}^{'} x r}{r - 1 + x^r} \tag{3.2}$$

where, $x = \frac{\epsilon_c}{\epsilon_{cc}}$; $r = \frac{E_c}{E_c - E_{sec}}$; $E_c = 5000\sqrt{f_{co}^{'}}$; $E_{sec} = \frac{f_{cc}^{'}}{\varepsilon_{cc}}$ and $f^{'}_{cc}$ is the peak strength expressed as follows:

$$f_{cc}^{'} = f_{co}^{'} \left[1 + 3.7 \left(\frac{0.5 k_e \rho_s f_y h}{f_{co}^{'}} \right)^{0.85} \right] \tag{3.3}$$

The expressions for critical compressive strains are expressed in this model as follows:

$$\varepsilon_{cu} = 0.004 + \frac{0.6 \rho_s f_{yh} \epsilon_{sm}}{f_{cc}^{'}} \tag{3.4}$$

$$\varepsilon_{cc} = \epsilon_{co} \left[1 + 5 \left(\frac{f_{cc}^{'}}{f_{co}^{'}} - 1 \right) \right] \tag{3.5}$$

The unconfined compressive strength ($f^{'}_{co}$) is $0.75 f_{ck}$, k_e having a typical value of 0.95 for circular sections and 0.75 for rectangular sections.

Fig. 3.5 shows a typical plot of stress-strain characteristics for M-20 grade of concrete as per Modified Mander’s model (Panagiotakos and Fardis, 2001). The advantage of using this model can be summarized as follows:

- A single equation defines the stress-strain curve (both the ascending and descending branches) in this model.
- The same equation can be used for confined as well as unconfined concrete sections.

- The model can be applied to any shape of concrete member section confined by any kind of transverse reinforcement (spirals, cross ties, circular or rectangular hoops).
- The validation of this model is established in many literatures (*e.g.*, Pam and Ho, 2001).

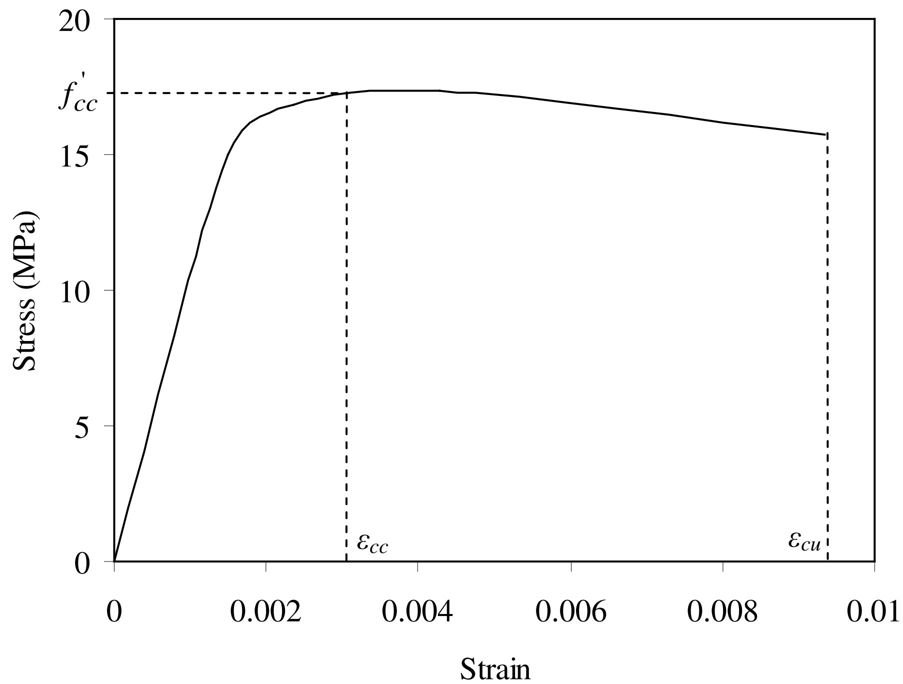


Fig. 3.5: Typical stress-strain curve for M-20 grade concrete (Panagiotakos and Fardis, 2001)

3.4.2 Stress-Strain Characteristics for Reinforcing Steel

The constitutive relation for reinforcing steel given in IS 456 (2000) is well accepted in literature and hence considered for the present study. The 'characteristic' and 'design' stress-strain curves specified by the Code for Fe-415 grade of reinforcing steel (in tension or compression) are shown in Fig. 3.6.

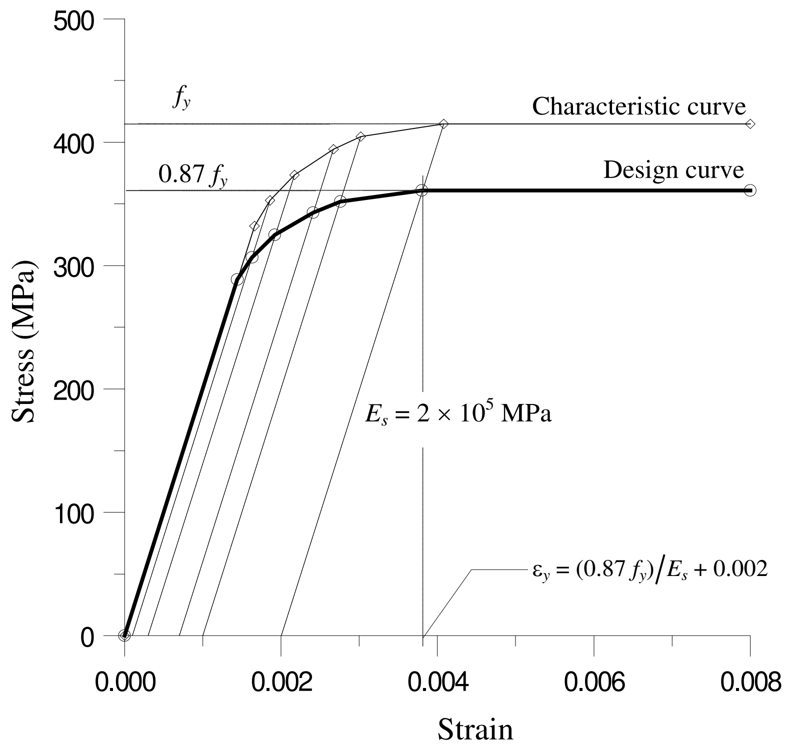


Fig. 3.6: Stress-strain relationship for reinforcement – IS 456 (2000)

3.4.3 Moment-Curvature Relationship

Moment-curvature relation is a basic tool in the calculation of deformations in flexural members. It has an important role to play in predicting the behaviour of reinforced concrete (RC) members under flexure. In nonlinear analysis, it is used to consider secondary effects and to model plastic hinge behaviour.

Curvature (φ) is defined as the reciprocal of the radius of curvature (R) at any point along a curved line. When an initial straight beam segment is subject to a uniform bending moment throughout its length, it is expected to bend into a segment of a circle with a curvature φ that increases in some manner with increase in the applied moment (M). Curvature φ may be alternatively defined as the angle change in the slope of the elastic curve per unit length $\left(\varphi = 1/R = d\theta/ds\right)$. At any section, using the 'plane sections remain plane' hypothesis under pure bending, the curvature can be computed as the ratio of the normal strain at any point across the depth to the distance measured from the neutral axis at that section (Fig. 3.7).

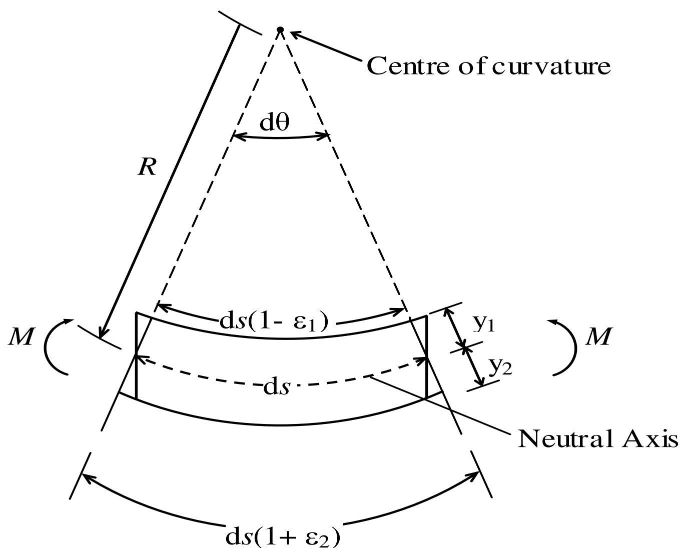


Fig. 3.7: Curvature in an initially straight beam section (Pillai and Menon, 2009)

If the bending produces extreme fibre strains of ε_1 and ε_2 at top and bottom at any section as shown in Fig. 3.7 (compression on top and tension at bottom assumed in this case), then, for small deformations, it can be shown that $\varphi = \left(\varepsilon_1 + \varepsilon_2\right)/D$. If the beam behaviour is linear elastic, then the moment-curvature relationship is linear, and the curvature is obtained as

$$\varphi = \frac{M}{EI} \tag{3.6}$$

The flexural rigidity (EI) of the beam is obtained as a product of the modulus of elasticity *E* and the second moment of area of the section *I*.

When a RC flexural member is subjected to a gradually increasing moment, it's behaviour transits through various stages, starting from the initial un-cracked state to the ultimate limit state of collapse. The stresses in the tension steel and concrete go on increasing as the moment increases. The behaviour at the ultimate limit state depends on the percentage of steel provided, i.e., on whether the section is 'under-reinforced' or 'over-reinforced'. In the case of under-reinforced sections, failure is triggered by yielding of tension steel whereas in over-reinforced section the steel does not yield at the limit state of failure. In both cases, the failure eventually occurs due to crushing of concrete at the extreme compression fibre, when the ultimate strain in concrete reaches its limit. Under-reinforced beams are characterised by 'ductile' failure, accompanied by large deflections and significant flexural cracking. On the other hand, over-reinforced beams have practically no ductility, and the failure occurs suddenly, without the warning signs of wide cracking and large deflections.

In the case of a short column subject to uniaxial bending combined with axial compression, it is assumed that Eq. 3.6 remains valid and that "plane sections before bending remain plane". However, the ultimate curvature (and hence, ductility) of the section is reduced as the compression strain in the concrete contributes to resisting axial compression in addition to flexural compression.

3.4.4 Modelling of Moment-Curvature in RC Sections

Using the Modified Mander model of stress-strain curves for concrete (Panagiotakos and

Fardis, 2001) and Indian Standard IS 456 (2000) stress-strain curve for reinforcing steel, for a specific confining steel, moment curvature relations can be generated for beams and columns (for different axial load levels). The assumptions and procedure used in generating the moment-curvature curves are outlined below.

