P-2622

Effect of Masonry Infill in Reinforced Multistoreyed Buildings By Linear Static And Dynamic Analysis

A Project Report

Submitted by

S.Suprageeth - **0720101013**

in partial fulfillment for the award of the degree of

Master of Engineering
in
Structural Engineering

DEPARTMENT OF CIVIL ENGINEERING

KUMARAGURU COLLEGE OF TECHNOLOGY
COIMBATORE – 641 006
(An Autonomous Institution Affiliated to Anna University, Coimbatore.)

ANNA UNIVERSITY: COIMBATORE

MAY 2009

ANNA UNIVERSITY: COIMBATORE

BONAFIDE CERTIFICATE

Certified that this project report titled **"Effect of Masonry Infill in Reinforced Multistoreyed Buildings by Linear Static and Dynamic analysis"** is the bonafide work of

Mr.S.Suprageeth Reg No: 0720101013

Dr.S.L.Narasimhan,
Head of the Department
Department of Civil Engineering
Kumaraguru College of Technology
Coimbatore-641 006

Mrs. K.Ramadevi, M.E, Lecturer,
Thesis Supervisor
Department of Civil Engineering
Kumaraguru College of Technology
Coimbatore-641 006

Certified that the candidate with university Register No. 0720101013 was examined in project viva voce Examination held on 05/05/2009

Internal Examiner

External Examiner 5/5/0

DEPARTMENT OF CIVIL ENGINEERING
KUMARAGURU COLLEGE OF TECHNOLOGY, COIMBATORE 641 006
(An Autonomous Institution Affiliated to Anna University, Coimbatore.)

KONGU ENGINEERING COLLEGE

(An Autonomous Institution affiliated to Anna University, Coimbatore)

PERUNDURAI ERODE 638 052 TAMILNADU INDIA

Estd : 1984

KONGU Assuring the Best

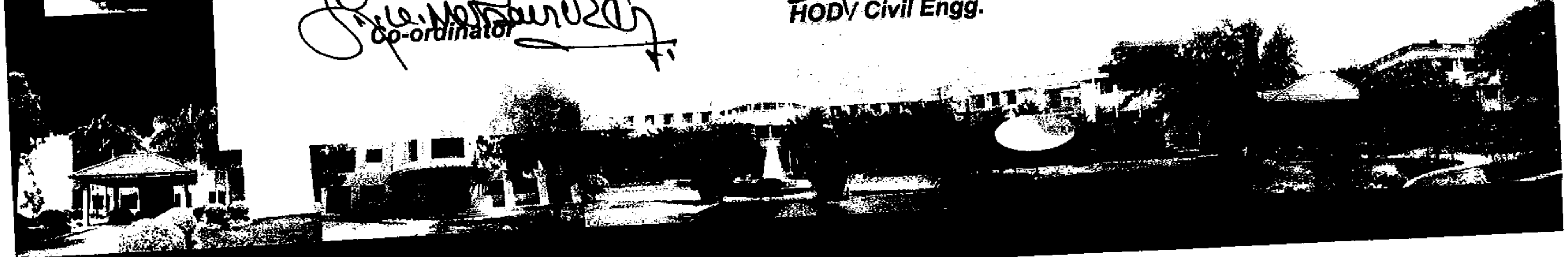
DEPARTMENT OF CIVIL ENGINEERING

Certificate

This is to certify that ~~Mr.~~/Ms. S. SUPRAGEETH

has attended in the National Conference on RECENT TRENDS IN CONCRETE COMPOSITES FOR STRUCTURAL SYSTEMS (RTCCSS-09) during 3rd & 4th April, 2009 and presented the paper entitled EFFECT OF MASONRY INFILL IN REINFORCED CONCRETE MULTISTOREY BUILDINGS BY LINEAR....

Co-ordinator HOD / Civil Engg. Principal 4/4/09



KUMARAGURU COLLEGE OF TECHNOLOGY
COIMBATORE , TAMILNADU

DEPARTMENT OF CIVIL ENGINEERING

INNOVATIONS IN CIVIL ENGINEERING - ICE'09

CERTIFICATE

This is to certify that Mr/Ms S. Suprageeth - PG Student of Kumaraguru College of Technology, Coimbatore has participated and presented a paper in the National Conference on "INNOVATIONS IN CIVIL ENGINEERING - ICE'09" during 21st, April 2009.

Title Effect of masonry Infill On Reinforced Concrete Multistorey Buildings By Linear Static And Dynamic Analysis

Dr.P.ESWARAMOORTHI & Mrs.K.RAMADEVI
COORDINATORS

Dr.S.L.NARASIMHAN
CONVENOR

Prof.R.ANNAMALAI
VICE PRINCIPAL

ABSTRACT

The masonry infill walls are mostly used in structures as non-structural elements and their stiffness contributions are greatly ignored in practice. But in reality, Masonry infill (MI) walls are remarkable in increasing the initial stiffness of reinforced concrete (RC) frames, and being the stiffer component, attract most of the lateral seismic shear forces on buildings, thereby reducing the demand on the RC frame members. The behavior of masonry infill under such lateral loads is extremely difficult to predict due to large number of variables associated in the structural behavior. The response of a structure to earthquake loads largely depends on the fundamental period of vibration and the associated mode shapes.

The presence of masonry infill walls in the frames of a building changes the lateral stiffness and strength of the structure. A frame with infill walls can be analysed by modeling it as equivalent diagonal struts. One approach of modelling the struts is based on the initial stiffness of the strut (Elastic analysis approach). Where as another approach is based on the strength of the strut (Ultimate load approach). The two approaches provide different values of the strut properties such as width, modulus of elasticity and strength.

The presence of walls in upper storeys makes them 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 the present study, seismic analysis of buildings having 4 to 15 stories was performed using response spectrum method as well as equivalent static load method. The seismic responses such as base shear, lateral forces on each floor are determined using the programme written in MATLABversion 7. The seismic response of the RC bare frame buildings and infilled buildings with and without open ground storey building are compared. The infill walls are modeled as equivalent diagonal struts. It is found that due to the presence of masonry infill bounded between the RC frame, there is a increased storey stiffness resulting in decreased the time period, which leads to increase in the base shear of the structure. For open ground storey building, there is a reduction in spectral acceleration and base shear due to increase of the natural time period of vibration of structure.

ACKNOWLEDGEMENT

First and foremost I submit my thanks to **The Almighty,** through whom all things are possible. This work was not by my might nor by power, but by **His Spirit**. This work not be possible without the gifts, **God** has given unto me.

I take pride and immense pleasure in expressing my deep sense of gratitude indebtedness to my guide **Mrs.K.Ramadevi,** Lecturer of Civil Engineering Department for his innovative ideas continued conduce, untiring efforts and encouragements which enabled the successful completion of thesis work.

I express my sincere gratitude to **Dr.P.Eswaramoorthi,** Professor of Civil Engineering Department for his essential support, valuable suggestions, and timely guidance for carrying out this research work.

I deem great pride in expressing heartfelt gratitude to **Dr.S.L.Narashimhan, H.O.D,** facilities extended throughout the project.

I express my sincere gratitude to all the **Faculty of Civil Engineering** for their timely suggestions and help rendered in connection with this thesis work.

I express my profound gratefulness to **Prof.V.Annamalai,** Principal for providing the necessary facilities for the successful completion of thesis work.

Last but not the least; I thank one and all those who have rendered help directly or indirectly at various stages of the thesis.

CONTENTS

LIST OF FIGURES

CHAPTER 1
INTRODUCTION

1.1 INTRODUCTION

Masonry is one of the oldest construction materials. It is commonly used around the world for reasons that include availability, functionality and cost. Masonry is used in buildings in load bearing walls, interior and exterior infill walls. In multi-storeyed framed buildings, the masonry infill walls are built after the frame is constructed. The infill walls are not designed to carry gravity loads. The term infilled frame is used to denote a composite structure formed by the combination of a moment resisting frame and infill walls (Fig. 1.1). Besides masonry, the infill walls can be made of concrete. The masonry can be of brick, concrete masonry units, light weight hollow blocks or stones. The primary function of masonry infill wall is to provide an enclosed space or to partition interior space.

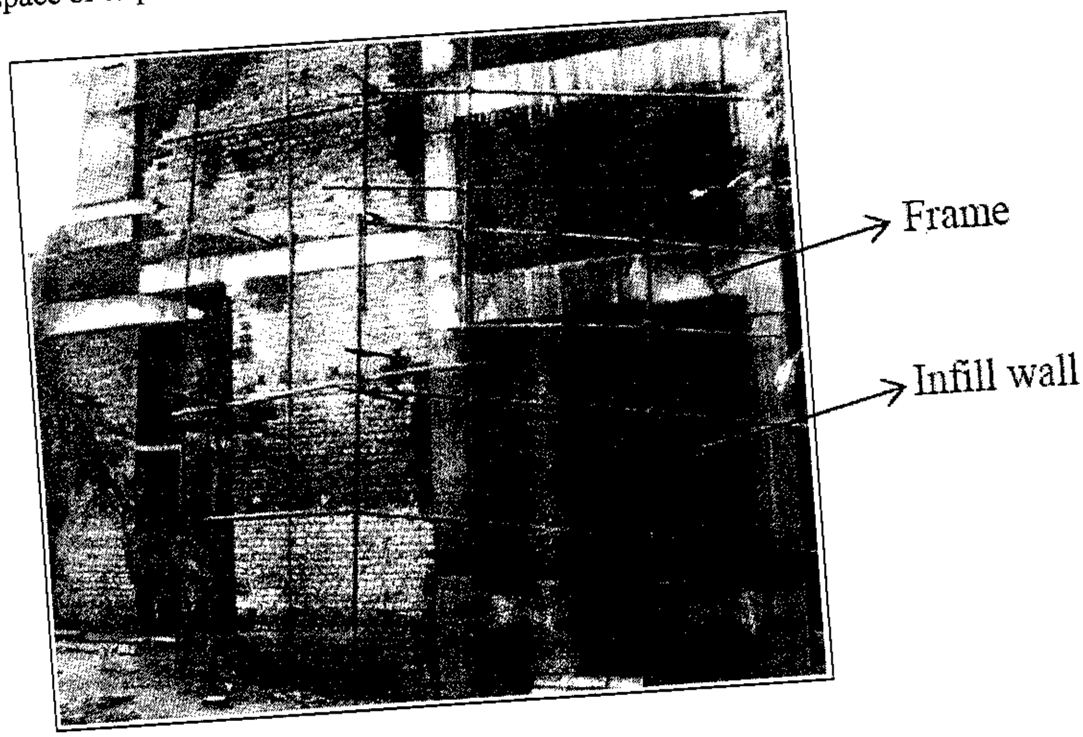


Fig. 1.1 Infilled frame

There are a few ways to classify infilled frames. It can be classified based on the following.

1. Bounding frame: steel or reinforced concrete (RC) frame.
2. Interface condition: integral or non integral. In an integral infilled frame, the infill

wall is connected to the frame by reinforcing bars or shear-connectors. In a non integral infilled frame, these connectors are absent.

3. Type of infill wall: plain masonry, reinforced masonry, reinforced concrete.

Non-integral infill walls are often found in the multi-storeyed buildings in India, where brick infill walls are present within the RC frames.

In multi-storeyed framed buildings, the horizontal loads are resisted by vertical lateral load resisting systems. These systems should ensure adequate strength and stiffness of the building in two orthogonal directions. Systems such as moment resisting frames, diagonal braces, and shear walls are widely used. Infill walls also provide significant lateral strength and stiffness to a building.

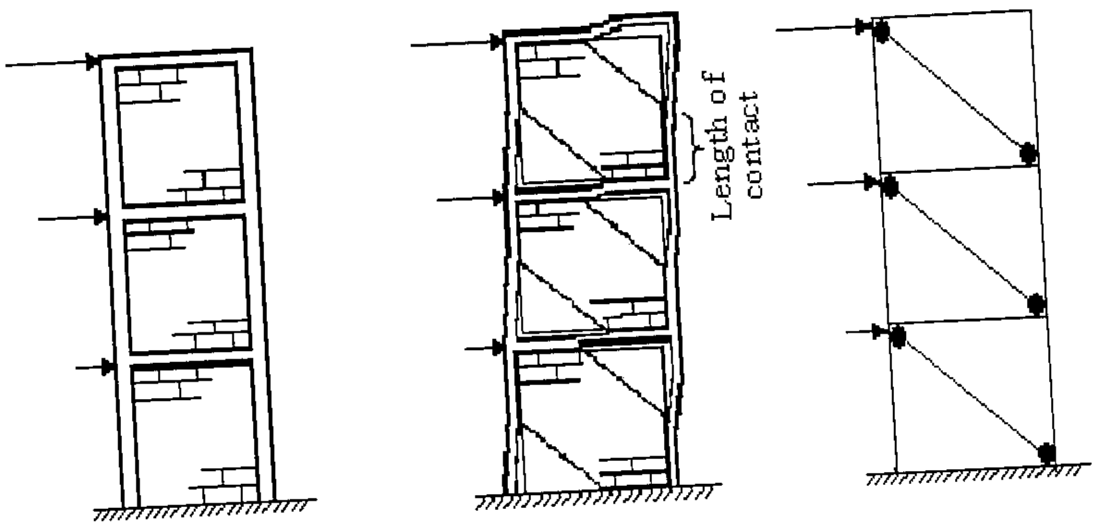
In the analysis and design of framed buildings, traditionally the infill walls are considered only for gravity loads. Any resistance by the walls to lateral loads is gnored. The walls are treated as non-structural elements. But the presence of infill walls in the frames alters the behaviour of the building under lateral loads. Hence, the modelling of infill walls in the seismic analysis of framed buildings is imperative.

Infill walls will interact with the bounding frame members when the building is subjected to seismic loads. Such interaction may or may not be beneficial for the performance of the building. Thus neglect of infill walls in lateral load analysis has two different aspects (negative and positive). Increased stiffness due to the presence of infill walls attracts more forces. This may cause, out of plane failure of the infill wall which will transfer large load to the frame all of a sudden. Hence, the frame members may not able to carry this extra moment and shear forces. Thus ignoring that may lead to an unconservative design of the building. Moreover, irregularly placed infill walls can generate plan and vertical irregularity in the building. Infill walls have been related to catastrophic failures in the past earthquakes, such as the development of soft storeys and the brittle shear failures of columns for partial height infill walls. Consideration of infill walls in the analysis can reveal these irregularities and the subsequent effects (such as soft-storey/weak-storey mechanism, torsional mode of vibration etc.).

But in many cases, ignoring the effect of infill walls in the analysis, works conservatively since extra strength and stiffness in the building. Masonry infill wallhas been used to strengthen existing moment resisting frame buildings. Past earthquake proved that regularly placed masonry infill walls improved the structural performance of several buildings. Thus, the inclusion of infill walls in the analysis makes a difference in the structural behaviour of the building under lateral loads. As many buildings located in seismically active areas in India have masonry infill walls, their vulnerability has become a major concern. The information available to predict the stiffness and strength of an infilled frame subjected to lateral loads, is not adequately compiled. A standard method for modelling an infill walls is yet to find a place in the Indian codes of practice.

1.2 BEHAVIOUR OF NON-INTEGRAL INFILLED FRAMES

The behaviour of a frame is influenced by the frame members as well as the infill walls. Separately, the infill wall is stiff and brittle but the frame is relatively flexible and ductile. The infilled frame combination is stronger and stiffer as compared to the bare frame. The composite action provides the additional strength and stiffness. The typical behaviour of an infilled frame subjected to equivalent static lateral loads is shown in Fig. 1.2. The introduction of masonry infill in RC frames changes the lateral-load transfer mechanism of the structure from predominant frame action to predominant truss action , which is responsible for reduction in bending moments and increase in axial forces in the frame members.



(a) Infilled frame (b) Deformed shape (c) Equivalent strut model

Fig. 1.2 Behaviour of infilled frames

With an initial bond between the frame and infill wall at their interface, the infilled frame behaves like a solid cantilever. After the bond is broken, separation between the frame and infill wall occurs at the tension corners (non loaded corner). Upon separation, the infill wall in a panel rests over certain lengths of the frame member. These lengths are called 'length of contact'. The load is transferred through this portion and behaves like a compression strut as shown in Fig. 1.2c. The struts are attached to the compression corners (loaded corners) acts similar to a diagonal strut of a braced frame.

In the past, different approaches have been adopted to model infill walls, such as strength of materials approach, methods based on equivalent strut, and lately, the finite element method. A non-integral infilled frame subjected to lateral loads can be analysed by replacing the infill walls by equivalent diagonal struts (Holmes, 1961). The modelling of the infill walls by struts is supported by the principal stress contours for the infill wall of a single panel from a finite element analysis (Fig. 1.3). The diagonal strut action is indicated by the principal compressive stress contours.

