"PUSHOVER ANALYSIS WITH AND WITHOUT INFILL STIFFNESS CONSIDERATION IN ZONE III AND ZONE IV"

Dissertation submitted in the partial fulfilment of the requirement for the award of

MASTER OF TECHNOLOGY
(STRUCTURAL ENGINEERING)

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CERTIFICATE

This is to certify that the work in this thesis entitled "**Pushover analysis with and without infill stiffness consideration in ZONE III and ZONE IV**"submitted by **Mr. Jitendra Arya** (2K14/STE/06) has been carried out under my supervision in partial fulfillment of the requirements for the degree of **Master of Technology** in Civil Engineering with **Structural Engineering** specialization during session 2014 - 2016 in the Department of Civil Engineering, Delhi Technological University, Delhi. To the best of my knowledge, this work has not been submitted to any other University/Institute for the award of any degree or diploma.

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I am extremely fortunate to be involved in an exciting and challenging research project like "**Pushover analysis with and without infill stiffness consideration in ZONE III and ZONE IV**". This project increased my thinking and understanding capability.

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ABSTRACT

The past earthquakes in which many reinforced concrete structures were severely damaged have indicated the need for evaluating the seismic adequacy of buildings. To make such an assessment, simplified linearelastic methods are not adequate. The aim of this study is to perform non linear static pushover analysis and assess the non linear behavior of building frame on altering the dimensions of the RCC frame structure (i.e. number of bay as well as storey height) for two different earthquake loading(i.e. for Siesmic Zone III and Zone IV) using software SAP2000.

In a building, the main purpose of masonry infilled walls is to fill the gap in between the building frame's horizontal and vertical resisting elements, where it is pre-assumed that Hence, while designing masonry infilled walls do not considered as a structural element, i.e. we design the structure as a bare frame only. But these masonry infilled walls, to a large extent affects structural strength and stiffness properties, and on the other hand, they are very brittle in nature. Some international publications like **FEMA-273/306/356**[5] contains methods for masonary infill walls stiffness calcultations and modelling them as equivalent diagonal pin-jointed strut which is also known as macro modelling of infill walls. And subsequently analyse the inplane effect of masonary infill wall designed as a equivalent diagonal strut on the Reinforced Concrete structure as compared to bare frame structure.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

Recent earthquakes in various parts of the world have exposed the issues pertaining to the seismic vulnerability of existing buildings. The existing building structures, which have been designed and constructed according to former code provisions, do not fulfil requirements of the current seismic code and design practices. Many reinforced concrete buildings in urban regions lying in active seismic zones, besides all four metro cities of India are located in either ZONE III or ZONE IV*, may suffer moderate to severe damages during future ground motions. Indian metro cities are the extremely dense populated cities and it is growing in the recent year. By the virtue of this demand of multi-storey buildings is increasing. So we cannot afford even a single failure of the structure during earthquake. So the optimal design will serve best in terms of performance during earthquake i.e. earthquake resistant design.

Using Indian codes we can design the building as earthquake resistant but how we check the performance of the building? So the Performance based design will serve our intention to pass over the catastrophic failure.

Reinforced concrete frames with masonry infill walls are widespread systems in many earthquake-prone regions of the world. The infill walls are used for insulation and partition purposes rather than structural purposes and generally considered as non-structural elements in structural design.

But various experiments and studies proves that it contributes to Lateral stiffness and resistance of buildings. There is variation of strength due to variation in mechanical properties used in infill (eg. brick masonry, concrete blocks, reinforced concrete etc.) and also the interaction between infill and frame. Additionally, the properties of the infill walls highly depend on the quality of the bricks, mortar and the workmanship. In addition, the properties of the frames, i.e. reinforcement detailing, member capacities, number of bays and stories are the factors that influence the behaviour of the in filled frames.

Effects of infill walls may be either beneficial or detrimental under seismic demands[3]. This improvement on the other hand largely affected by the distribution of infill through the building stories. It is known that existence of infill walls lead to substantial

* The capital of the country (**Delhi**) lies in **Zone** IV, Kolkata lies at the seismic Zones III, **Mumbai** lie in **Zone** III, **Chennai** lies in **Zone** III.

increases in lateral strength and stiffness of the frames compared to those of the bare frames while decreasing the average drifts. However, these effects may or may not be advantageous depending on the case. Infill walls are stiff but brittle elements. If the surrounding frame is not strong enough, infill walls can cause unforeseen damages such as premature failures in columns such as shear, compression or tension failures. Another negative effect may be the development of soft-story mechanism in the structure. This mechanism is more likely to occur in the structures without infill walls at bottom story. Particularly, soft story mechanisms may occur due to drift concentrations at lower stories. An organized stiffness distribution along the height of the structure may help mitigating these negative effects [4].

Nowadays in literature some methodologies are available to simulate the behaviour of in-filled structures; experimental and numerical studies demonstrated that a diagonal strut with appropriate geometrical and mechanical characteristics could be a good solution to take into account the influence of in-fills in the seismic behaviour [2].

1.1 Performance Based Seismic Engineering (PBSE)

1.1.1 Description:

PBSE termed was coined by SEAOC vision 2000 committee in 1995. It covers design, construction, occupancy and maintenance, but particularly emphasis on design which is denominated as PERFORMANCE BASED SEISMIC DESIGN (PBSD)[5]. It is iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved.

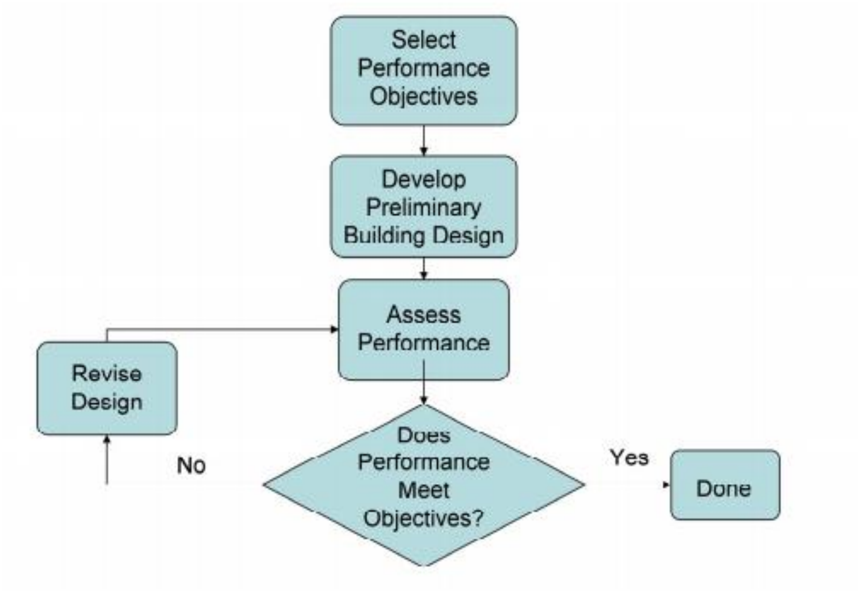


Fig. 1.1: Flowchart of Performance Based Siemic Engineering

The main intention to study PBSE is that the most effect way to minimize the earthquake hazard to building is

- Development of more reliable seismic standards and code provision than those currently available.
- Seismic vulnerability assessment in existing structure i.e.:
    - To obtain a measure of over strength.
    - To obtain a sense of the general capacity of the structure to sustain inelastic deformation.
- Provides a systematic methodology for assessing the performance capability of a building, system or component.
- It can be used to verify the equivalent performance of alternatives.
- It constitutes a terminology that facilitates significant discussion between stakeholders and design professionals on the development and selection of design options.
- It provides a framework for determining what level of safety and what level of property protection, at what cost, are acceptable to stakeholders based upon the specific needs of a project.
- Design individual buildings with a advanced level of confidence that the performance proposed by present building codes will be achieved.

- Assess the potential performance of current prescriptive code requirements for new buildings, and serve as the basis for improvements

1.1.2 AVALAIBLE DOCUMENTS for PBSD (FOR NEW AND EXISTING BUILDINGS)

- SEAOC(1995, 1996, 1996),
- ATC-40(1996)
- FEMA-273,274(1997)
- FEMA 356(2000)
- FEMA350 (2000)
- NEHRP Recommended provision for seismic regulations of buildings and other str. covers in FEMA-302,303(1997) & FEMA-368(2001)

1.1.3 FIRST GENERATION PERFORMANCE BASED PROCEDURES

- Vision 2000 Report, Performance-Based Seismic Engineering of Buildings (SEAOC, 1995).
- FEMA 273 Report, NEHRP Guidelines for the Seismic Rehabilitation of Buildings (ATC, 1997a).
- FEMA 274 NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (ATC, 1997b).
- Introduced the concept of performance in terms of discretely defined performance levels with names proposed to connote the expected level of damage: Collapse, Collapse Prevention, Immediate Occupancy, Life Safety and Operational Performance.
- Introduced the concept of performance connected to damage of both structural and non-structural components.
- Performance Objectives were developed by linking one of these performance levels to a specific level of earthquake hazard.

- Introduced a set of analytical procedures of altering levels of difficulty that could be used to simulate the seismic response of buildings, and provided a comprehensive set of guidelines on nonlinear analysis techniques and acceptance criteria.

1.1.4 PRESENT SECOND GENERATION PERFORMANCE BASED PROCEDURES

- Based on implementation of procedures and criteria contained within the FEMA 356.
- FEMA 356 shows incremental improvement to the first generation procedures of FEMA 273 (depends on information obtained from the use of the procedures in engineering practice).
- FEMA-445 / August 2006)**: (**Next-Generation Performance-Based Seismic Design Guidelines Program Plan for New & Existing Buildings.

1.2 OBJECTIVE, SCOPE OF THE STUDY AND METHODOLOGY

1.2.1 OBJECTIVE :-

The main purpose of the study is to analyze the performance on changing the dimension of the RCC frame structure as well as the effects by including the infill walls on the seismic response of reinforced concrete frames. Particularly, change of inter storey drift allocation with the inclusion of the Infill walls are investigated. And calculate the response reduction factor from that obtained from non-linear Analysis using SAP2000 v15.

In the first part, Pushover analysis without infill stiffness consideration for the following frames in ZONE III and ZONE IV.

STOREY	Bays		
	5X5	6X6	7X7
G+10	✓	✓	✓
G+12	✓	✓	✓
G+14	✓	✓	✓

And then Pushover analysis with infill stiffness consideration for the above mentioned frames (5x5 bays) and seismic zones to get the in plane effect of Infill wall.

Seismic response and the drift allotment of the frames are determined and compared.

1.2.2 SCOPE OF THE STUDY:

This study is done on a R.C.C frame building, which is taken regular in plan.

b) In this study the effects of soil-structure interaction is not taken in account.

c) The column base are taken as fixed in support.

d) Out of plane action of masonry infilled walls is ignored.

e) Building torsional response is not considered.

f) The slabs are are assumed to be rigid diaphragms.

g)No types of irregularities are considered in study.

i) Building has no basement, and soft storeys.

j) Only analysis, not designing of frame elements is the part of the study.

1.2.3 METHODOLOGY:-

(i) Review of the previous studies and literatures and study various Indian Standard Codes available, related to the project.

(ii) Selection of building plans for carrying out study.

(iii) Creating models using SAP2000V18 software for modelling bare frame, and masonry infilled wall frames.

(iv) Applying the dead, live and seismic load as per respective Indian Standard Codes.

(v) Analysis of models created and carrying out comparative study on the basis of results obtained.

(vi) Observation of results.

(vii) Conclusion made from the above study.

1.3 ORGANIZATION OF THE THESIS

Chapter 1, an introductory chapter, dealing with the basic overview of Pushover Analysis, Masonry Infilled Wall Frames and also deals with objective, scope of the study, and methodology adopted to carry out the study.

Chapter 2, contains the various literatures surveyed/studied to develop the understanding required to carry out the project.

Chapter 3, includes the modelling part of structure, as bare frame and masonry infilled wall with varying plan dimension as well as number of storey. It also mentions the various building parameters adopted in the study, different loads applied to structure for carrying out analysis, the modelling of masonry infilled walls by macro modelling method.

Chapter 4, deals with results obtained from carrying out the analysis.

And finally **Chapter 5**, discusses the conclusion made from the results.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

It has been observed by most researchers, that infill masonry panals are rarely included in the analysis of reinforced concrete (R/C) frames. Mainly engineers consider masonry panels as non-structural elements especially in skeleton structure. However, experimental tests showed a important difference in structural response of R/C masonry infilled frames compared to R/C bare frames especially under lateral loading.[6]

Consequently, a lot of research efforts have been directed in recent years to explore the effect of masonry infill panals on the structural behaviour of R/C frames under seismic loads. Additionally, other research efforts have been directed to find the most convenient methods to numerical modelling of masonry infilled R/C frames. The subsequent sections reconsider both the theoretical background and the existing literature for pushover analysis methods for R/C frames and modelling of masonry infill panels.

2.2 PUSH-OVER ANALYSIS

2.2.1.1 DESCRIPTION OF PUSHOVER ANALYSIS

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or tri-linear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is dispersed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve, as shown in Fig. (2.1). Pushover analysis can be performed as force-controlled or displacement controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e, force-controlled procedure should be used when the load is known (such as gravity loading). Moreover, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

Generally, pushover analysis is performed as displacement-controlled proposed by Allahabadi [7] to overcome these problems. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the centre of mass of structure is chosen as the control displacement.

The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check. In this paper we have done the pushover analysis using guidelines of FEMA356 and ATC40.

2.2.1.2 MODELLING OF MASONARY INFILL

Infill walls are two dimensional elements that can be modelled with orthotropic plate element for linear analysis of building with infill wall. The analytical models that represent the behaviour of the infill walls may be categorized into two main groups as micro-models and macro-models.

1. **Micro-models** are based on the finite element method. In this approach, detailed modelling of the infill walls is established by modelling masonry units, mortar and interface elements independently to represent the behaviour of the infill wall more precisely. On the other hand, major computational effort and calibration of high amount of parameters are the disadvantages of the method. Therefore, this approach may be effective for local analyses such as frame-infill interaction or failure modes of the walls, but impractical for global analyses.

2. **Macro-models** use equivalent struts to model the contribution of the infill walls to the response of the in-filled frame. This method replaces the infill panel by two diagonal, compression-only struts as seen in Figure 2.5.

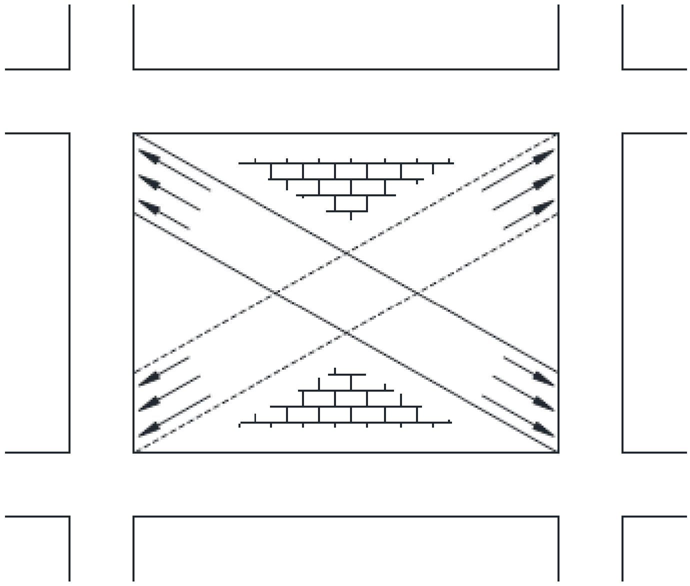


Fig.2.1: Equivalent diagonal strut representation

This approach is beneficial since the masonry is a extremely heterogeneous material and it is hard to predict the material properties of the constituent members accurately. As well, it is possible to obtain mechanical properties of the infill walls from prism tests to model the equivalent struts. IDARC utilizes macro-modelling for infill walls.

A masonry infill strut model is generally defined by an axial stress-strain relationship in the case of monotonic loading, and by the consequent hysteretic rule in the case of cyclic loading, as well as by the strength and stiffness properties, which are closely related to the evaluation of the equivalent strut width[8].

Experimental evidence shows that detachment of the frame from the infill occurs, Holmes [9] has proposed replacing the panel by an equivalent diagonal strut made of the same material as the infill and having a width equal to 1/3 of the infill diagonal length. Based on experimental investigation on diagonally and laterally loaded square infilled steel frames, Stafford Smith [10] has subsequently developed furthermore the idea of an equivalent strut as suggested by Holmes, and provided a numerical procedure to evaluate its dimensions. The procedure proposed in [10] for the evaluation of the geometrical dimensions of the equivalent strut that represents the stiffening effect of the infill is nowadays well accepted. It was found to be sufficient in many situations.

In FEMA 273 [11], FEMA 306 [12] and FEMA 356 [13] it is suggested that the stiffness of the infills is represented in the structural model by equivalent diagonal struts based on the work of Mainstone . The equivalent strut width is given by

$$w = 0.201 \frac{\sqrt{H'^2 + L'^2}}{H^{0.4}\left(E_d\, s \sin(2\theta)\right)^{0.1}} \left(E_f I_c H'\right)^{0.1} \qquad (1)$$

with

$$\theta = \tan^{-1}\left(\frac{H'}{L'}\right) \qquad (2)$$

where is the actual infill thickness that is in contact with the frame, d' the diagonal length of the infill, E_d is the Young modulus of the infill along the diagonal, E_f the Young modulus of the reinforced concrete, H and L are the height and the length of the frame, H' and L' are the height and the length of the infill as shown in Fig.1, finally I_c is the entire inertia moment of the cross-sectional area of the column.