Assumptions

i. The strain is linear across the depth of the section ('plane sections remain plane').

ii. The tensile strength of the concrete is ignored.

iii. The concrete spalls off at a strain of 0.0035.

iv. The initial tangent modulus of the concrete, E_c is adopted from IS 456 (2000), as $5000\sqrt{f_{ck}}$.

v. In determining the location of the neutral axis, convergence is assumed to be reached within an acceptable tolerance of 1%.

Algorithm for Generating Moment-Curvature Relation

i. Assign a value to the extreme concrete compressive fibre strain (normally starting with a very small value).

ii. Assume a value of neutral axis depth measured from the extreme concrete compressive fibre.

iii. Calculate the strain and the corresponding stress at the centroid of each longitudinal reinforcement bar.

iv. Determine the stress distribution in the concrete compressive region based on the Modified Mander stress-strain model for given volumetric ratio of confining steel. The resultant concrete compressive force is then obtained by numerical integration of the stress over the entire compressive region.

v. Calculate the axial force from the equilibrium and compare with the applied axial load (for beam element both of these will be zero). If the difference lies within the specified tolerance, the assumed neutral axis depth is adopted. The moment capacity and the corresponding curvature of the section are then calculated. Otherwise, a new neutral axis is determined from the iteration (using bisection method) and steps (iii) to (v) are repeated until it converges.

vi. Assign the next value, which is larger than the previous one, to the extreme concrete compressive strain and repeat steps (ii) to (v).

vii. Repeat the whole procedure until the complete moment-curvature is obtained.

3.4.5 Moment-Rotation Parameters

Moment-rotation parameters are the actual input for modelling the hinge properties and this can be calculated from the moment-curvature relation. This can be explained with a simple cantilever beam AB shown in Fig. 3.8(a) with a concentrated load applied at the free end B. To determine the rotation between the ends an idealized inelastic curvature distribution and a fully cracked section in the elastic region may be assumed. Figs 3.8(b) and 3.8(c) represent the bending moment diagram and probable distribution of curvature at the ultimate moment.

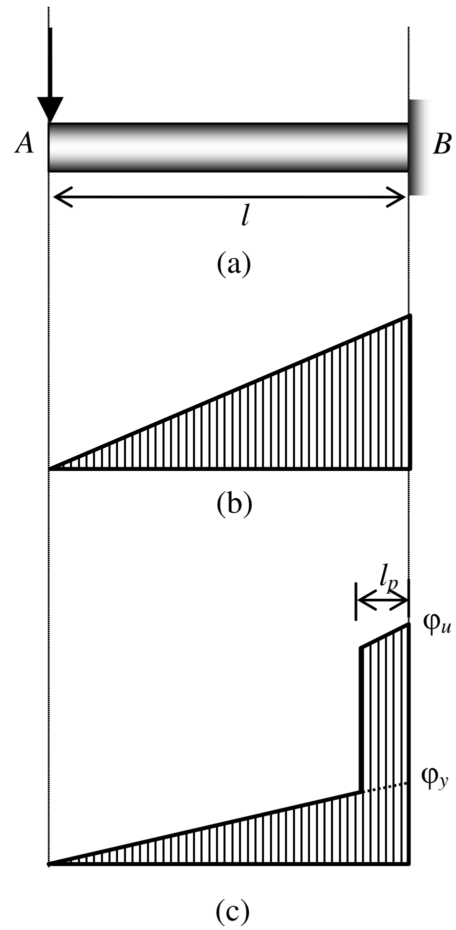


Fig. 3.8: (a) cantilever beam, (b) Bending moment distribution, and (c) Curvature distribution (Park and Paulay 1975)

The rotation between *A* and *B* is given by

$$\theta = \int_A^B \varphi \, dx \tag{3.7}$$

The ultimate rotation is given by,

$$\theta_u = \varphi_y \frac{1}{2} + (\varphi_u - \varphi_y) l_p \tag{3.8}$$

The yield rotation is,

$$\theta_y = \varphi_y \frac{1}{2} \tag{3.9}$$

And the plastic rotation is,

$$\theta_p = (\varphi_u - \varphi_y) l_p \tag{3.10}$$

l_p is equivalent length of plastic hinge over which plastic curvature is considered to be constant. The physical definition of the plastic hinge length, considering the ultimate flexural strength developing at the support, is the distance from the support over which the applied moment exceeds the yield moment. A good estimate of the effective plastic hinge length may be obtained from the following equation (Paulay and Priestley, 1992)

$$l_p = 0.08l + 0.15 d_b f_y \tag{3.11}$$

The yield strength of the longitudinal reinforcement should be in 'ksi'. For typical beam and column proportions Eq. 2.11 results in following equation (FEMA-274; Paulay and Priestley, 1992) where *D* is the overall depth of the section.

$$l_p = 0.5D \tag{3.12}$$

The moment-rotation curve can be idealised as shown in Fig. 3.9, and can be derived from the moment-curvature relation. The main points in the moment-rotation curve shown in the figure can be defined as follows:

- The point 'A' corresponds to the unloaded condition.
- The point 'B' corresponds to the nominal yield strength and yield rotation θ_y.

- The point 'C' corresponds to the ultimate strength and ultimate rotation θ_u, following which failure takes place.
- The point 'D' corresponds to the residual strength, if any, in the member. It is usually limited to 20% of the yield strength, and ultimate rotation, θ_u can be taken with that.
- The point 'E' defines the maximum deformation capacity and is taken as $15\theta_y$ or θ_u, whichever is greater.

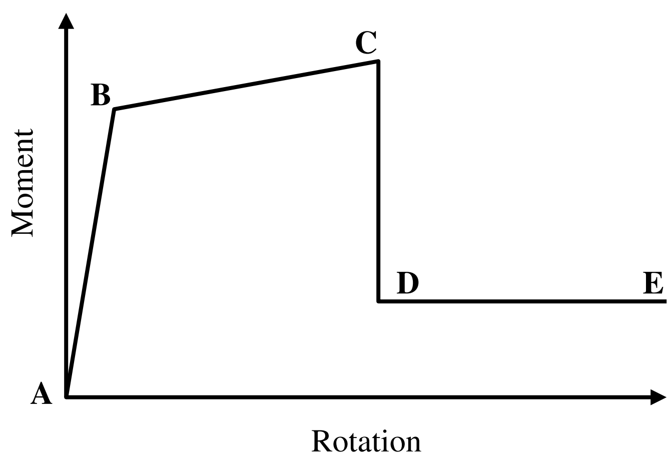


Fig. 3.9: Idealised moment-rotation curve of RC elements

While applying eqs. 3.9 and 3.10 to determine the ultimate and yield rotations, care must be taken to adopt the correct value of the length *l*, applicable for cantilever action. In the case of a frame member in a multi-storey frame subject to lateral loads, it may be conveniently assumed that the points of contra flexure are located (approximately) at the mid-points of the beams and columns. In such cases, an approximate value of *l* is given by half the span of the member under consideration.

3.5 MODELLING OF EQUIVALENT STRUT

For an infill wall located in a lateral load-resisting frame, the stiffness and strength contribution of the infill has to be considered. Non-integral infill walls subjected to lateral load behave like diagonal struts. Thus an infill wall can be modelled as an equivalent 'compression only' strut in the building model. Rigid joints connect the beams and columns, but pin joints connect the equivalent struts to the beam-to-column junctions. This section explains the procedure based on Smith and Carter (1969) to calculate the modelling parameters (effective width, elastic modulus and strength) of an equivalent strut. This method is elaborated in Section 2.3.1 of chapter 2.

The length of the strut is given by the diagonal distance (d) of the panel and its thickness is equal to the thickness of the infill wall. The elastic modulus of the strut is equated to the elastic modulus of masonry (E_m)

For the estimation of width (w) of the strut, a simple expression as given in Eq. 2.1 to Eq. 2.3 (Chapter 2) is adopted.

3.6 STRENGTH OF EQUIVALENT STRUT

The strength of the equivalent strut is governed by the lowest of the failure loads corresponding to the following failure modes.

a) Local crushing of the infill at one of the loaded corners.

b) Shear cracking along the bedding joints of the brickwork.

The diagonal tensile cracking need not be considered as a failure mode, as higher load can be carried beyond tensile cracking.

3.6.1 Local Crushing Failure

The diagonal load causing local crushing (R_c) is given by the following equation (Smith and Carter, 1969).

$$R_c = \alpha_c t sec\theta f_m \tag{3.13}$$

The length of contact at the column (α_c) at the compression diagonal corner is calculated using the following formula.

$$\frac{\alpha_c}{h} = \frac{\pi}{2\lambda h} \tag{3.14}$$

Other variables are as defined earlier.

3.6.2 Shear Failure

Following relationship of R_s proposed by Govindan (1986) using the curves given by Smith and Carter (1969) is chosen, as it is simple and non-dimensional.

$$\frac{R_S}{f'_{bs}\ ht} = 1.65(l'/h')^{0.6}\ (\lambda h)^{-0.05(l'/h')0.5} \tag{3.15}$$

Where, f'_{bs} = The bond shear strength between the masonry and mortar. It is varies from 0.24 MPa for low strength mortar to 0.69 MPa for high strength mortar (Ramesh 2003). Again to be in conservative side f'_{bs} is taken as 0.24 in the calculation.

3.7 ALGORITHM FOR GENERATING THE EQUIVALENT STRUT MODEL

Asokan (2006) conducted a comprehensive study of various existing approaches for equivalent diagonal strut modelling and developed a computer program based on the study by Smith and Carter (1969). This program is used to calculate the parameters of equivalent strut model in the present study.