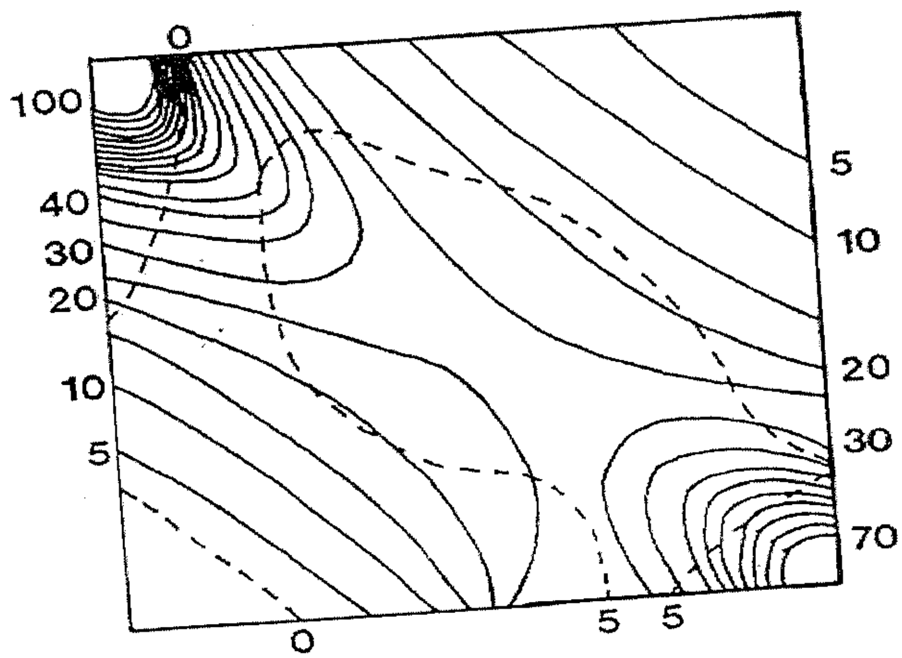


Fig. 1.3 Principal compressive stress contours (Riddington and Smith, 1977)

When the frame is strong enough to cause an infill wall failure first, the observed failure modes are as follows.

1. By local crushing at the compression corners.
2. By shear cracking along the bedding joints of the brickwork.
3. By diagonal compression failure in a slender infill wall.

The diagonal tensile cracking need not be considered as a failure mode, as higher load can be carried beyond the cracking. The failure load depends upon the strength of infill material and to some extend upon the aspect ratio of the infill wall. If he frame is relatively weak, these can be shear or flexural failure of the beams or columns near
the compression corners.

1.3 MODELLING OF INFILL WALLS

The equivalent strut method is convenient for modelling the infill wall in the analysis of a building. Substantial research work has been carried out on the modelling of equivalent struts. The approach based elastic analysis (Smith and Carter, 1969), the approach based plastic analysis (Liauw and Kwan, 1983), and the approach based ultimate load (Saneinejad and Hobbs, 1995) are among them. These analyses aim at calculating the geometric properties and strength of an equivalent strut.

The approach based on elastic analysis is based on initial stiffness of the equivalent strut. Where as the approach based on ultimate load is based on the strength of the equivalent strut. The two approaches of modelling the equivalent strut provide different values of the strut properties. The suitability of the two approaches for different types of analysis of building is to be investigated.

In a linear analysis of a building, the required properties of an equivalent strut are the width, modulus of elasticity and strength. But for a nonlinear analysis of a building such as pushover analysis, in addition to these properties, the axial load versus deformation curve is also required to define the axial hinge property of a strut. A linear axial load versus deformation curve is simple and frequently used. But there is a considerable nonlinearity before the failure of masonry infill wall. The nonlinearity in the infill wall is due to the formation and development of cracks under lateral load and the confinement of the infill wall within the surrounding frame. So a linear model does not represent the material nonlinearity and leads to reduced drift. Also if the struts fail prior to the frame member, there is abrupt termination of the pushover analysis. Hence, it is necessary to incorporate the nonlinearity in the axial load versus deformation curve of a strut.

1.4 OBJECTIVES

The main objectives of the study are as follows.

1. Study of seismic response of various RC Structure.
2. Modelling of infill wall as the equivalent diagonal strut
3. Contribution of stiffness of infill wall to the lateral load resistance
4. Soft Storey effects with the consideration of Stiffness of the storey

1.5 SCOPE OF THE WORK

In the present study, seismic analysis of buildings having Four to fifteen stories were performed using response spectrum method as well as equivalent static load method. The building parameters such as fundamental time period, storey stiffness and seismic responses such as base shear, lateral forces on each floor are determined using the programme written in Matlab. The seismic response of the RC bare frame buildings and infilled buildings with and without open ground storey building are compared. The infill walls are modeled as equivalent diagonal struts. The effect of openings in the infilled wall has not been considered in the present study.

CHAPTER 2
REVIEW OF LITERATURE

2.1 PRELIMINARY REMARKS

There are different approaches to model infill walls, such as strength of material approach methods based on equivalent strut and the finite element method. The literature on equivalent strut method is emphasized for review. Earlier research includes experimental and theoretical evaluation of single storey and multi-bay multistoreyed frames subjected to different types of lateral loads such as static, cyclic and dynamic loads. The available literature is broadly grouped into the following

1. Studies which mainly concentrated on the behaviour of infilled frames
2. Studies which aimed at developing the method of analysis of infilled frames.

2.2 STUDIES OF BEHAVIOUR OF INFILLED FRAMES

In order to determine the lateral strength of infilled frames, **Polyakov (1956)** performed a number of large scale tests with various aspect ratios of infill wall. Parameters investigated include the effects of the type of masonry units, mortar mixes, methods of load application (monotonic or cyclic) and the effect of openings. Based on observation of the infill wall boundary separation, it was suggested that the infilled frame system is equivalent to a braced frame with a compression diagonal brace replacing the infill wall.

Holmes (1961) experimentally studied the strength and stiffness of steel frames infilled with both concrete and masonry. It was suggested to consider the infill wall as an equivalent diagonal strut acting directly between the loaded corners of the frame (Fig. 1.2c). **Holmes (1963)** extended the earlier research to study the effect of vertical loads in infilled frames. A study of the behaviour of unframed mortar panel subjected to diagonal loading was done by **Smith (1962)**. The results of unframed mortar panel were used for the calculation of width of the equivalent strut in infilled frames. **Smith (1966)** studied the influence of the stiffnesses of frame and infill wall, panel aspect ratio and the magnitude of lateral loads on the behaviour of an infilled frame. **Liauw (1970)** pointed out that the contact stresses, slip and separation at the frame-infill wall interface were not modelled in the equivalent strut method proposed by **Smith (1966)**. The theoretical approach used Airy's stress function to determine the stresses and deformations in the infill wall and the frame. In 1972, Liauw introduced an equivalent frame method to model integral infill wall. **Liauw and Lee** (1977) discussed the importance of the connectors in improving the lateral strength and stiffness. A strain energy method was used to get the area of an equivalent strut for a non-integral infill wall.

Rao et al. (1982) presented the influence of various conventional methods of construction on the behaviour of infilled frames. It was recommended that the infill wall and frames should be constructed in a particular order in stages to achieve perfect bond at the interface of frame and infill wall. Tests conducted by **Dawe and Seah** (1989) revealed the effects of various parameters such as the effects of column ties, mortar strength, interface friction and bond, initial gap, openings in infill walls and rigidity of connectors of the bounding frame. Later **Dawe et al. (1989)** extended the study to the dynamic and out of plane behaviour of infilled frames.

Mehrabi et al. (1994, 1996) made an investigation on various parameters such as the strength of infill walls with respect to bounding frame, panel aspect ratio, distribution of vertical loads and lateral load history. It was found that specimens with strong frames and strong infill walls exhibit a better performance than those with weak frames and weak infill walls. **Mosalam et al**. (1997) carried out a series of tests on steel frames with concrete masonry infill walls under pseudo-dynamic loading. The effect of various parameters such as the number of bays, number of storeys, the relative strength of the concrete block to the mortar and the effect of openings were studied. It was found that weak blocks lead to corner crushing whereas stronger blocks lead to shear cracking.

Al-Chaar et al. (2002) investigated the seismic vulnerability of masonry infilled RC frames which were designed for gravity loading alone. Tests were conducted on brick and concrete masonry infilled RC frames (single bay, single storey). The behaviour of two bay, single storey frames was also studied. There are many studies conducted on the infilled frames under cyclic and dynamic loading conditions. **Mallick and Severn** (1968) studied the dynamic characteristics of an infilled frame. Liauw (1979) investigated the effect of shear connectors between the frame and infill wall and effect of openings in the infilled frames under dynamic load. **Brokken and Bertero** (1981) studied the effects of infill walls in seismic performance of RC frames using a series of quasi-static, cyclic and monotonic load tests. It was concluded that infill walls can be effectively used to enhance the seismic performance of reinforced concrete frames in terms of strength and ductility.

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 load. **Govindan et al.** (1985), Govindan et al. (1986) and Govindan (1986) explored the strength, ductility and energy dissipation characteristics of the infilled frame under cyclic loads. Girish et al. (1992) investigated the free vibration characteristics and hysteretic behaviour of reinforced concrete bare and brick infilled frames. **Sobaih and Abdin** (1988) investigated the different parameters such as the presence and continuity of infill walls, infill material and height of the infilled frames under earthquake excitation. **Fardis and Panagiotakos** (1997) found that the infill walls reduce the spectral displacements and forces mainly through their high damping in the first large post cracking excursion. Buonopane and White (1999) found that, under seismic load, infill-frame interaction changes the shear force and bending moment patterns of the frame members as compared to that of the bare frame. **Gnanappa (2004)** studied the performance of multi-bay RC frames with various types of infill walls made of fly ash brick masonry, hollow block masonry and ferrocement panel under cyclic loads.

2.3 STUDIES OF MODELLING OF INFILL WALLS

The composite action of infill wall and frame makes the behaviour of an infilled frame very complex. Researchers attempted to simplify this complexity in the modelling of infill wall. The equivalent strut method was found to be convenient to use in conventional analysis of buildings. There are two approaches in the modelling of equivalent strut. These approaches are reviewed here.

2.3.1 APPROACH BASED ON ELASTIC ANALYSIS

The concept of representing an infill wall as an equivalent compression member was put forward by **Holmes (1961).** It was suggested to model the infill wall as equivalent diagonal strut acting directly between the corners of the frame (Fig. 1.2c). This assumption was known to be a simplification because the infill wall, although acting as a diagonal strut, clearly reacts against each side of the frame over finite lengths extending from the loaded corner. The thickness of the strut was assumed to be the thickness of the infill wall and width to be one third of the diagonal length. Airy's stress function was used to find out the effective width of equivalent strut by Smith (1962). It was found that the strut width depends upon the panel proportions. The width was found to vary from $d/4$ to $d/3$ for different panels. In 1966, Smith related the frame and infill wall stiffnesses with the width and strength of the equivalent strut by conducting a number of tests. 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 with different aspect ratios. This approach of modelling the struts is based on the initial stiffness of a strut.

2.3.2 EFFECTIVE WIDTH OF EQUIVALENT STRUT

The effective width (w) was found to depend on the following variables.

1. Relative stiffness of the infill wall to the frame
2. Instantaneous lateral load, expressed as the diagonal load in the infill wall
3. Aspect ratio of the infilled panel

The relative stiffness of the infill wall to the frame influences the frame and wall interaction. Similar to the beam on elastic foundation formulation, the relative stiffness expressed as a non-dimensional variable ?h as follows.

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

Here,

Es = elastic modulus of the equivalent strut

Ec = elastic modulus of the column in the bounding frame

Ic = moment of inertia of the column

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

h = height of column between centerlines of beams

t = thickness of infill wall

θ = slope of the infill wall diagonal to the horizontal

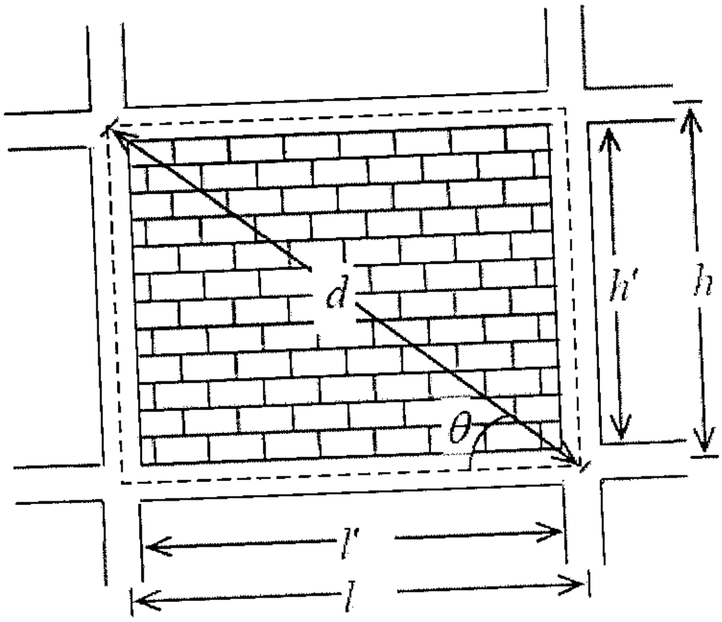


Fig. 2.1 A typical panel of an infilled frame

The width is expressed as w/d with increasing λh (Fig. 2.2). For flexible frames the value of λh will be higher so the width of the strut reduces as length of contact decreases.

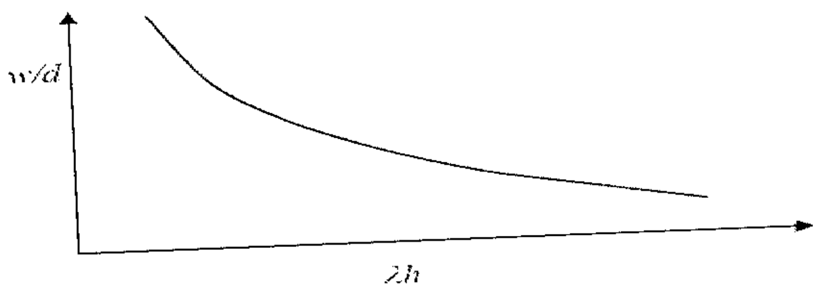


Fig. 2.2 Variation of w/d with λh

Since the length of contact changes with increasing load, the effective width of the equivalent strut (w) decreases. The variation of w/d for various panel aspect ratios was plotted as a function of λh, as shown in Fig. 2.3. The panel aspect ratio is given as l'/h', where l' and h' are the clear length and height of the infill wall, respectively. The reduction of w with increasing load includes the effect of reduction of the modulus of the infill wall.

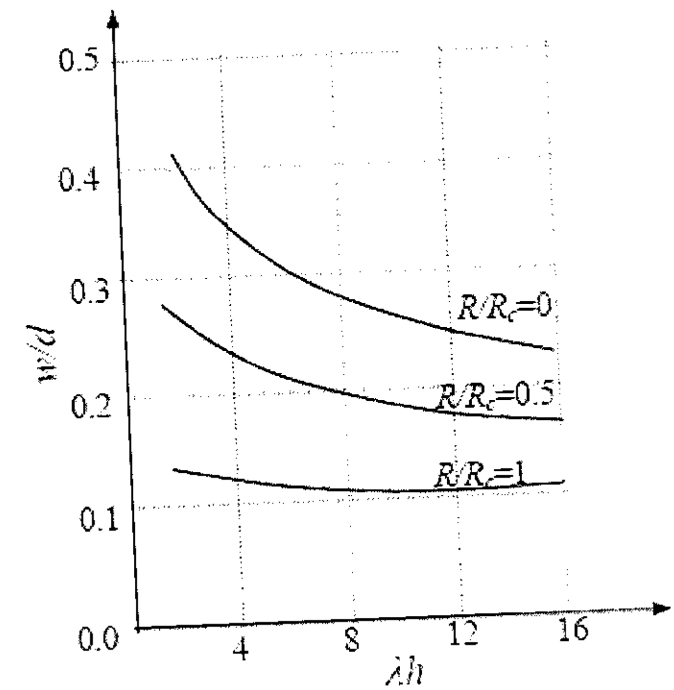


Fig. 2.3 Variation of w/d (l' : h' = 1 : 1)

(Smith and Carter, 1969)

Here, R is the instantaneous diagonal load and Rc is the diagonal load at corner crushing failure. These curves were developed for concrete infill walls. It was recommended that the above curves could be used for masonry infill wall. From the curves given by Smith and Carter (1969), the following relationship was derived using a regression analysis (Ramesh, 2003).

2.3.3 STRENGTH OF EQUIVALENT STRUT

The strength of the equivalent strut is governed by the lower of the failure loads corresponding to the following failure modes (Fig. 2.4).

1. Local crushing of the infill wall at one of the loaded corners
2. 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. The tensile cracking can be viewed as a serviceability limit state.

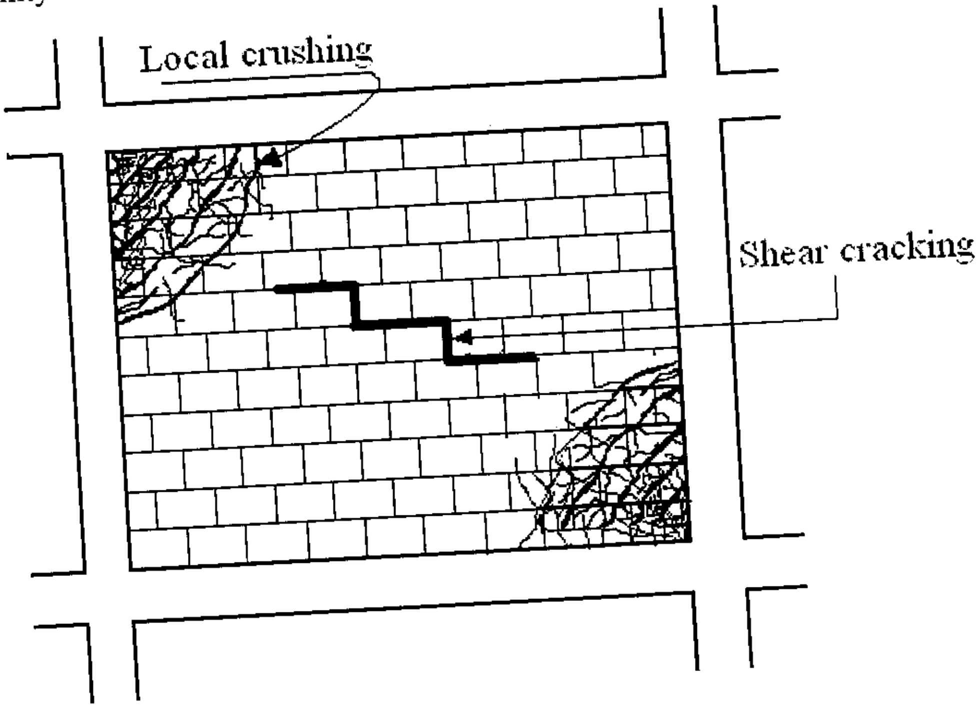


Fig. 2.4 Modes of infill wall failure

Al-Chaar (2002) proposed an eccentric equivalent strut (Fig. 2.6) to model the masonry infill wall. The infill wall forces were assumed to be resisted by columns. This helps to obtain the extra shear and bending moment in the columns in the presence of infill walls. This eccentric strut is pin connected to the column at a distance *le* from the face of the beam.