2.2.2 LITERATURE REVIEW ON CALCULATION/DETERMINE THE EFFECT OF INFILL WALL

Testing of infill frames began with monotonic loading. The earliest attempt to perform static experiments on infill frames with monotonically increasing lateral load took place in 1952 (Thomas). Since that time, several researchers (e.g., Benjamin and Williams (1957, 1958a, b), Wood (1958), Sachanski (1960), Holmes (1961, 1963), Stafford-Smith (1962, 1966, 1968), Mallick and Severn (1967), Polyakov (1967) and Zarnic and Tomazevic (1985)) have performed experiments on steel or RC frames infilled with different materials such as mortar, bricks, clinker blocks, hollow, grouted, or RC block masonry and clay blocks.

Fiorato et al. (1970) tested several 1/8 scale, masonry brick infill non-ductile frames and concluded that the infill added significant stiffness and strength to the frames but caused a decrease in their ductility.[15]

T. Elouali at el., (1998) This paper presents the results of an experimental program investigating the behaviour of frame with masonry infill panels subjected to cyclic loadings. Two types of masonry frequently used were tested. The experimental results have been used to develop an analytical model for the determination of the stress-strain relationship to predict the inelastic behaviour of each type of infill.

It shows that the addition of the masonry panels reduces the fundamental periods of the structures. There is a considerable increase in the horizontal base shear forces due to reduction of fundamental period. The displacement may be reduced or increased depending on the frequency contents. The equivalent diagonal representing the confined panels transform the rigid frame into trussed frame, and there is a definite change in the form in which the frame will resist lateral loads; flexural effects will decrease substantially. There is a radical change in bending moments and axial forces. Then the presence of infill should be considered in the design of the frame structures in order to give the strength of the structure and to avoid the possible harmful effects.

Hossain Mohammad Muyeed-Ul-Azam1 et al., (2005) The structural effect of brick infill is usually not considered in the design of columns as well as other structural components of RC frame structures. The lateral deflection is reduced considerably in the infilled frame compared to the deflection of the frame without infill. This leads to different steel requirements for frame structures considering infill. In order to understand the behaviour of frames and steel requirements of column having brick masonry infill and without infill a finite element investigation is performed.

A detailed investigation is performed using a variety of loads and load combinations of the building considering infill and without infill to find out steel requirements and to see the effect of infill in the sway characteristics of the building. It is observed that frames with infill produce much smaller deflections as compared to frames without infill. It is also observed that there is no significant difference in steel requirements of interior column but there is moderate difference in steel requirements in exterior column and significant difference in steel requirements in corner column. This indicates considering stiffness of the infill may not result in an economy in the design of multi-storied buildings if the number of interior columns is considerably greater compared to the number of exterior and corner columns.

Kasım Armagan Korkmaz et al., (2007) The diagonal strut approach is adopted for modelling masonry infill walls. Pushover curves are obtained for the structures using nonlinear analyses option of commercial software SAP2000. Nonlinear analyses are realized to sketch pushover curves and results are presented in comparison and the effects of irregular configuration of masonry infill wall on the performance of the structure are studied. Present study shows that infill walls are under investigation via nonlinear analyses. To determine the earthquake performance of the structural systems, nonlinear static pushover analyses are used instead of time history analyses. The results of elastic analysis show that the presence of nonstructural masonry infill walls can modify the global seismic behavior of framed buildings to a large extent. Irregular distributions of masonry infill walls in elevation can result in unacceptably elastic displacement in the soft storey frame.

Salah El-Din Fahmy Taher at el., (2008) The influence of partial masonry infilling on the seismic lateral behavior of low, medium, and high rise buildings is addressed. The effect of number of stories, number of bays, infill proportioning, and infill locations are investigated. The most simple equivalent frame system with reduced degrees of freedom is proposed for handling multi-story multi-bay infilled frames. The system is composed of a homogenized continuum for the reinforced concrete members braced with unilateral diagonal struts for each bay, which are only activated in compression.

R. Vicente, H. Rodrigues, A. Costa at el., (2010) In this paper, appropriate measures are proposed to improve both in-plane and out-of-plane integrity and the performance behaviour under seismic actions of external leaf of double leaf cavity walls as well as premature disintegration of the infill walls. The infill masonry panels are commonly used in the reinforced concrete (RC) structures as interior or exterior partition walls. They are not considered structural elements; however it is recognized the influence in the global behaviour of RC frames subjected to earthquake loadings

T.C. Nwofor at el., (2012) Reinforced concrete frames are usually infilled by masonry walls, but in most designs, the shear strength response of these walls and also the contribution of the infill panel openings in the reduction of the shear strength of the infilled frame are ignored. In this work, two kinds of numerical models are used in order to validate the finite element micro-modeling method and the basic stiffness method for macro-modeling of infilled frames.

The macro-modeling technique which analyses an equivalent one-strut model used to replace the infill panel gave results which were validated against that of the micro-modeling procedure. From the foregoing both models will able to model the shear response of the frame up to a failure load. Finally the procedure for macro-modeling used in this work is not computationally tedious and gives quick results, hence is recommended for non-linear analysis of infilled frame structures.

The shear strength of infilled frames is reduced with an increase in the opening ratio and remains relatively constant as the opening ratio exceeds 0.5. For a frame without infill panel (i.e. a bare frame) the decrease in the shear strength may reach 75%, decrease in the lateral displacements. Shear strength response of the column was considerably lower than those obtained from a bare frame.

Prof. P.B Kulkarni at el., (2013) In the present study, it is attempt to access the performance of masonry infilled reinforced concrete (RC) frames with open first storey of with and without opening. In this paper, symmetrical frame of college building (G+5) located in seismic zone-III.

From this present result, deflection is very large in case of bare frame as compare to that of Infill frame with opening. If the effect of infill wall is considered then the deflection has reduced drastically. And also deflection is more at last storey because earthquake force acting on it more effectively. Deflection in case of centre opening is large compare to corner opening.

Waleed Abo El-Wafa Mohamed at el., (2012) In this study, a nonlinear numerical investigation on the lateral behavior of masonry infilled RC buildings is carried out. Variety of parameters for both MI (main infill) walls and buildings are considered. The MRF buildings have 6 floors, while the SW-MRF buildings have 5 different heights represented by the number of floors (from six to twenty floors). To check the behavior of infill walls taking into consideration the effect of opening sizes. Nonlinear static push-over analysis is carried out for the applied on buildings. While they can drastically reduce the displacement capacity of MRF buildings to values up to 50.0 %, the existence of uniform RC shear walls can highly restrict the reduction of peak displacement capacity to less than 8.0 %.Masonry infill walls with small thickness equal 0.12 m can significantly alter the response of the buildings, either MRF or SW-MRF, to which they are applied. The variation of masonry infill wall thickness between 0.12 m and 0.2 m yields relatively, minor change in the results of nonlinear lateral response.

MagarPatil H.R. at el., (2012) In this paper, the seismic vulnerability of building with soft storey is shown with an Example of G+10 three dimensional (3D) Steel Frame. The open first storey is an important functional requirement of almost all the urban multi-storey buildings, and hence, cannot be eliminated. Hence some special measures need to be adopted for this specific situation like to increasing the stiffness's of the first storey. In this paper, stiffness balancing is proposed between the first and second storey of a steel moment resisting frame building with open first storey and brick infills as described in models. The stiffness effect on the first storey is demonstrated through the lateral displacement profile of the building.

Dr. S.S.Jamkar et al., (2013) In this present paper to study the behaviour of RC frames with various arrangement of infill when subjected to dynamic earthquake loading. The result of bare frame, frame with infill, soft ground floor and soft basement are compared and conclusion are made in view of IS 1893(2002) code. It is observed that, providing infill below plinth improves earthquake resistant behaviour of the structure when compared to soft basement. Software (ETAB) is used as a tool for analyzing effect of infill on the structural behaviour. It is observed and which provide overestimated values of fundamental period.

Hemchandra Chaulagain at el., (2014) In this context, the paper presents an extensive case study of existing RC-framed buildings in a high seismic risk area in Nepal. A sensitivity analysis of the structures with masonry infill is performed. For this, the influence of different material properties is studied, namely diagonal compressive stress, modulus of elasticity and tensile stress of masonry infill panels. Result shows the influence on the structural behaviour particularly by variation of the diagonal compressive strength of infill masonry panels.

The results of the sensitivity analysis indicate that the variation of diagonal compressive stress on the structure is clearly apparent in all building models. The maximum

IS drift is decreased by 34% and 64% when the diagonal compressive stress of masonry is increased by 25% and 50% respectively.

Pujol and Fick (2010) performed pseudo-static tests on a full-scale, three-story, flat-plate reinforced concrete building which was designed according to modern codes only for gravity loads. Initially, the bare frame was subjected to four cycles of lateral loading showing a triangular distribution. The structure was pushed to roof drifts of 0.22%, 0.45%, 1.5% and 3.0% in consecutive cycles. After the roof drift ratio reached to 2.8%, shear failure observed at a column-slab connection on the third story. After the first test was completed, the infill walls were added into one of the two bays in each story. The structure with infill walls is shown in Figure 1.5. The walls were made out of modular-cored clay bricks and type N mortar. The infilled structure was subjected to twenty cycles of increasing roof drift ratios ranging from 0.025% to 1.25%. Each drift target was applied twice.

CHAPTER 3

BUILDING CONFIGURATION AND MODELLING FOR ANALYSIS

3.1 INTRODUCTION

This chapter deals with the details and design of selected building frames as per the design code procedures. Then the frames modelled for non linear analysis.

The parameters defining building models, basic assumptions and the geometry of the selected building for the study is discussed. This includes the development of concentrated plastic hinges at the critical sections of beams and columns.

3.2 BUILDING CONFIGURATIONS AND MATERIAL PROPERTIES DETAILS

The buildings are assumed which are regular in plan selected with respect to variation in number of bays , no of storey and configuration of infill masonry. Description of buildings are given in the tables as below:

Length of each bay(centre to centre in both direction) = 4 m

Height of each storey = 3m

Table 3.1: Details Of Reinforced Concrete Moment Resisting Frames Without Infill In Zone-III

SL NO	FRAME TITLE	SEISMIC ZONE	No. OF STOREY	No. OF BAYS	FRAME TYPE
1	5B10S-BF-III	III	10	5	BARE
2	5B12S-BF-III	III	12	5	BARE
3	5B14S-BF-III	III	14	5	BARE
4	6B10S-BF-III	III	10	6	BARE
5	6B12S-BF-III	III	12	6	BARE
6	6B14S-BF-III	III	14	6	BARE
7	7B10S-BF-III	III	10	7	BARE
8	7B12S-BF-III	III	12	7	BARE
9	7B14S-BF-III	III	14	7	BARE

Table 3.2: Details of reinforced concrete moment resisting frames with infill in Zone-III

SL NO	FRAME TITLE	SEISMIC ZONE	No. OF STOREY	No. OF BAYS	FRAME TYPE
1	5B10S-IF-III	III	10	5	WITH INFILL
2	5B12S-IF-III	III	12	5	WITH INFILL
3	5B14S-IF-III	III	14	5	WITH INFILL

Table 3.3: Details of reinforced concrete moment resisting frames without infill in Zone-IV

SL NO	FRAME TITLE	SEISMIC ZONE	No. OF STOREY	No. OF BAYS	FRAME TYPE
1	5B10S-BF-IV	IV	10	5	BARE
2	5B12S-BF-IV	IV	12	5	BARE
3	5B14S-BF-IV	IV	14	5	BARE
4	6B10S-BF-IV	IV	10	6	BARE
5	6B12S-BF-IV	IV	12	6	BARE
6	6B14S-BF-IV	IV	14	6	BARE
7	7B10S-BF-IV	IV	10	7	BARE
8	7B12S-BF-IV	IV	12	7	BARE
9	7B14S-BF-IV	IV	14	7	BARE

Table 3.4: Details Of Reinforced Concrete Moment Resisting Frames With Infill In Zone-IV

SL NO	FRAME TITLE	SEISMIC ZONE	No. OF STOREY	No. OF BAYS	FRAME TYPE
1	5B10S-IF-IV	IV	10	5	WITH INFILL
2	5B12S-IF-IV	IV	12	5	WITH INFILL
3	5B14S-IF-IV	IV	14	5	WITH INFILL

Further it is R.C.C building which is specified as Special Moment Resisting Frame (SMRF).Various other details related to building and materials used is mentioned in the Table.3.5 below.

Table 3.5: Building Geometry And Material Property

S.no.	**Design parameter**	**Value**
1	Floor Height (c/c)	3m
2	Size of Beam	350X 450 mm
3	Size of Column	500 X500mm
4	Unit Weight of Concrete	25 KN/m^3
5	Unit Weight of Masonry Infilled Walls	20 KN/m^3
6	Characteristic Strength of Concrete (f_{ck})	30 MPa
7	Characteristic Strength of Masonry Infilled Walls	3.89 MPa
8	Modulus of Elasticity of Concrete (E_c)	$5000\sqrt{f_{ck}}$
9	Modulus of Elasticity of Masonry Infilled Walls (E_m)	5500 MPa
10	Poison’s Ratio for Concrete	0.20
11	Poison’s Ratio for Masonry Infilled Walls	0.17
12	Slab Thickness	150 mm
13	Masonry Infilled Walls Thickness	230 mm
14	Angle made by Strut with the Horizontal (θ)	36.07°

3.3 LOAD CALCULATIONS

3.3.1 SIESMIC DESIGN DATA

The designed seismic data for assumed SMRF building is shown in Table.3.6

Table 3.6: Seismic Design Data

S.NO.	Design Parameter	Values	Values
1	Seismic Zone	III	IV
2	Zone Factor	0.16	0.24
3	Response Reduction Factor (R)	5	5
4	Importance Factor (I)	1	1
5	Soil Type	Medium Soil (type II)	Medium Soil (type II)
6	Damping Ratio	5%	5%
7	Frame Type	Special Moment Resisting Frame	Special Moment Resisting Frame

3.3.2 GRAVITY LOAD CONSIDERED FOR DESIGN

DEAD LOAD (IS875:Part 1)

(i) Dead load of Beams and Columns: As per unit weight of material and dimensions.

(ii) Dead Load on floor slabs (Flooring Load) : 1.5 KN/m²

(iii) Dead Load on roof slab (Flooring Load) : 2 KN/m²

(iv) Dead Load on Periphery Beams (Exterior Wall Load,230mm thick) : 11.73KN/m

(v) Dead Load on Interior Beams (Interior Wall load,115 mm thick) : 5.865KN/m

(vi) Dead on Periphery Beams of Roof (Parapet Wall load,1m high) : 4.6 KN/m

LIVE LOAD (IS875:Part 2)

(i) Live Load on Floor Slabs (except roof) : 2 KN/m²

(ii) Live Load on Roof Floor Slab : 1.5 KN/m²

As per IS 1893:2002, clause 7.3.1, the percentage of live load considered for seismic load calculation is 25%.

3.4 MODELLING OF FRAME MEMBERS AND MASONRY INFILLED WALLS:-

For modelling of masonry infilled wall in the frame, two types of modelling are as mentioned below:

a. Micro Modeling: It is based on finite element method. In this approach, thorough modeling of the infill walls is established by modeling masonry units, mortar and boundary elements separately to represent the performance of the infill wall more accurately.

But major computational effort and calibration of large amount of parameters are the demerits of the method. Therefore, this approach may be efficient for local analyses such as frame-infill interaction or failure modes of the walls, but impractical for global analyses.

b. Equivalent diagonal Strut (Macro Modelling):

In this method the masonry infill walls are modelled as equivalent diagonal pin-jointed strut having an effective width as proposed by different researchers. Here in this study the method given in FEMA 356 [13] has been used to find out the effective width of equivalent diagonal pin-jointed strut. The proposed method is:

$$W = 0.175(\lambda H)^{-0.4} d_m \qquad \text{.......3.1}$$

where,

$$\lambda = \sqrt[4]{\frac{E_m t \sin 2\theta}{4 E_c I_c h_m}} \qquad \text{.......3.2}$$

Where, E_m is modulus of elasticity of masonry infilled walls, t is thickness of masonry infilled walls, E_c is modulus of elasticity of concrete, I_c is moment of inertia of columns, h_m is height of masonry infill, θ is the angle made by strut with the horizontal, H is the height of the floor (c/c) and d_m is the length of diagonal pin-jointed strut.

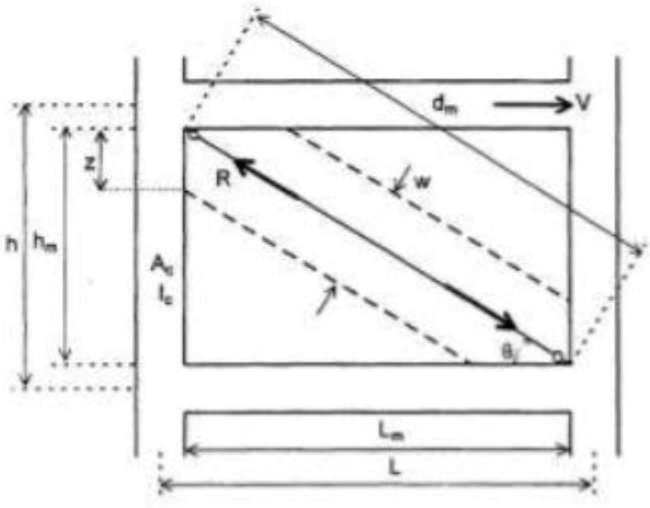


Fig.3.1: Equivalent Diagonal Pin-Jointed Strut

In this paper our scope limited to analyse the building using above mentioned method (i.e.macro modeling of infill masonary).

Table 3.7 Calculation Of Equivalent Diagonal Strut

LENGTH OF BAY @EACH DIRECTION IN PLAN(m)	**4**
HEIGHT OF STOREY(m)	**3**
BEAM DEPTH(mm)	**450**
SHEAR STRESS(N/mm2)	**0.2**
ELASTIC MODULUS INFILL(E_{me} , N/mm^2)	**2300**
ELASTIC MODULUS frame material(E_{fe}, N/mm^2)	**25000**
POSSIONS RATIO (μ)	**0.15**
WIDTH OF INFILL(t)	**0.23**
square COL. DIMENSION (m)	**0.5**
I_{col} (in^4)	**12513.0**
DIAGONAL LENGTH OF INFILL(r)	**4.33**
θ (radians)	**0.63**
Λ	**0.02**
WIDTH OF STRUT(m)	**0.49**

3.5 PUSHOVER ANALYSIS METHODOLOGY

Pushover analysis is a static, nonlinear procedure in which the level of the structural loading is incrementally increased in harmony with a certain predefined pattern. The ATC-40 and FEMA-356 documents have developed modeling parameters, acceptance criteria and procedures of pushover analysis. These documents also describe the method to determine the yielding of frame member during the analysis. Two methods as shown in Figure (3.2) are used to govern the inelastic behavior of the member during the pushover analysis, that are deformation-controlled (ductile action) or force-controlled (brittle action).