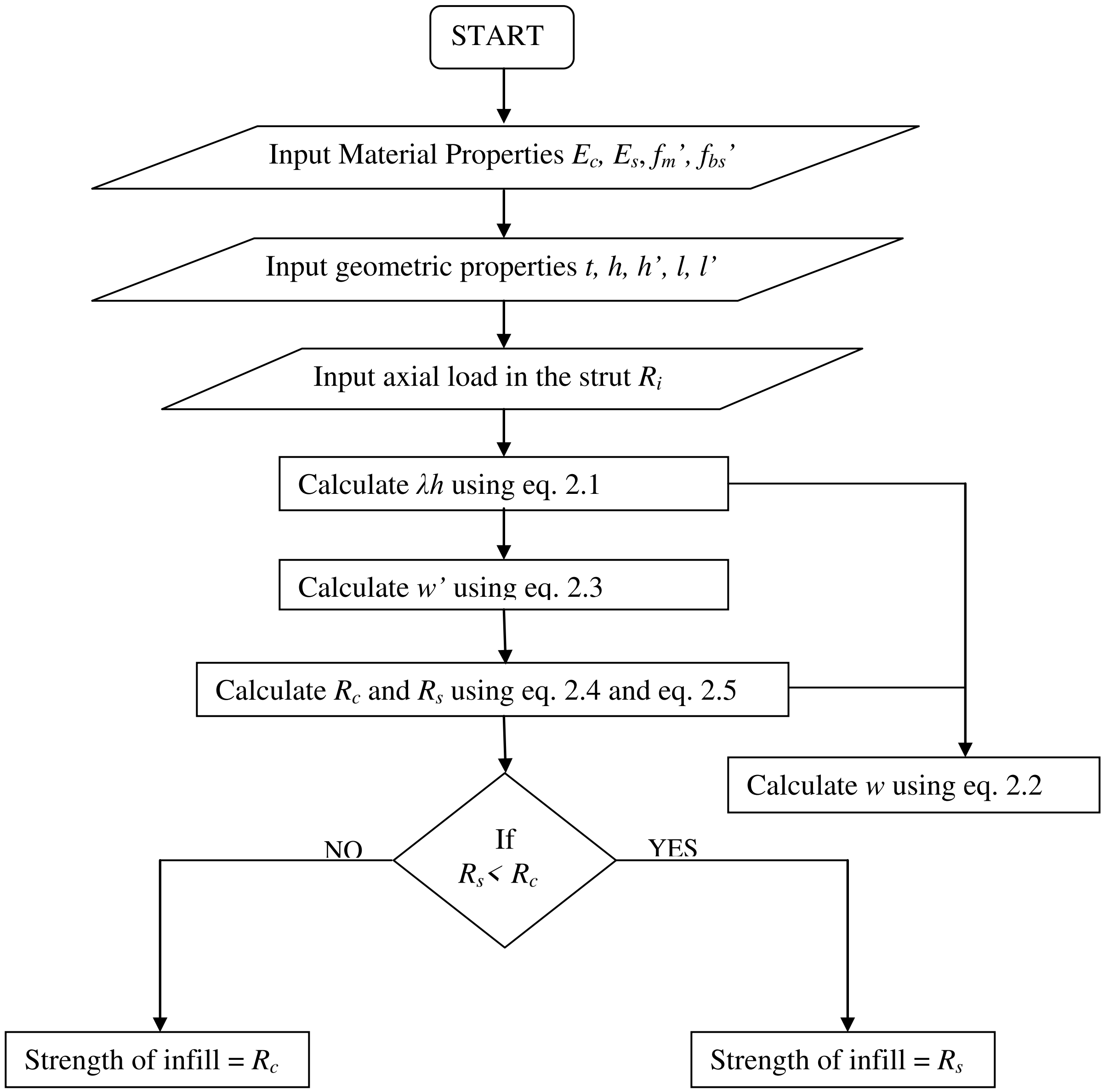


Fig. 3.10: Algorithm for the calculation of equivalent strut width as per Smith and Carter (1969)

The algorithm for calculating the strut width and strength is as given below

Step1. Specify material properties

Step2. Specify geometric properties

Step3. Calculate λh and w' using Eqs. 2.1 and 2.3

Step4. Calculate the failure load R_c and R_s of the equivalent strut using Eqs. 2.4 and 2.5

Step5. Calculate initial width of the strut using Eq. 2.2 assuming the axial load in the strut

$R_i = 0$

Step6. Calculate strength of the equivalent strut = minimum of R_c and R_s

Computational model of the building can be analysed using the obtained values of w and E_m for the struts. Revised value of w corresponding to the axial force R_i is obtained using Eq. 2.2. This procedure can be repeated to get a converged value of w. It is observed that two iterations are sufficient to get the converged value of w, Asokan (2006). The above steps in the form of the flow chart are as presented in Fig. 3.10.

3.8 MODELLING OF AXIAL HINGES FOR EQUIVALENT STRUTS

3.8.1 Nonlinearity of Axial Hinge Property

The nonlinearity in the infill wall is due to the formation and development of cracks under lateral load. As soon as a diagonal crack develops within an infill wall (usually at much lower load and deflection values than those at ultimate) the behaviour becomes nonlinear. The wall stays confined within the surrounding frame and bears against it over the contact lengths. The wall carries more loads until the existing crack continues to widen and new cracks appear, leading eventually to ultimate failure due to corner crushing or shear cracking. Even though the masonry and mortar are brittle in nature, the behaviour of infill wall is nonlinear (Asokan 2006). However for convenience brittle load deformation behaviour as per the elastic analysis approach is considered to model the axial hinges in equivalent strut. The following procedure presented in Section 3.8.2 is used to get the axial load versus deformation curve of equivalent strut.

3.8.2 Elastic Analysis Approach

The axial load versus deformation behaviour of the equivalent struts under compression can be modelled with axial hinges. In absence for data, an elastic behaviour up to the failure load can be assumed. Any tensile load carrying capacity of the strut is neglected. For a nonlinear analysis of a building such as pushover analysis, in addition to the strut width, modulus and strength, the axial load versus deformation curve is also required to define the axial hinge property of a strut. ATC 40 gives simplified expressions for the different hinge properties for different structural elements such as beams and columns. But for equivalent struts, there is a need to develop refined axial hinge properties. The axial hinge properties commonly used are explained in the following section. Fig. 3.11 shows a typical load-deformation relation for the axial hinge in strut. R and δ_y represent the failure load and the corresponding deformation, respectively, of the strut. In the linear model, it is assumed that the equivalent strut is elastic till the failure load.

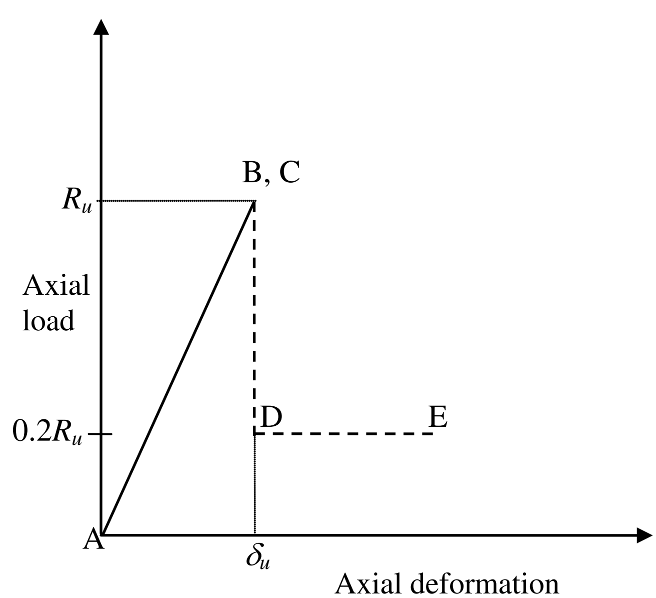


Fig. 3.11 Axial load versus deformation curve for equivalent strut

The failure load (R) is calculated from the lower of the failure loads corresponding to local crushing and shear cracking as described above.

The deformation corresponding to the failure load can be calculated based on the initial stiffness as follows.

$$\delta_u = \frac{R_u}{wtE_s}d \tag{3.16}$$

Here

E_m = elastic modulus of the infill material

t = thickness of infill wall

d = length of the strut between the beam-to-column joint nodes

w = effective width of the equivalent strut

δ_u = axial deformation of the strut at failure

R_u = strength of the equivalent strut

It is to be noted that IO, LS and CP are defined at the same point. This is a very simplified approach of defining the axial load versus deformation curve. But it does not represent the material nonlinearity and ignore the inelastic drift.

3.9 SUMMARY

The first part of the chapter presents the geometry, section sizes, reinforcement details and other important information about the selected OGS building. The next part of this chapter describes the issues related to computational modelling of a framed building followed by a detailed procedure on nonlinear frame element modelling with point plastic flexural hinges. This includes generating uncoupled moment-rotation parameters for beams and coupled axial load-biaxial moment-rotation interaction parameters for columns. The last part of the chapter discusses the modelling of infill wall as equivalent diagonal strut element as per Smith and Carter (1969). Also, modelling of nonlinear axial hinge properties for equivalent strut is explained here based on elastic analysis approach.

CHAPTER 4

RESULTS FROM LINEAR ANALYSIS

4.1 INTRODUCTION

Seismic analysis is a subset of structural analysis and is the calculation of the response of the building structure to earthquake and is a relevant part of structural design where earthquakes are prevalent. The seismic analysis of a structure involves evaluation of the earthquake forces acting at various level of the structure during an earthquake and the effect of such forces on the behaviour of the overall structure. The analysis may be static or dynamic in approach as per the code provisions.

Thus broadly we can say that linear analysis of structures to compute the earthquake forces is commonly based on one of the following three approaches.

1. An equivalent lateral procedure in which dynamic effects are approximated by horizontal static forces applied to the structure. This method is quasi-dynamic in nature and is termed as the *Seismic Coefficient Method* in the IS code.
2. *The Response Spectrum Approach* in which the effects on the structure are related to the response of simple, single degree of freedom oscillators of varying natural periods to earthquake shaking.
3. *Response History Method* or *Time History Method* in which direct input of the time history of a designed earthquake into a mathematical model of the structure using computer analyses.

Two of the above three methods of analysis, *i.e*. Seismic Coefficient Method and Response Spectrum Method, are considered for the analysis of buildings studied here. Details of these methods are described in the following section. The seismic method of analysis based on Indian standard 1893:2002 (Part – 1) is described as follows:

4.1.1 Equivalent Static Analysis

This is a linear static analysis. This approach defines a way to represent the effect of earthquake ground motion when series of forces are act on a building, through a seismic design response spectrum. This method assumes that the building responds in its fundamental mode. The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces. In the equivalent static method, the lateral force equivalent to the design basis earthquake is applied statically. The equivalent lateral forces at each storey level are applied at the design 'centre of mass' locations. It is located at the design eccentricity from the calculated 'centre of rigidity (or stiffness)'.