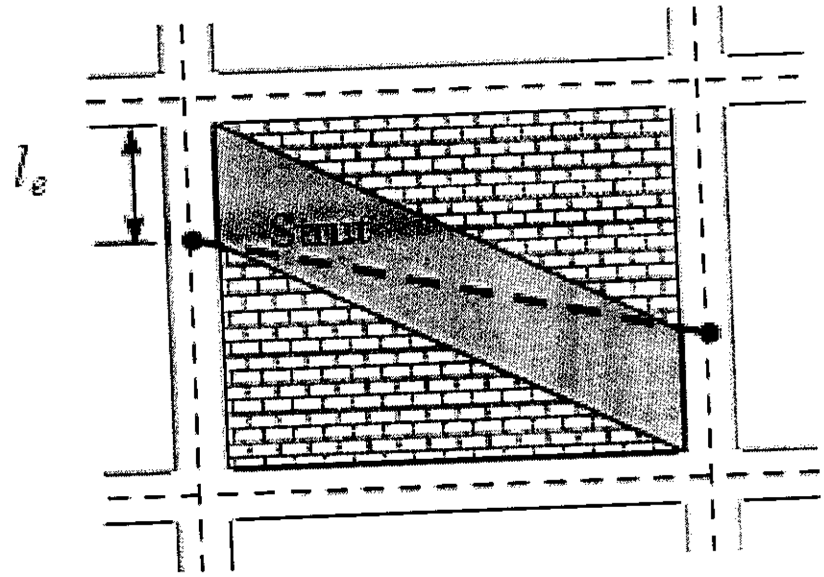


Fig. 2.6 Position of eccentric strut (Al-Chaar, 2002)

This distance *le* is calculated using the strut width (w) by the following equation.

$$le=(w/Cos\theta) \tag{2.2}$$

Here w is calculated based on Eq 2.6.

A three strut model (Fig. 2.7) was suggested for modelling infill wall by **El-Dakhakhni (2002)** and **El-Dakhakhni et al. (2003)**. This model found to be a better representation as it able to model the shear force attracted to the column more realistically. Modelling of three struts is little involved than the simpler single strut. The width of the off diagonal struts are taken as half of the of diagonal strut width.

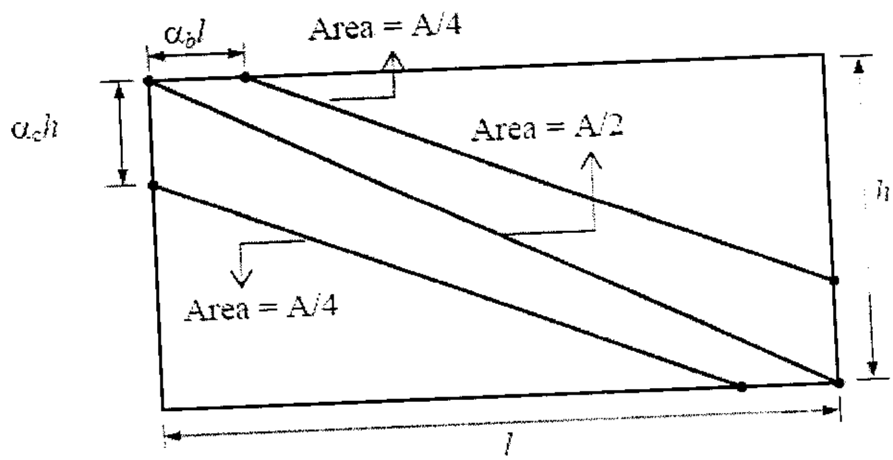


Fig. 2.7 Three strut model (El-Dakhakhni et al., 2003)

Paulay and Priestley (1992) suggested treating the infill walls as diagonal bracing members connected by pins to the frame members. It was suggested to calculate the stiffness and hence natural period of the infilled frame by considering the width of the strut to be 1/4th of the diagonal length.

2.4 SUMMARY

A review of the behaviour of infill walls under lateral loads was presented. It was found that there are different methods to model infill walls such as approach based on elastic analysis, plastic analysis, ultimate load and finite element analysis. A brief review of modelling of infill wall as equivalent diagonal struts was discussed. A frame with infill walls can be analysed for lateral loads with replacing the infill walls by equivalent diagonal struts. The different approaches provide different values of the strut properties.

From the review of literature it was found that, the presence of infill affects the distribution of lateral load in the frames of building because of the stiffness of some of the frames. The distribution of lateral forces in the frames of the building basically depends upon the center of rigidity of the building and the resultant of the applied lateral loads. If both nearly coincide, distribution of the lateral load remains straightforward i.e. in the ratio of their relative stiffness. If it is not the case, large torsional forces are introduced in the building. These types of structures can be analysed on the basis of 3D analysis of building after considering the increased stiffness of the infilled frames.

The study of the interaction of the infill with frames has been attempted by using sophisticated analysis like finite element analysis or theory of elasticity. But due to uncertainty in defining the interface conditions between the infilled with the frames, an approximate analysis method may be better acceptable. One of the most common approximation of infilled walls is on the basis of equivalent diagonal strut i.e. the system is modeled as a braced frame and the infill walls as web element. The main problem in this approach is to find the effective width for the equivalent diagonal strut. Various investigators have suggested different values of width of equivalent diagonal strut.

Originally proposed by **Polyakov(1956)** and subsequently developed by many investigators, the width of strut depends on the length of contact between the wall and the column (ah), and between the wall and beams (al). **Stafford Smith (1966)** developed the formulations for ah and al .The infill parameters are effective width, elastic modulus and strength were calculated using the method recommended by Smith, the length of the strut is given by the diagonal distance (L_d) of the panel and thickness (t) is given by the thickness of the infill wall. Parameters ah and al are given below are used to calculate the effective width.

Here,

$$ah = \pi /2 \, [4 \, E_f \, Ic \, h \;/\; E_m \, t \sin(2\theta)]^{1/4}$$

$$al = \pi \, [4 \, E_f \, I_b \, l \;/\; E_m \, t \sin(2\theta)]^{1/4}$$

The Stiffness of the infill due to diagonal strut is given by,

$$K_{infill} = (A_{strut} \; E_i \; Cos^2\theta \,/\, L_d)$$

$$A_{strut} = w \, t$$

$$w = 1/2 \, \{ah^2 + al^2\}^{1/2}$$

Where, E_m and E_f are elastic moduli of wall and frame material respectively; t, h and l are respectively the thickness, height and length of the infill panel. The angle θ is defined as, $\theta = \tan^{-}(h/l)$.

Hendry (1998) has proposed recommended a width of the diagonal strut equal to one-third of the diagonal panel, whereas New Zealand Code (NZS 4230) specifies a width equal to one quarter of its length.

CHAPTER 3

MODELLING OF INFILLWALLS

3.1 INTRODUCTION

The modelling of infill walls in the seismic analysis of framed buildings is imperative. The behaviour of infilled frames under lateral loads can be estimated by two levels of modelling. The estimation at the element level, based essentially on the finite element method, requires modelling of the masonry blocks and the mortar joints as link elements between them. However, these models are complex and numerically intensive. In the member level, a macro model can be substituted for the micro model without substantial loss in accuracy, but with significant gain in computational efficiency. In a macro model, the infill walls are replaced by equivalent diagonal struts (Fig. 1.2c). The equivalent strut method is convenient for modelling the infill wall in the analysis of a building.

Substantial research work has been done on the equivalent strut method. There are two approaches to model the equivalent struts. One approach is based on an initial stiffness of the infill wall (Smith and Carter, 1969). The other one is based on ultimate load of the infill wall (Saneinejad and Hobbs, 1995).

3.2 MODELLING OF EQUIVALENT STRUTS

In a linear structural analysis, the required properties of an equivalent strut are the effective width, thickness, length and elastic modulus. The thickness (t) is assumed to be same as that of the infill wall. The length (L_d) is the diagonal length of the frame. The remaining properties to be determined are the effective width (w) and elastic modulus (Es) of the equivalent strut. The simplest form w and Es are taken equal to $L_d/3$ or $L_d/4$ and Em (modulus of masonry), respectively. Originally proposed by Polyakov(1956) and subsequently developed by many investigators, the width of strut depends on the length of contact between the wall and the column (ah), and between the wall and beams (al). The proposed range of contact length is between one-fourth and one-tenth of the length of panel. Stafford Smith (1966) developed the formulations for ah and al.The approach proposed by Smith and Carter (1969) is the basis for the conventional linear analyses of infilled frames (Drysdale et al. (1994), FEMA 273). Since this was developed for an elastic analysis of the frame and wall interaction, this approach is subsequently referred to as the elastic analysis (EA) approach. The EA approach is based on initial stiffness of the equivalent strut. On the contrary, the approach proposed by Saneinejad and Hobbs (1995) is based on the ultimate load of the infill wall, which is beyond the linear elastic range of the wall. Henceforth, this approach is referred to as ultimate load (UL) approach. The UL approach is based on the strength of the equivalent strut.

3.3 ELASTIC ANALYSIS APPROACH

The effective width, elastic modulus and strength of the equivalent strut were calculated as follows.

3.3.1 EFFECTIVE WIDTH OF EQUIVALENT STRUT

The simplest form of effective width of diagonal strut is taken as $L_d/3$ or $L_d/4$. But actually, effective width of strut depends on the length of contact between the wall and the column (ah), and between the wall and beams (al). Stafford Smith (1966) developed the formulations for ah and al.

Here,

$$ah = \pi/2\,[4\,E_f\,Ic\,h\;/\;E_m\,t\,\sin(2\theta)]^{1/4}$$

$$al = \pi\,[4\,E_f\,I_b\,l\;/\;E_m\,t\,\sin(2\theta)]^{1/4}$$

$$w = 1/2\,\{ah^2 + al^2\}^{1/2}$$

3.3.2 ELASTIC MODULUS OF EQUIVALENT STRUT

The elastic modulus of the equivalent strut *Es* can be equated to *Em*, the elastic modulus of the masonry. Krishnakedar (2004) conducted a series of experiments on masonry prisms on various types of bricks in India. Following range of values for *Em* were obtained.

Em = 350 to 800 MPa for table moulded bricks

Em = 2500 to 5000 MPa for wire cut bricks

In absence of information on *Em* in IS 1905: 1987, the IBC (2000) guidelines can be followed. As per IBC (2000), *Em* = 750*f'm*, where *f'm* is the compressive strength of the masonry.

3.3.3 STIFFNESS OF EQUIVALENT STRUT

The Stiffness of the infill due to diagonal strut is given by,

$$K_{infill} = (A_{strut} \ E_i \ Cos^2\theta / L_d)$$

$$A_{strut} = w\,t$$

$$w = 1/2 \{ah^2 + al^2\}^{1/2}$$

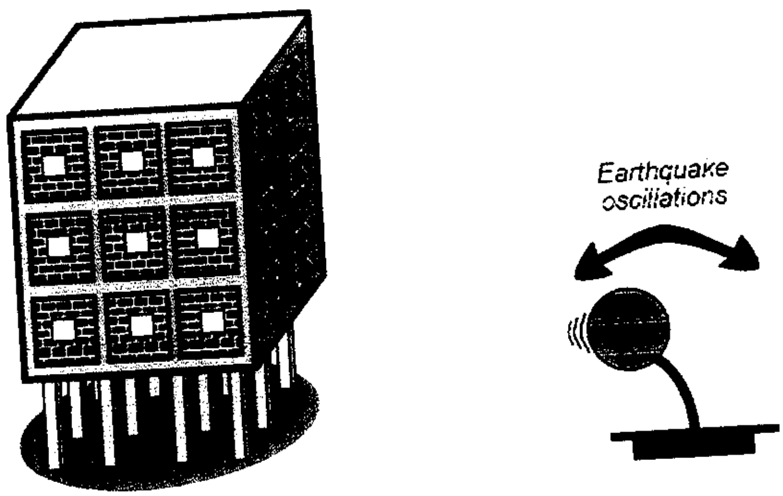
Where, A_{strut} ,E_m and E_f are area of strut, elastic moduli of wall and frame material respectively; t, h and l are respectively the thickness, height and length of the infill panel. The angle θ is defined as, $\theta = \tan^{-}(h/l)$.

CHAPTER 4
OPEN GROUND STOREY

4.1 INTRODUCTION

In general, multi-storeyed buildings in metropolitan cities require open taller first storey for parking of vehicles and/or for retail shopping, large space for meeting room or a banking hall owing to lack of horizontal space and high cost.

An open ground storey building, having *only columns* in the ground storey and *both partition walls and columns* in the upper storeys,



a)Open ground storey

b) Inverted pendulum

Fig 4.1 Behaviour of open ground storey building

4.2 SOFT AND WEAK STOREY

An open ground storey have two distinct characteristics, namely:

(a) It is relatively *flexible* in the ground storey, *i.e.*, the relative horizontal displacement it undergoes in the ground storey is much larger than what each of the storeys above it does. This flexible ground storey is also called *soft storey*.

(b) It is relatively *weak* in ground storey, *i.e.*, the total horizontal earthquake force it can carry in the ground storey is significantly smaller than what each of the storeys above it can carry. Thus, the open ground storey may also be a *weak storey*. Often, open ground storey buildings are called *soft storey buildings*, even though their ground storey may be *soft and weak*. Generally, the soft or weak storey usually exists at the ground storey level, but it could be at any other storey level too.

4.3 SOFT STOREY FAILURE

Due to this functional requirements, the first storey has lesser strength and stiffness as compared to upper stories, which are stiffened by masonry infill walls. This characteristic of building construction creates "weak" or "soft" storey problems in multi-storey buildings. Increased flexibility of first storey results in extreme deflections, which in turn, leads to concentration of forces at the second storey connections accompanied by large plastic deformations. In addition, most of the energy developed during earthquake is dissipated by the columns of the soft stories. In this process the plastic hinges are formed at the ends of the columns, which transform the soft storey into a mechanism. In such case the collapse is unavoidable. Therefore, the soft stories deserve a special consideration in analysis and design.

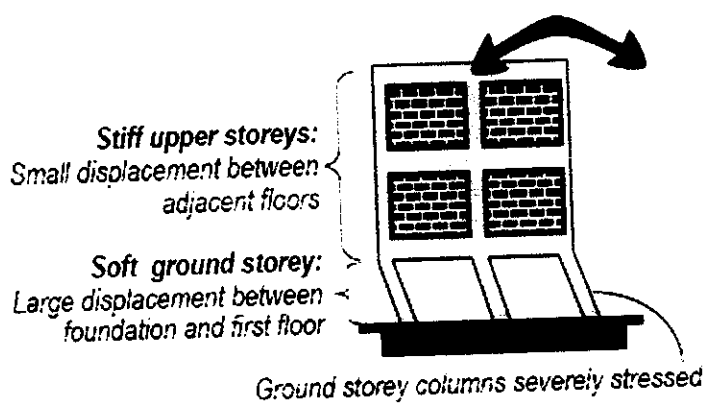


Fig 4.2 Failure of soft storey

It is recognized that this type of failure results from the combination of several other unfavorable reasons, such as torsion, excessive mass on upper floors, P-delta effects and lack of ductility in the bottom storey.