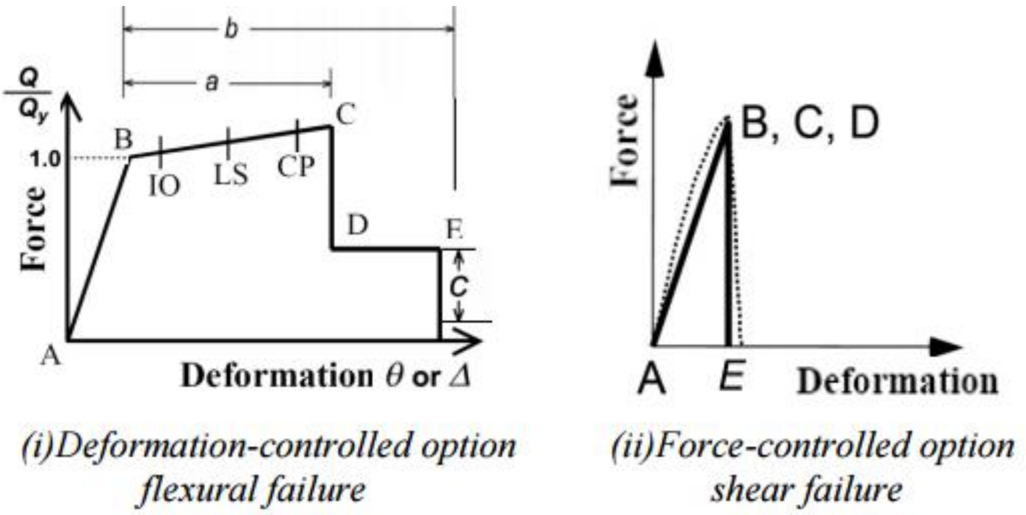


Fig. 3.2: Force- Deformation Behavior Of Hinges

3.5.1 ACCEPTANCE CRITERIA

3.5.1.1 PERFORMANCE LEVEL OF BEAMS AND COLUMNS:

When the structure is analyzed with three loading conditions (GRAV, EQX and EQY), pushover curve of the structure is obtained. The curve is the base shear vs deformation curve.

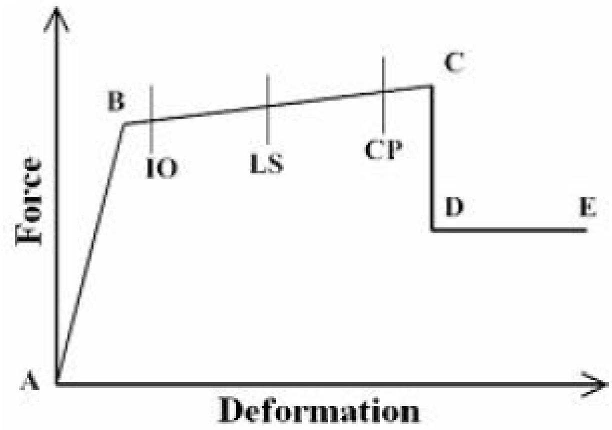


Figure 3.3: Component Force-Deformation Curve

A common component behaviour curve is shown in figure3.3. The points marked on the curve are expressed as follows:

- Point A is the origin
- Point B represents yielding. No deformation occurs in the hinge up to point B, not considering the deformation value specified for point B, the deformation (rotation) at point B will be deducted from the deformations at points C, D, and E. Only the plastic deformation beyond point B will be exhibited by the hinge.
- Point C represents the ultimate capacity for pushover analysis. However, a positive slope from C to D may be specified for other purposes.
- Point D represents a residual strength for pushover analysis. However, a up slope from C to D or D to E may be specified for other purposes. However in the nonexistence of the modelling of descending branch of a load vs deformation curve, the residual strength can be supposed to be 20% of the yield strength.
- Point E shows total failure. Beyond point E on the horizontal axis, if it is not required that the hinge to fail this way, a large value for the deformation at point D may be provided.

Three points categorized IO, LS and CP as referred in Figure (3.3) are used to describe the Acceptance Criteria or performance level for the plastic hinge formed in the vicinity of the joints (at the ends of beams and columns). IO, LS and CP are abbriviated form of Immediate Occupancy, Life Safety and Collapse Prevention, respectively. The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA- 273 documents. Tables (3.9) and (3.10) show the values of Acceptance Criteria for both beams and columns, whereas Table 3.11 describes the structural performance levels of the concrete frames.

3.4.1 NON-LINEAR MODELLING OF INFILL WALLS:-

The simplified tri-linear stress strain model for masonry infill proposed by Kaushik [16] is used in this study (Figure 3.4).

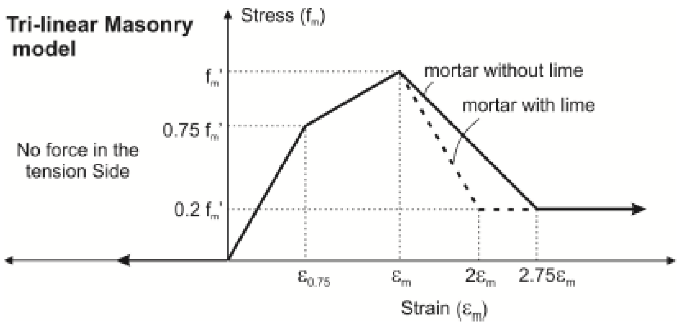


Fig. 3.4: Trilinear Model suggested by Kuashik et. al.

The control points on stress-strain curves for infill model used in SAP 2000v18 program is summarized in Table 3.8

Table 3.8: Stress -Strain Relationship For Brick Masonary (1:6) Suggested By Kuashik Et. Al.[16]

Stress level	Brick Masonary	
	Stress(MPa)	Strain
0.75fm’	3.075	0.0015
1.00fm’	4.1	0.003
0.20fm’	0.82	0.006
0.20fm’	0.778	0.008

3.5 MODELLING OF STRUCTURAL ELEMENTS:

In SAP2000, the structural frame section (beam and Column) are considered as line elements. Slab acts as rigid diaphragm which shows integral behavior of vertical load resisting elements.

3.5.1 NON-LINEAR MODELLING OF BEAM & COLUMNS

In pushover analysis it is essential to model all elements load - deformation curve for all elements. The beams and columns are modelled as frame elements and the infill walls are modelled as equivalent diagonal pin jointed strut by truss elements. Since the deformations are likely to go beyond the elastic range in a pushover analysis so it is essential to model all elements load versus deformation curve for all elements. The non-linear behaviour is integrated in the load versus deformation property of a concentrated hinge attached to the member. A beam is assigned with a moment versus rotation for a section where hinge is expected to form. In addition to that a shear force versus shear deformation curve is defined to model the possible shear failure at a section. Similarly, a column is also assigned with flexible and shear hinges. For equivalent strut, the hinge is placed at the middle length of the strut with an axial load versus deformation curve.

Table 3.9: Modelling parameter of beams [FEMA 356]

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases[5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.

Table 3.10: Modelling Parameter Of Column [FEMA 356]

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
			Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians — Performance Level				
							Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 3.11 Description Of Performance Levels Of The Concrete Frame [FEMA 356]

Elements	Type	Structural Performance Levels: Collapse Prevention	Life Safety	Immediate Occupancy
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16" width.
	Drift[2]	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

3.6 BEHAVIOR PARAMETER OF THE BUILDING

In force based seismic design procedure, R is force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra.(Maheri and Akbari,2011). Hence, structure is designed for seismic force much less than what is expected under strong shaking if the structure were to remain linearly elastic.

$$R = V_e / V_d$$

The factor R is an empirical response reduction factor intended to account for damping, over strength, and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system [1].

Now, the IS code provides the realistic force for elastic structure and divides those forces by (2R)

$$\text{Force reduction factor (2R)} = \frac{\text{Elasticstrengthdemand}}{\text{Designstrength}} = R\mu\ \Omega$$

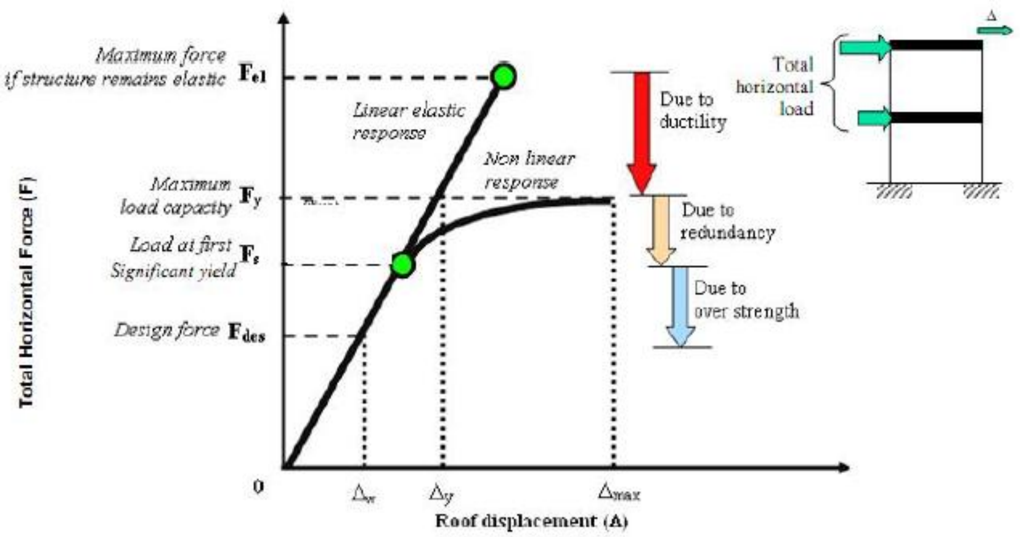


Fig. 3.5: Concept Of Response Reduction Factor

3.6.1 RESPONSE REDUCTION FACTOR FORMULATION

ATC 19 describe R is constitute by three factors

$$R = R_S \cdot R_\mu \cdot R_R \qquad \text{.................3.3}$$

R_s represent over strength and calculated to be equal to the maximum base shear force at the yield level (V_y) divided by the design base shear force (V_d).

Rμ is ductility factor and calculated as the base shear (V_e) for elastic response divided by the yield base shear (V_y).

R_r is redundancy factor.

3.6.1.1 OVER STRENGTH FACTOR

The structure has ultimately reached its strength and deformation capacity. The supplementary strength beyond the design strength is called the overstrength.

Over strength factor (Ω) = apparent strength/design strength

$$\Omega = V_u/V_d \qquad ......3.4$$

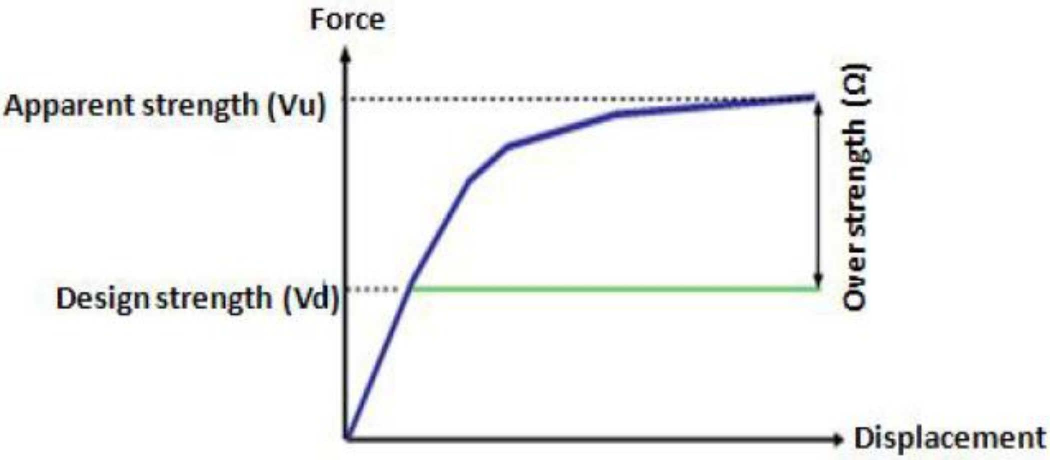


Fig. 3.6: Force Displacement relationship for overstrength

3.6.1.2 Ductility Reduction Factor:-

T. Paulay and M. J. N. Priestley [17]

This theory define the ductility factor is the ratio of maximum deformation to the yield deformation and proposed the following equations for the determination of ductility reduction factor (Rμ).This theory divides the time period of the structure for calculating ductility reduction factor.

$$\left.\begin{array}{ll} R_\mu = 1.0 & \text{for zero-period structures} \\ R_\mu = \sqrt{(2\mu - 1)} & \text{for short-period structure} \\ R_\mu = \mu & \text{for long period structure} \\ R_\mu = 1 + (\mu-1)\, T/0.70 & (0.70\ s < T < 0.3) \end{array}\right\} \qquad ....3.5$$

3.6.1.3 REDUNDANCY FACTOR

Redundant is usually defined as: exceeding what is required or naturally unnecessary. Building should have a high degree of redundancy for lateral load resistance. More redundancy in the structure leads to amplified level of energy dissipation and more overstrength. In a non-redundant system the failure of a member is equivalent to the failure of the entire structure however in a redundant system failure will occur if more than one member fails. Thus, the reliability of a system will be a function of the system's redundancy meaning that the reliability depends on whether the system is redundant or non-redundant. Overstrength, redundancy and ductility together contribute to the fact that an earthquake resistant structure can be designed for much lower force than is implied by the strong shaking.

3.6.2 FORMULATION USED FOR THIS STUDY

For the determination of Overstrength factor (Ω) concept of FEMA P695 is
used, which gives

$$\Omega = Vu / Vy \text{ X } Vy/Vd = R_s \text{ X } R_r$$

$$\Omega = Vu/Vd \qquad \text{...........3.6}$$

The expression of equation (3.6) is same as the indication given by IS 1893-2002. For the determination of displacement ductility following expression is used

$$\mu = \Delta u / \Delta y \qquad \text{...........3.7}$$

For determination of ductility reduction factor $R\mu$, equation (3.5) is used i.e. depends on time period.

For the determination of Response Reduction Factor (R), the main concept given by ATC-19 is used, which is given in equation (3.3)

$$R = R_s \times R_R \text{ X } R_\mu$$

But in our case, Overstrength and redundancy factor is taken as single term i.e overstrength factor and the IS 1893-2002 gives the value of
Force Reduction Factor = (2R),
Same concept is used to determine Response Reduction Factor of the study structures.

$$2R = \Omega \times R\mu$$

$$R = \Omega \times R\mu/2$$

SUMMARY

All the data regarding the analysis and design is mentioned in above tables. Pushover methodology, procedure for modelling structural elemnts and infill wall strut is also discussed in this chapter. And also the concept of building behaviorical parameter and ccalculation procedure is also discussed.