The base dimension of the building at the plinth level along the direction of lateral forces is represented as d (in meters) and height of the building from the support is represented as h (in meters). The response spectra functions can be calculated as follows:

For Type I soil (rock or hard soil sites):

$$\frac{S_a}{g} = \begin{cases} 1 + 15T & 0.00 \leq T \leq 0.10 \\ 2.5 & 0.10 \leq T \leq 0.40 \\ \dfrac{1}{T} & 0.40 \leq T \leq 4.00 \end{cases} \tag{4.1a}$$

For Type II soil (medium soil):

$$\frac{S_a}{g} = \begin{cases} 1 + 15T & 0.00 \leq T \leq 0.10 \\ 2.5 & 0.10 \leq T \leq 0.55 \\ \dfrac{1.36}{T} & 0.55 \leq T \leq 4.00 \end{cases} \qquad (4.1b)$$

For Type III soil (soft soil):

$$\frac{S_a}{g} = \begin{cases} 1 + 15T & 0.00 \leq T \leq 0.10 \\ 2.5 & 0.10 \leq T \leq 0.67 \\ \dfrac{1.67}{T} & 0.67 \leq T \leq 4.00 \end{cases} \qquad (4.1c)$$

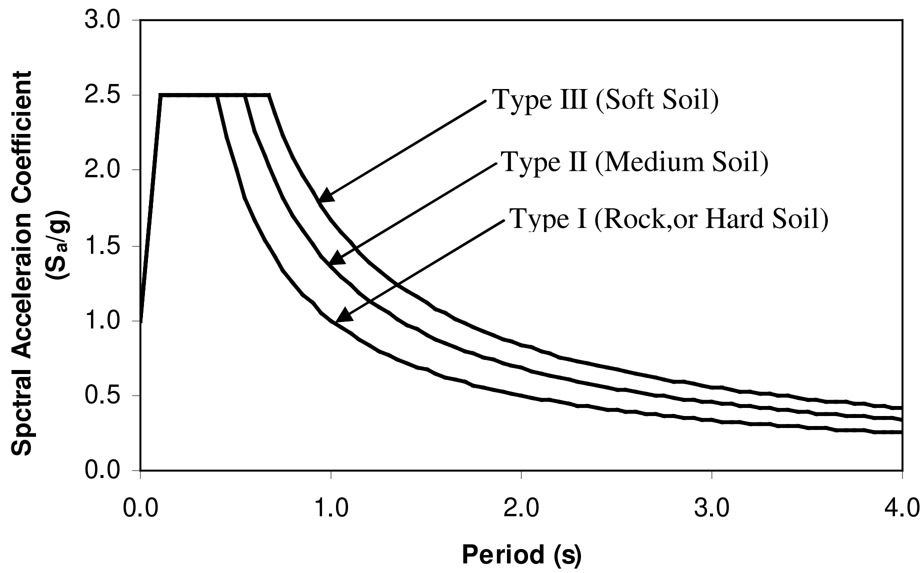


Fig. 4.1: Response spectra for 5 percent damping (IS 1893: 2002)

The design base shear is to be distributed along the height of building as per Clause 7.7.1 of IS 1893: 2002. The design lateral force at floor *i* is given as follows,

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \qquad (4.2)$$

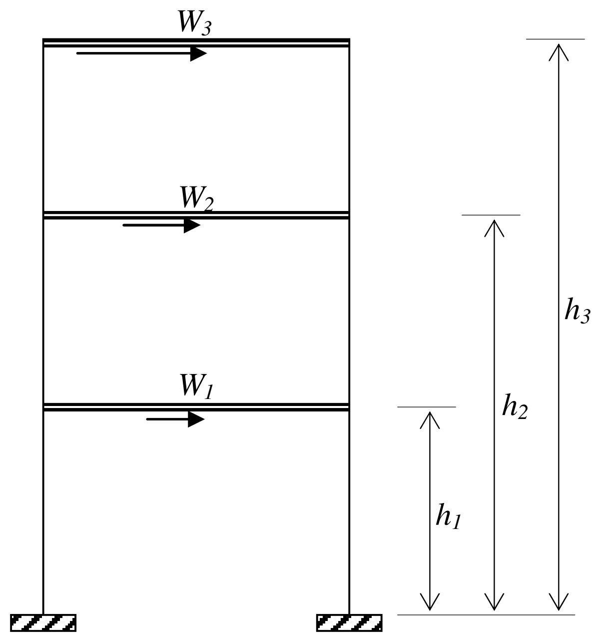


Fig. 4.2: Building model under seismic load

4.1.2 Response Spectrum Analysis

The equations of motion associated with the response of a structure to ground motion are given by:

$$\ddot{u}(t)_n + 2\zeta_n\omega_n\dot{u}(t)_n + \omega_n^2u(t)_n = p_{ni}\ddot{u}(t)_g \tag{4.3}$$

Where p_{ni} the Mode Participation Factor are defined by modal participation factor of mode I of vibration is the amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions.

For a specified ground motion$\ddot{u}(t)_g$, damping value and assumingp_{ni}. It is possible to solve above equation at various values of ω and plot a curve of maximum peak response $u(\omega)_{MAX}$. For this acceleration input, the curve is defined as Displacement

Response Spectrum for earthquake motion. A different curve will exist for each different value of damping.

A plot of ω $u(\omega)_{MAX}$ is defined as the pseudo-velocity spectrum and plot of $\omega^2 u(\omega)_{MAX}$ is defined as the pseudo-acceleration spectrum. These pseudo values have minimum significance and are not essentially a part of a response spectrum analysis. The true values for maximum velocity and acceleration must be calculated from the solution of above equation. There is a mathematical relationship, however, between the pseudo-acceleration spectrum and the total acceleration spectrum. The total acceleration of the unit mass, single degree-of-freedom system is given by,

$$\ddot{u}(t)_T = \ddot{u}(t) + \ddot{u}(t)_g \tag{4.4}$$

$\ddot{u}(t)$from first equation is substituted in the above equation which yields,

$$\ddot{u}(t)_T = -2\zeta_n \omega \dot{u}(t) - \omega^2 u(t) \tag{4.5}$$

Therefore, for the special case of zero damping, the total acceleration of the system is equal to $\omega^2 u(t)$. For this reason, the displacement response spectrum curve is normally not plotted as modal displacement ω vs $u(\omega)_{MAX}$. It is standard to present the curve in terms of $S(\omega)_n$ vs a period T in seconds.

Where,

$$S(\omega)_n = \left(\frac{2\pi}{T}\right)^2 u(\omega)_{MAX} \tag{4.6}$$

The pseudo-acceleration spectrum, $S(\omega)_n$ curve has units of acceleration vs period which has some physical significance for zero damping only. It is apparent that all response spectrum curves represent the properties of the earthquake at specific site and are not a function of the properties of the structural system. After estimation is made of linear viscous damping properties of the structure, a specific response spectrum curve is selected.

It is the linear dynamic analysis. This approach permits the multiple modes of response of a building to be taken into account (in the frequency domain) or where modes other than the fundamental one significantly affect the response of the structure. In this method the response of Multi-Degree of Freedom (MDOF) is expressed as the superposition of each Single-Degree of Freedom (SDOF) system, which is then combined to compute the total response. This is required in many building codes for all except for very simple or very complex structures. Computer analysis can be used to determine these modes for a structure. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass, and they are then combined to provide an estimate of the total response of the structure. Combination methods include the following:

- Maximum Absolute Response (ABS) - peak values are added together
- Square Root of Sum of Squares (SRSS) - method of combining modal maxima for two-dimensional structural system.
- Complete Quadratic Combination (CQC) - a method that is an improvement on SRSS for closely spaced modes

In cases where structures are either too irregular, too tall or of significance to a community in disaster response, the response spectrum approach is no longer appropriate, and more complex analysis is often required, such as non-linear static or dynamic analysis.

4.2 RESULTS OF LINEAR ANALYSIS AND DISCUSSIONS

As mentioned earlier the selected OGS building is analyzed for following two different cases and two end support conditions (fixed and pinned end support)

(a) Considering infill strength and stiffness (with infill/infilled frame)

(b) Without considering infill strength and stiffness (without infill/bare frame).

Therefore there are a total of four building models: (a) building modelled without infill and fixed end support, (b) building modelled with infill and fixed end support, (c) building modelled without infill and pinned end support and (d) building modelled with infill and pinned end support.

Equivalent static and response spectrum analyses of these four building models are carried out to evaluate the effect of infill on the seismic behaviour of OGS building for two different support conditions. Following sections presents the results obtained from these analyses.

4.2.1 Calculation of Time Period and Base Shear

The design base shear (V_B) was calculated as per IS 1893: 2002 corresponding to the fundamental period for moment-resisting framed buildings with brick infill panels as follows:

$$V_B = A_h W \tag{4.7}$$

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} \tag{4.8}$$

where W ≡ seismic weight of the building, Z ≡ zone factor, I ≡ importance factor, R ≡ response reduction factor, S_a /g ≡ spectral acceleration coefficient corresponding to an approximate time period (T_a) which is given by:

$$T_a = \frac{0.09h}{\sqrt{d}} \quad \text{for RC frame with masonry infill} \tag{4.9}$$

The base dimension of the building at the plinth level along the direction of lateral forces is represented as d (in meters) and height of the building from the support is represented as h (in meters). Same base shear were applied in the two building models. The equivalent lateral forces at each storey level are applied statically at the design centre of mass locations for equivalent static analysis (ESA). The building models also analyzed using Response Spectrum analysis (RSA). The first five modes were considered in the dynamic analysis, which give more than 90% mass participation in both of the horizontal directions. The base shears for the equivalent static method and the response spectrum methods are given in Table 4.1. This table indicates that there is no considerable difference between two models with regards to the global stiffness and design forces.

Table 4.1: Comparison of fundamental time periods for with and without infill for pinned and fixed end support condition

	With infill		Without infill	
	V_x (kN)	V_y (kN)	V_x (kN)	V_y (kN)
Equivalent Static $\left(\overline{V_B}\right)$	1566	1566	1566	1566
Response Spectra $\left(V_B\right)$	1427	1427	1300	1310
$\overline{V_B}/V_B$	1.10	1.10	1.20	1.19

4.2.2 Shift in Period

When the infill stiffness is considered in the OGS building model the global stiffness is bound to increase, reducing the fundamental period of the building. This reduction may attract additional seismic force and this is one of the factors that make difference between buildings modeled with and without infill stiffness. Therefore shift in fundamental period can be considered as an important parameter to describe how much the infill stiffness contributes to the global stiffness of the OGS building. The fundamental time periods in the predominant direction of vibration and the spectral acceleration coefficients corresponding to medium soil for the building for various cases are given in Table 4.2(a) and 4.2(b) for building models with fixed and pinned end supports respectively.

Table 4.2 (a): Shift in period for fixed end support condition.

Fixed End	Empirical formula		Computational Value	
	With infill	Without infill	With infill	Without infill
Tx (s)	0.28	0.47	0.28	0.47
Ty (s)	0.33	0.47	0.33	0.47
(Sa/g)x	2.50	2.50	2.50	2.50
(Sa/g)y	2.50	2.50	2.50	2.50

From Table 4.2(a) and 4.2(b) we can see that there is not much considerable difference in the time periods of the building irrespective of the directions considered according to the empirical formula. From the computational value we can see that there is a considerable shift of period for buildings modelled with fixed end support conditions. But the period shift is found to be very little in case of buildings modelled with pinned end support conditions

Hence it can be said that the IS 1893:2002 (Part-1) does not take into account the support conditions for the calculation of fundamental period. It always gives a lower bound solution to be conservative for force calculation.