4.4 IRREGULARITY IN STRENGTH AND STIFFNESS

A "weak" storey is defined as one in which the storey's latreral strength is less than 80 percent of that in the storey above. The storey's lateral strength is the total strength of all seismic resisting elements sharing the storey shear for the direction under consideration i.e. the shear capacity of the column or shear walls or the horizontal component of the axial capacity of the diagonal braces. The deficiency that usually makes a storey weak is inadequate strength of frame columns. A "soft storey is one in which the lateral stiffness is less than 70% of that in the storey immediately above, or less than 80% of the combined stiffness of the three stories above".

i.e. Soft Storey when

$$K_i<0.7k_{i+1} \text{ or } k_i<0.8\{1/3(k_{i+1} + k_{i+2} + k_{i+3})\}$$

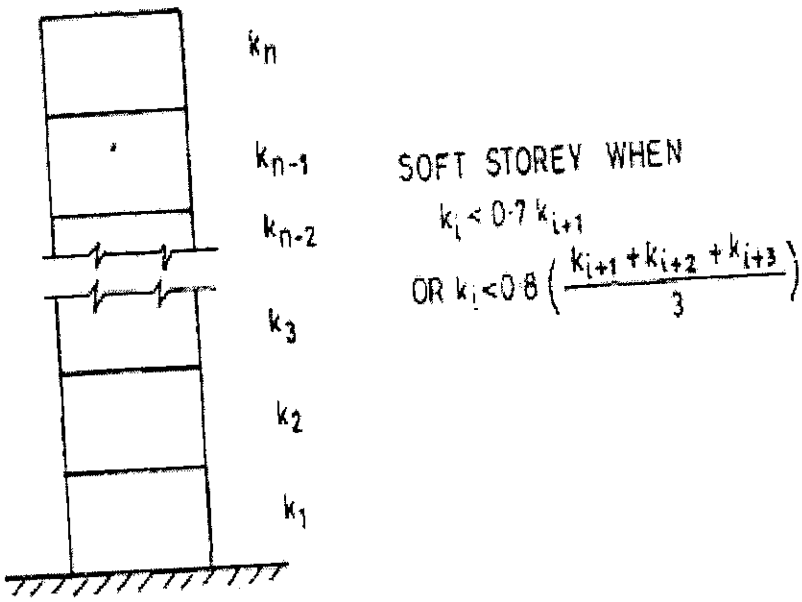
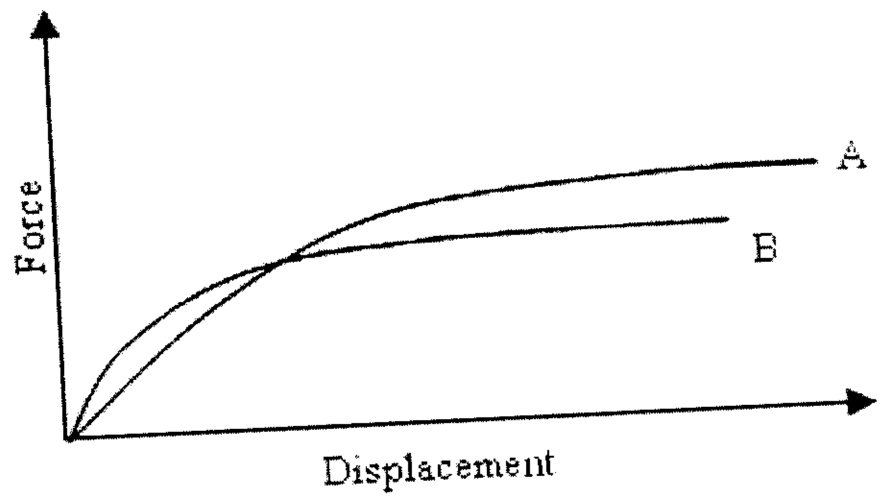


Fig 4.3 Soft storey based on stiffness

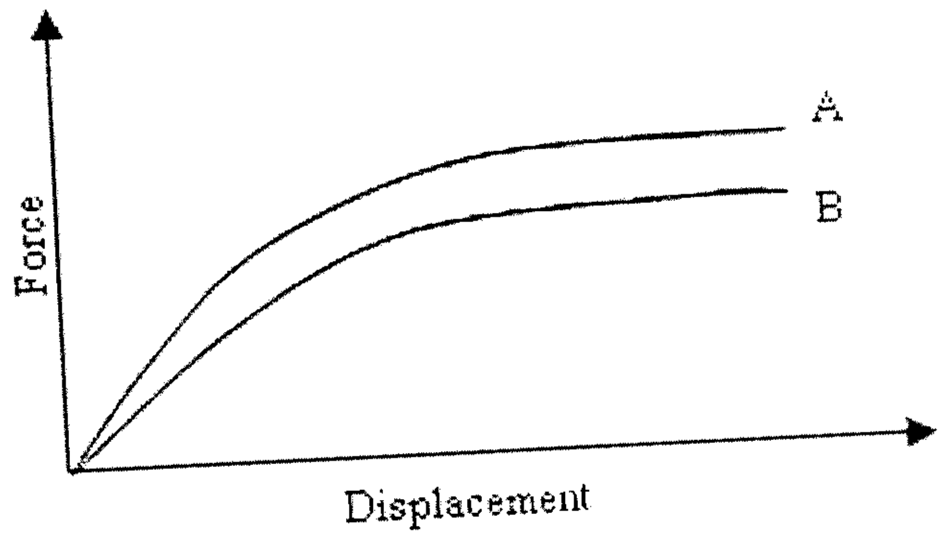
The soft storey concept has technical and functional advantage over the conventional construction. First, is the reduction in spectral acceleration and base shear due to increase of the natural time period of vibration of structure as in a base isolated structure. However, the price of this force reduction is paid in the form of an increase in structural displacement and inter-storey drift, thus entailing a significant P-delta effect, which is a threat to the stability of the structure.

4.5 WEAK STOREY FAILURE

There is a clear distinction between stiffness and strength. Stiffness is a force needed to cause a unit displacement and is given by the slope of the force-displacement relationship, whereas strength is the maximum force that a system can take. Soft storey refers to stiffness and weak storey refers to strength. Usually, a soft storey may also be a weak storey.



Structure A has higher strength and lower stiffness as compared to structure B



Structure A has higher strength and higher stiffness as compared to structure B

Fig 4.5 Stiffness versus Strength

4.6 EARTHQUAKE BEHAVIOUR

Open ground storey buildings have consistently shown poor performance during past earthquakes across the world (for example during *1999 Turkey, 1999 Taiwan* and *2003 Algeria* earthquakes); such buildings are *extremely* vulnerable under earthquake shaking.

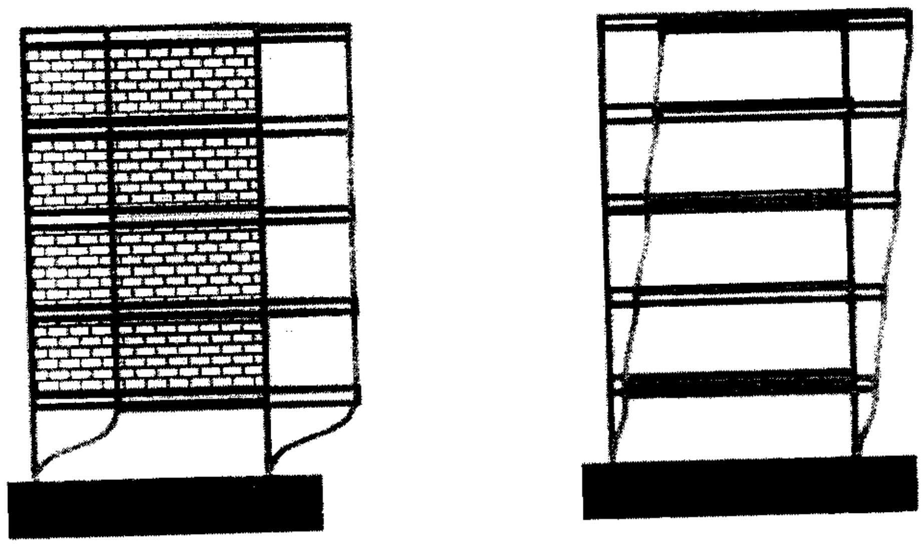


Figure 4.6 - Soft-storey is subject to severe deformation demands during seismic shaking (From Murty et al, 2002)

4.6.1 THE PROBLEM

Open ground storey buildings are inherently poor systems with sudden drop in stiffness and strength in the ground storey. In the current practice, stiff masonry walls are neglected and only bare frames are considered in design calculations. Thus, the inverted pendulum effect is not captured in design.

4.7 IMPROVED DESIGN STRATEGIES

After the collapses of RC buildings in 2001 Bhuj earthquake, the Indian Seismic Code IS:1893 (Part 1) - 2002 has included special design provisions related to soft storey buildings. Firstly, it specifies when a building should be considered as a soft and a weak storey building. Secondly, it specifies higher design forces for the soft storey as compared to the rest of the structure. The Code suggests that the forces in the columns, beams and shear walls (if any) under the action of seismic loads specified in the code, may be obtained by considering the bare frame building (without any infills). However, beams and columns in the open ground storey are required to be designed for 2.5 times the forces obtained from this bare frame analysis. For all new RC frame buildings, the best option is to avoid such sudden and large decrease in stiffness and/or strength in any storey; it would be ideal to build walls (either masonry or RC walls) in the ground storey also (Figure 5). Designers can avoid dangerous effects of flexible and weak ground storeys by ensuring that too many walls are not discontinued in the ground storey, i.e., the drop in stiffness and strength in the ground storey level is not abrupt due to the absence of infill walls The existing open ground storey buildings need to be strengthened suitably so as to prevent them from collapsing during strong earthquake shaking.

CHAPTER 5

SEISMIC ANALYSIS OF STRUCTURE

5.1 SEISMIC METHODS OF ANALYSIS:

Once the structural model is selected, it is possible to perform analysis to determine the seismically induced forces in the structures. There are different methods of analysis which provide different degrees of accuracy. The analysis process can be categorized on the basis of three factors: the type of externally applied loads, the behaviour of structure/structural materials, and the type of structural model selected (Fig 5.1).Based on the type of external action and behaviour of the structure, the analysis can be further classified as linear static analysis, linear dynamic analysis, non-linear static analysis or non-linear dynamic analysis.

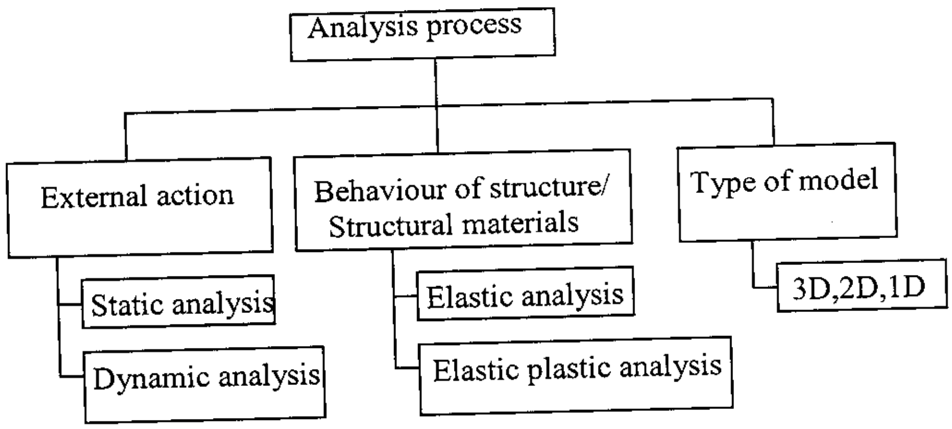


FIG:5.1 Methods of analysis process

5.1.1 LINEAR STATIC ANALYSIS

This method is mainly suitable for regular buildings which respond primarily within the elastic range. Equivalent static load procedure or seismic coefficient method is specified in most of the design codes. A set of static loads are calculated based on the fundamental period of the structure and the seismic conditions at site (zone, importance factor, soil type). The loads are distributed along the height of the building in a manner consistent with the first mode shape.

Higher mode effects are approximated by additional fraction of load applied at the roof level in many of the seismic codes. This analysis is normally performed either by manual calculations or using any analysis software.

5.1.2 LINEAR DYNAMIC ANALYSIS

Linear dynamic analysis can be performed in two ways either by mode superposition method or response spectrum method and elastic time history method. This analysis will produce the effect of the higher modes of vibration and the actual distribution of forces in the elastic range in a better way. They represent an improvement over linear static analysis. The significant difference between linear static and dynamic analysis is the level of force and their distribution along the height of the structure.

5.1.3 NON-LINEAR STATIC ANALYSIS

Non-linear static analysis is an improvement over the linear static or dynamic analysis in the sense that it allows the inelastic behaviour of the structure. The methods still assume a set of static incremental lateral load over the height of structure. The method is relatively simple to be implemented, and provides information on the strength, deformation and ductility of the structure and the distribution of demands. This permits to identify critical members likely to reach limit states during earthquake, for which attention should be given during the design and detailing process. But this method contains many limited assumptions, which neglect the variation of loading patterns, the influence of higher modes, and the effect of resonance. This method, under the name push over analysis has acquired a great deal of popularity nowadays and in spite of these deficiencies this method provides reasonable estimation of the global deformation capacity, especially for structures which primarily respond according to first mode.

5.1.4 NON-LINEAR DYNAMIC ANALYSIS

A non-linear dynamic analysis or inelastic time history analysis is the only method to describe the actual behaviour of the structure during an earthquake. The method is base on the direct numerical integration of the motion differential equations by considering the elasto-plastic deformation of the structural element. This method captures the effect of amplification due to resonance, the variation of displacements at diverse levels of a frame, an increase of motion duration and a tendency of regularization of movements as far as the level increases from bottom to top.

5.2 CODE-BASED PROCEDURE FOR SEISMIC ANALYSIS

Main features of seismic method of analysis base on Indian Standard 1893 (Part 1):2002 are described as follows:

5.2.1 SEISMIC CO-EFFICEINT METHOD

This is the simplest method of analysis and requires less computational effort because the forces depend on the code based fundamental period of structures with some empirical modifier. The design base shear shall be first computed as a whole, then be distributed along the height of the buildings based on simple formulas appropriate for buildings with regular distribution of mass and stiffness. The design lateral force obtained as each floor level shall then be distributed to individual lateral load resisting elements depending upon on floor diaphragm action. In case of rigid diaphragm (reinforced concrete monolithic slab-beam floors or those consisting of pre-fabricated/ precast elements with topping reinforced screed can be taken as rigid diaphragm) action, the total shear in any horizontal plane shall be distributed to the various elements of lateral force resisting system on the basis of relative rigidity (Clause 7.7.2 of IS1893 (Part 1):2002). The following are the major steps for determining the forces by equivalent static procedures.

DETERMINATION OF BASE SHEAR

The total design lateral force or design base shear along any principal direction shall be determined by the following expression, Clause 7.5 of IS1893 (Part 1):2002.

$$V_B = A_h W$$

where,

A_h - Design horizontal seismic co-efficient for a structure.

W - Seismic weight of the building

Seismic weight of a building is the sum of the seismic weight of all the floors. The seismic weight of each floor is its full dead load plus percentage of imposed load as given Table 8 of IS1893 (Part 1):2002 as per clause 7.3.1. Imposed load on roof level need not be considered. The basic reasons for considering the percentage of live load as specified in Table 8 is that only a part of maximum live load will probably be present at the time of earthquake.

Percentage of Imposed Load to be considered in Seismic weight calculation

(Clause 7.3.1)

Imposed Uniformly Distributed Floor Loads (kN / m2)	Percentage of Imposed Load %
Up to and including 3.0	25
Above 3.0	50

Design Horizontal Seismic Coefficient (Ah) of a structure for each mode of vibration is determined by the equation given below:

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

Provided that for any structure with T = 0.1 s, the value of Ah will not be taken less than Z/2 whatever the value of I/R

where

Z = Zone factor

I = Importance Factor

R = Response Reduction Factor

Sa/g = Average Response Acceleration Coefficient

ZONE FACTOR (Z)

The our country is classified into four seismic zones for the purpose of determining the seismic forces and is given in the code.

Zone Factor (Z) is given in Table 2 in IS1893 (Part 1):2002 as per clause 6.4.2 for the Maximum Considered Earthquake (MCE) and service life of the structure in a zone. In Eqn. the factor 2 in the denominator of Z is used to reduce the

Maximum Considered Earthquake zone factor to the factor for Design Basis Earthquake (DBE). The maximum intensity is fixed in such a way that the lifeline/critical structures will remain functional and there is low probability of collapse for structures with the provisions provided in the code even with for an event of occurrence of earthquake with higher intensity

Zone factor ,Z

(Clause 6.4.2)

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very severe
Z	0.10	0.16	0.24	0.36

IMPORTANCE FACTOR (I)

Importance Factor (I) given in Table 6 in IS1893 (Part 1):2002 as per clause 6.4.2 is depending upon the functional use of the structure. This value is characterised by hazardous consequences of failure of the structure, post-earthquake functional needs historical value or economical importance

Importance factor,I

(Clause 6.4.2)

Sl.no	**Structure**	**Importance Factor**
(1)	(2)	(3)
i)	Important service and community buildings, such as hospitals, schools, monumental structures, emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings, large community halls like cinema, assembly halls, and subway stations, power stations	1.5
ii)	All other buildings	1.0

RESPONSE REDUCTION FACTOR (R)

Response reduction factor (R) depends on the seismic damage performance of the structure for ductile or brittle deformation. However the ratio I/R shall not be greater than 1.0. The values of R for the buildings are given in Table 7 in IS1893 (Part 1):2002 as per Clause 6.4.2.

Response Reduction Factor , R for building systems

(Clause 6.4.2)

Sl.no	**Lateral load resisting system**	**R**
(1)	**(2)**	**(3)**
	Building frame systems	
i)	Ordinary RC moment resisting frame (OMRF)	3.0
ii)	Special moment resisting frame(SMRF)	5.0
iii)	Steel frame with a) Concrete braces	4.0
	b) Eccentric braces	5.0
iv)	Steel moment resisting frame designed as per SP 6	5.0
	Building with shear walls	
v)	Load bearing masonry wall buildings a) Un reinforced	1.5
	b) Reinforced with horizontal RC bands	2.5
	c) Reinforced with horizontal RC bands and vertical bars at corners of rooms and jambs of openings	3.5
vi)	Ordinary reinforced concrete shear walls	3.0
vii)	Ductile shear walls	4.0
	Building with dual system	
viii)	Ordinary shear wall with OMRF	3.0
ix)	Ordinary shear wall with SMRF	4.0
x)	Ductile shear wall with OMRF	4.5
xi)	Ductile shear wall with SMRF	5.0

AVERAGE RESPONSE ACCELERATION CO-EFFICIENT

The *average response acceleration coefficient,* Sa/g is based on the

- appropriate natural periods,
- type of soil and
- damping of the structure.

FUNDAMENTAL TIMEPERIOD (Ta)

Although dynamic analysis is the preferred method for the estimation of the fundamental period, the value may be unrealistically large when ignoring filler walls resulting in low values for the seismic design force. Hence empirical relationships for the estimation of fundamental period are suggested in codes.

The fundamental period may be assumed equal to the number of storeys divided by ten.

$$T= 0.1N$$

where,

N = number of stories

Bureau of Indian Standards IS 1893 - 2002 specifies some empirical expressions for finding the approximate fundamental natural period of vibration in R.C. frame Building.