CHAPTER 4

RESULT AND DISCUSSIONS

4.1 LINEAR ANALYSIS RESULTS

4.1.1. INTERSTOREY DRIFT RATIO:

Inter-story drift is one of the particularly useful engineering response quantity and indicator of structural performance, especially for high-rise buildings. It is relative translational displacement between two consecutive floors

4.1.1.1 DRIFT COMPARASION OF TEN STOREY FRAMES

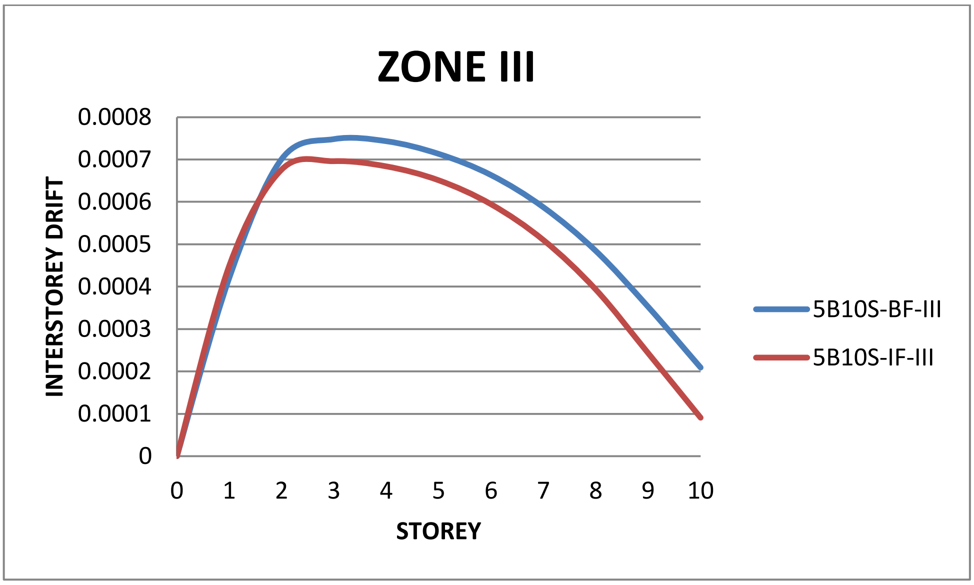


Fig. 4.1: Storey Drift Comparasion Between Bare Frame And Infill Frame In Zone III

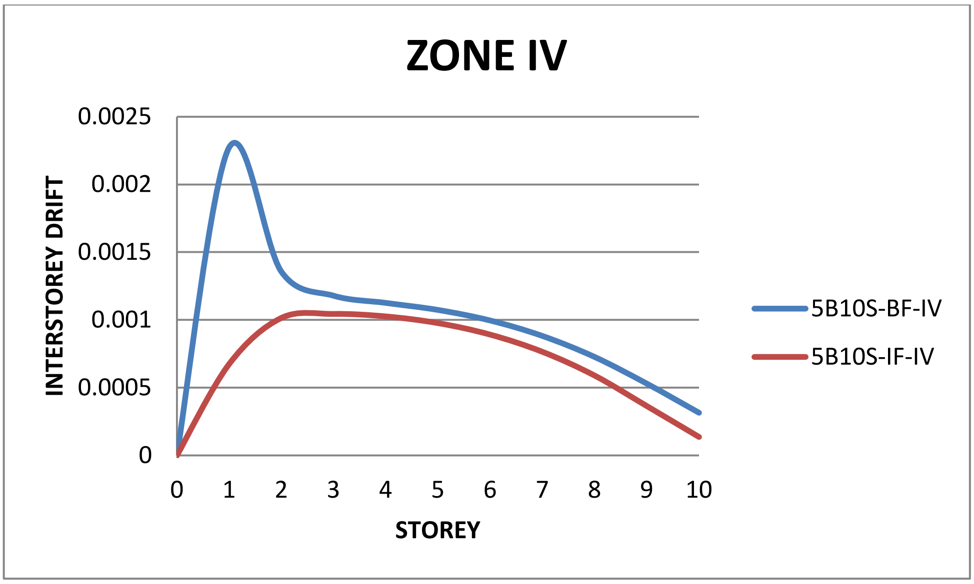


Fig. 4.2: Storey Drift Comparasion Between Bare Frame And Infill Frame In Zone IV

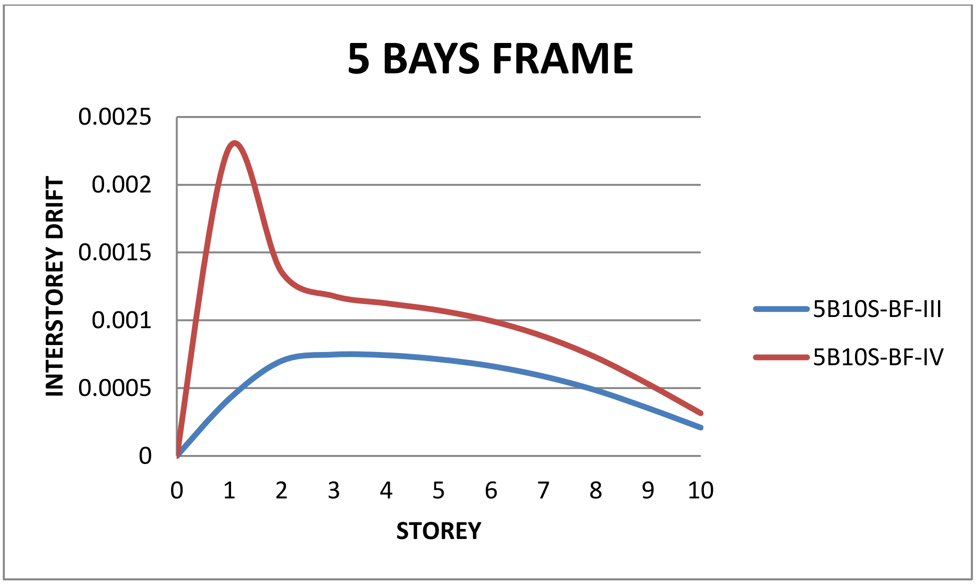


Fig. 4.3: Storey Drift Comparasion For Five Bays Bare Frame In Zone III & Zone IV

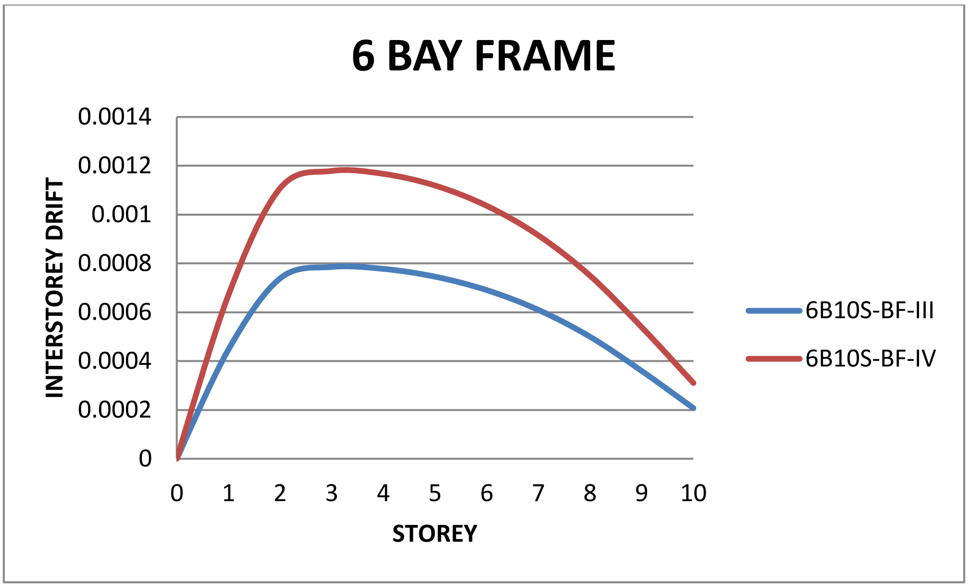


Fig. 4.4: Storey Drift Comparasion For Six Bays Bare Frame In Zone III &Zone IV

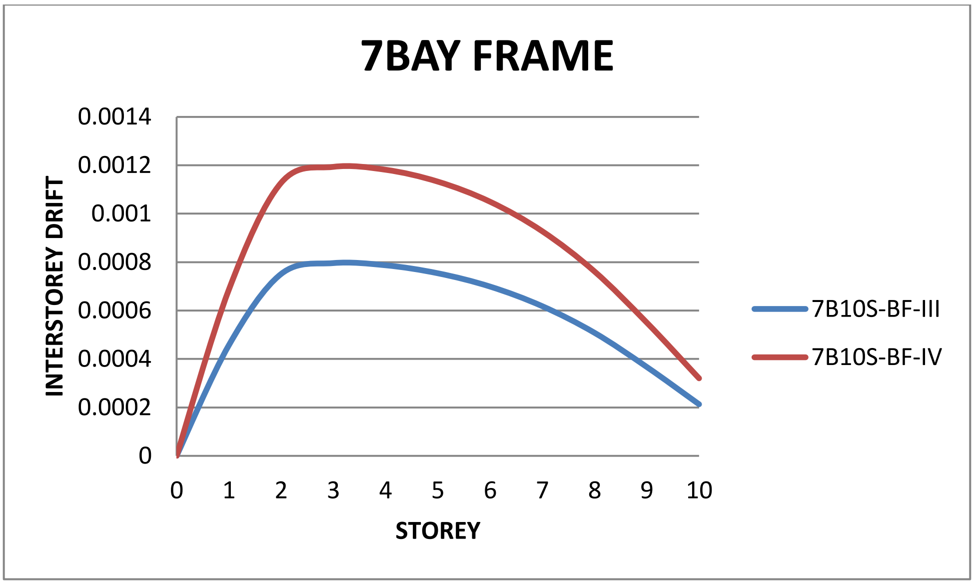


Fig. 4.5: Storey Drift Comparasion For Seven Bays Bare Frame In Zone III &Zone IV

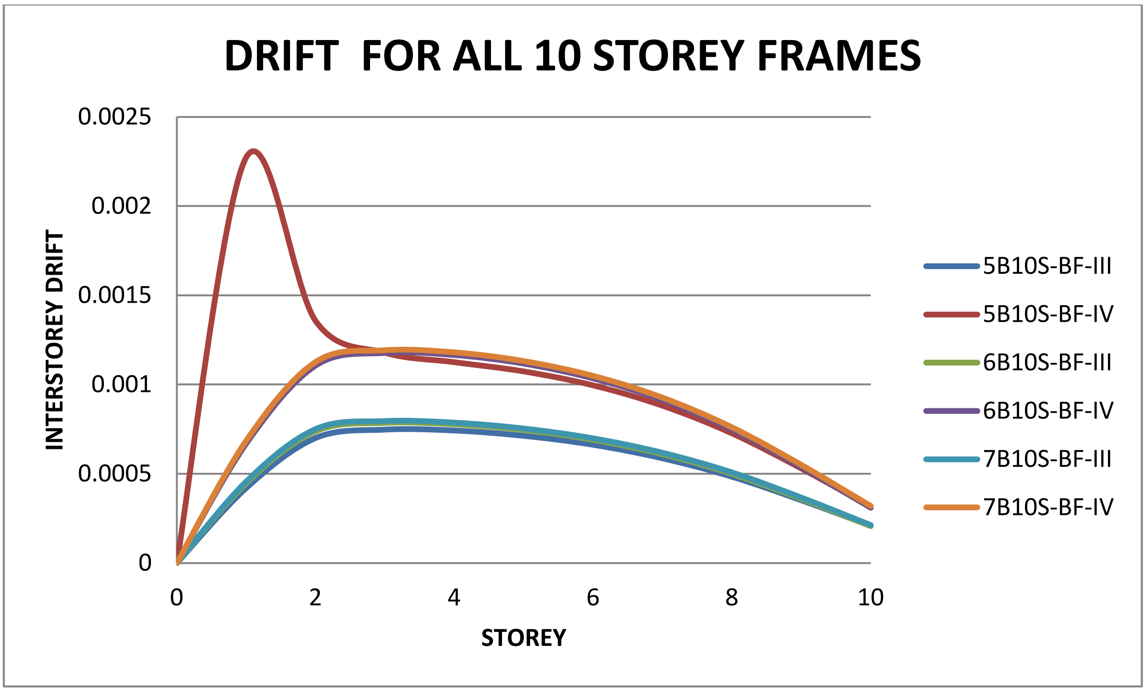


Fig. 4.6(a): Storey Drift Comparasion For All Ten Storey Bare Frames

4.1.1.2 DRIFT COMPARASION OF TWELVE STOREY FRAMES

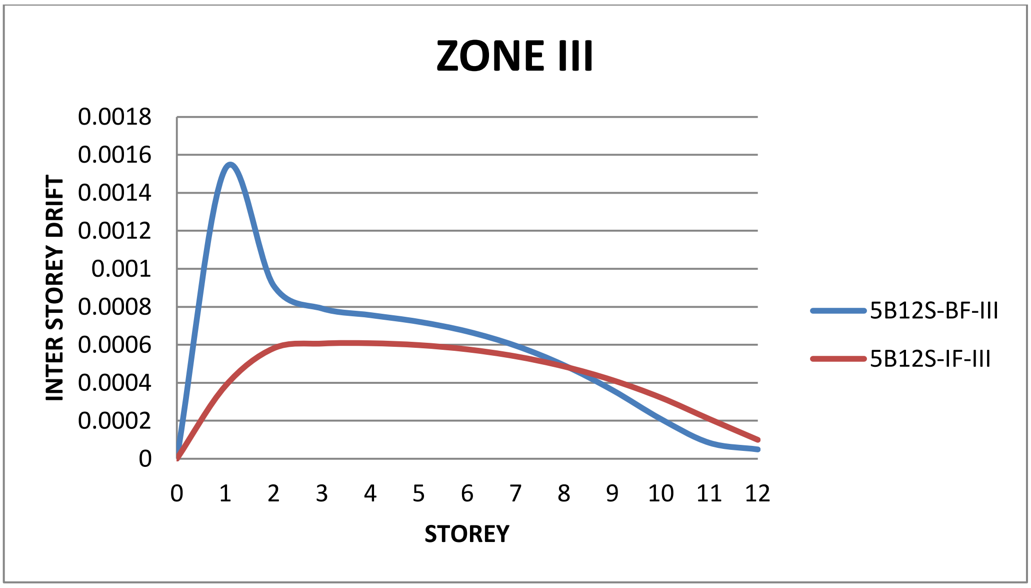


Fig. 4.6(b): Storey Drift Comparasion Between Bare Frame And Infill FRAME IN ZONE III

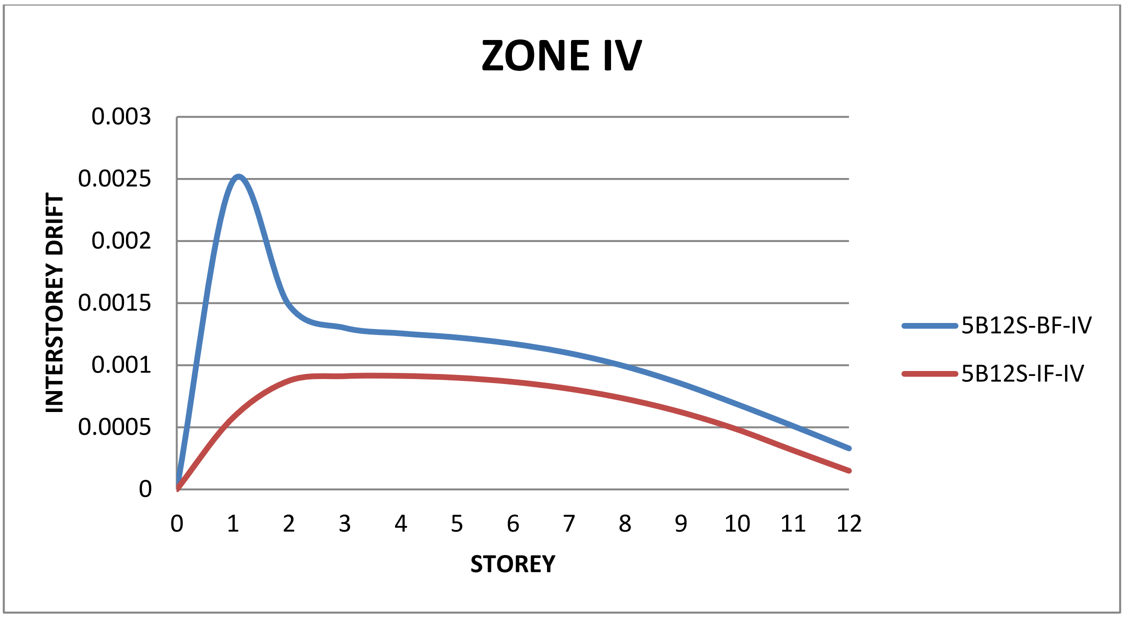


Fig. 4.7: Storey Drift Comparasion Between Bare Frame And Infill Frame In Zone IV