Table 4.2 (b): Shift in period for pinned end support condition

Pinned End	Empirical formula		Computational Value	
	With infill	Without infill	With infill	Without infill
Tx (s)	0.28	0.47	0.52	0.61
Ty (s)	0.33	0.47	0.52	0.60
(Sa/g)x	2.50	2.50	2.50	2.23
(Sa/g)y	2.50	2.50	2.50	2.28

4.2.3 Column Interaction Ratios

All four building models were analyzed with lateral force associated with 'with infill' case for linear static analyses. However, for response spectrum analyses the base shear is a function of the respective structural natural periods. The demands (moments, axial forces) obtained at the critical sections from the linear (static and dynamic) analyses are compared with the capacities of the individual elements. For a column, the moment demand due to bi-axial bending under axial compression is checked using the P-Mx-My interaction surface, generated according to IS 456: 2000. The demand point is plotted in the P-Mx-My space and a straight line is drawn joining the demand point to the origin. This line (extended, if necessary) will intersect the interaction surface at the capacity point. The ratio of the distance of the demand point (from the origin) to the distance of the capacity point (from the origin) is termed as the interaction ratio (IR) for the column. The interaction ratio was found out using the formula as given in the IS code 456:2000:

$$IR = \left(\frac{M_x}{M_{x,c}}\right)^{\alpha} + \left(\frac{M_y}{M_{y,c}}\right)^{\alpha} \tag{4.10}$$

$$\alpha = 1.0\ for\ \ \frac{P}{P_c} < 0.2$$

$$\alpha = 2.0\ for\ \frac{P}{P_c} > 0.8$$

Where, M_x is moment about X- axis and M_y is moment about Y- axis

$M_{x,c}$ and $M_{y,c}$ are maximum uniaxial bending moment capacity about X- and Y- axes respectively

P is the axial load on the member

Table 4.3 (a) and Table 4.3 (b) presents the interaction ratio (IR) for all ground storey columns for buildings modelled with pinned end and fixed end support conditions respectively. These tables also show the ratio of IR for similar columns to show how the ground floor column forces increases for modelling infill stiffness at the upper storeys.

Table 4.3(a): Comparison of Ground Storey Column Interaction Ratio for Pinned End Case

Col. ID	IR (ESA)		Ratio of IR	IR (RSA)		Ratio of IR
	WI	WOI		WI	WOI	
C1	1.13	1.53	0.74	2.05	1.78	1.15
C2a	1.94	1.93	1.01	2.45	2.24	1.09
C2b	1.84	1.82	1.01	2.49	2.09	1.19
C3	1.84	1.91	0.96	3.9	3.34	1.17

Table 4.3(b): Comparison of Ground Storey Column Interaction Ratio for Fixed End Case

Col. ID	IR (ESA)		Ratio of IR	IR (RSA)		Ratio of IR
	WI	WOI		WI	WOI	
C1	0.89	0.93	0.96	1.24	1.19	1.04
C2a	1.04	1.13	0.92	1.26	1.23	1.02
C2b	1.01	1.07	0.94	1.24	1.93	1.04
C3	1.41	1.52	0.82	2.04	1.93	1.06

This table clearly shows that for a low rise OGS building model with fixed-end support the ground storey column forces actually reduced when infill stiffness is considered in Equivalent Static Analysis. It marginally increases (less than 10%) in the case of response spectrum analysis. This is because the forces applied to building model with infill stiffness is little more compared to that applied to building model without infill stiffness in Response Spectrum Analysis. But the applied forces to these two buildings are same in case of Equivalent Static Analyses. Therefore using a multiplication factor of 2.5 for ground floor columns of low rise OGS buildings as per Indian Standard IS 1893:2002 (Part-1) is not justified.

4.2.4 Beam Demand-to-Capacity Ratios

The forces and displacement resulting from an elastic analysis for design earthquake load are called elastic demand. A ratio of the demand to the corresponding capacity of a member is termed as Demand-to-Capacity Ratios (DCR). DCR values are calculated for all the beam elements of the buildings studied here. A DCR value more than one for a member indicates member capacity is less than the demand posed by the design earthquake. Therefore, the DCR values for each beam element should be less than 1.0 for code compliance. Maximum (positive) and minimum (negative) bending moment demands at the two ends of each beam element have been compared with the corresponding capacities and calculated DCR values for first floor beams (top of the open ground storey) are presented in Table 4.4. Table 4.4 (a) presents the case of pinned-end OGS building whereas Table 4.4 (b) presents the case of fixed-end OGS building.

Table 4.4 (a): Comparison of Beam DCR (Pinned-End)

Beam ID	DCR (ESA)		Ratio of DCR	DCR (RSA)		Ratio of DCR
	WI	WOI		WI	WOI	
B1	1.88	2.81	0.67	1.32	1.65	0.80
B4	1.84	2.63	0.70	1.15	1.45	0.80
B5	1.03	1.53	0.67	0.62	0.81	0.77
B7	1.33	1.93	0.69	0.86	1.09	0.79
B8	1.77	2.52	0.70	1.24	1.52	0.82
Average			0.69			0.80
Standard Deviation			0.01			0.02

Table 4.4 (b): Comparison of Beam DCR (Fixed-End)

Beam ID	DCR (ESA)		Ratio of DCR	DCR (RSA)		Ratio of DCR
	WI	WOI		WI	WOI	
B1	1.04	1.74	0.60	0.65	0.95	0.68
B4	1.16	1.78	0.65	0.60	0.88	0.68
B5	0.76	1.08	0.70	0.34	0.52	0.65
B7	0.82	1.29	0.64	0.46	0.66	0.70
B8	1.01	1.59	0.64	0.64	0.89	0.72
Average			0.65			0.69
Standard Deviation			0.04			0.03

Table 4.4 presents results from both equivalent static analyses (ESA) and response spectrum analyses (RSA). The table presented above shows that the conclusion drawn for the columns hold good for beams also. Force demands in all first floor beams are found to be lower when infill stiffness modelled in OGS building. It can be concluded from this results that it is conservative to analyse low-rise OGS building without considering infill

stiffness. Table 4.4 shows that the average ratio of DCR values (ratio of DCR in WI model to DCR in WOI model) for first floor beams is below 0.70 for both pinned-end and fixed-end building in Equivalent Static Analyses. Table 4.4 (b) shows that the average ratio of DCR values for first floor beams is 0.80 for pinned-end building in Response Spectrum Analyses although this lies within 0.70 for fixed-end building model. A statistical analysis of the DCR ratios shows that the DCR ratios for all the beams are very consistent (standard deviation is within 0.04 for all cases). A conclusion can be drawn from these results that amplification factor of 2.5 need not be multiplied to the beam forces even when infill stiffness is not modelled in analysis. However, this statement is valid for low-rise OGS building and cannot be used for high-rise OGS buildings.

It is observed from Table 4.3 (a), 4.3 (b), 4.4(a), and 4.4 (b) that analysis of the model without considering infill strength and stiffness gives a conservative estimation for all beam and column elements in a low-rise open ground storey building. This is true for equivalent static analysis as well as response spectrum analysis. The response spectrum analyses present slightly different results for column. As per the response spectrum analyses, model without considering infill strength and stiffness gives marginally un-conservative estimate for the columns. This is due to the applied load, in response spectrum analyses, is more in the model with infill strength and stiffness. This is to be noted that the applied force were same for both of the two models in equivalent static analyses. Therefore it can be concluded that if the applied load is fixed the building analysis ignoring infill strength and stiffness will be conservative even for a low-rise OGS building.

4.3 SUMMARY

This chapter starts with a detailed description of equivalent static analysis procedure and response spectrum analysis procedure as per Indian Standard IS 1893 (Part -1): 2002 followed by the analysis results of selected OGS buildings obtained by these two methods of analyses.

It is observed from the results presented here that analysis of the model without considering infill strength and stiffness gives a conservative estimation for all beam and column elements in a low-rise open ground storey building. This is true for equivalent static analysis as well as response spectrum analysis. Therefore, amplification factor of 2.5 as recommended in Indian Standard IS 1893 (Part -1): 2002 need not be multiplied to the beam forces even when infill stiffness is not modelled in analysis. This conclusion based on linear analysis needs to be validated by nonlinear analysis.

CHAPTER 5

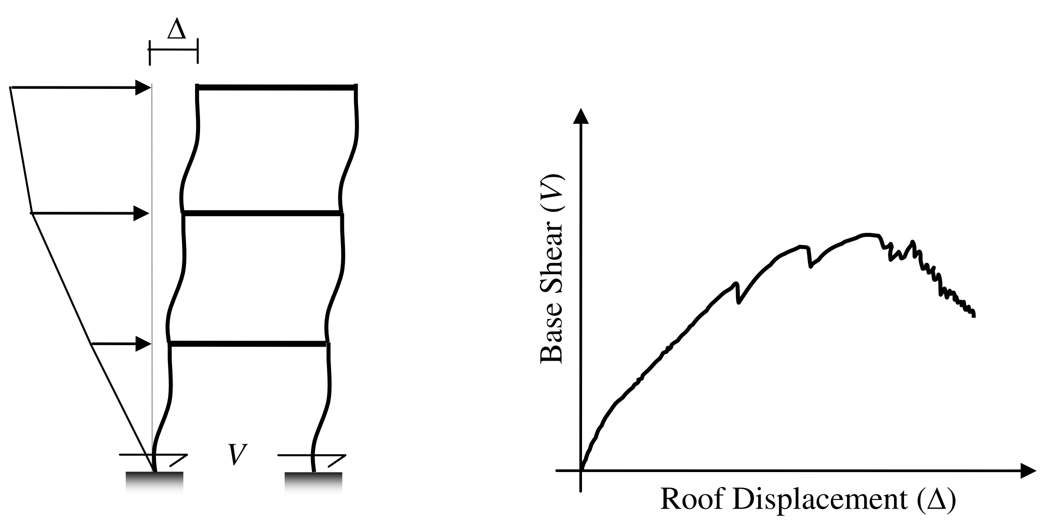
RESULTS FROM NON-LINEAR ANALYSIS

5.1 INTRODUCTION

It is found from the linear (static and dynamic) analyses that the amplification factor of 2.5 as recommended in Indian Standard IS 1893 (Part -1): 2002 for designing open ground storey beams and columns are too conservative for low-rise OGS buildings. An effort has been made to verify this conclusion from nonlinear analysis. Pushover Analysis is selected as it is the simplest among the different nonlinear analysis procedures. First half of this chapter presents a detailed description on Pushover Analysis and its procedure. Later half of this chapter presents the results obtained from the pushover analyses of selected open ground storey building for both pinned and fixed end condition. Nonlinear analysis requires modelling of all load resisting elements with material nonlinearity. Modelling nonlinearity for frame elements and infill walls is discussed in Chapter 3.

