“SEISMIC ANALYSIS OF REINFORCED CONCRETE SYMMETRICAL FRAMES WITH DIFFERENT ARRANGEMENTS OF INFILL WALLS”

A PROJECT

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

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of

Mr. Anil Kumar

(Assistant Professor)

By

Nikhil Bisen (121620)

Abhijit Vikram Singh (121621)

to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173234

HIMACHAL PRADESH, INDIA

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CERTIFICATE

This is to certify that the work which is being presented in the project titled **"Seismic Analysis of Reinforced Concrete Symmetrical Frames with Different Arrangements of Infill Walls"** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering and submitted to the Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Nikhil Bisen** (121620) and **Abhijit Vikram Singh** (121621) during the period from July 2015 to June 2016 under the supervision of Mr. Anil Kumar, Assistant Professor, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of our knowledge.

Date: - ………………………

Dr. Ashok Kumar Gupta	Mr. Anil Kumar Dhiman	External Examiner
Professor and Head	Assistant Professor	
Department of Civil Engineering	Department of Civil Engineering	
JUIT Waknaghat	JUIT Waknaghat	

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Abhijit Vikram Singh (121621)

Nikhil Bisen (121620)

ABSTRACT

One of the major developments in seismic design over the past decade has increased because of the continuing desire to limit excessive damage and maintain function of the structure after a moderate earthquake. Masonry infill walls have many beneficial and disadvantageous effects on seismic performances of RC frames. This study aims to demonstrate that neglecting the effects of infill walls during the linear dynamic analysis of the RC frames may lead to the dramatic misunderstanding of the seismic performance of the structure. In this work, a frame was analysed with different arrangement of infill walls using response spectrum and time history method. For modelling of infill walls single strut model strategy was used. Results of this study revealed that changing the arrangement of infill walls may change the damage state of the building during an earthquake. With the immense loss of life and property witnessed in the last couple of decades alone in India, due to failure of structures caused by earthquakes, attention is now being given to the evaluation of the adequacy of strength in framed RC structures to resist strong ground motions.

CONTENTS

LIST OF FIGURES

LIST OF TABELS

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Performance of infilled structures during the past earthquakes revealed that masonry infill walls play a vital role in seismic performance of structures. Researchers demonstrated that infill walls increase the seismic vulnerability of structures. Masonry infill walls noticeably increase the initial stiffness of the frame and thus change the lateral load transfer mechanism. Sudden decrease in stiffness due to failure of infill walls may cause several damages to buildings. Thus, the beneficial and disadvantageous effects of infill walls on seismic performance of the frames may be neglected during the design procedure. The dynamic analysis of a G+5 reinforced concrete building is carried out in STAAD.ProV8i software. Many models will be considered for analysis as frame with infill walls and frame without infill walls with different arrangements of it. In general, for frame without infill effect, the base shear is lower as compared with frame with infill walls. The displacement of storey is much higher for frame without infill walls as compared to frame with infill walls. Performance based design technique helps to determine how the building will perform under seismic effect. The infill walls effect shows major change in the performance of building and the needs to be considered in the analysis.

1.2 INTRODUCTION TO STAAD.Pro

STAAD or (STAAD.Pro) is a structural analysis and design computer program originally developed by Research Engineers International at Yorba Linda, CA in year 1997. In late 2005, Research Engineers International was bought by Bentley Systems. An older version called Staad-III for windows is used by Iowa State University for educational purposes for civil and structural engineers. Initially it was used for DOS-Window system. The commercial version, STAAD.Pro is one of the most widely used structural analysis and design software. It supports several steel, concrete and timber design codes. It can make use of various forms of analysis from the traditional 1st order static analysis, 2nd order p-delta analysis, geometric non linear analysis or a buckling analysis. It can also make use of various forms of dynamic analysis from modal extraction to time history and response spectrum analysis. In recent years it has become part of integrated structural analysis and design solutions mainly using an exposed API called OpenSTAAD to access and drive the program using a VB macro system included in the application or other by including OpenSTAAD functionality in applications that themselves include suitable programmable macro systems.