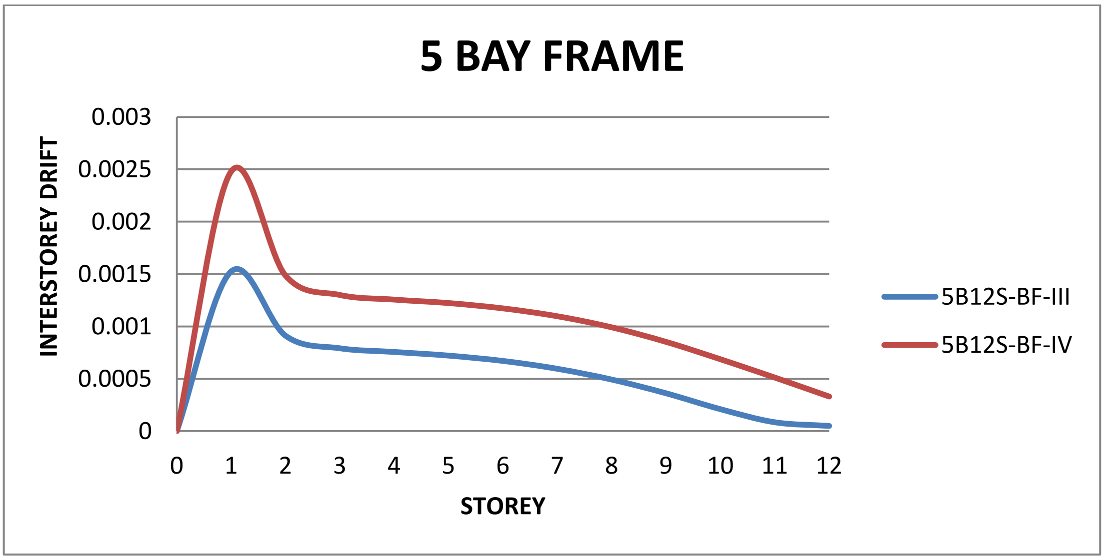


Fig. 4.8: Storey Drift Comparasion For Five Bays Bare Frame In Zone III &Zone IV

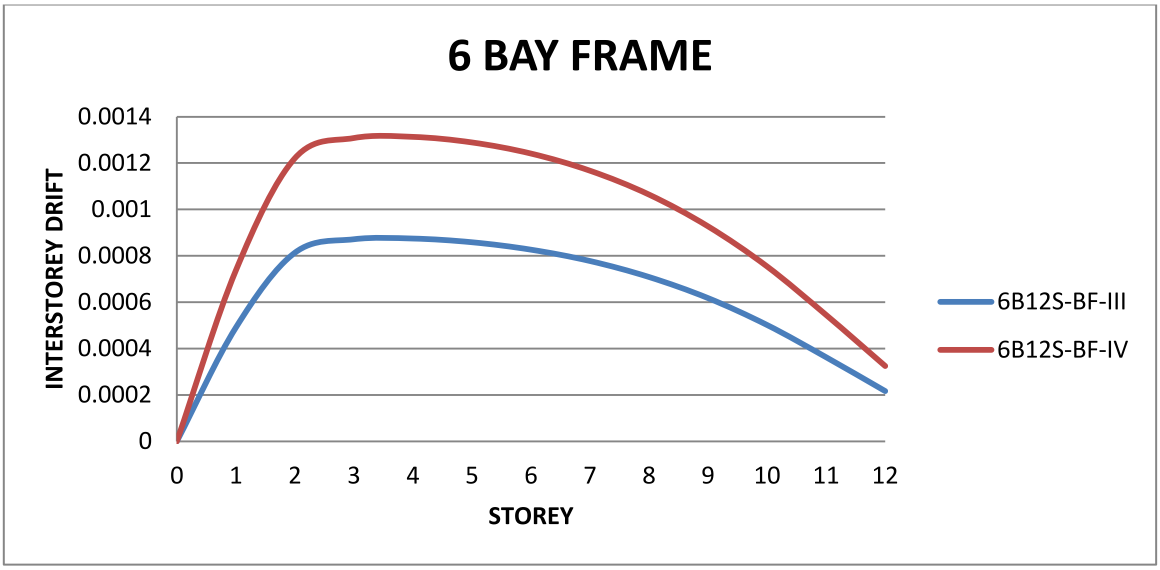


Fig. 4.9 Storey Drift Comparasion For Six Bays Bare Frame In Zone III &Zone IV

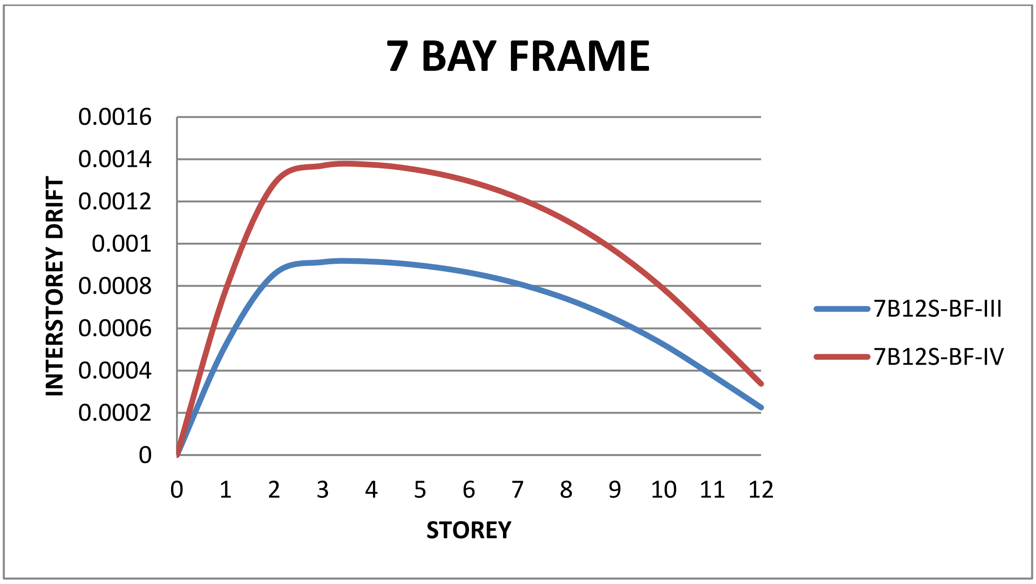


Fig. 4.10: Storey Drift Comparasion For Seven Bays Bare Frame In Zone III &Zone IV

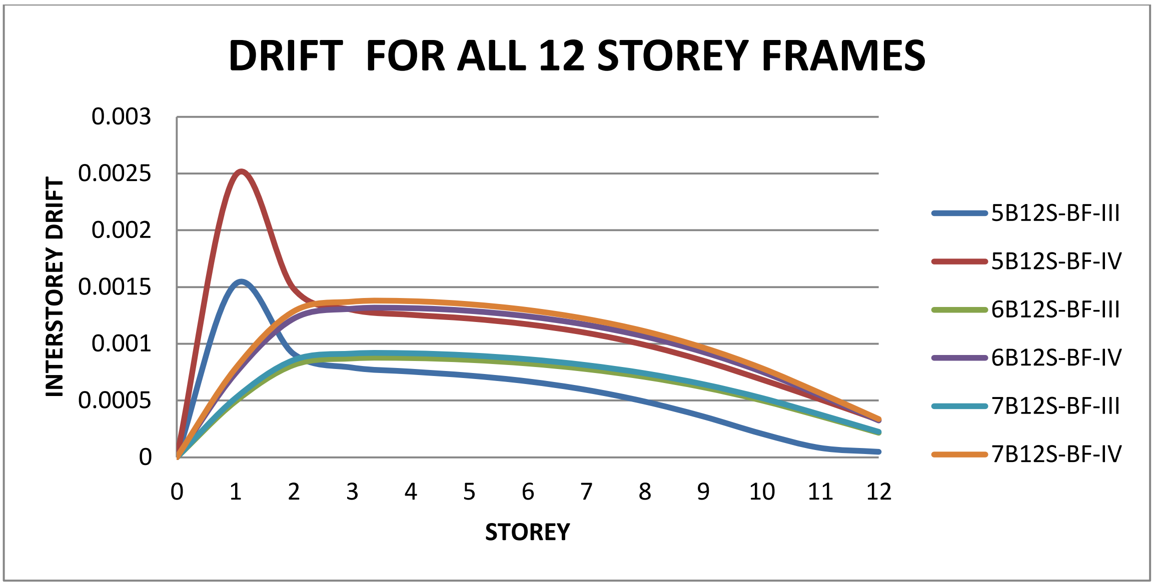


Fig. 4.11: Storey Drift Comparasion For All Twelve Storey Bare Frames

4.1.1.3 DRIFT COMPARASION OF FORTEEN STOREY FRAMES

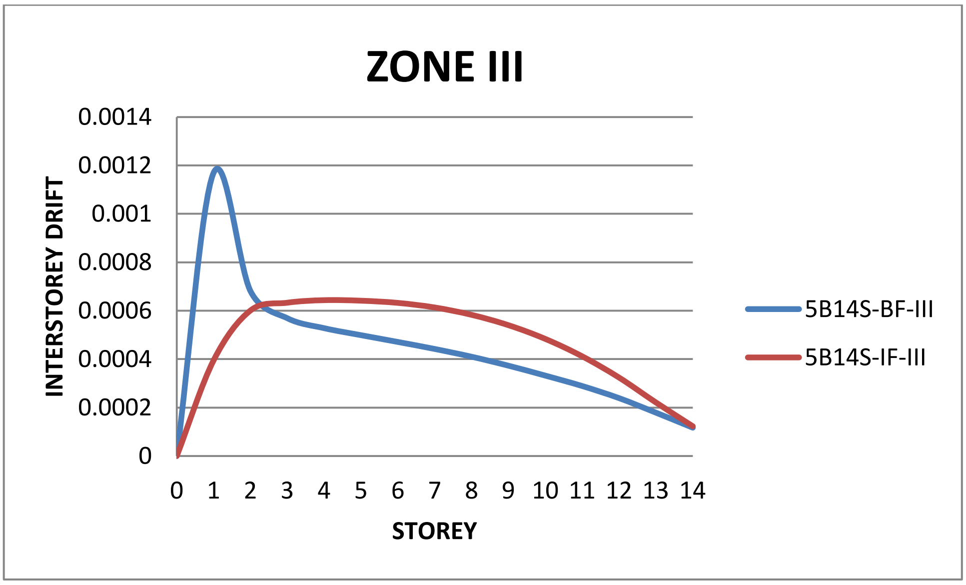


Fig 4.12: Storey Drift Comparasion Between Bare Frame And Infill Frame In Zone III

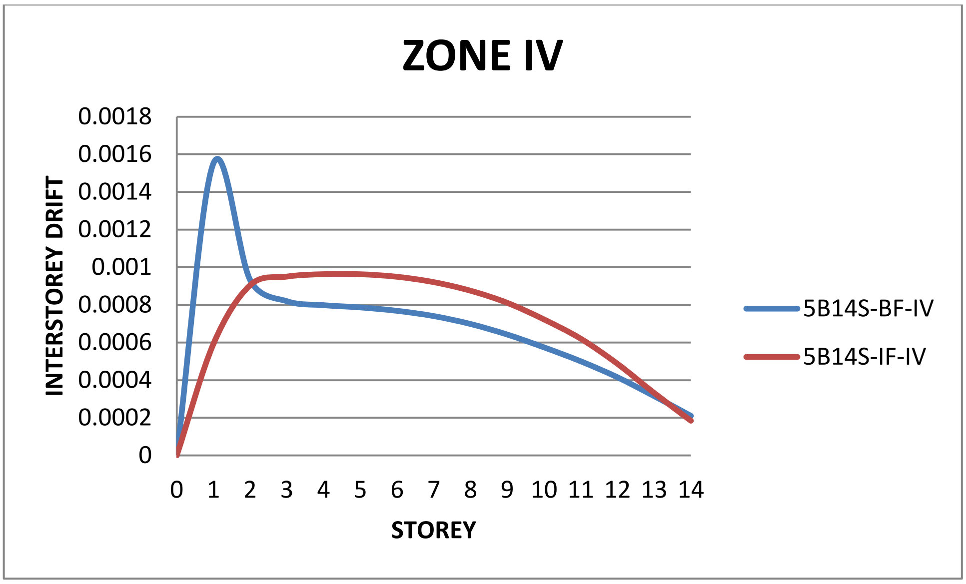


Fig 4.13: Storey Drift Comparasion Between Bare Frame And Infill Frame In Zone IV

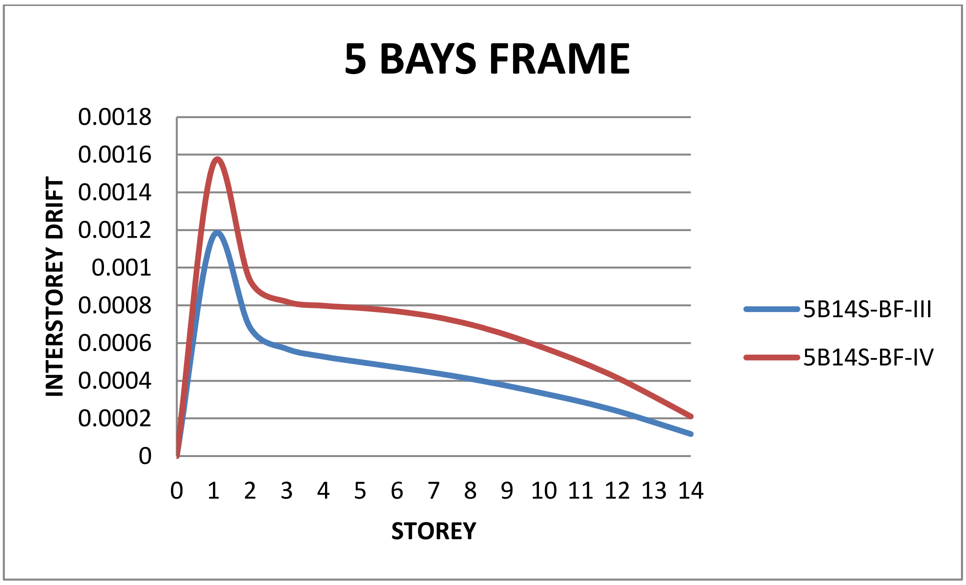


Fig 4.14: Storey Drift Comparasion For Five Bays Bare Frame In Zone III & Zone IV

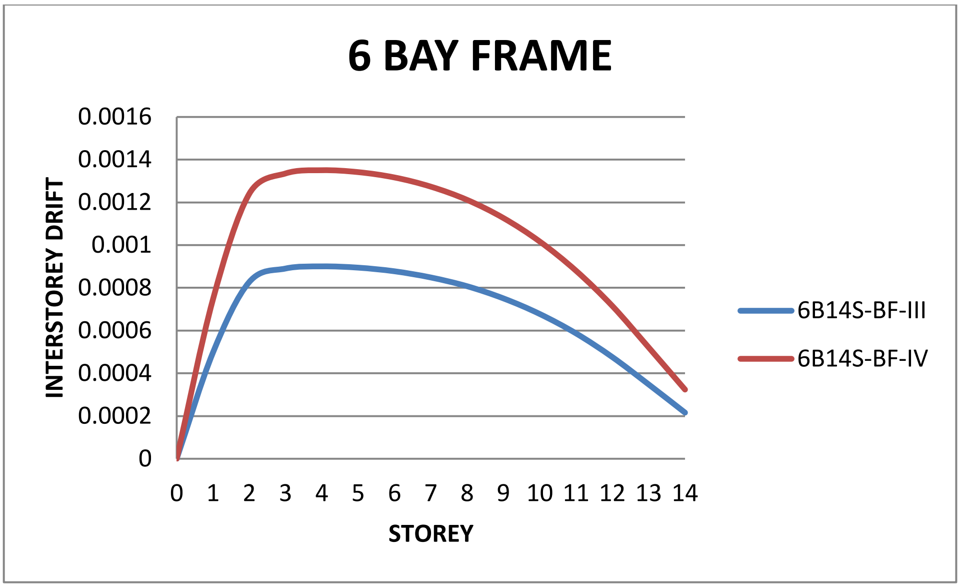


Fig 4.15 Storey Drift Comparasion For Six Bays Bare Frame In Zone III &Zone IV

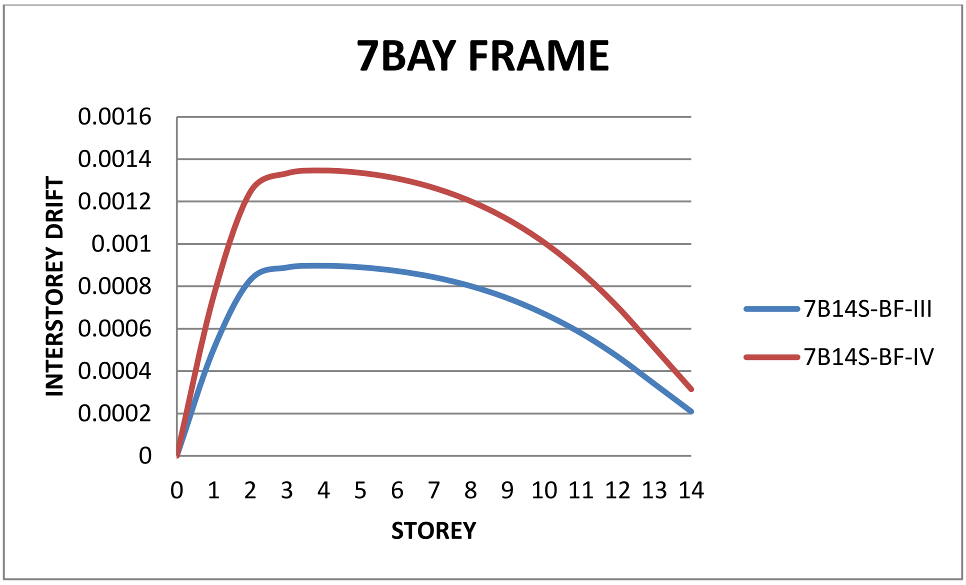


Fig 4.16: Storey Drift Comparasion For Seven Bays Bare Frame In Zone III & Zone IV

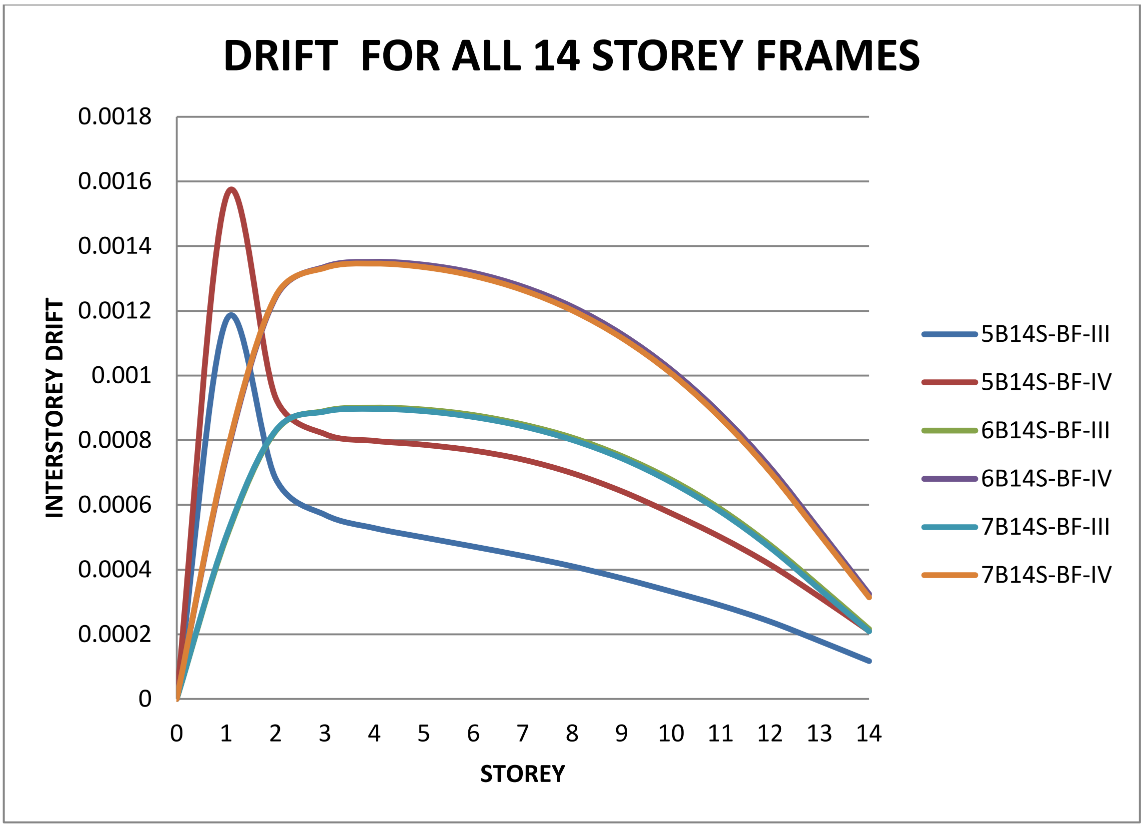


Fig 4.17 Storey Drift Comparasion For All Forteen Storey Bare Frames

4.1.2 MODAL TIME PERIOD:

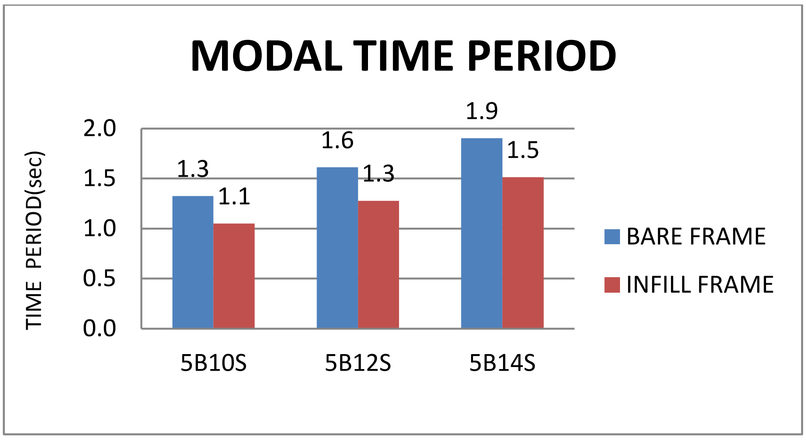


Fig 4.18: Comparasion Of Modal Time Period Between Bare Frame And Infill Frame

It is evident from above figure that inclusion of stiffness of infill wall decrease the modal time period.