5.2 PUSHOVER ANALYSIS

The pushover analysis is a nonlinear static method which is used in a performance based analysis. The method is relatively simple to be implemented, and provides information on strength, deformation and ductility of the structure and distribution of demands which help in identifying the critical members likely to reach limit states during the earthquake and hence proper attention can be given while designing and detailing. This method assumes a set of incremental lateral load over the height of the structure. Local nonlinear effects are modelled and the structure is pushed until a collapse mechanism is developed. With the increase in the magnitude of loads, weak links and failure modes of the buildings are found. At each step, the base shear and the roof displacement can be plotted to generate the pushover curve (Fig. 5.1). This method is relatively simple and provides information on the strength, deformation and ductility of the structure and distribution of demands. This permits to identify the critical members likely to reach limit states during the earthquake by the formation of plastic hinges. On the building frame load/displacement is applied incrementally, the formation of plastic hinges, stiffness degradation, and lateral inelastic force versus displacement response for the structure is analytically computed. But some limitations of this method is that it neglects the variation of loading pattern, influence of higher modes and effect of resonance. In spite of the above deficiencies still this method has gained a wide acceptance as it provides reasonable estimation of global deformation capacity. And also the decision to retrofit can be taken on the basis of such studies.



a) Building model b) Pushover curve

Fig. 5.1: Schematic representation of pushover analysis procedure

It gives an idea of the maximum base shear that the structure is capable of resisting and the corresponding inelastic drift. For regular buildings, it also gives an estimate of the global stiffness of the building.

In pushover analysis, it is necessary to model the nonlinear load versus deformation behaviour of every element. The beams and columns are modelled as frame elements and the infill walls are modelled as equivalent struts by truss elements. Since the deformations are expected to go beyond the elastic range in a pushover analysis, it is necessary to model the nonlinear load versus deformation behaviour of the members. The nonlinear behaviour is incorporated in the load versus deformation property of a concentrated hinge attached to the member.

5.3 PUSHOVER ANALYSIS PROCEDURE

Pushover analysis involves the application of increasing lateral forces or displacements to a nonlinear mathematical model of a building. The nonlinear load-deformation behaviour of each component of the building is modelled individually. In a force-controlled push, the forces are increased monotonically until either the total force reaches a target value or the building has a collapse mechanism. In a displacement-controlled push, the displacements are increased monotonically until either the displacement of a predefined control node in the building exceeds a target value or the building has a collapse mechanism. For convenience, the control node can be taken at the design centre of mass of the roof of the building. The target displacement is intended to represent the maximum displacement likely to be experienced during the earthquake.

Initially, the gravity loads are applied in a force-controlled manner till the total load reaches the target value. The target value can be same as the design gravity load for the linear analysis. Next, the lateral loads are applied in the X- or Y- direction, in a displacement controlled manner. The direction of monitoring of the behaviour is same as the push direction. The effect of torsion can be considered. As the displacement is increased, some beams, columns and 'equivalent struts' may undergo in-elastic deformation. The non-linear in-elastic behaviour in flexure, shear or axial compression is modelled through assigning appropriate load-deformation properties at potential plastic hinge locations.

5.4 PERFORMANCE LEVELS OF STRUCTURE AND ELEMENTS

The structural and non- structural components of the buildings together comprise the building performance. The performance levels are the discrete damage states identified from a continuous spectrum of possible damage states. The structural performance levels based on the roof drifts are as follows:

i) Immediate occupancy (IO)

ii) Life safety (LS)

iii) Collapse prevention (CP)

The three levels are arranged according to decreasing performance of the lateral load resisting systems.

i) Point 'A' corresponds to the unloaded condition.

ii) Point 'B' corresponds to the onset of yielding.

iii) Point 'C' corresponds to the ultimate strength.

iv) Point ‘D’ corresponds to the residual strength. For the computational stability, it is recommended to specify non-zero residual strength beyond C. In absence of the modelling of the descending branch of a load versus deformation curve, the residual strength can be assumed to be 20% of the yield strength.

v) Point ‘E’ corresponds to the maximum deformation capacity with the residual strength. To maintain computational stability, a high value of deformation capacity is assumed.

The performance levels (IO, LS, and CP) of a structural element are represented in the load versus deformation curve.

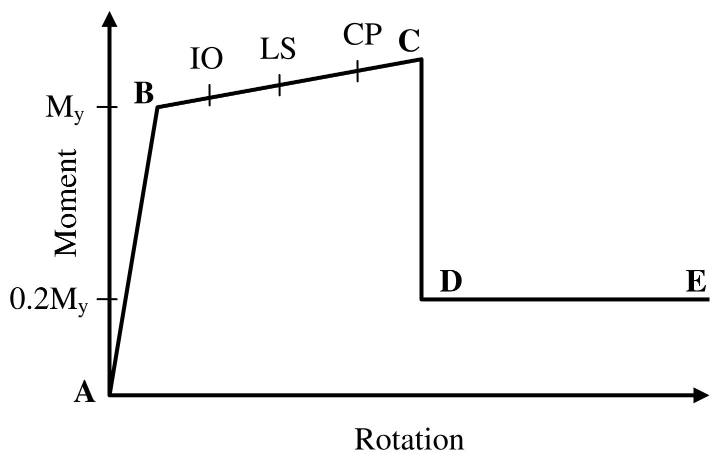
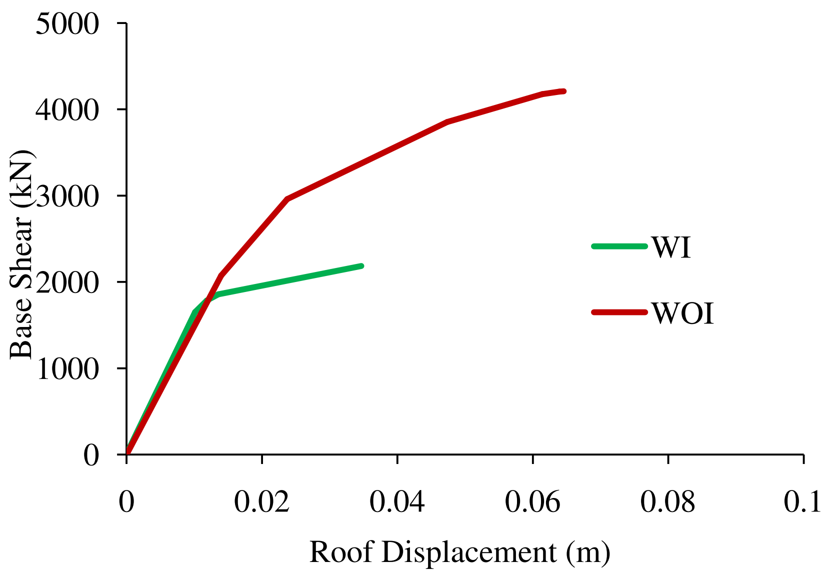


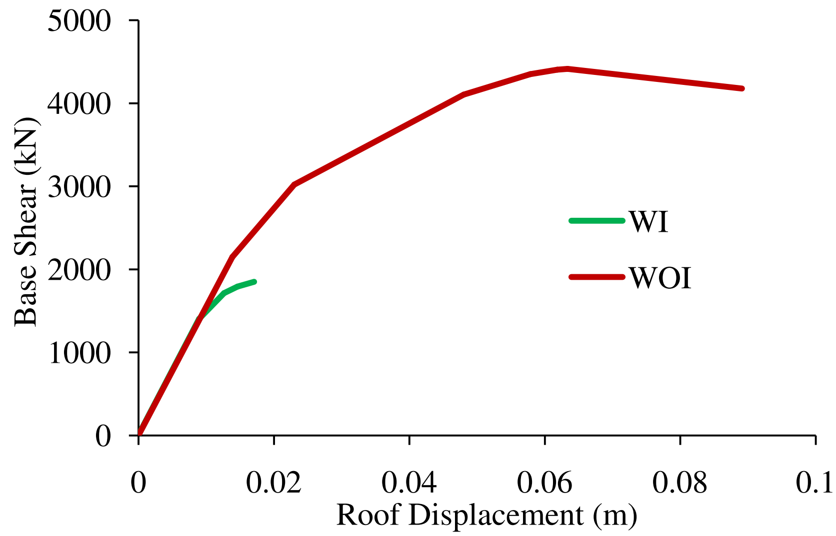
Fig. 5.2 Performance Level of Pushover Analysis

5.5 RESULTS FROM PUSHOVER ANALYSIS

Pushover analysis is carried out for both of the two building models. First pushover analysis is done for the gravity loads (DL+0.25LL) incrementally under load control. The lateral pushover analysis (PUSH-X and PUSH-Y) is followed after the gravity pushover, under displacement control. The building is pushed in lateral directions until the formation of collapse mechanism. The capacity curve (base shear versus roof displacement) is obtained in X- and Y- directions and presented in Figs. 5.3(a) and 5.3(b). These figures clearly show that global stiffness of an open ground storey building hardly changes even if the stiffness of the infill walls is ignored. If there is no considerable change in the stiffness elastic base shear demand for the building will also not change considerably if the stiffness of the infill walls is ignored. The variation of pushover curves in X- and Y- directions is in agreement with the linear analysis results presented in the previous section with regard to the variation of elastic base shear demand for different building models.



(a) X-direction Push



(b) Y-direction Push

Fig.5.3: Pushover curves for pinned-end building

The above figures are the results from the pushover analysis for pinned end support condition in both X- and Y- direction respectively. It is found that both of the maximum base shear and roof displacement capacity for without-infill case is higher than that of with-infill case. This is true for both X- and Y- direction push. Also, it is clear from these figures that building modelled without infill stiffness has more ductility compared to the building modelled with infill stiffness.

Figs. 5.4 (a) and (b) show plastic hinge distribution in a typical X-Z frame at collapse under X-direction push. It is clear that without-infill model utilizes the full capacity of the building before collapse as the hinges are evenly distributed in all storeys of the building. Whereas the plastic hinges, for with-infill model, are concentrated only in the ground storey columns and building model fails by storey mechanism. There is a major difference in mode of failure for the two building models. Similar conclusion can be made from the Y-direction pushover analysis.