1.3 BENEFITS OF STAAD.Pro

1. STAAD has a extremely flexible 2D/3D Modelling environment. STAAD revolutionized the concurrent use of spreadsheets, a 3D CAD graphical modeller, and a text-based input language editor. With over 40 step-by-step movie tutorials and hundreds of examples and verification problems, even a novice user can become productive in a matter of days.
2. It covers all aspects of Structural Engineering STAAD.Pro V8i is a solution for all types of structures and includes tools designed to aid specific structural engineering tasks. For example, for the bridge engineer, STAAD.beava incorporates a powerful influence surface generator to assist in locating vehicles for maximum effects.
3. It has a broad Spectra of Design Codes. Since the 1980s, STAAD.Pro V8i has encompassed concrete and steel design, making it a true one-stop-shop structural environment.
4. International coding since its introduction into the market in 1981, STAAD was thrown onto the international scene with its implementation of British codes. Currently, it supports over 70 international codes and approximately 20 US codes. Forty-seven out of the top 50 ENR companies actively use STAAD.Pro V8i in their offices worldwide.
5. Interoperability and open architecture unlike most structural software, STAAD.Pro V8i can be customized by you to exactly fit your design needs. STAAD.Pro V8i is developed on an open architecture called OpenSTAAD.
6. STAAD.Pro V8i is the only structural analysis software that has gone through ISO 9001 certification and has passed the stringent software validation requirements of the nuclear industry (10CFR Part 50, 10CFR 21 and ASME NQA-1-2000).
7. Extremely Scalable STAAD.Pro V8i is best known for its accurate 3D linear static and P-delta analysis of multi-material structures. But the most powerful and widely used facets of STAAD.Pro V8i are its soil-structure interaction, push-over and dynamic analyses.

8. Reports and Documentation STAAD.Pro V8i has one of the most powerful, and customizable and high quality reports available so that you are able to provide your clients and engineers with exactly the information that is required, whether it is a two page summary of pictures exactly as seen onscreen or complete, fully detailed reports.
9. STAAD.Pro V8i offers a dynamic website with new tips and tricks and a discussion board for STAAD users around the world.

1.4 RESPONSE SPECTRUM ANALYSIS

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response spectrum analysis provides insight into dynamic behaviour by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period. Response spectrum analysis is useful for design decision-making because it relates structural type selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

1.5 OBJECTIVES OF THE STUDY

- The response spectrum analysis and of a G+5 reinforced concrete building is carried out in STAAD.ProV8i software.
- Two models will be considered for analysis as frame with infill walls (different arrangements) and frame without infill walls.
- Comparison between both the model will be done on parameters such as storey drift, base shear, top storey drift, peak storey shear, etc.
- Response history of a particular RC frame with different possible arrangements of infill walls will be investigated.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Murthy and **Jain (2000)** investigated that masonry infill wall panels increase strength, stiffness, overall ductility and energy dissipation of the building. More importantly, they help in drastically reducing the deformation and ductility demand on RC frame members. This explains the excellent performance of many such buildings in moderate earthquakes even when the buildings were not designed or detailed for earthquake forces. The reinforcement in the infills does not contribute significantly towards stiffness and strength; in fact, it may lead to reduction in stiffness and strength due to increased mortar thickness in the layers containing the reinforcement. It should be possible to develop suitable detailing schemes for anchoring masonry reinforcement into the frames and thereby improve the out of plane behavior of the infills. In such situations, the infill could be relied upon to ensure good seismic performance.

Pujari and **Madhekar (2007)** considered two cases, one is bare frame analysis (without infill) and another one is considering infill walls with open ground storey. Due to inclusion of infill, behaviour and failure modes of buildings change. The results show the importance of considering infill walls in modelling, to get the real scenario of damage. The ductility requirements and displacement are greater for bare frame. This leads to more consumption of material. Pushover analysis result shows the effect of open ground storey and true failure mechanism. This effect in not captured in structural design and leads to catastrophic failure of structure .From pushover analysis results the weak links in the structure are identified and the performance level achieved by structure is known. This helps to find the retrofitting location to achieve the performance objective. Performance based seismic design helps to understand the behaviour of building well in advance and redesigning can help to improve the performance of structure.

Cuiqiang, Ying, Zhou Deyuan and **Xilin (2011)** investigated that for the RC frame with infill walls, the dynamic properties are apparently changed and the period of the RC frame is also changed. Because of the increase in the stiffness to the RC frame, the RC frame will carry more inertia force. From the hysteresis loops of the inter-story, for RC frames with discontinuous infill walls in elevation, the infill walls not only change the stiffness distribution of the entire structure, but also change the distribution of the strength. Therefore, a weak story can easily be formed in the RC frame without an infilled wall. For the RC frame in this paper, the weak story is developed on the first floor. The diagonal strut model considering the co-work of the beam and infill walls can simulate the damage of the RC frame with in filled walls. Regardless of the co work of the beam and infill walls, there are discrepancies between the simulation and the actual damage. From the point of view of the simulation and observed seismic damage, both the stiffness contribution of the infill walls to the beam and the strength contribution should be considered.

Nautiyal, Singh and **Batham (2013)** investigated that the Indian standard provides different expressions for the estimation of the natural period of the building structure considering or neglecting the stiffness of the infill wall. The consideration of stiffness of masonry infill increases the stiffness of the structure and hence reduces the natural period and consequently increases the response acceleration and hence the seismic forces i.e. base shear and correspondingly the lateral forces at each storey.