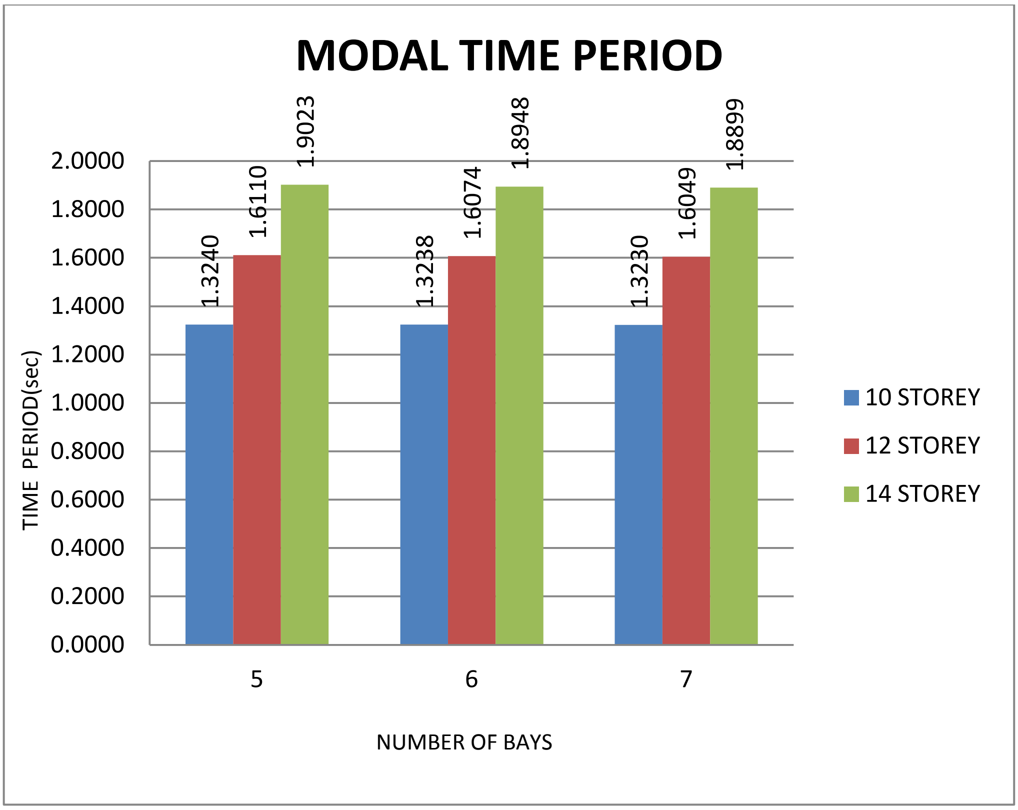


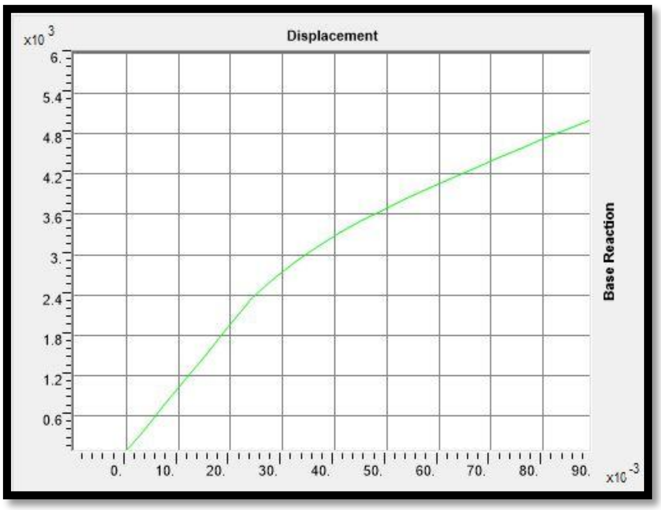
Fig. 4.19: Comparasion Of Modal Time Period By Changing Bays And Number Of Storeys Of Bare Frame

As we increase the height of the structure time period increases but increase in number of bays reduces the modal time period very trivial.

4.2 NON LINEAR ANALYSIS RESULTS

4.2.1 COMPARASION OF PUSHOVER CURVE OF RC FRAME WITH INFILL WALL AND WITHOUT INFILL WALL IN ZONE III

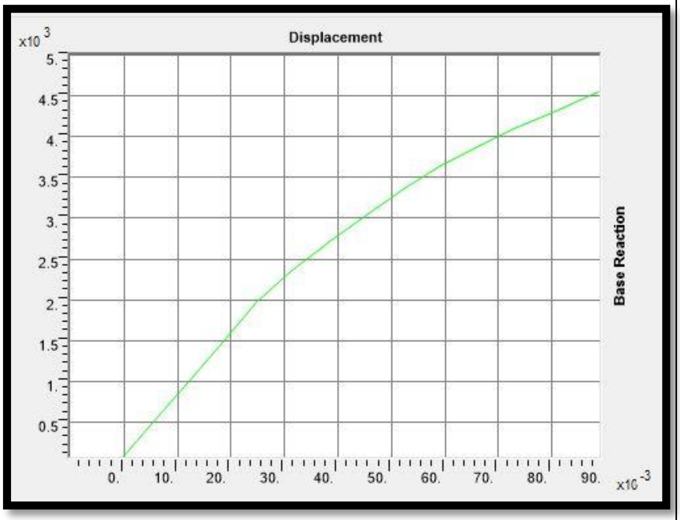
5B10S-IF-III



5B10S-BF-III

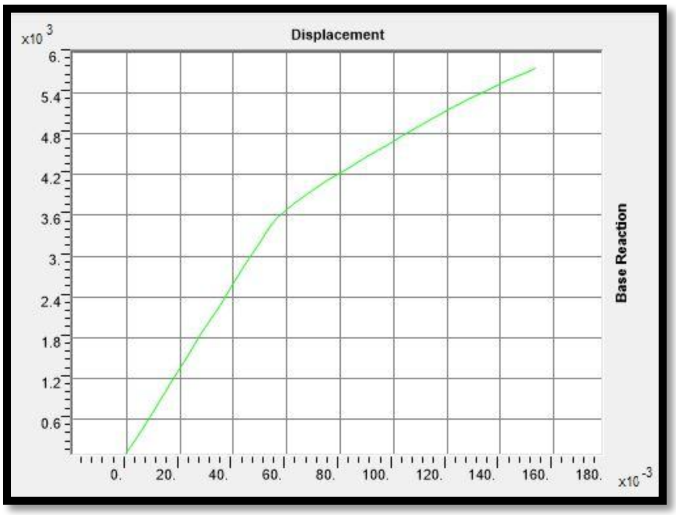
FRAME	5B10S-IF-III	5B10S-BF-III
BASE SHEAR AT COLLPASE(KN)	4917.23	3101.23
ROOF DISPACEMENT(m)	90	401

Fig 4.19 (i): Comparasion Of Pushover Curve Of RC Frame With Infill Wall And Without Infill Wall For 5 Bay 10 Storey In Zone III

5B12S-IF-III	5B12S-BF-III
	

FRAME	5B12S-IF-III	5B12S-BF-III
BASE SHEAR AT COLLPASE(KN)	4549.83	3112.40
ROOF DISPACEMENT(m)	88.8	0.3404

Fig 4.20: Comparasion Of Pushover Curve Of RC Frame With Infill Wall And Without Infill Wall For 5 Bay 12 Storey In Zone III

5B14S-IF-III	**5B14S-BF-III**
	

FRAME	**5B14S-IF-III**	**5B14S-BF-III**
BASE SHEAR AT COLLPASE(KN)	**5768.488**	**2797.42**
ROOF DISPACEMENT(mm)	**153**	**500**

Fig.4.21: Comparasion Of Pushover Curve Of RC Frame With Infill Wall And Without Infill Wall For 5 Bay 14 Storey In Zone III

From the above shown figures it is seen in each case that at collapse condition the infill wall stiffness consideration in analysis shows the frame more stiffness than the frame analysed without infill wall stiffness consideration.

4.2.1.1 COMPARASION OF PUSHOVER PARAMETERS BETWEEN BARE AND INFILL FRAME IN ZONE III

Table 4.1: Comparison Of Pushover Base Shear Between Bare Frame And Infilled Frame in Zone III

PUSHOVER BASE SHEAR		
	BARE FRAME	INFILLED FRAME
5B10S-III	3101.23	4917.23
5B12S-III	3112.4	4549.83
5B14S-III	2979.42	5768.488

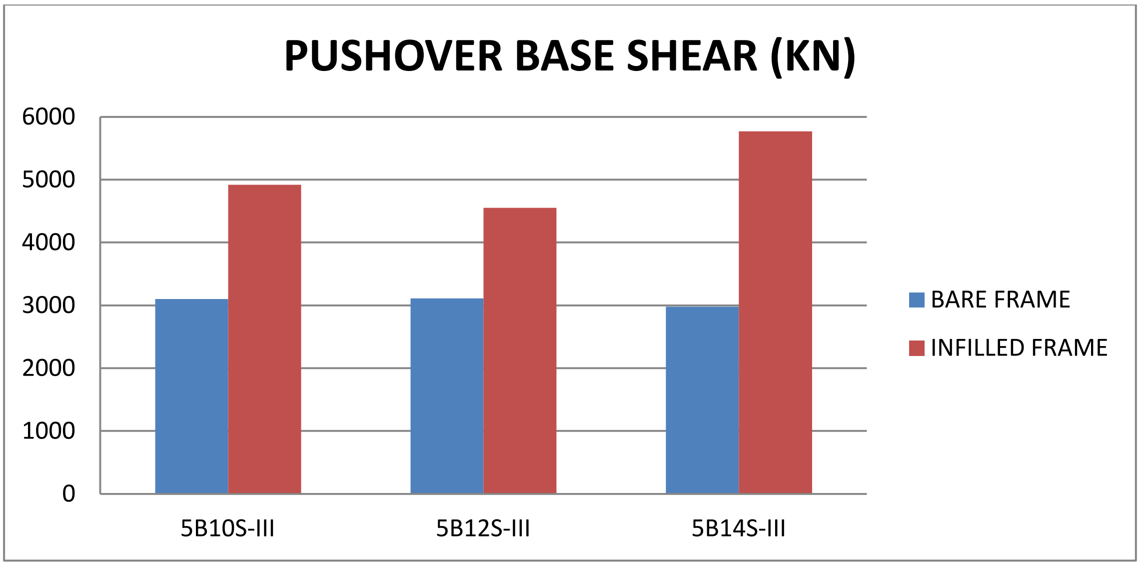


Fig4.22: Comparasion Of Pushover Base Shear Between Bare Frame And Infilled Frame In Zone III.

Table4.2: Comparasion Of Collapse Displacement Between Bare Frame And Infilled Frame In Zone IV

COLLAPSE DISPLACEMENT(m)		
	BARE FRAME	INFILLED FRAME
5B10S-III	0.401	0.09
5B12S-III	0.3404	0.0888
5B14S-III	0.5	0.153

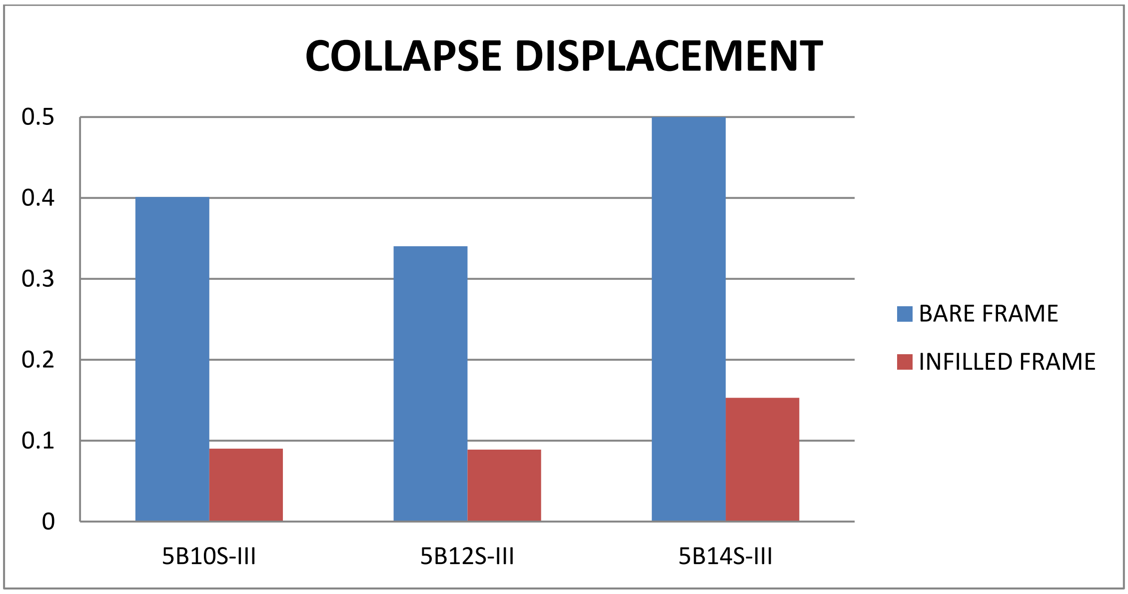
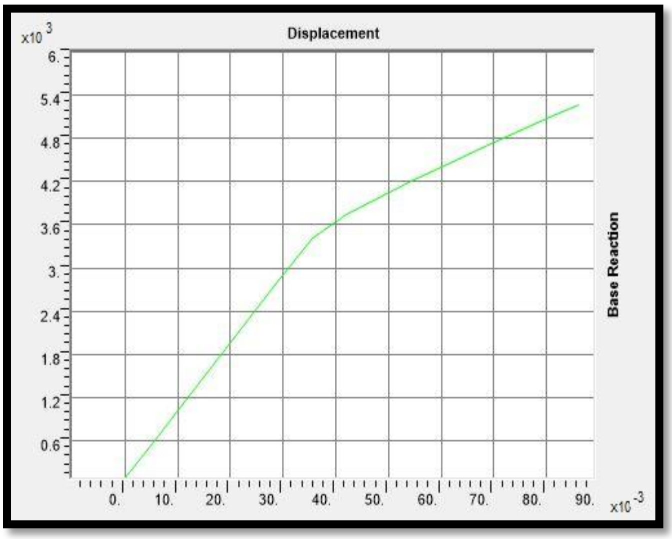


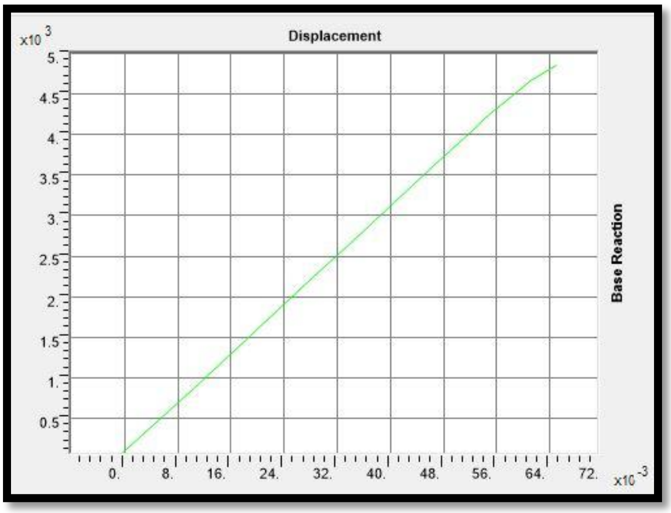
Fig. 4.23: Comparasion Of Collapse Displacement Between Bare Frame And Infilled Frame In Zone IV

4.2.2 COMPARASION OF PUSHOVER CURVE OF RC FRAME WITH INFILL WALL AND WITHOUT INFILL WALL IN ZONE IV

5B10S-IF-IV	**5B10S-BF-IV**
	

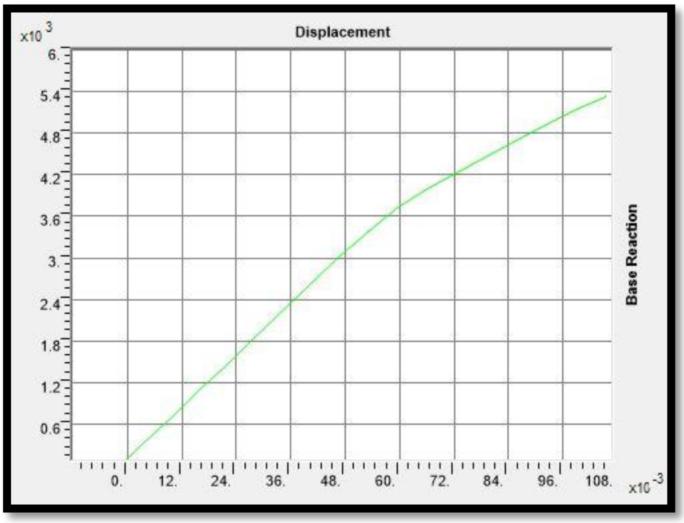
FRAME	5B10S-IF-IV	5B10S-BF-IV
BASE SHEAR AT COLLPASE(KN)	5266.88	4308.68
ROOF DISPACEMENT(mm)	85.8	300

Fig 4.24 Comparasion Of Pushover Curve Of Five Bay Ten Storey RC Frame With Infill Wall And Without Infill Wall In Zone IV

5B12S-IF-IV	5B12S-BF-IV
	

FRAME	5B12S-IF-IV	5B12S-BF-IV
BASE SHEAR AT COLLPASE(KN)	4855.3057	4244.373
ROOF DISPACEMENT(m)	64.9	399.8

Fig 4.25: Comparasion Of Pushover Curve Of Five Bay Twelve Storey RC Frame With Infill Wall And Without Infill Wall In Zone IV

5B14S-IF-IV	5B14S-BF-IV
	

	5B14S-IF-IV	5B14S-BF-IV
BASE SHEAR AT COLLPASE(KN)	5344.051	4019.29
ROOF DISPACEMENT(m)	105.6	446.1

Fig. 4.26: Comparasion Of Pushover Curve Of Five Bay Ten Storey RC Frame With Infill Wall And Without Infill Wall In Zone IV

From the above shown figures it is seen in each case that at collapse condition the infill wall stiffness consideration in analysis shows the frame more stiffness than the frame analysed without infill wall stiffness consideration.