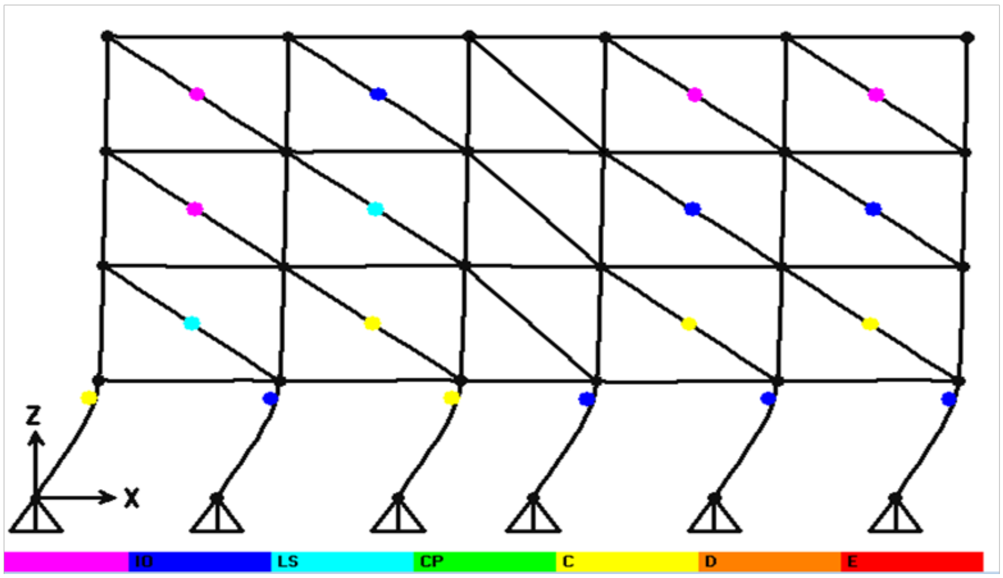


Fig. 5.4 (a): Distribution of plastic hinges for WI building model

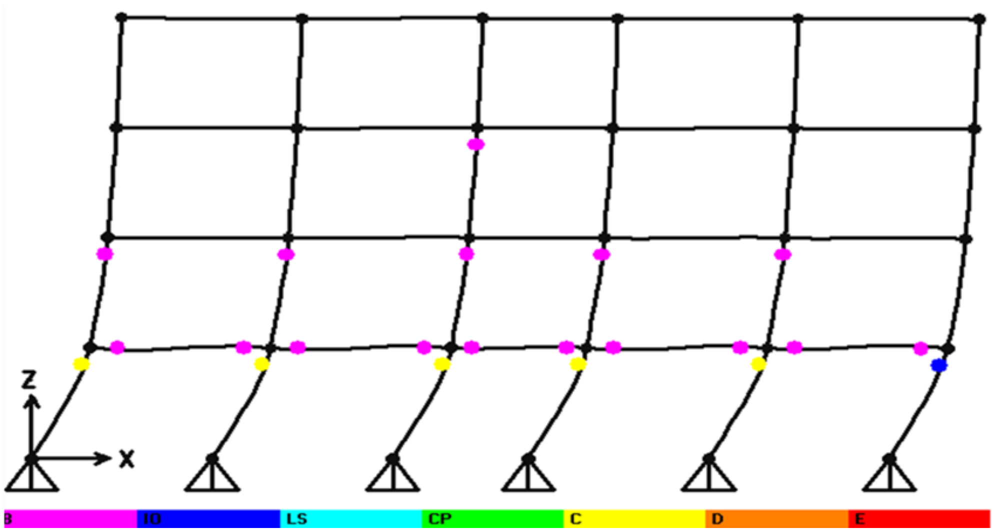


Fig. 5.4 (b): Distribution of plastic hinges for WOI building model

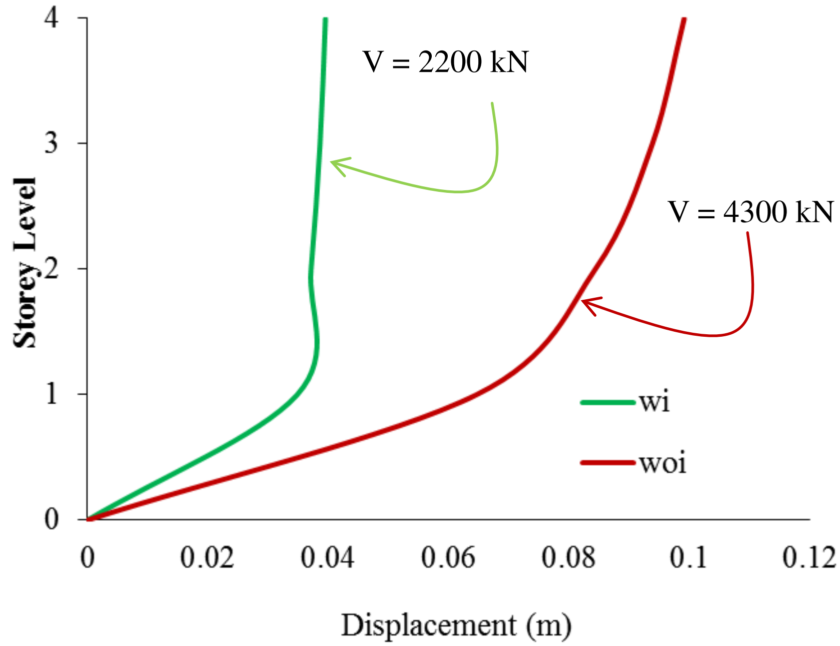
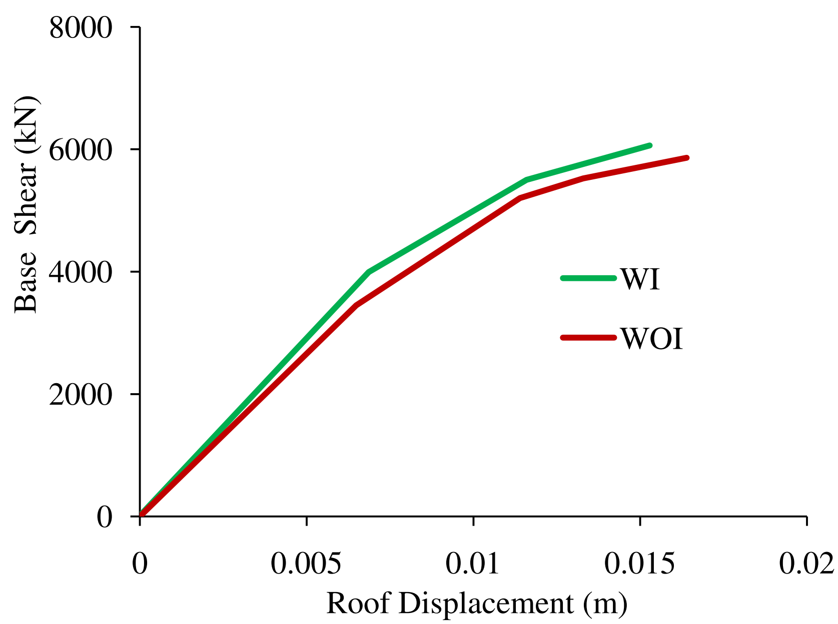


Fig. 5.5: Storey displacement at collapse for pinned end case (Push X)

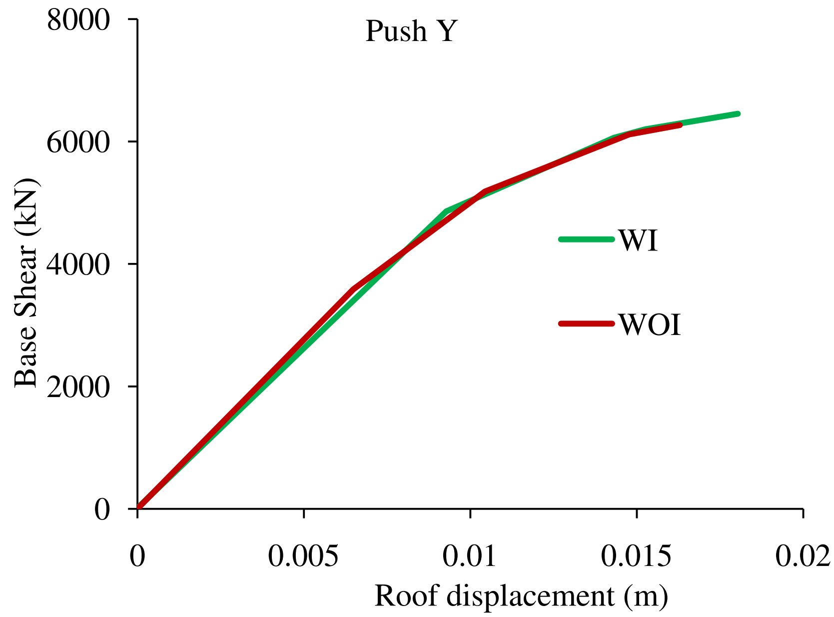
Fig. 5.5 presents the displacement profile of the two pinned-end building models. The building modelled with infill stiffness has negligible change in storey displacement at 2nd, 3rd and 4th storey as seen from the graph above, moreover its base shear capacity is less than that of WOI model, and this is because the WI model failed earlier with plastic hinges formed only in the ground storey columns as evident from the Fig. 5.5 above. Similar results were also observed for Y- direction pushover analysis.

Figs. 5.6(a) and 5.6(b) show the results of the building modelled with fixed-end support condition. The pushover curves presented in these figures indicates similar results. WOI model has higher base shear capacity compared to WI model. Obviously in case of fixed end condition the maximum base shear capacity (around 6900kN in X direction) is much higher compared to that of pinned-end building model (4300kN in X direction).

Moreover the curves in the initial linear stages seem to vary marginally for both X and Y direction and this is because marginal change in stiffness hardly affects the elastic analysis results. The results presented here show that modelling infill strength and stiffness in nonlinear analysis of an open ground storey building may reveal the appropriate failure mode but this may not change the estimated demand and capacity of the building considerably. The results of pushover analyses along with the linear analyses (ESA and RSA) do not support the IS 1893:2002 criteria for open ground storey building that requires to upgrade design forces of ground storey frame elements 2.5 times when infill stiffness and strength is not considered.