For a moment-resisting frame building without brick infill panels, the approximate fundamental natural period of vibration (Ta) in seconds may be estimated by the empirical expression

$$Ta = \mathbf{0.075}\ h^{0.75} \quad \text{for RC frame building}$$

$$Ta = \mathbf{0.085}\ h^{0.75} \quad \text{for steel frame building}$$

For all other buildings including moment-resisting frame building with brick infill panels the approximate fundamental natural period of the vibration (Ta) in seconds may be estimated by using the empirical expression

$$Ta = \frac{0.09h}{\sqrt{d}}$$

where,

h - Height of the building in meters. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected.

d - Base dimension of the building at the plinth level, in meters, along the considered direction of the lateral force.

The values of spectral acceleration coefficient for different natural periods has be taken from *Response spectrum.* A response spectrum is defined as a curve which shows the peak response of a single degree freedom oscillator (having a certain damping) to a given input ground motion. It shows the variation between peak acceleration as a function of acceleration due to gravity (Sa/g) on the Y axis and the natural period (T) on the X axis.

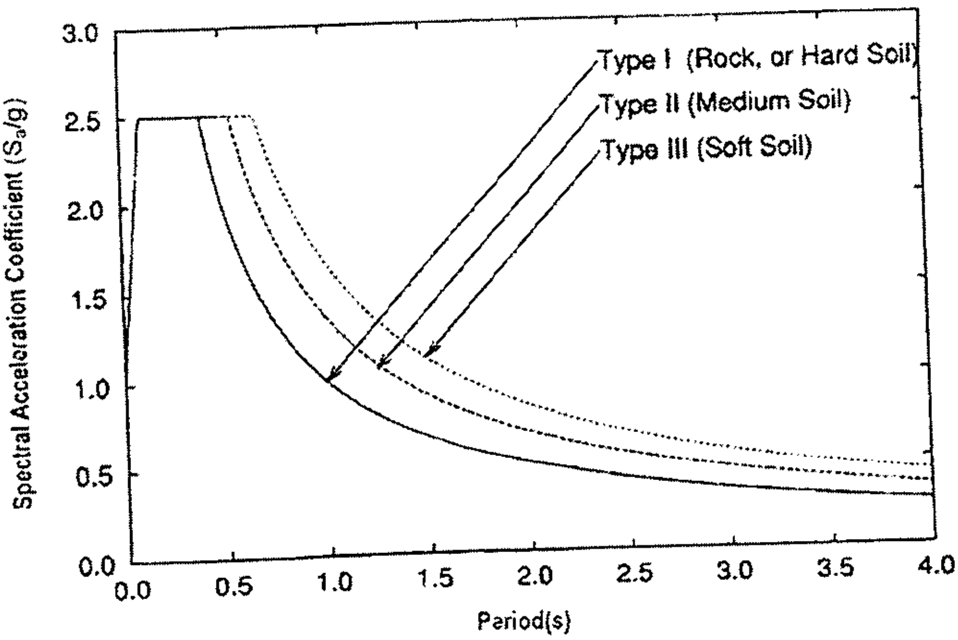


Fig: 5.2 Response spectra for 5 percent damping

FOR ROCKY OR HARD SOIL SITES

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T & 0.00 = T = 0.10 \\ 2.50 & 0.10 = T = 0.40 \\ 1.00/T & 0.40 = T = 4.00 \end{cases}$$

FOR MEDIUM SOIL SITES

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T & 0.00 = T = 0.10 \\ 2.50 & 0.10 = T = 0.55 \\ 1.36/T & 0.55 = T = 4.00 \end{cases}$$

FOR SOFT SOIL SITES

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T & 0.00 = T = 0.10 \\ 2.50 & 0.10 = T = 0.65 \\ 1.67/T & 0.65 = T = 4.00 \end{cases}$$

The figure shows the proposed 5 percent spectra for rocky and soil sites and the Table 6.5 in IS1893 (Part 1):2002 as per clause 6.4.2 gives the multiplying factors for obtaining spectral values for different percentage of damping.

Multiplying factors for obtaining values for other damping

(Clause 6.4.2)

Damping percent	0	2	5	7	10	15	20	25	30
Factors	3.20	1.40	1.00	0.90	0.80	0.70	0.60	0.55	0.50

DISTRIBUTION OF DESIGN LATERAL FORCE

The computed base shear is now distributed along the height of the building. The shear force, at any level, depends on the mass at that level. IS1893 (Part 1):2002 uses parabolic distribution of lateral force along the height of the building as per the following expression.

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

Where,

Q_i – design lateral force at floor i.

W_i – Seismic weight of floor i.

h_i – Height of floor i measured from base.

n – Number of storeys in the building is the number if levels at which the masses are located.

5.2.2 RESPONSE SPECTRUM ANALYSIS

Dynamic analysis is carried out either by modal analysis procedure or dynamic analysis procedure (Clause 7.8 of IS 1893 (Part 1):2002).Dynamic analysis is recommended to obtain the design seismic force, and its distribution levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

a) Regular buildings – those greater than 40 m in height in zones IV and V, and those greater than 90 m in height in zone II, and III.
b) All Irregular buildings(Tables 4,5) and all framed buildings higher than 12 m in zones IV and V, those greater than 40 m in height in zones II and III. For irregular buildings, less than 40 m in height located in zones II and III, dynamic analysis, though not mandatory, shall be preferred.

The purpose of dynamic analysis is to obtain the design seismic forces, with its distribution to different levels along the height of the building and to various lateral load resisting elements similar to the seismic co-efficient method. The procedure of dynamic analysis described in Code is valid only for regular type of buildings, which are almost symmetrical in plan and elevation about the axis having uniform distribution of the lateral load resisting elements. It is assumed that all masses are lumped at the storey level and only sway displacement is permitted in each storey. The dynamic analysis procedure for regular type building is divided into several distinctive steps, which are as follows:

Using the eigen-values and eigen-vectors determined by the modal analysis for the multi storey shear frame, modal participation factors and effective masses for all the modes are calculated.

Modal participation factor (P_k)

Modal participation factor of mode k of vibration is the amount by which the mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions.

$$P_k = \frac{\sum_{i=1}^{N} W_i \phi_{ik}}{\sum_{i=1}^{N} W_i [\phi_{ik}]^2}$$

Modal mass, M_k

Modal mass of a structure subjected to horizontal or vertical, as the case may be, ground motion is a part of the total seismic mass of the structure that is effective in mode k of vibration.

$$M_k = \frac{\left[\sum_{i=1}^{N} W_i \phi_{ik}\right]^2}{g \sum_{i=1}^{N} W_i [\phi_{ik}]^2}$$

where,

g - Acceleration due to gravity.

ϕ_{ik} - Mode shape co-efficient at floor i in mode k.

W_i - Seismic weight at floor i.

Modal contributions for various modes, Clause 7.8.4.2

It is clear from the values of the participation factors and effective mass, their value decreases as mode number increases. The practical significance of this fact is that in general it is necessary to include all the modes in the calculation. Only a few significant modes need to be included in order to obtain reasonable results for practical problems. Therefore, the Clause 7.8.4.2 of IS1893 (Part 1):2002 states, that "The number of modes to be used in the analysis should be such that the sum of total modal masses of all modes considered is at least 90% of the total seismic mass and missing mass correction beyond 33 Hz are considered, modal combination shall be carried out only for modes upto 33 Hz"

Modal contribution of various modes, for mode i = $\frac{Mi}{M}$ %

Design lateral force at each floor in each mode

The design lateral force Q_{ik} at floor i in mode k i as per Clause 7.8.4.5 is given by

$$Q_{ik} = A_k \phi_{ik} P_k W_i$$

where

A_k - design horizontal acceleration spectrum value as per 6.4.2 using the natural period of vibration T_k of mode k.

The design horizontal seismic co-efficient A_k for various modes are worked out using

$$Ah = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

Design lateral force in each mode

$$Qi1 = (A1\ P1\ \phi_{ik}\ Wi) = \begin{bmatrix} A1P1\phi 11W1 \\ A1P1\phi 21W1 \\ \text{.........} \quad \text{.......} \\ A1P1\phi n-1W1 \\ A1P1\phi n1W1 \end{bmatrix} \text{kN}$$

Similarly $Q_{i2}, Q_{i3}, Q_{i4} \ldots\ldots Q_{in}$

Storey shear forces in each mode

The peak shear force V_{ik} acting in storey i in mode k as per Clause 7.8.4.5 is given by

$$V_{ik} = \sum_{j=i+1}^{N} Q_{jk}$$

The storey shear force for the first mode is,

$$V_{i1} = \sum_{j=i+1}^{n} Q_{\overline{j1}} \begin{bmatrix} V11 \\ V21 \\ Vn-1 \\ Vn \end{bmatrix} = \begin{bmatrix} Q11 + Q21 + Qn-1 + Qn \\ \ldots Q21 + Qn-1 + Qn \\ \ldots\ldots Qn-1 + Qn1 \\ \ldots\ldots\ldots\ldots Qn \end{bmatrix}$$

Storey shear forces due to all modes considered

The peak storey shear force (V_i) in storey i due to all modes considered is obtained by combining those due to each mode in accordance with modal combination as per clause 7.8.4.4. The combinations are usually achieved by using statistical methods.

The design values for the total base shear are obtained by combining the corresponding modal responses. In general these modal maximum values will not occur simultaneously. To overcome this difficulty, it is necessary to use an approximate method.

An upper limit for the maximum response may be obtained by the Sum of the **AB**solute values **(ABS)** of the maximum modal contributions. This is very conservative method and is very seldom used except in some codes for say two or three modes for very short period structures. If the system does not have closely spaced modes, another estimate of the maximum response, which is widely accepted and which usually provides a reasonable estimate is the **S**quare **R**oot of the **S**um of **S**quares (**SRSS**). Application of the SRSS method for combining modal responses generally provides an acceptable estimation of the total maximum response. However , when some of the modes are closely spaced i.e. the difference between two natural frequencies is within 10% of the smallest of two frequencies, the use of SRSS method may either grossly underestimate or overestimate the maximum response. A formulation known as the **C**omplete **Q**uadratic **C**ombination (**CQC**), based on the theory of random vibration and is also considered as the extension of SRSS method. For an undamped structure CQC estimate is identical to SRSS method.

MAXIMUM ABSOLUTE REPONSE (ABS)

The Maximum Absolute Response (ABS) for any system response quantity is obtained by assuming that the maximum response in each mode occurs at the same instant of time. Thus the maximum value of the response quantity is the sum of the maximum absolute value of the response associated with each mode. Therefore using ABS, maximum storey shear for all modes shall be obtained as

$$\lambda^* = \sum_{c}^{r} \lambda_c$$

where, the summation is for closely spaced modes only. The peak response quantity due to the closely spaced modes (λ^*) is then combined with those of the remaining well separated modes by the SRSS method.

5.2.3 SQUARE ROOT OF THE SUM OF SQUARES (SRSS)

A more reasonable method of combing modal maxima for two-dimensional structural system exhibiting well-separated vibration frequencies is the square root of the sum of squares (SRSS). The peak response quantity (λ) due to all modes considered shall be obtained as

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2}$$

where,

λ_k - The absolute value of a quantity in mode k.

r - Number of modes being considered.

Using the above method the storey shears are as follows,

$V_1 = [(V_{11})^2+(V_{12})^2+\ldots\ldots\ldots(V_{1(n-1)})^2+(V_{1n})^2]^{1/2}$ kN

$V_2 = [(V_{21})^2+(V_{22})^2+\ldots\ldots\ldots(V_{2(n-1)})^2+(V_{2n})^2]^{1/2}$ kN

$V_3 = [(V_{31})^2+(V_{32})^2+\ldots\ldots\ldots(V_{3(n-1)})^2+(V_{3n})^2]^{1/2}$ kN

....

....

$V_n = [(V_{n1})^2+(V_{n2})^2+\ldots\ldots\ldots(V_{n(n-1)})^2+(V_{nn})^2]^{1/2}$ kN

5.2.4 COMBINED QUADRATIC COMBINATION (CQC)

For three dimensional structural systems exhibiting well-separated vibration frequencies, the peak response quantities shall be combined as per Complete Quadratic Combination (CQC) method.

$$\lambda = \sum_{i=1}^{r} \sum_{j=1}^{r} \lambda_i \rho_{ij} \lambda_j$$

where,

r - Number of modes being considered.

λ_i - Response quantity in mode i (including sign).

λ_j - Response quantity in mode j (including sign).

ρ_{ij} - Cross modal co-efficient.

$$\rho_{ij} = \frac{8\zeta^2(1+\beta_{ij})\beta^{1.5}}{(1-\beta_{ij})^2 + 4\zeta^2\beta_{ij}(1+\beta_{ij})^2}$$

where,

ζ - Modal damping ratio (in fraction)

β_{ij} - Frequency ratio ω_j / ω_i .

ω_i - Circular frequency in i^{th} mode.

ω_j - Circular frequency in j^{th} mode.

Here the terms λ_i and λ_j represent the response of different modes of a certain storey level. Using matrix notation the storey shears V_1, V_2, ...V_n are worked out respectively

Lateral forces at each storey due to all modes

The design lateral forces F_{roof} and F_i at roof and at i^{th} floor are calculated as,

$F_{roof} = F_i$, and $F_i = V_i + V_{i+1}$

$F_n = V_n$ kN

....

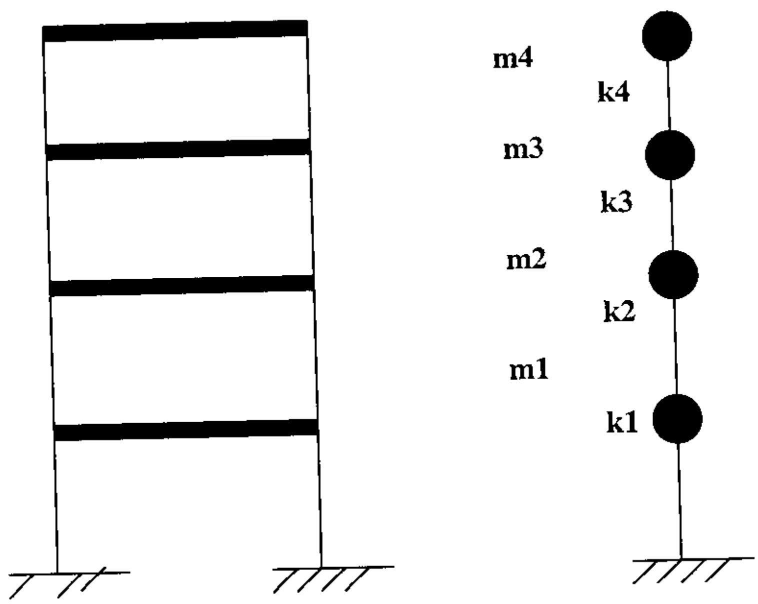
$F_2 = F_2$ kN

$F_1 = V_1$ kN

CHAPTER 6

MODAL ANALYSIS OF MULTI-STOREYED FRAMES

A structure can be modeled and its response analyzed using a SDOF model if the mass is essentially concentrated at a single point that can move, translate, or rotate only in one direction, or if the system is constrained in such a way as to permit only a single mode of displacement. A realistic description of the dynamic response of large building or structural systems generally requires the use of a number of independent displacement coordinates, and modeling of the system as a multi degree of freedom (MDOF) system. Free vibration of the structure is initiated by disturbing the structure from its equilibrium position by some initial displacements and/or by imparting some initial velocities



Shear frame **Equivalent system**

Fig 6.1 Multi-storey frame idealized as Multi-degree of freedom system

For an undamped free vibration of structures, the equation of motion is given by

$$[M]\{\ddot{u}\} + [K]\{u\}=0$$

the above equation represents N homogeneous differential equations that are coupled through the mass matrix, the stiffness matrix, or both matrices; N is the number of DOFs.

Where k1, k2 and k3 are the respective storey stiffness which is equal to the sum of the stiffness of column as well as that of infill walls acts as a diagonal strut.

i.e. ., k1 = n x column stiffness + (n-1) x infill stiffness

k1 = n x (12EI / l^3) + (n-1) (AEm / Ld) x Cos^2 theta.

Where n = No. of columns .

E= Young's modulus of Concrete ie 5000x(fck)^0.5

Em = Young's modulus of masonry wall..

An undamped structure would undergo simple harmonic motion without change of deflected shape, however, if free vibration is initiated by appropriate distributions of displacements in the various DOFs. n characteristic deflected shapes exist for n DOF. If this system is displaced in one of these shapes and released, it will vibrate in simple harmonic motion, maintaining the initial deflected shape. All the floors reach their extreme displacements at the same time and pass through the equilibrium position at the same time. Each characteristic deflected shape is called a natural mode of vibration of an MDF system.

A natural period of vibration T_n of an MDF system is the time required for one cycle of the simple harmonic motion in one of these natural modes. The corresponding natural circular frequency of vibration is $?_n$ and the natural cyclic frequency of vibration is f_n, where

$$T_n=2\pi/\omega_n \qquad fn=1/T_n$$

Assume solutions of form $u_i = a_i$ (sin? t-a), where i=1, 2, 3 ...n. a_i is the amplitude of motion of i^{th} coordinate and n is the no of degree of freedom.

$$[[K]-[M]\omega 2]\{a\}=0$$

which is the homogenous algebraic system of linear equations with n unknown displacements a_i, and unknown parameters ω^2. The formulation of above equation is an important mathematical problem known as eigen problem. In order to have a non trivial solution

$$|[K]-\omega^2 [m]|=0$$

In general, above equation results in a polynomial equation of degree 'n' in ω^2 this should be satisfied for n values of ω^2. The above equation is termed as the characteristic equation. This equation has n real and positive roots for ω_n^2 because **m** and **k** matrices are symmetric and positive definite. The positive definite property of **k** is assured for all structures supported in a way that prevents rigid body motion. The positive definite property of **m** is also assured because the lumped masses are non zero in all DOFs retained in the analysis. The roots of this characteristic equation are called eigen values and the positive square root of the eigen values are known as natural frequency of the MDOF system.