4.2.1.1 COMPARASION OF PUSHOVER PARAMETERS BETWEEN BARE AND INFILL FRAME IN ZONE IV

Table 4.3: Comparasion Of Pushover Base Shear Between Bare Frame And Infilled Frame In Zone IV

PUSHOVER BASE SHEAR		
	BARE FRAME	INFILLED FRAME
5B10S-IV	4308.68	5266.88
5B12S-IV	4244.373	4855.3
5B14S-IV	4019.29	5344.051

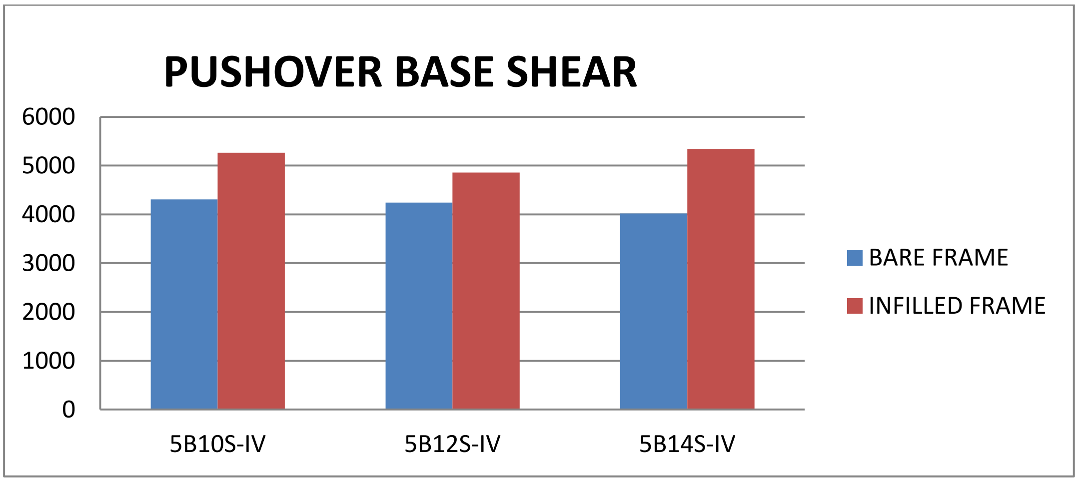


Fig. 4.27: Comparasion Of Pushover Base Shear Between Bare Frame And Infilled Frame In Zone IV

Table 4.4: Comparasion Of Collapse Displacement Between Bare Frame And Infilled Frame In Zone IV

COLLAPSE DISPLACEMENT(m)		
	BARE FRAME	INFILLED FRAME
5B10S-IV	0.3	0.0858
5B12S-IV	0.399	0.0649
5B14S-IV	0.446	0.1056

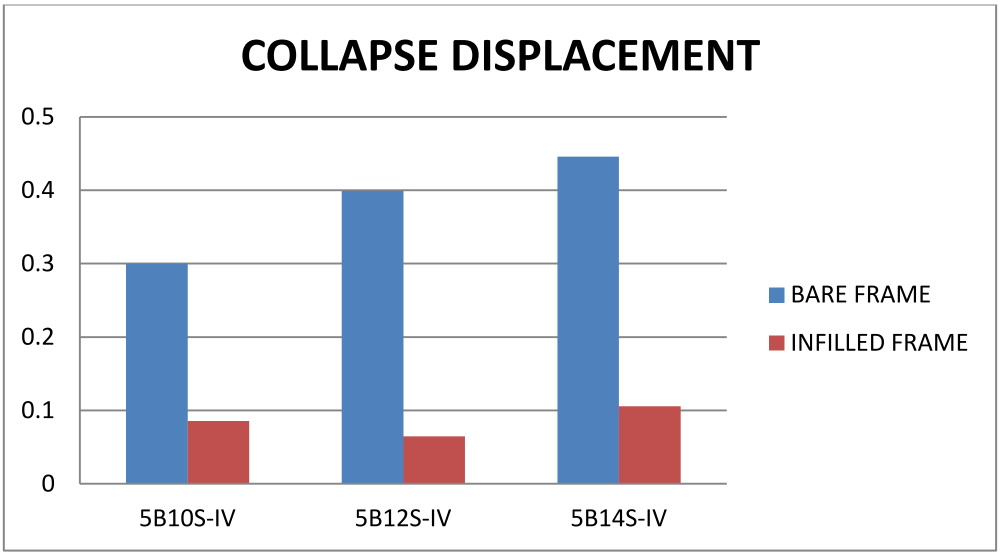


Fig. 4.28: COMPARASION OF COLLAPSE DISPLACEMENT BETWEEN BARE FRAME AND INFILLED FRAME IN ZONE IV

Table 4.5: Comparasion Of Performance Point Base Shear Between Bare Frame In Zone III And Zone IV

PERFORMANCE POINT BASE SHEAR				
S. NO.	**FRAME TITLE**	**ZONE III**	**ZONE IV**	**RATIO**
1	**5B10S-BF**	**3086.03**	**4395.02**	**1.42**
2	**5B12S-BF**	**3069.54**	**4182.83**	**1.36**
3	**5B14S-BF**	**2798.26**	**3994.42**	**1.43**
4	**6B10S-BF**	**4280.39**	**5432.84**	**1.27**
5	**6B12S-BF**	**4268.84**	**5925.41**	**1.39**
6	**6B14S-BF**	**3889.51**	**5559.80**	**1.43**
7	**7B10S-BF**	**5203.40**	**7338.04**	**1.41**
8	**7B12S-BF**	**5666.82**	**7870.39**	**1.39**
9	**7B14S-BF**	**5159.08**	**7379.29**	**1.43**

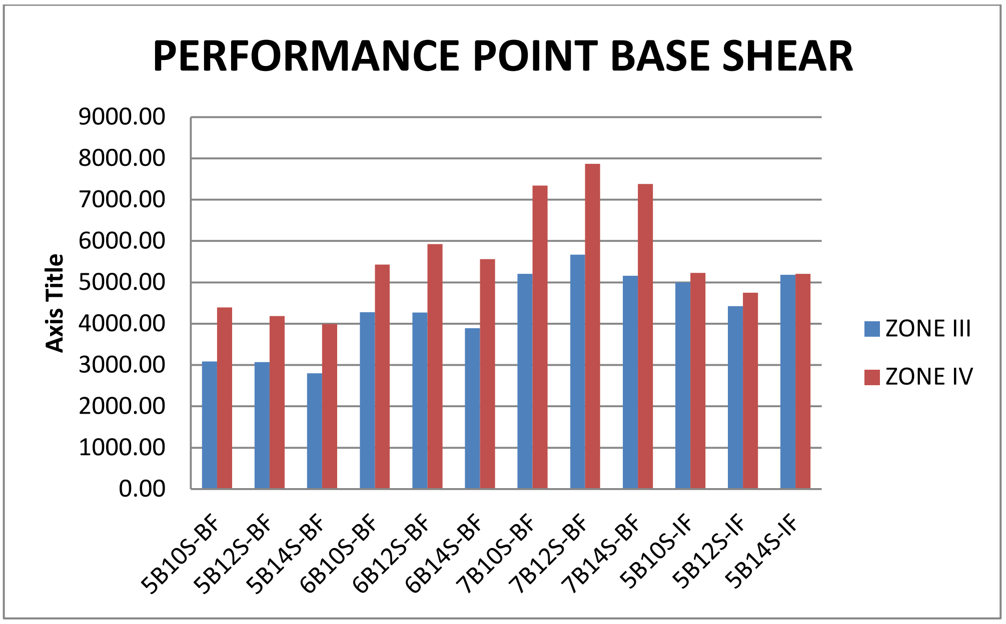


Fig 4.29: Comparasion Of Performance Point Base Shear Between Bare Frame In Zone III And Zone IV

Table 4.6: Comparasion Of Performance Point Displacement Between Bare Frame In Zone III And Zone IV

PERFORMANCE POINT DISPLACEMENT				
S. NO.	**FRAME TITLE**	**ZONE III**	**ZONE IV**	**RATIO**
1	**5B10S-BF**	**0.235**	**0.166**	**0.706383**
2	**5B12S-BF**	**0.264**	**0.21**	**0.795455**
3	**5B14S-BF**	**0.334**	**0.251**	**0.751497**
4	**6B10S-BF**	**0.227**	**0.186**	**0.819383**
5	**6B12S-BF**	**0.264**	**0.208**	**0.787879**
6	**6B14S-BF**	**0.335**	**0.25**	**0.746269**
7	**7B10S-BF**	**0.25**	**0.184**	**0.736**
8	**7B12S-BF**	**0.264**	**0.208**	**0.787879**
9	**7B14S-BF**	**0.336**	**0.25**	**0.744048**
10	**5B10S-IF**	**0.089**	**0.085**	**0.955056**
11	**5B12S-IF**	**0.084**	**0.063**	**0.75**
12	**5B14S-IF**	**0.122**	**0.101**	**0.827869**

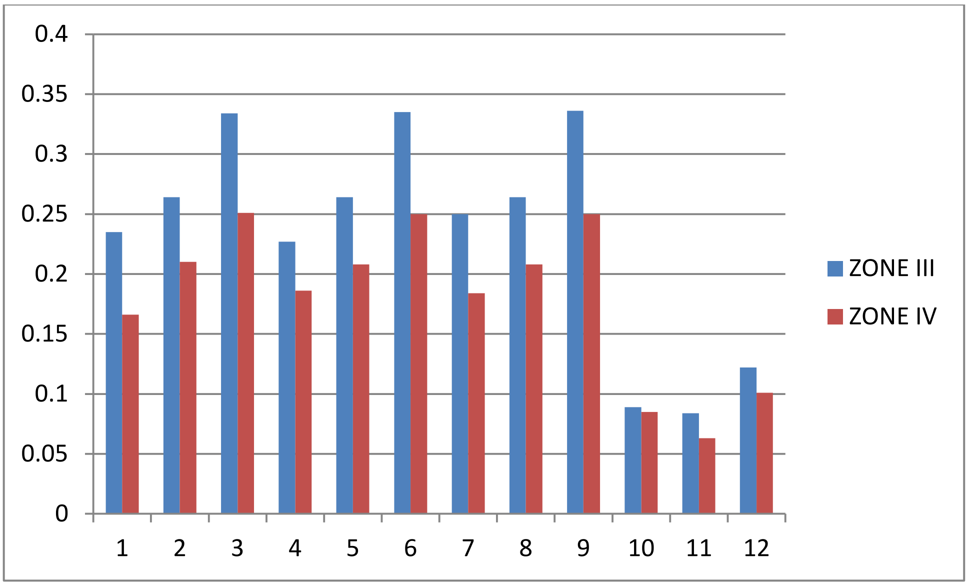


Fig. 4.30: Comparasion Of Performance Point Base Shear Between Bare Frame In Zone III And Zone IV

4.2.3 BEHAVIOURICAL PARAMETES OF NON LINEAR ANALYSIS:

4.2.3.1. DISPLACEMENT DUCTILITY RATIO(μ):

The calculation of ductility ratio is done by equation (3.5) & (3.7) mention in CHAPTER 3. Calculated data of ductility ratio is shown in table as follows:

Table 4.7: Calculation of Displacement Ductility Ratio

FRAME TITLE	YIELD DISPLACEMENT (Δy) (mm)	ULTIMATE DISPLACEMENT (Δu) (mm)	ductily ratio(μ) = $\Delta u/\Delta y$	Fundamental Time Period(T) (sec)	Ductility reduction factor ($R\mu$)
5B10S-BF-III	0.04	0.284	7.1	0.961	7.10
5B12S-BF-III	0.0529	0.34	6.4	1.02	6.43
5B14S-BF-III	0.06	0.392	6.5	1.237	6.53
6B10S-BF-III	0.0404	0.28	6.9	0.961	6.93
6B12S-BF-III	0.0527	0.336	6.4	1.02	6.38
6B14S-BF-III	0.06	0.39	6.5	1.237	6.50
7B10S-BF-III	0.038	0.28	7.4	0.961	7.37
7B12S-BF-III	0.0515	0.335	6.5	1.02	6.50
7B14S-BF-III	0.0588	0.388	6.6	1.237	6.60
5B10S-IF-III	0.0304	0.09	3.0	0.603	2.69
5B12S-IF-III	0.0341	0.084	2.5	0.724	2.46
5B14S-IF-III	0.0588	0.153	2.6	0.845	2.60
5B10S-BF-IV	0.0529	0.256	4.8	0.961	4.84
5B12S-BF-IV	0.0755	0.346	4.6	1.02	4.58
5B14S-BF-IV	0.087	0.402	4.6	1.237	4.62
6B10S-BF-IV	0.0537	0.283	5.3	0.961	5.27
6B12S-BF-IV	0.0748	0.344	4.6	1.02	4.60
6B14S-BF-IV	0.0853	0.398	4.7	1.237	4.67
7B10S-BF-IV	0.0559	0.284	5.1	0.961	5.08
7B12S-BF-IV	0.0735	0.343	4.7	1.02	4.67
7B14S-BF-IV	0.0853	0.397	4.7	1.237	4.65
5B10S-IF-IV	0.0363	0.0858	2.4	0.603	2.17
5B12S-IF-IV	0.0406	0.0649	1.6	0.724	1.60
5B14S-IF-IV	0.0553	0.1056	1.9	0.845	1.91

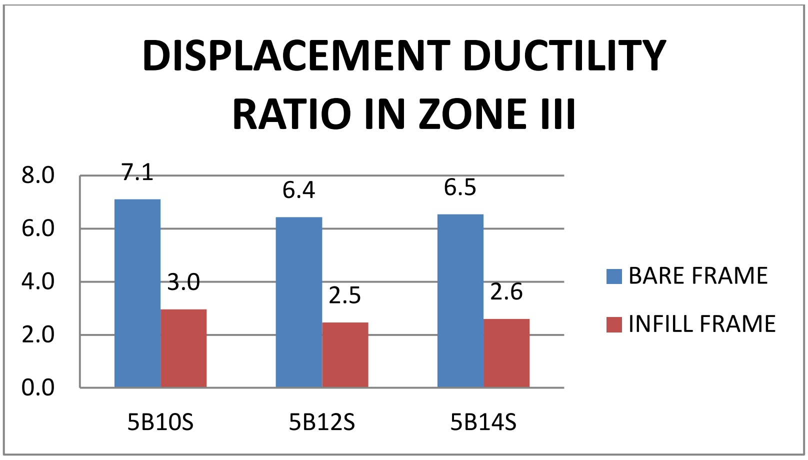


Fig. 4.31: Comparasion Ductility Ratio Between Bare Frame And Frame With Consideration Of Infill Wall In Zone III

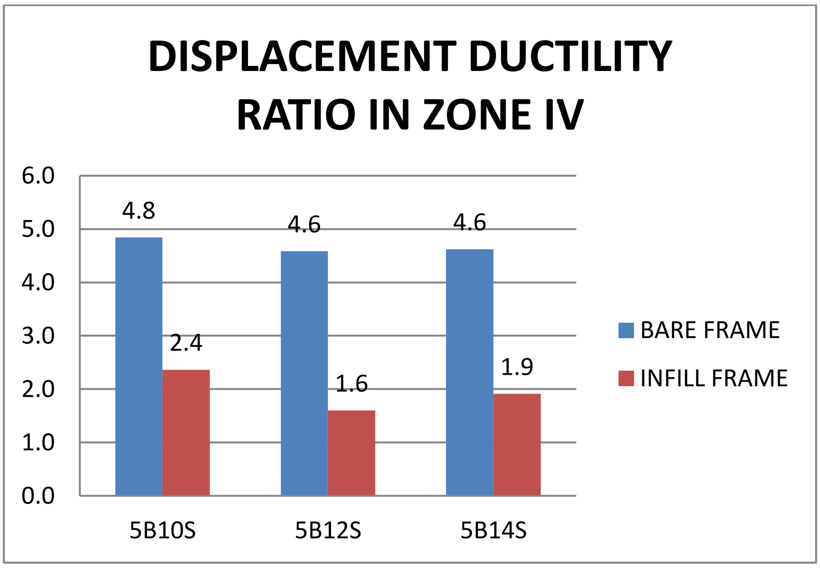


Fig 4.32: Comparasion Ductility Ratio Between Bare Frame And Frame With Consideration Of Infill Wall In Zone IV

It is observed that the ductile behavior of the building is significantly reduced in both zones due to the consideration of Infill wall stiffness in design consideration.