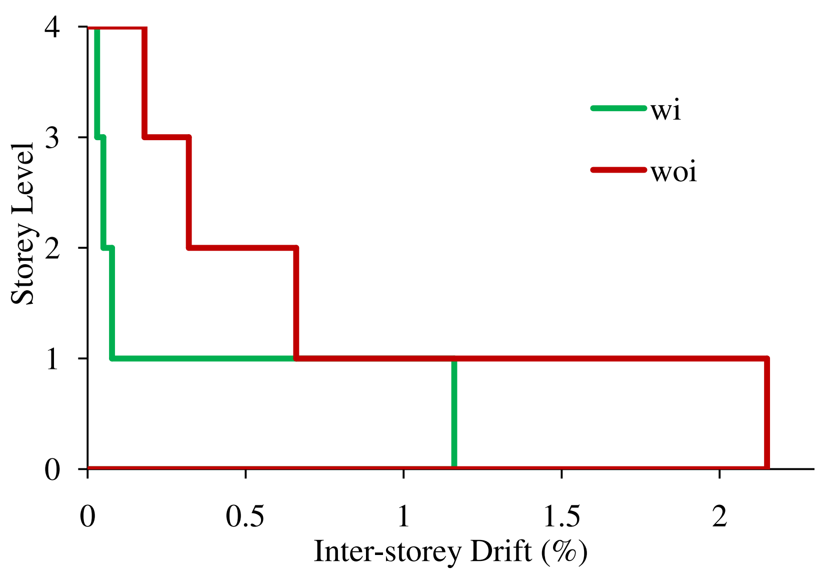


(a) X-direction Push

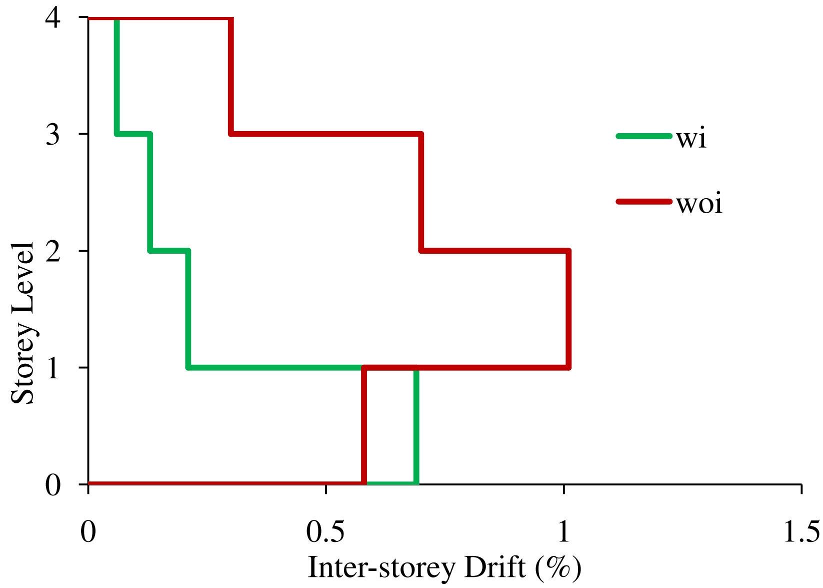


(b) Y-direction Push

Fig. 5.6: Pushover curves for fixed-end building model



(a) for pinned-end building model



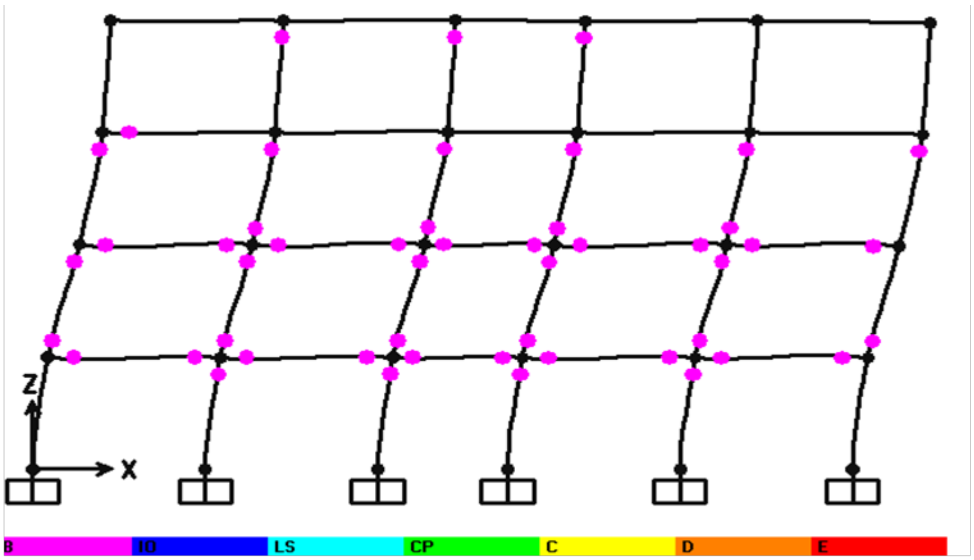
(b) for fixed-end building model

Fig. 5.7: Inter-storey drift at collapse as obtained from pushover analysis in X direction

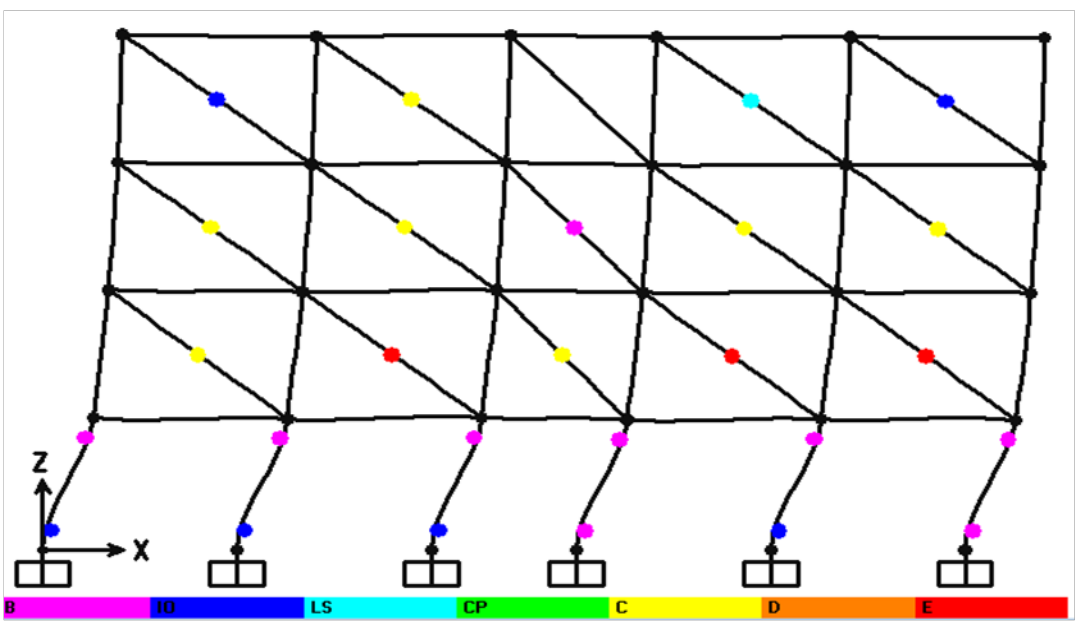
Fig. 5.7 presents the inter-storey drift of the different building models at the final stage of the pushover analysis for both the support conditions. This figure shows model with infill strength and stiffness has lesser inter-storey drift, as expected, for both the support conditions. However, the model without infill strength and stiffness for fixed end case shows unexpectedly higher inter-storey drift in the first floor compared to the ground floor. This is due to the failure of the diagonal struts at different nonlinear steps that makes the first floor equivalent to that of a bare frame building.

Figs. 5.8(a) and 5.8(b) show the distribution of plastic hinges formed during collapse for fixed end support condition. Similar to the previous case WOI model found to utilize the full capacity of the building as plastic hinges are distributed almost equally in all storeys.

Also, in WI model, the compressive struts fail prior to the column which may be advantageous from seismic point of view.



(a) Modelled without infill stiffness



(b) Modelled with infill stiffness

Fig. 5.8: Hinge distribution at collapse for fixed-end building model (X-direction push)

5.6 SUMMARY

Details of Pushover analysis procedure has been presented in this chapter. Pushover analysis results of the selected building models are discussed. The main conclusion that can be drawn from the pushover analysis results presented here is estimated elastic force demand in the frame elements of OGS building do not change much by ignoring the infill stiffness. But one can wrongly estimate a higher inelastic base shear and displacement capacity of an OGS building by ignoring infill stiffness. The analysis shows that OGS building can have a brittle mode of failure when the infill stiffness is correctly modelled. This cannot be captured when the model ignores the infill stiffness.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

Open ground storey buildings are considered as vertically irregular buildings as per IS 1893: 2002 that requires dynamic analysis considering strength and stiffness of the infill walls. IS 1893: 2002 also permits Equivalent Static Analysis (ESA) of OGS buildings ignoring strength and stiffness of the infill walls, provided a multiplication factor of 2.5 is applied on the design forces (bending moments and shear forces) in the ground storey columns and beams. The objective of the present study is to review the rationality of this approach. An existing RC framed building (G+3) with open ground storey located in Seismic Zone-V is analyzed for two different cases: (a) considering infill strength and stiffness and (b) without considering infill strength and stiffness (bare frame). Infill weights (and associated masses) were modelled in both the cases through applying static dead load. Non-integral infill walls subjected to lateral load behave like diagonal struts. Thus an infill wall can be modelled as an equivalent 'compression only' strut in the building model. Rigid joints connect the beams and columns, but pin joints connect the equivalent struts to the beam-to-column junctions. Infill stiffness was modelled using a diagonal strut approach as per Smith and Carter (1969).

Linear static and dynamic analyses of the two building models are carried out to compare the force demand in the open ground storey frames. The code specified multiplication factor is compared with the ratio of their force demands. Two different support conditions are considered for the analysis to check the effect of the support conditions on the relative frame force demand. The support conditions considered are: pinned-end and fixed-end conditions.

Nonlinear static (pushover) analysis is carried out for all the building models considered. First pushover analysis is done for the gravity loads incrementally under load control. The lateral pushover analysis is followed after the gravity pushover, under displacement control.

6.2 CONCLUSIONS

Followings are the salient conclusions obtained from the present study:

i) IS code gives a value of 2.5 to be multiplied to the ground storey beam and column forces when a building has to be designed as open ground storey building or stilt building. The ratio of IR values for columns and DCR values of beams for both the support conditions and building models were found out using ESA and RSA and both the analyses supports that a factor of 2.5 is too high to be multiplied to the beam and column forces of the ground storey. This is particularly true for low-rise OGS buildings.

ii) Problem of OGS buildings cannot be identified properly through elastic analysis as the stiffness of OGS building and Bare-frame building are almost same.

iii) Nonlinear analysis reveals that OGS building fails through a ground storey mechanism at a comparatively low base shear and displacement. And the mode of failure is found to be brittle.

iv) Both elastic and inelastic analyses show that the beams forces at the ground storey reduce drastically for the presence of infill stiffness in the adjacent storey. And design force amplification factor need not be applied to ground storey beams.

v) The linear (static/dynamic) analyses show that Column forces at the ground storey increases for the presence of infill wall in the upper storeys. But design force amplification factor found to be much lesser than 2.5.

vi) From the literature available it was found that the support condition for the buildings was not given much importance. Linear and nonlinear analyses show that support condition influences the response considerably and can be an important parameter to decide the force amplification factor.

6.3 SCOPE FOR FUTURE WORK

i) The proposed results need to be validated by further case studies. Building models considered in this study are of low height and therefore influence of period-shift is negligible. For high-rise buildings shift-in-period can be an additional parameter what is not accounted in the present study.

ii) Another field of wide research could be the design of the infill walls considering the door and the window openings which has not been considered in this research work.

iii) It is found in the present study that the multiplication factor of 2.5 as given in IS 1893:2002 is not justified through elastic force demand. However this factor may be required to achieve a ductile mode of failure and to avoid localised storey mechanism. This can be studied elaborately.

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