For each eigen value the resulting (synchronous) motion has a distinct shape known as natural mode shapes or normal mode shape or eigen vector. There might be a number of eigen vectors. The n eigen vector can be displayed into a single square matrix, each column of which is a natural mode:

$$\phi = \begin{bmatrix} \phi_{11} & \phi_{12} & \cdots & \phi_{1n} \\ \phi_{21} & \phi_{22} & \cdots & \phi_{2n} \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot\ \cdot & \cdot \\ \phi_{n1} & \phi_{n2} & & \phi_{nn} \end{bmatrix}$$

The matrix ϕ is called the modal matrix for the eigen value problem. Then eigen values can be assembled into a diagonal matrix O, which is known as the spectral matrix of the eigen value problem. In order to obtain a unique solution, eigen vectors are normalized using certain normalization conditions. Such a normalization using mass matrix is known as mass renormalization and the resulting mode shape is known as mass orthonormal mode shape.

CHAPTER 7
NUMERICAL INVESTIGATIONS

7.1 ANALYTICAL METHODOLOGY

The effect of with and without masonry infill wall in a RC frame structure is studied by analyzing the structure with static and dynamic analysis. The analyses procedures are followed as per IS 1893:2002. The generalized software programme was created using MATLAB 7 for both static and dynamic analysis. The masonry infill walls are modeled as Equivalent diagonal strut method. Seismic coefficient method for bare frame and masonry infill frame are analyzed. Modal analysis procedure is used in the matlab programme to find the soft storey effect in the RC structure.

7.2 INTRODUCTION TO THE MATLAB 7

MATLAB- stands for **MAT**rix **LAB**oratory . Starting the program START-→PROGRAMS-→MATLAB 7 or by double clicking the icon in the desktop. Mat lab stores most of its numerical results as matrices. Unlike C, it dynamically allocates memory to store variables. Therefore, it is not necessary to declare variables before using them.

EXAMPLE:

In the command window the following programme shows to calculate the sum of two numbers.

Enter the following commands

Type in "x = 3" then hit "enter"

Type in "y = 2;" then hit "enter"

Type "z = x + y" then hit "enter"

The result will display as z=5

7.3ADVANTAGES OF USING MATLAB 7

Mat lab is nowadays widely used in civil engineering field. Matrix can be easily calculated with in four steps, where as in C program we need to define these steps with large amount. Matlab can easily plot the graphs with the available results.

7.4GENERALISED PROGRAMME CREATED IN MATLAB7

The following programme was created in explicit matlab programme.

GENERALISED PROGRAMME CREATED IN THE MATLAB 7

GETTING INPUTS FOR THE PLAN DIMENSIONS

```
disp('NOTE: The geometric dimension parameter is to be given in m  ')
disp('                                                    ')
Bx=input('Enter the no of bays in X direction -');
for i=1:Bx
  i
  l(i)=input('Enter the bay width in X direction-');
end
dx=0;
for i=1:Bx
  dx=l(1,i)+dx;
end
dx;
By=input('Enter the no of bays in Y direction -');
for i=1:By
  i
  b(i)=input('Enter the bay width in Y direction-');
end
dy=0;
for i=1:By
  dy=b(1,i)+dy;
end
dy;
Ns=input('Enter the no of storey -');
for i=1:Ns
  i
  h(i)=input('Enter the storey height-');
end
H=0;
for i=1:Ns
  H=h(1,i)+H;
end
H;
```

GETTING INPUTS FOR COLUMN,BEAMS, WALLS AND LIVELOAD OF THE STRUCTURE.

```
%COLUMN DIMENSIONS
disp('Enter the column dimensions in m: ')
bc=input('b = ');
dc=input('d = ');
C=bc*dc;
%BEAM DIMENSIONS
disp('Enter the beam dimensions in longer direction in m: ')
bl=input('b = ');
dl=input('d = ');
BSX=bl*dl;
disp('Enter the beam dimensions in shorter direction in m: ')
bs=input('b = ');
ds=input('d = ');
BSY=bs*ds;
OWT=input('Enter the outer wall thickness in m -');
IWT=input('Enter the inner wall(infill)thickness in m -');
LL=input('Enter the live load in kN/sq.m - ');
FST=input('Enter the floor slab thickness in m  - ');
WCT=input('Enter the weathering coarse thikness in m - ');
```

GETTING INPUT FOR THE TYPE OF SEISMIC ZONE

```
%CALCULATION OF Z
%for i=2:5
SZ=input('Enter the seismic zone - ');
if (SZ==2)
   Z=.10;
 else
   if (SZ==3)
     Z=.16;
   else
     if (SZ==4)
       Z=.24;
     else
       if (SZ==5)
         Z=.36;
       end
     end
   end
 end
```

```
%CALCULATION OF I
i=input('If the Structure is important service building enter 1 else 2
- ');
if (i==1)
  I=1.5;
else
  if(i==2)
  I=1;
  end
end
%CALCULATION OF R
r=input('If Structure is SMRF,Enter 1 else 2 - ');
if(r==1)
   R=5.0;
else
   if(r==2)
   R=3.0;
   end
end
```

GETTING THE VALUES FOR 'I' AND 'R'

```
%CALCULATION OF Tax
k=input('if the structure is without brick infill Enter 1 else 2 - ');
if(k==1)
   j=input('If the structure is RC Frame enter 1 else 2 - ');

 if(j==1)
    Tax=.075*(H^.75);
    Tay=.075*(H^.75);
 else
    if (j==2)
       Tax=.085*(H^.75);
       Tay=.085*(H^.75);
    end
 end
 else
   if(k==2)
      Tax=.09*H/sqrt(dx);
      Tay=.09*H/sqrt(dy);
   end
 end
```

CALCULATION OF THE FUNDAMENTAL NATURAL PERIOD

```
%CALCULATION OF Sax
disp(' Enter 1-rocky or hard soil sites; 2-medium soil; 3-soft soil')
s=input('Enter the type of soil - ');
d=input('Enter the percentage of damping - ');
if(s==1)
  if (Tax>=0)&(Tax<=.1)
     Ssax=1+15*Tax;
  else
   if (Tax>0.1)&(Tax<=.4)
     Ssax=2.5;
   else
    if (Tax>0.4)&(Tax<=4)
      Ssax=1/Tax;
    end
   end
  end
  else
   if(s==2)
     if (Tax>=0)&(Tax<=.1)
      Ssax=1+15*Tax;
     else
     if (Tax>0.1)&(Tax<=.55)
      Ssax=2.5;
    else
     if (Tax>0.55)&(Tax<=4)
      Ssax=1.36/Tax;
     end
    end
   end
   else
   if(s==3)
     if (Tax>=0)&(Tax<=.1)
     Ssax=1+15*Tax;
     else
    if (Tax>0.1)&(Tax<=.67)
     Ssax=2.5;
    else
     if (Tax>0.67)&(Tax<=4)
      Ssax=1.67/Tax;
     end
    end
```

GETTING THE INPUTS FOR TYPE OF SOIL AND % OF DAMPING.

```
Ahsx=(Z/2)*(I/R)*Ssax;
Ahsy=(Z/2)*(I/R)*Ssay;
```

HORIZONDAL ACCELERATION SPECTRUM

```
%LOAD CALCULATIONS
SWF=dx*dy*FST*25;
SWLb=dx*BSX*(By+1)*25;
SWCb=dy*BSY*(Bx+1)*25;
SWC=C*h(1,Ns)*(Bx+1)*(By+1)*25;
SWOw=((dx+dy)*2)*OWT*h(1,Ns)*19;
SWCx=dx*IWT*h(1,Ns)*(By-1)*19;
SWCy=dy*(Bx-1)*IWT*h(1,Ns)*19;
TLL=SWF+SWLb+SWCb+SWC+SWOw+SWCx+SWCy;
%TLSS=SWLb+SWCb+SWC+SWF;
```

SELF WEIGHT CALCULATION

```
SWW=dx*dy*WCT*19;
%CALCULATION OF IMPOSED LOADS
IFL=(dx*dy*LL);
if(LL<=3)
  ISW=0.25*IFL
else
  ISW=(.5*IFL)
end
SL=(TLL-(SWC/2)+SWW);
SLO=(TLL+ISW);
SLP=(SWLb+SWCb+SWC+SWOw*0.5+SWCx*0.5+SWCy*0.5);
TW=((SLO*(Ns-2))+SL+SLP);
Vbx=Ahsx*TW;
Vby=Ahsy*TW;
disp(' ')
disp('TOTAL SEISMIC WEIGHT OF ALL FLOORS IN kN:')
TW
disp('BASE SHEAR IN X DIRECTION IN kN:')
Vbx
disp('BASE SHEAR IN Y DIRECTION IN kN:')
Vby
```

CALCULATION OF IMPOSED LOAD AND BASE SHEAR

```
% Design Lateral forces
for i=2:(Ns-1)
  W(1,Ns)=SL;
  W(1,i)=SLO;
  W(1,1)=SLP;
end
ht(1,1)=h(1,1);
for i=1:Ns-1
  ht(i+1)=ht(i)+h(i+1);
end
for i=1:Ns
  Wh(i)=W(i)*(ht(i)^2);
end
W=0;
for i=1:Ns
  W=Wh(1,i)+W;
end
for i=1:Ns
   V(i)=Wh(i)/W;
   Qix(i)=Vbx*V(i);
   Qiy(i)=Vby*V(i);
end
disp('DESIGN LATERAL FORCE IN X DIRECTION IN kN:')
Qix'
disp('DESIGN LATERAL FORCE IN Y DIRECTION IN kN:')
Qiy'
```

DESIGN LATERAL FORCE CALCULATIONS

```
%SHEAR FORCE DISTRIBUTION
SFx(Ns)=Qix(Ns);
for i=Ns-1:-1:1
   SFx(i)=SFx(i+1)+Qix(i);
end
disp('SHEAR FORCE DISTRIBUTION IN X DIRECTION IN kN:')
SFx'
SFy(Ns)=Qiy(Ns);
for i=Ns-1:-1:1
   SFy(i)=SFy(i+1)+Qiy(i);
end
disp('SHEAR FORCE DISTRIBUTION IN Y DIRECTION IN kN:')
SFy'
```

SHEAR FORCE CALCULATIONS

MATLAB PROGRAMMING FOR MODAL ANALYSIS

```
%MODAL ANALYSIS
fck=input('Enter the grade of concrete to be used - ')
E=(5000*sqrt(fck))*(10^6);
Ix=(bc*(dc^3))/12;
for i=1:Ns
  kx(i)=(12*E*Ix)/((h(i))^3);
  kx(Ns+1)=0;
end
for i=1:Ns
  mx(i)=(SLO*(10^3))/9.81;
end
for i=1:Ns
  for j=1:Ns
    Kx(i,i)=((Bx+1)*(By+1))*(kx(i)+kx(i+1));
    if (j-i==1)
      Kx(i,j)=-((Bx+1)*(By+1))*(kx(j));
    else
      if (i-j==1)
        Kx(i,j)=-((Bx+1)*(By+1))*(kx(i));
      end
    end
  end
end
for i=1:Ns
  Mx(i,i)=mx(i);
end
[xx] = eye(Ns);
for i = 1 : 200
[dx]=[xx']*[Mx]*[xx];
[cx]=chol(dx);
[yx]=[xx]*[inv(cx)];
rx=[Mx]*[yx];
[xx]=[inv(Kx)]*[rx];
[lx]=[yx']*[Kx]*[yx];
end
for i=1:Ns
  wnx(i)= sqrt(lx(i,i));
  Fx(i)=wnx(i)/(2*pi);
  Tx(i)=1/Fx(i);
end

disp('STIFFNESS MATRIX - Stiffness in N/m:')
Kx
disp('MASS MATRIX:')
Mx
disp('EIGEN VALUES:')
lx
disp('EIGEN VECTORS:')
yx
disp('NATURAL FREQUENCY IN EACH MODE in rad/sec:')
wnx
disp('NATURAL TIME PERIOD in sec:')
Tx
Iyy=(dc*(bc^3))/12;
for i=1:Ns
  ky(i)=(12*E*Iyy)/((h(i))^3);
  ky(Ns+1)=0;
end
for i=1:Ns
  my(i)=(SLO*(10^3))/9.81;
end
for i=1:Ns
  for j=1:Ns
```

Contd..

```
Ky(i,i)=((Bx+1)*(By+1))*(ky(i)+ky(i+1));
    if (j-i==1)
      Ky(i,j)=-((Bx+1)*(By+1))*(ky(j));
    else
      if (i-j==1)
        Ky(i,j)=-((Bx+1)*(By+1))*(ky(i));
      end
    end
  end
end
for i=1:Ns
  My(i,i)=my(i);
end

[xy] = eye(Ns);
for i = 1 : 200
[dy]=[xy']*[My]*[xy];
[cy]=chol(dy);
[yy]=[xy]*[inv(cy)];
ry=[My]*[yy];
[xy]=[inv(Ky)]*[ry];
[ly]=[yy']*[Ky]*[yy];
end
for i=1:Ns
  wny(i)= sqrt(ly(i,i));
  Fy(i)=wny(i)/(2*pi);
  Ty(i)=1/Fy(i);
end

disp('STIFFNESS MATRIX- Stiffness in N/m:')
Ky
disp('MASS MATRIX:')
My
disp('EIGEN VALUES:')
ly
disp('EIGEN VECTORS:')
yy
disp('NATURAL FREQUENCY IN VARIOUS
MODES in rad/sec:')
wny
disp('NATURAL TIME PERIOD in sec:')
Ty
```

RC structures varying from four storeys to fifteen storeys are considered for investigation to compare the results of response spectrum method with seismic coefficient method. The seismic responses of the building such as base shear, lateral forces on each floor are determined from the explicit programme written using Matlab.

7.5 DESCRIPTION OF THE STRUCTURE

The plan dimensions of the considered RC framed regular building is 16m x 16m. (3 bays in X and Y direction). Height of each storey is 3m.The depth of footing is 1.5m. Number of storeys is varied from 4 to 15. It is considered to be located in seismic zone II with importance factor of 1.0. The response reduction factor is taken as 3 for OMRF. It is built on medium soil as per IS 1893. The size of the beam is 0.23mX0.45m in both directions. Thickness of the slab is 120mm. Thickness of weathering course on roof is 100mm. Live load intensity is taken as 3KN/m2 at each floor level and 2 KN/m2 on the roof. Grade of concrete is M25. Columns are square in plan and their sizes are varied from 0.3m to 0.5 m. Width of the diagonal strut derived by using Brayan Stafford Smith formula(1969) is 1.1397 m along both X and Y-directions.

The external and internal wall thickness are taken as 0.23m and 0.12m respectively for all the walls. The seismic weight on the RC structure for the analysis conforms to IS:1893-2002 wherein 100% of dead load as well as 25% of the live load is considered for floor and no live load for terrace has been considered.

The analysis takes into account of an important structure for which the importance factor is assigned to be 1.5.The results have been compared for the RC frame structures with and without brick infill. Percentage of damping considered is 5% for all the structures during the seismic analysis .