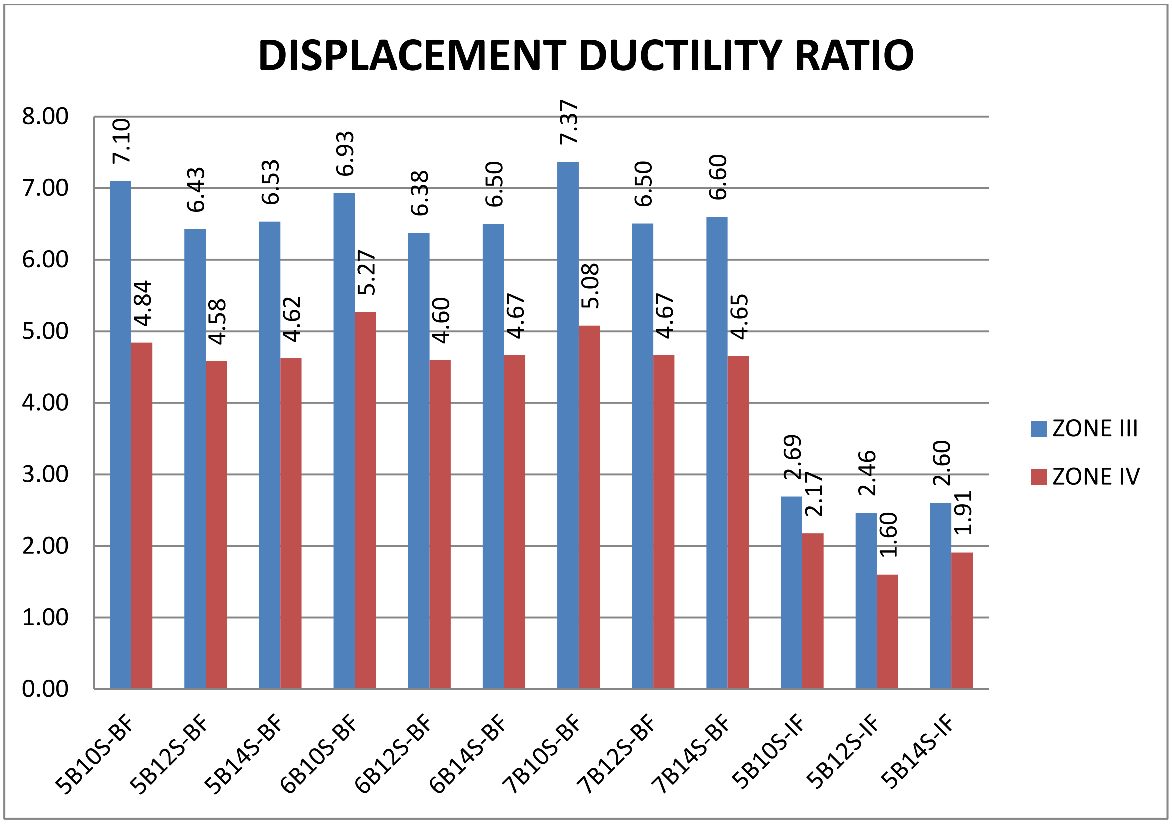


Fig. 4.33: Comparasion Of Ductility Ratio Of Frames Due To Change In Seimic Zone .

As compared to Zone III the ductility ratio is reduces for each Reinforced Concrete frame in Zone IV. It is due to the increment in yield displacement more than the increment of ultimate displacement from ZONE III to ZONE IV. Becuase of the increase in the design force level building yields later.

4.2.3.2 OVERSTRENGTH FACTOR:

The calculation of over strength factor is done by equation(3.26) mention in CHAPTER 3. Calculated data of over strength factor is shown in table as follows:

Table 4.8: Calculation of Overstrength factor

FRAME TITLE	DESIGN BASE SHEAR(Vd) (KN)	Maximum Base Shear (Vmax) (KN)	Overstrength factor (Ω)= Vmax/Vd
5B10S-BF-III	1275.386	3138.271	2.46
5B12S-BF-III	1447.423	3112.761	2.15
5B14S-BF-III	1397.905	2795.455	2.00
6B10S-BF-III	1780.475	4340.394	2.44
6B12S-BF-III	2020.392	4323.92	2.14
6B14S-BF-III	1950.474	3890.7	1.99
7B10S-BF-III	2368.218	5301.695	2.24
7B12S-BF-III	2687.81	5735.60	2.13
7B14S-BF-III	2594.705	5170.438	1.99
5B10S-IF-III	2030.465	5004	2.46
5B12S-IF-III	2039.618	4548.99	2.23
5B14S-IF-III	2046.14	5763.249	2.82
5B10S-BF-IV	1913.29	4421.2217	2.31
5B12S-BF-IV	2171.24	4239.932	1.95
5B14S-BF-IV	2105.302	4003.992	1.90
6B10S-BF-IV	2670.1	5498.39	2.06
6B12S-BF-IV	3348.794	6058.379	1.81
6B14S-BF-IV	3233.6	5567.542	1.72
7B10S-BF-IV	3552.32	7459.8	2.10
7B12S-BF-IV	4031.721	8041.343	1.99
7B14S-BF-IV	3892.58	7384.573	1.90
5B10S-IF-IV	3045.697	5263.248	1.73
5B12S-IF-IV	3059.308	4850.2	1.59
5B14S-IF-IV	3069.21	5338.36	1.74

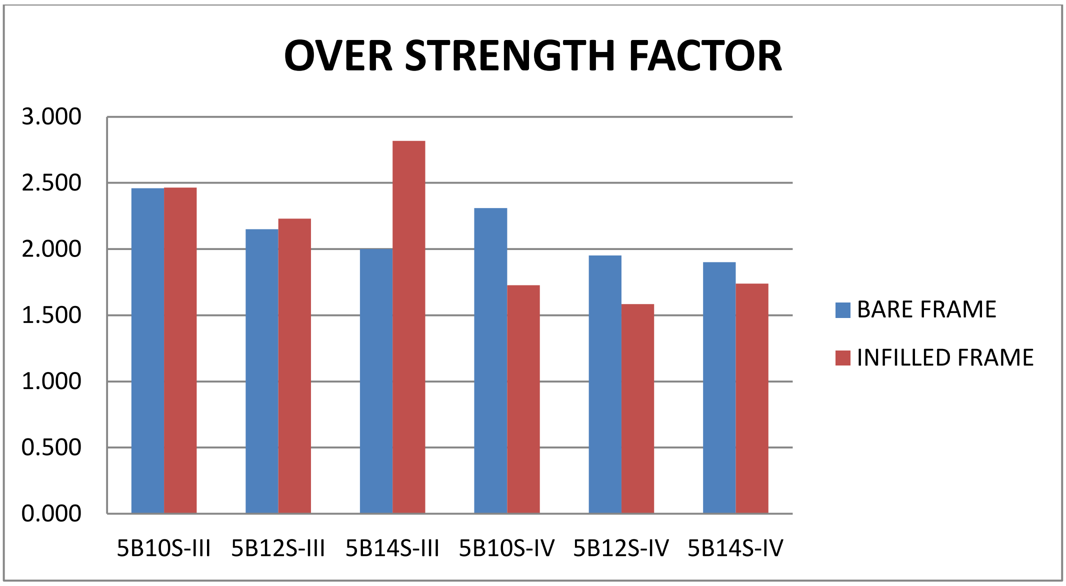


Fig4.34: Comparison Over Strength Ratio Between Bare Frame And Frame With Consideration Of Infill Wall In Zone Iii & Zone Iv

It is visible that overstrength factor of Bare frame as compared to Infilled frame is less in Seismic Zone III but more in Seismic Zone IV. It is due to high increase in the design base shear in Seismic Zone IV.

Table 4.9: Variation Of Over Strength Factor For Reinforced Concrete Frame In Zone III

FRAME TITLE	OVER STRENGTH FACTOR		
	10 STOREY	12 STOREY	14 STOREY
5B-BF-III	2.461	2.151	2.000
6B-BF-III	2.438	2.140	1.995
7B-BF-III	2.239	2.134	1.993

From the Table 4.9 it can be concluded that as we increase bay as well as height over strength factor reduces. That will lead to reduction of reserve strength respectively.

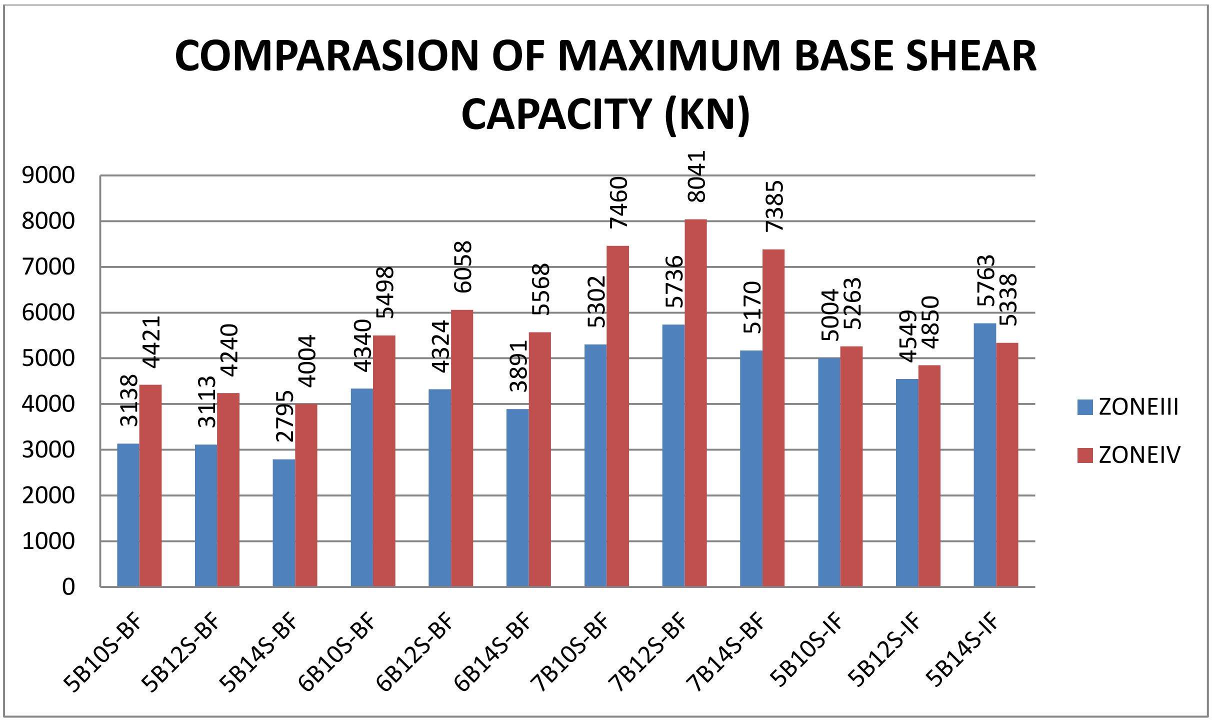


Fig.4.35: Comparasion Of Maximum Base Shear Capacity In Zone III And Zone IV

All the frame shows increase in base shear capacity in ZONE IV as compare to ZONE III except the last frame(5B14S-IF),it may be the result of of brittle failure of building frame in ZONE IV becuse of highest design base shear.

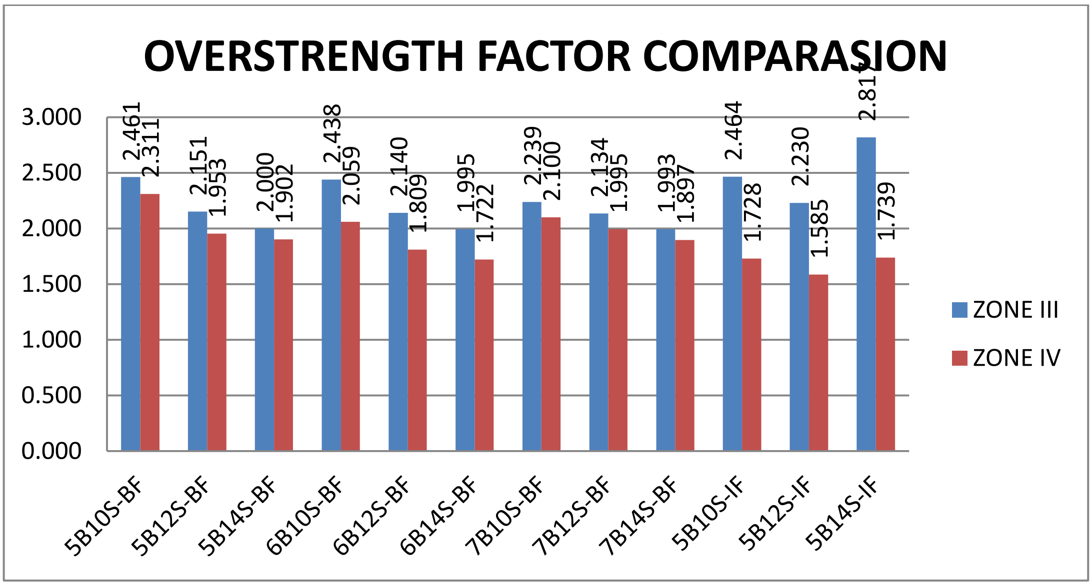


Fig.4.36 : Effect On Overstrength Factor Due To Change In Seismic Zone

Over strength factor decreases from Zone III to Zone IV. It is due to the design seismic base shear increases more as compared to the increment in maximum base shear capacity in Zone IV as compared to Zone III. It is also seen that in general the over strength factor is decreasing with increase in height of the building.

4.2.3.3 RESPONSE REDUCTION FACTOR:

The calculated data is given in table as shown below:

Table 4.10: Calculation Of Response Reduction Factor

FRAME TITLE	Ductility reduction factor ($R\mu$)	Overstrength factor (Ω)= Vmax/Vd	RESPONSE RESUCTION FACTOR (R)=$R\mu$ X Ω/2
5B10S-BF-III	7.10	2.46	8.74
5B12S-BF-III	6.43	2.15	6.91
5B14S-BF-III	6.53	2.00	6.53
6B10S-BF-III	6.93	2.44	8.45
6B12S-BF-III	6.38	2.14	6.82
6B14S-BF-III	6.50	1.99	6.48
7B10S-BF-III	7.37	2.24	8.25
7B12S-BF-III	6.50	2.13	6.94
7B14S-BF-III	6.60	1.99	6.57
5B10S-BF-IV	4.84	2.31	5.59
5B12S-BF-IV	4.58	1.95	4.47
5B14S-BF-IV	4.62	1.90	4.39
6B10S-BF-IV	5.27	2.06	5.43
6B12S-BF-IV	4.60	1.81	4.16
6B14S-BF-IV	4.67	1.72	4.02
7B10S-BF-IV	5.08	2.10	5.33
7B12S-BF-IV	4.67	1.99	4.65
7B14S-BF-IV	4.65	1.90	4.41

It is observed that in most of the cases Response Reduction factor is more that the assumed. Building has good inelastic capacity to sustain large inelastic deformations without collapse(ductile behavior) and develop lateral strength in excess of their design strength(often termed reserve strength) .

CHAPTER 5

CONCLUSION

1. It is evident that in linear static analysis maximum interstorey drift are found in 5 bays frames. So we can say that 5 bay bare frame has least stiffness compared to other frames with more number of bays (refer fig. 4.6 and 4.11).

2.On Considering the infill stiffness the maximum drift significantly for the frame without infill.(refer fig.4.12 and 4.13).

3. Inclusion of stiffness of infill wall in analysis shows decrease the modal time period (refer fig. 4.18).

4. As we increase the height of the structure time period increases but increase in number of bays reduces the modal time period very trivial.(refer fig. 4.19).

5. From the above shown figures it is seen in each case that at collapse condition the infill wall stiffness consideration in analysis shows the frame more global stiffness than the frame analysed without infill wall stiffness consideration(fig. 4.19 ,4.20 and .4.21). The infilled structure show brittle behaviour at collapse.

6. Inclusion of stiffness of infill wall in analysis results in increase base shear at collapse 1.58, 1.46 ,1.93 times in Zone III and 1.222,1.143 , 1.329 times in Zone IV as we increase the number of storeys(referTable4.1 and 4.3) but decrease in increment due to change in high design base shear Zone(i.e. ZoneIII to ZoneIV).

7. Inclusion of stiffness of infill wall in analysis results in decrease in collapse displacement significantly (refer Table 4.2 and 4.4).

8. At prformance point base shear increase on an average 1.3922 in Zone IV with respect to Zone III(Refer table 4.6).

9. It is observed that the ductile behavior of the building is significantly reduced in both zones due to the consideration of Infill wall stiffness in design consideration(fig 4.31 and 4.32).

10. As compared to Zone III the ductility ratio is reduces for each Reinforced Concrete frame in Zone IV. It is due to the increment in yield displacement more than the increment of ultimate displacement from ZONE III to ZONE IV. Becuase of the increase in the design force level building yields later(fig. 4.33).

11. It is visible that overstrength factor of Bare frame as compared to Infilled frame is less in Seismic Zone III but more in Seismic Zone IV. It is due to high increase in the design base shear in Seismic Zone IV.(fig 4.34).

12. From the Table 4.9 it can be concluded that as we increase bay as well as height over strength factor reduces. That will lead to reduction of reserve strength respectively.

13. All the frame shows increase in base shear capacity in ZONE IV as compare to ZONE III except the last frame(5B14S-IF),it may be the result of of brittle failure of building frame in ZONE IV because of highest design base shear.(fig.4.35)

14. Overstrength factor decreases from Zone III to Zone IV.It is due to the design seismic base shear increases more as compared to the increment in maximum base shear capacity in Zone IV as compared to Zone III. It is also seen that in general the overstength factor is decreasing with increase in height of the building.(fig.4.36)

15. It is observed that in most of the cases Response Reduction factor is more that the assumed. Building has good inelastic capacity to sustain large inelastic deformations without collapse(ductile behavior) and develop lateral strength in excess of their design strength(often termed reserve strength) .

Hence we can conclude that using pushover analysis we can get suffice data to analyse the behaviour of structure in siesmic analysis.It seems to be more rational method for estimatingthe lateral strength and distribution of inelastic deformations.

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