No of storey – 4 to 12

No of bays in X direction –3

No of bays in Y direction – 3

Storey height – 3 m

Bay width in X direction – 4 m

Bay width in Y direction – 4 m

Column dimensions - .30 x .30 m to 0.60 to 0.60

Beam dimensions in longer (X) direction - 0.23 x .45 m

Beam dimensions in shorter (Y) direction - 0.23 x .45 m

Outer wall thickness - .23 m

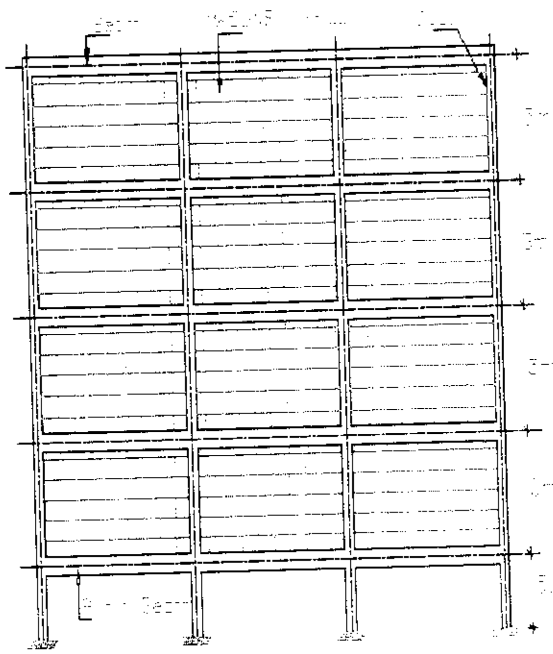
Inner wall thickness - .15 m

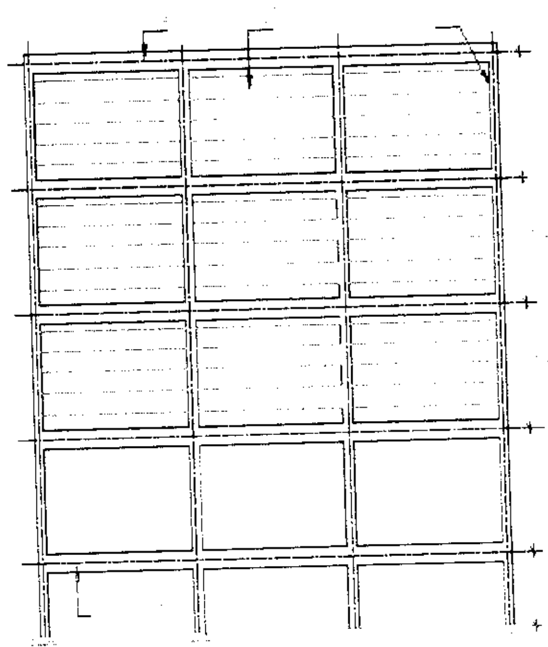
Live load - 3 kN/sq.m

Floor slab thickness - .12 m

Weathering coarse thickness - .1 m

Seismic zone – 2





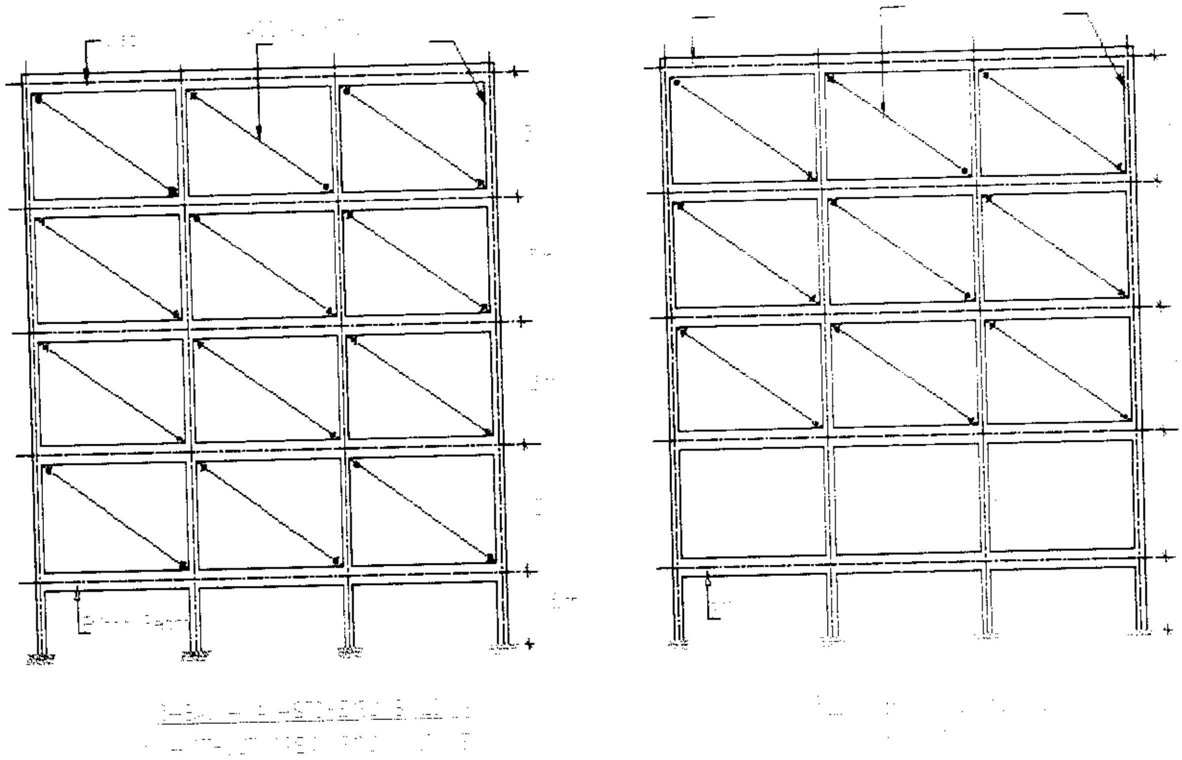


Fig.7.1.Typical RC Frame showing with and without open Ground Storey
Fig.7.2.Typical RC Frame with diagonal strut

The lateral forces at each floor level are determined by both the methods and the output is tabulated as below:

7.6 Comparison of Storey Stiffness

The storey stiffness for a five storey building is compared as shown in table 7.1. It is found that masonry infill contributes higher percentage of storey stiffness.

Table 7.1 Comparison of Storey Stiffness 'kN/m'for a four storey building

Storey no	Bare frame	Masonry Infilled frame	
		Without open ground storey	With open ground storey
1	1.50E+09	1.50E+09	1.50E+09
2	1.88E+08	8.57E+09	1.88E+08
3	1.88E+08	8.57E+09	8.57E+09
4	1.88E+08	8.57E+09	8.57E+09

7.7 Fundamental time period

It is the longest (first) modal time period of vibration. Comparison of fundamental time periods obtained from both methods with different stories for bare frame, infilled framed structure with open ground storey and infilled frame without open ground storey is tabulated in Table 7.2 and represented in Fig 7.3. In both methods, the fundamental time period for bare framed structure gives higher values as compared to infilled framed structure. In empirical methods, fundamental time period for bare framed structure is only based on the height of the building. As expected, the increases in number of storeys increase the fundamental time period.

But in response spectrum method, fundamental time period Ta, is based on stiffness and mass of all storeys as well as their distribution along the height of the building.

Table 7.2 comparison of Time period for different stories

No. of Stories	Height in m	T in 'sec'				
		Seismic co-efficient method		Modal analysis		
				Without open ground storey		With open ground storey
		BR	MI Frame	BR	SS	SS
4	10.5	0.4375	0.2363	0.6041	0.1867	0.4776
5	13.5m	0.5282	0.3038	0.7661	0.2171	0.5515
6	16.5m	0.614	0.3713	0.9282	0.2459	0.6174
7	19.5m	0.6959	0.4387	1.0905	0.2736	0.6776
8	22.5m	0.7748	0.5062	1.2528	0.3006	0.7336
9	25.5m	0.851	0.5737	1.4152	0.327	0.7862
10	28.5m	0.9251	0.6412	1.5776	0.3529	0.8361
11	31.5m	0.9972	0.7087	1.74	0.3786	0.8837
12	34.5m	1.0676	0.7762	1.9025	0.404	0.9294
13	37.5m	1.1365	0.8437	2.065	0.4292	0.9735
14	40.5m	1.204	0.9112	2.2274	0.4542	1.0162
15	43.5m	1.2703	0.9787	2.3899	0.4791	1.0576

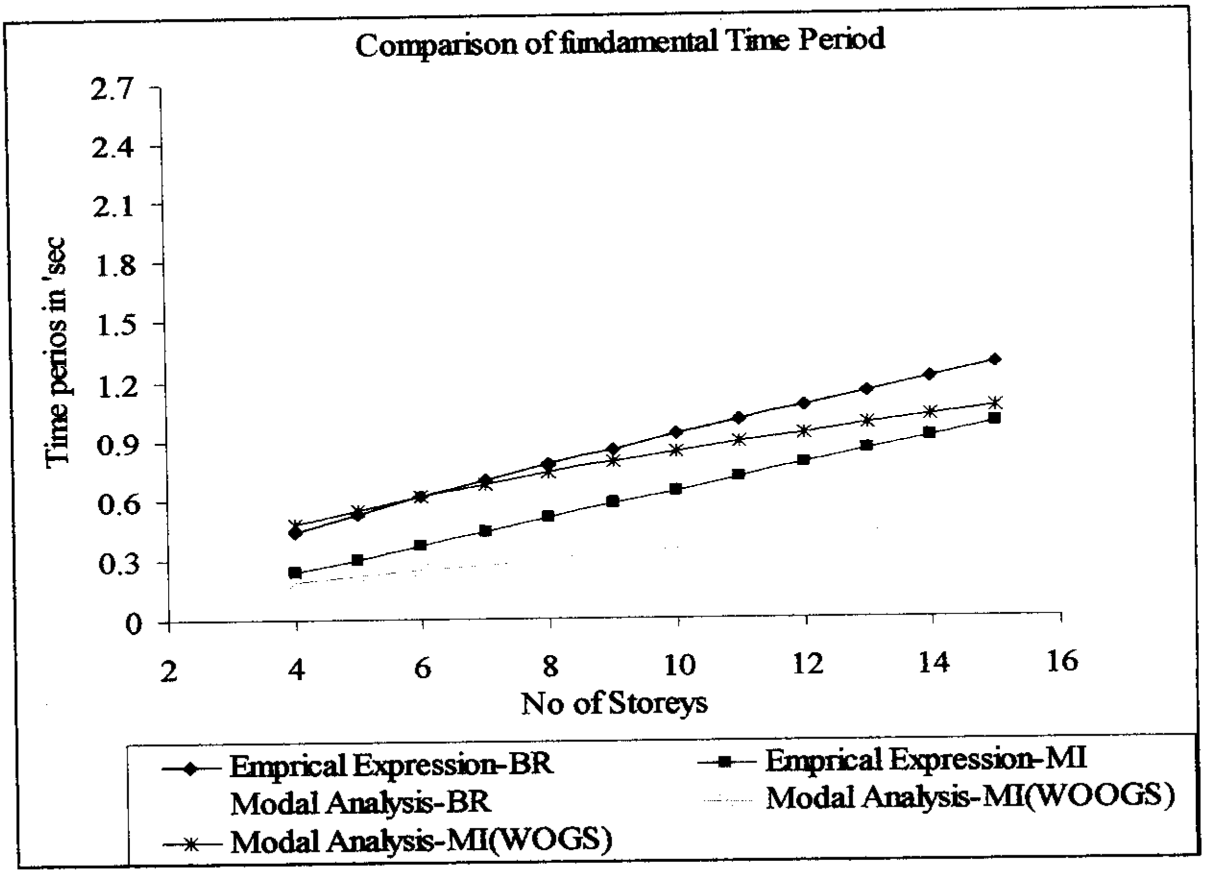


Fig.7.3 Comparison of Fundamental Time Period in 'sec'

7.8 Base Shear

The comparison of base shear obtained from both the methods with different storeys is tabulated in Table 7.3 and represented in Fig. 7.4. The highest value of base shear has been predicted for masonry infilled structure unlike bare framed structure by both of the methods. For masonry infilled framed structure, the seismic co-efficient method gives more or less same value of base shear for above eight stories. The reason may be deduced as follows: Base shear is a function of horizontal seismic co-efficient value (Ah) and seismic weight of the building (W). The seismic co-efficient value is mainly based on the zone factor (Z) , importance factor (I) , response reduction factore (R) and spectral acceleration co-efficient value (Sa/g). here the values of Z, I and R are constant for particular case. But the Sa/g value is not constant, whose variation is based on fundamental time period.

For our case,

FOR MEDIUM SOIL SITES

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T & 0.00 = T = 0.10 \\ 2.50 & 0.10 = T = 0.55 \\ 1.36/T & 0.55 = T = 4.00 \end{cases}$$

For above eight stories, value of time period for infilled framed structure in seismic co-efficient method lies between **0.5737** to **0.9787.**

$(Sa/g) = 1.36/T$ when $0.55 = T = 4.00$

From the above formula, the (Sa/g) value is decreased due to the increased time period of the structure. Time period is increased due to the increased height of the structure. At the same time seismic weight of the building is increased due to increased height of the structure. The base shear which is increased due to seismic weight is being compensated by the base shear decreased due to (Sa/g) values.

In response spectrum analysis for bare frame gives the approximately same values (ie there is no much difference) of base shear for all stories. (fig 7.4, for bare frame stories , the CQC method gives the almost the horizontal line .)

From Fig7.4, it is found that the CQC method gives the linear variation of base shear from fourth stories to fifteen stories for buildings without open ground storey.

Table 7.3. Comparison of Base Shear

BASE SHEAR IN 'kN'								
No of stories	SEISMIC COEFFICENT METHOD		RESPONSE SPECTRUM METHOD					
			Bare frame		Without open ground storey		With open ground storey	
	BARE FRAME	MI-FRAME	SRSS	CQC	SRSS	CQC	SRSS	CQC
4	443.99	444.00	332.81	333.10	440.70	440.71	401.57	401.60
5	570.60	570.60	339.34	339.78	564.60	564.61	520.51	520.55
6	617.71	697.20	344.09	344.71	687.48	687.50	576.24	576.34
7	643.92	823.80	348.25	349.07	809.31	809.35	626.62	626.65
8	667.28	950.40	352.29	353.34	930.12	930.18	672.57	672.58
9	688.41	1020.00	356.44	357.73	1050.00	1050.00	715.00	715.02
10	707.76	1020.00	360.80	362.36	1170.00	1170.00	754.30	754.53
11	726.00	1020.00	362.92	364.69	1290.00	1290.00	791.54	791.59
12	742.29	1020.00	364.14	366.11	1400.00	1400.00	826.52	826.54
13	757.89	1020.00	365.31	367.50	1520.00	1520.00	859.61	859.62

14	772.58	1020.00	366.48	368.88	1640.00	1640.00	891.00	891.04
15	786.48	1020.00	367.65	370.29	1750.00	1750.00	920.77	920.98

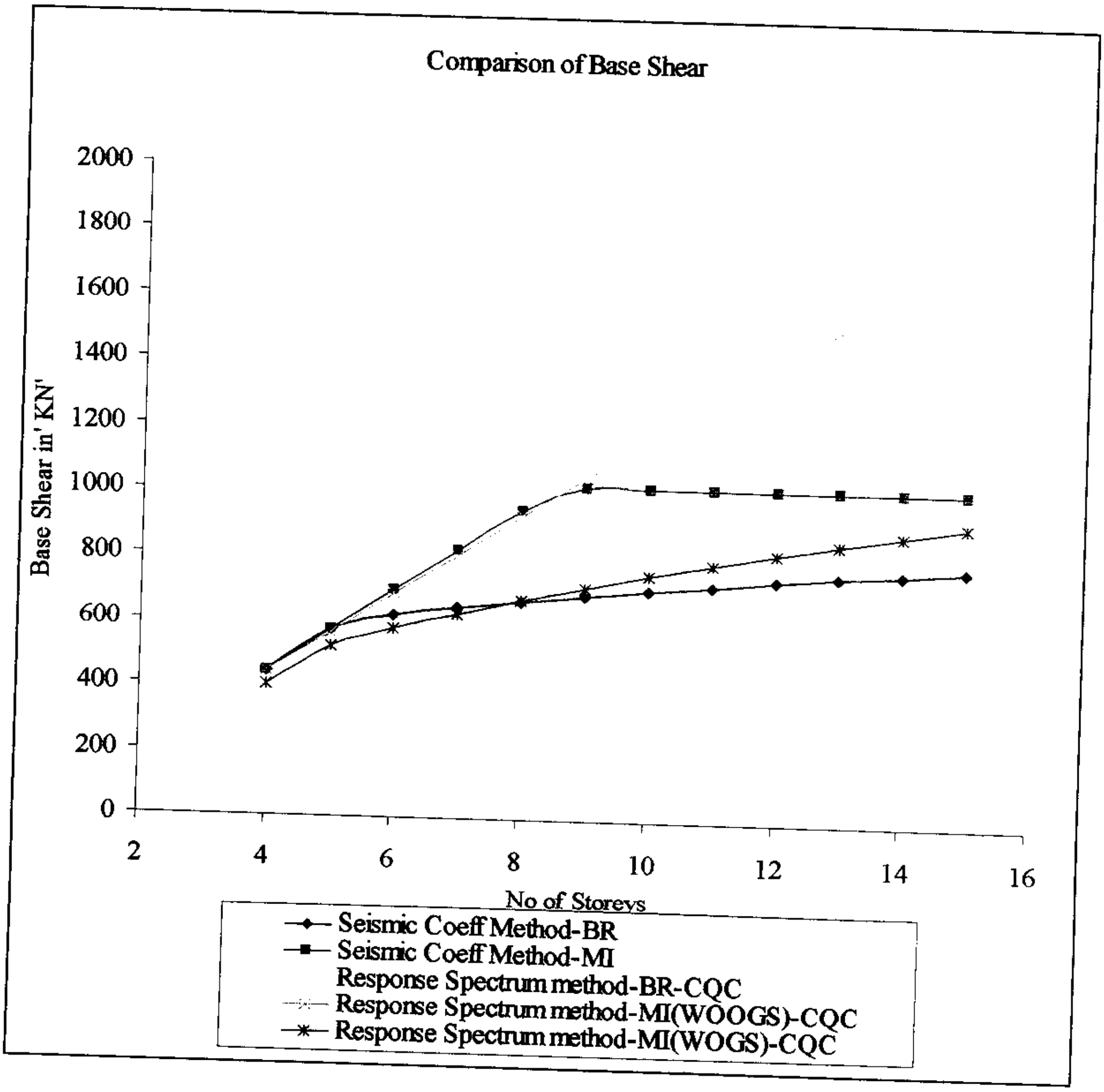


Fig.7.4 Comparison of Base Shear in 'KN'

7.9 Storey shear distribution

In seismic co-efficient method, the computed base shear is distributed along the height of the building. The shear force, at any level of the building, depends on the mass at that level. IS1893 (Part 1):2002 uses parabolic distribution of lateral force along the height of the building as per the following expression.

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

Where,

Q_i – design lateral force at floor i.

W_i – Seismic weight of floor i.

h_i – Height of floor i measured from base.

n – Number of storeys in the building is the number if levels at which the masses are located.

In response spectrum method, the design lateral force Q_{ik} at floor i in mode k i as per Clause 7.8.4.5 of IS 1893 is given by

$$Q_{ik} = A_k \phi_{ik} P_k W_i$$

where

A_k - design horizontal acceleration spectrum value as per 6.4.2 using the natural period of vibration T_k of mode k.

The shear force distribution along the height of the building for 5, 8, 12 and 15 storeyed buidings are depicted in Fig.7.5- 7.7. From Figures, it is observed that the storey shear force in seismic co-efficient method for bare framed structure and infilled structure is parabolic. Storey shear force found by using response spectrum method, for infilled framed structure without open ground storey is trapezoidal thus contradicting the parabolic assumption made in seismic co-efficient method.

For infilled structure with open ground storey it is found that the shear force is almost constant for all the stories above the open ground storey i.e. all the storeys above open ground storey will act as a single rigid mass like an inverted pendulum.

Much of the deformation is concentrated in the ground floor which is the main cause of failure during past earthquakes.

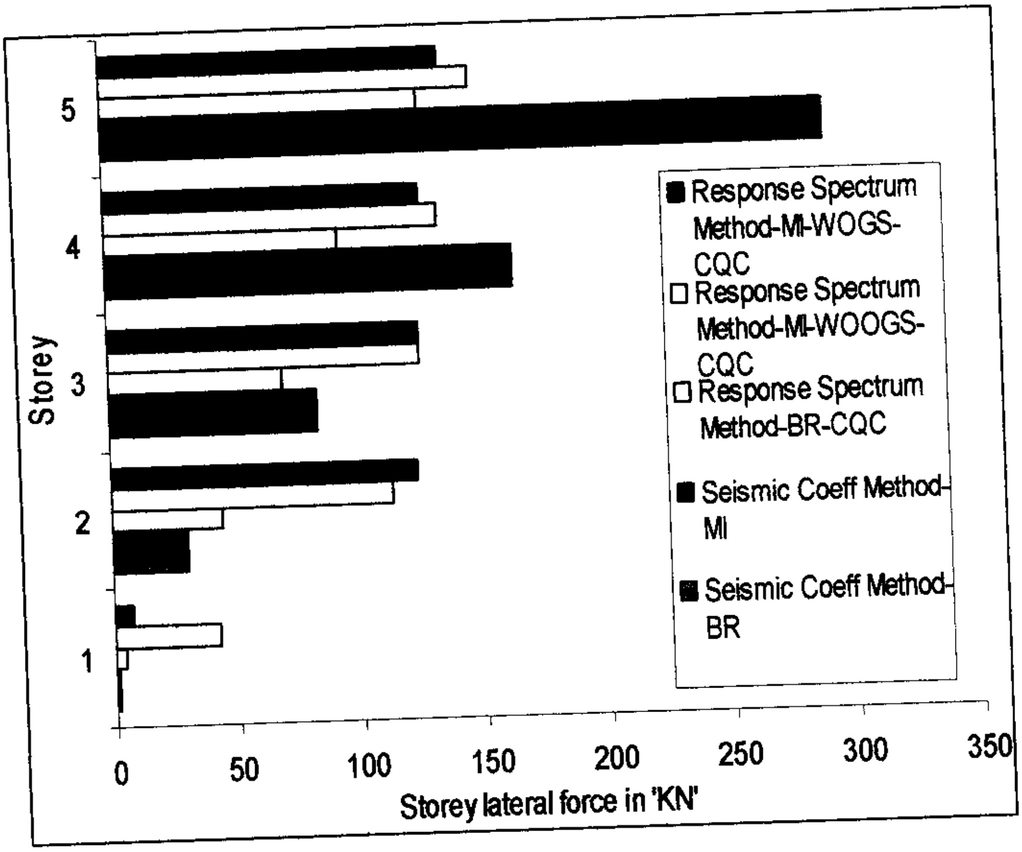


Fig : 7.5 a) 5 Storey Building

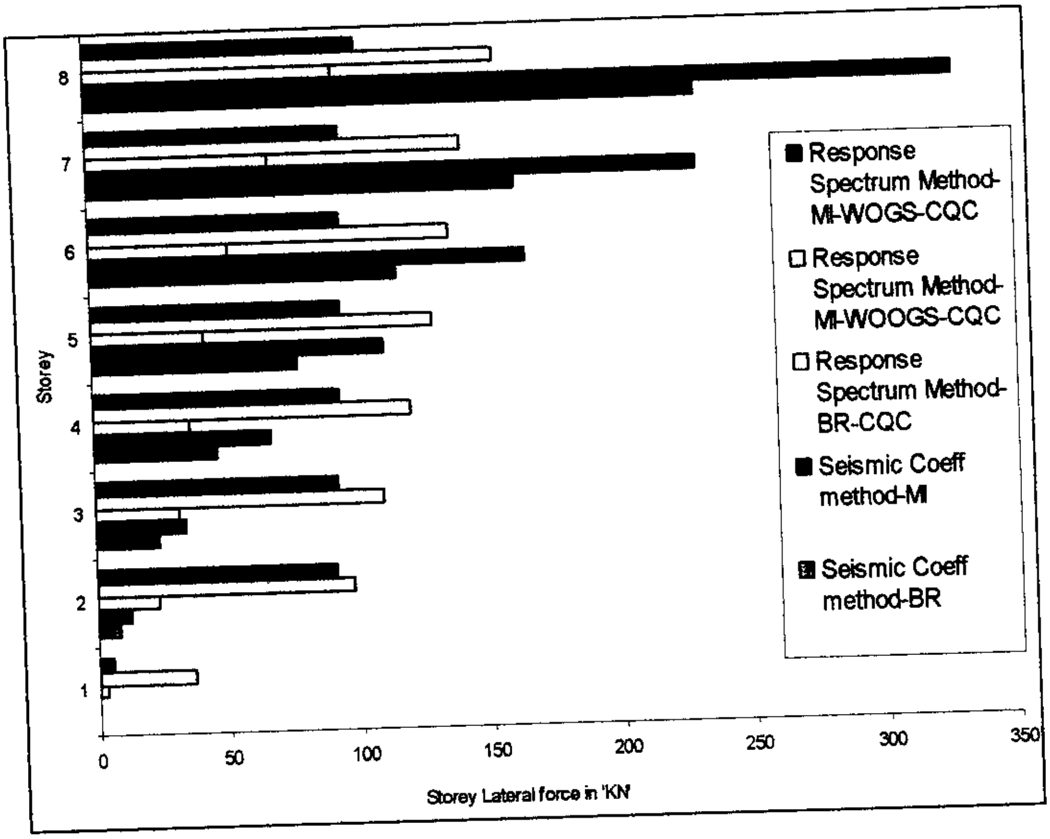


Fig : 7.6 b) 8 Storey Building

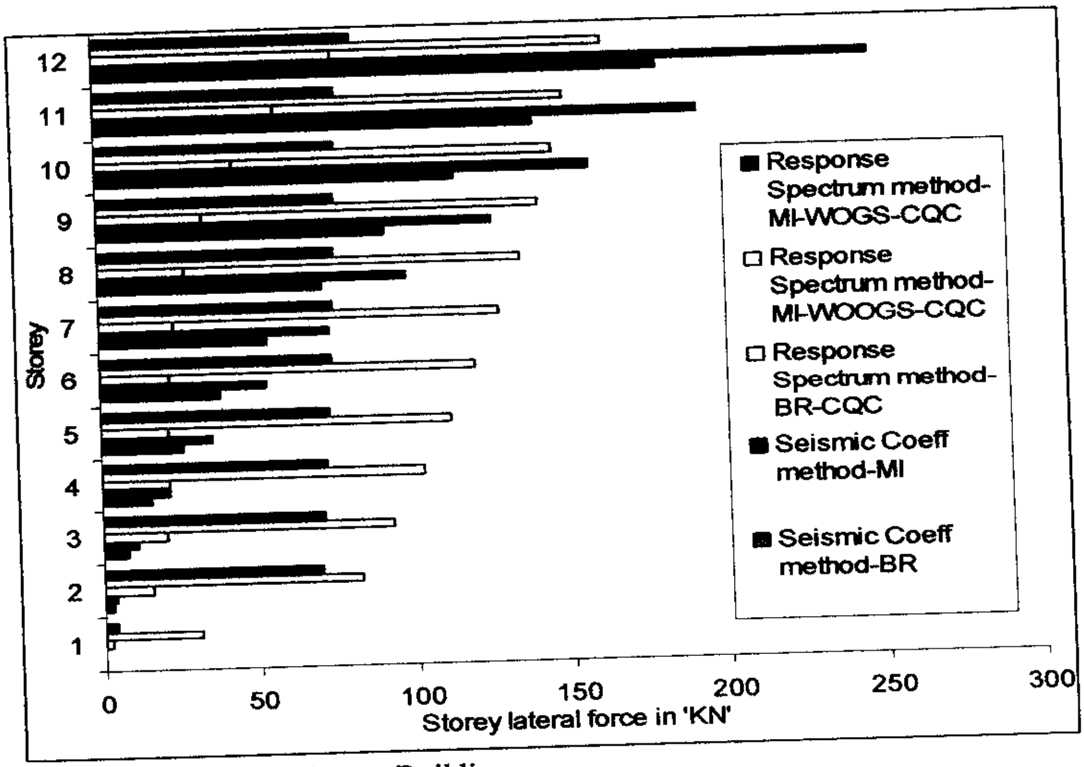


Fig : 7.7 c) 12 Storey Building

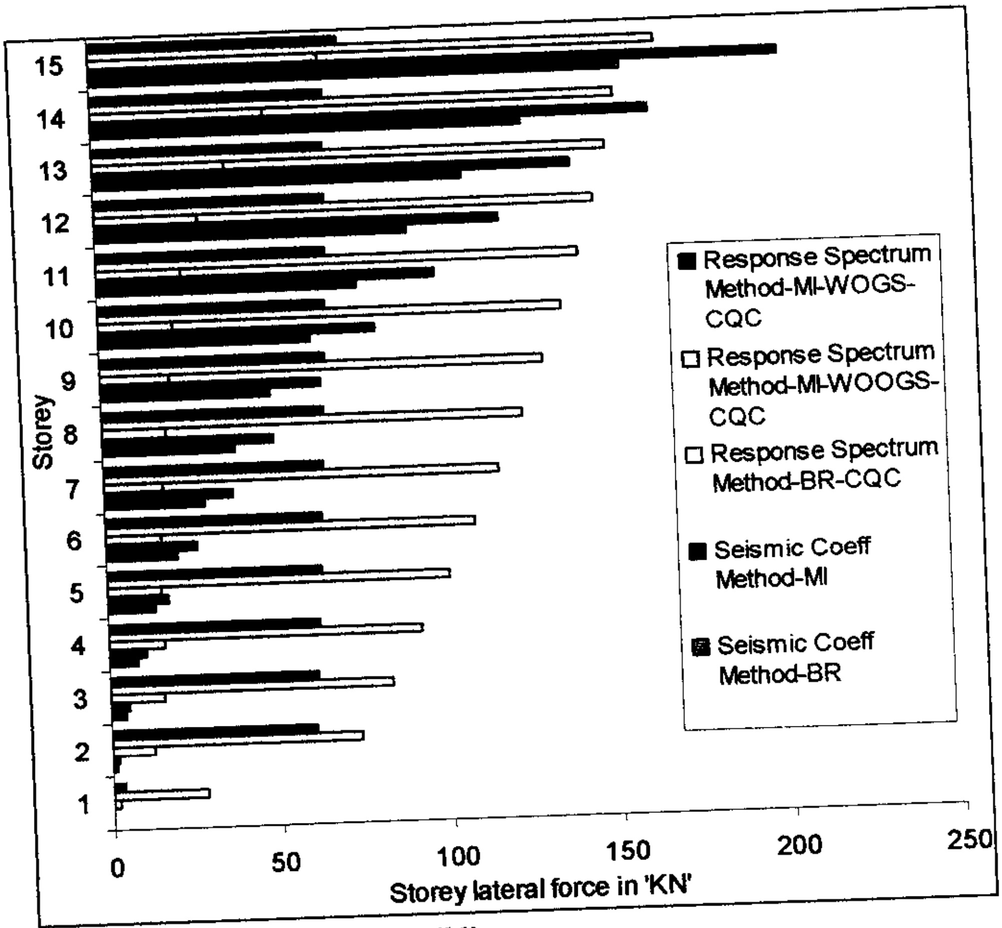


Fig : 7.8 D) 15 Storey Building

Fig.7.5 to 7.8 showing Distribution of Lateral Forces along the height of the building.

It is observed from this study that the lateral force at various floor levels as per Square Root of Sum of Squares (SRSS) method and Combined Quadratic Combination (CQC) method are almost equal.

CHAPTER 8

CONCLUSIONS

Masonry infill in RC frames acts as a diaphragm in vertical plane that imparts significant lateral strength and stiffness to RC frames under lateral loads. Infilled frames also tend to be substantially stronger, but less deformable, than otherwise identical bare frames. In symmetric buildings with vertically continuous infilled frames, the increased stiffness and strength may protect a building from damage associated with excessive lateral drift or inadequate strength. Because of its higher stiffness, infill panels may attract significantly greater forces that may lead to premature failure of infill, and possibly of the whole structure. Therefore, it is essential for designers to consider the effects of infill in the design of RC buildings.

Although masonry infill attracts most of the lateral forces coming on buildings, RC frames must have sufficient strength to prevent the premature failure of buildings in case of failure of masonry walls because of their brittle behavior.

For a linear analysis of a building by the equivalent static force method or the response spectrum method, the modeling of infill walls by the simpler elastic analysis approach is adequate. In this analysis, the elastic member forces are of primary interest. Since the elastic analysis approach gives higher strut width and hence higher stiffness, it leads to a higher base shear.

From this study the following conclusions are made

1. It is found that masonry infill contributes higher percentage of storey stiffness. The Stiffness contribution of masonry infill wall to the RC frame is mainly based on width of the diagonal strut, young's modulus of the masonry as well as thickness of the wall.
2. As expected, the increase in number of storeys increases the fundamental time period.
3. For the given number of storeys, fundamental natural period calculated from masonry infill RC frame analysis gives the least value while the fundamental natural period calculated from bare RC frame analysis gives the highest value.

4. Due to the presence of masonry infill bounded between the RC frame, there is a increased storey stiffness resulting in decreased the time period, which leads to increase in the base shear of the structure.
5. From this study, it is found that the storey shear force in seismic co-efficient method for bare framed structure and infilled structure is parabolic. Storey shear force found by using response spectrum method, for infilled framed structure without open ground storey is trapezoidal thus contradicting the parabolic assumption made in seismic co-efficient method.
6. For infilled structure with open ground storey it is found that the shear force is almost constant for all the stories above the open ground storey i.e. all the storeys above open ground storey will act as a single rigid mass like an inverted pendulum. Much of the deformation is concentrated in the ground floor which is the main cause of failure during past earthquakes. The soft storey concept has technical and functional advantage over the conventional construction. First, is the reduction in spectral acceleration and base shear due to increase in the natural time period of vibration of structure as in a base isolated structure. However, the price of this force reduction is paid in the form of an increase in structural displacement and inter-storey drift, thus entailing a significant P-delta effect, which is a threat to the stability of the structure.

REFERENCES

1. Criteria for Earth Quake Resistant Design Of Structures IS 1893:2002
2. Ductile Detailing Of Reinforced Concrete Structures Subjected To Seismic Forces. IS 13920:1993
3. **IS 456-2000** *Indian Standard Code of Practice for Plain and Reinforced Concrete*. Bureau of Indian Standards. New Delhi. 2000.
4. **Pankaj Agarwal And Manish Shrikhande** (2006) 'Earthquake Resistant Design Of Structures'**Seshu.P** (2007) 'Textbook Of Finite Element Analysis'
5. **Paulay, T.** and **M. J. N. Priestley** *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons. 1992.
6. **Smith, S. B.** (1962) Lateral Stiffness of Infilled Frames. *Journal of the Structural Division*. American Society of Civil Engineers. 88. 183-199.
7. **Smith, S. B.** and **J. R. Riddington** (1978) The Design of Masonry Infilled Steel Frames for Bracing Structures. *The Structural Engineer*. 56B. 1-7.
8. *Manual for Seismic Performance of Reinforced concrete Frame Buildings with Masonry Infill Walls*. Developed by World Housing Encyclopedia. Earthquake Engineering Research Institute , November 2006
9. Proposed Draft Provisions and commentary on Indian Seismic Code IS 1893 (Part 1) By Dr .Sudhir K Jain & Dr. C.V.R. Murthy Department of civil Engineering , IIT Kanpur.
10. **Gnanappa, B., S.** *Behaviour of Multi-Bay RC Frames with Various Infills under Cyclic Loading*. Ph. D. Thesis. Coimbatore Institute of Technology. Tamilnadu.
11. **El-Dakhakhni, W. W., M. Elgaaly** and **A. A. Hamid** (2003) Three Strut Model for Concrete Masonry Infilled Steel Frames. *Journal of Structural Engineering*.American Society of Civil Engineers. 129. 177-185.

12. **Achintya, P. Dayaratnam** and **S. K. Jain** (1991) Behaviour of Brick Infilled RC Frame under Lateral Load. *Indian Concrete Journal.* 65. 453–457

13. **Das, D.** and **C. V. R. Murty** (2004) Brick Masonry Infills in Seismic Design of RC Frame Buildings: Part 2- Behaviour. *Indian Concrete Journal.* 78. 31-38.

14. **Dhanasekar, M.** and **A. W. Page** (1986) The Influence of Brick Masonry Infill Properties on the Behaviour of Infilled Frames. *Proceedings of Institution of Civil Engineers.* Part 2. 81. 593–605.

15. **B. Stafford-Smith**,1966, Behaviour *of square infilled frames*, ASCE Journal the structural engineering division. 91,381-403

16. **IS 1893: Part 1 (2002)**; " *Criteria for earthquake resistant design of structures*", Bureau of Indian Standards, New Delhi.

17. **Pankaj Agarwal And Manish Shrikhande** (2006) '*Earthquake resistant Design of Structures*', Prentice hall India Private Limited, New Delhi.

18. **Mario Paz** (2004) '*Structural Dynamics Theory and computations*', CBS Publishers and Distributors, New Delhi.

19. **Hemant B.Kaushik, Durgesh C.Rai, M.Eeri And Sudhir K.Jain, M.Eeri**, *Code Approaches to Seismic design of Masonry Infilled RC frames*: A State-of-the –art-Review,Earthquake Spectra, Volume 22,No.4,pages 961-983, November 2006, Engineering Research Institute