Kangra, Mehare and **Meshram (2015)** investigated by considering different models of G+3, G+6, G+9 and G+12 which are modeled in STAAD, that for G+3 building whose plan area is increases by 177.78% (15m×9m = 135m^2 and 25m×9m = 375m^2) the increase in base shear is153.62%.For G+6, G+9 and G+12 the increase in base shear is by 152.28%, 220.35% and 214.4% respectively. Moreover for plan area (15m×9m) and varying height the base shear is increased by 81.79%, 97.85% and 115.67% for G+6, G+9and G+12. For plan area (25mx15m) and varying height the base shear is increased by 80.8%, 150% and 167.41% for G+6, G+9and G+12. Thus base shear has drastic effect on 25x15 plan area building in comparison to 15mx9m building. The lateral forces result for G+3 building show that for plan area 15mx9m and 25mx15m the average increase in lateral force is by 149.61%.For G+6, G+9 and G+12 buildings it can be observed that the average increase in storey shear is by 147.75%, 212.71% and 221.19% respectively. Thus,G+12 is the most critical one. It is interesting to note from spring mass model that the worst hit floor is eight floor of G+9 building having plan area 25mx15m as it is subjected to a lateral force of 465.76kN.The storey drift results suggest that for G+3 building the average increase in drift is by 4.69%.For G+6, G+9 and G+12 the average increase in storey drift is 3.77%, 30.13% and 27.18% respectively. The twelfth floor of G+12 building having plan area 25mx9m is drifted maximum by 47.92mm.The maximum base shear is also borne by G+12 building plan area 25mx9m and its value is 2222.79kN.

2.2 SUMMARY OF LITERATURE REVIEW

- Masonry infill wall panels increase strength, stiffness, overall ductility and energy dissipation of the building
- They help in drastically reducing the deformation and ductility demand on RC frame members.
- The consideration of stiffness of masonry infill increases the stiffness of the structure and hence reduces the natural period and consequently increases the response acceleration and hence the seismic forces i.e. base shear and correspondingly the lateral forces at each storey.
- The ductility requirements and displacement are greater for bare frame. This leads to more consumption of material.
- Performance based seismic design helps to understand the behaviour of building well in advance and redesigning can help to improve the performance of structure.

2.3 ANALYSIS METHODS

Seismic Engineering is a sub discipline of the broader category of Structural engineering. Its main objectives therefore are:

- To understand interaction of structures with the shaky ground.
- To foresee the consequences of possible earthquakes.
- To design, construct and maintain structures to perform at earthquake exposure up to the expectations and in compliance with building codes.

The methodologies available so far for the evaluation of existing buildings can be divided into two categories-(i) Qualitative method (ii) Analytical method.

In the same realm, seismic analysis is a subset of structural analysis and is the calculation of the response of a structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent. Structural analysis methods can be divided into the following categories-

- Equivalent Static Analysis
- Response Spectrum Analysis
- Linear Dynamic Analysis
- Nonlinear Static Analysis
- Nonlinear Dynamic Analysis

In this study we have used **"Response Spectrum Analysis"** and **"Time History Analysis"** for the assessment of the considered G+5 RC structure.

CHAPTER 3

BACKGROUND AND PROBLEM DEFINITION

3.1 EARTHQUAKE

The term **earthquake** can be used to describe any kind of seismic event which may be either natural or initiated by humans, which generates seismic waves. Earthquakes are caused commonly by rupture of geological faults; but they can also be triggered by other events like volcanic activity, mine blasts, landslides and nuclear tests. An abrupt release of energy in the Earth's crust which creates seismic waves results in what is called an earthquake, which is also known as a tremor, a quake or a temblor). The frequency, type and magnitude of earthquakes experienced over a period of time define the seismicity (seismic activity) of that area. The observations from a seismometer are used to measure earthquake. Earthquakes greater than approximately 5 are mostly reported on the scale of moment magnitude. Those smaller than magnitude 5, which are more in number, as reported by the national seismological observatories are mostly measured on the local magnitude scale, which is also known as the Richter scale. There are many buildings that have primary structural system, which do not meet the current seismic requirements and suffer extensive damage during the earthquake. At present time the methods for seismic evaluation of seismically deficient or earthquake damaged structures are not yet fully developed. The buildings which do not fulfill the requirements of seismic design, may suffer extensive damage or collapse if shaken by a severe ground motion. The seismic evaluation reflects the seismic capacity of earthquake vulnerable buildings for the future use.

3.1.1 Seismic Zones of India

The varying geology at different locations in the country implies that the likelihood of damaging earthquakes taking place at different locations is different. Thus, a seismic zone map is required to identify these regions. Based on the levels of intensities sustained during damaging past earthquakes, the 1970 version of the zone map subdivided India into five zones – I, II, III, IV and V. The maximum Modified Mercalli (MM) intensity of seismic shaking expected in these zones were *V or less*, *VI*, *VII*, *VIII*, and *IX and higher*, respectively. Parts of Himalayan boundary in the north and northeast, and the Kachchharea in the west were classified as zone V. The seismic zone maps are revised from time to time as more understanding is gained on the geology, the seismic tectonics and the seismic activity in the country. The Indian Standards provided the first seismic zone map in 1962, which was later revised in 1967 and again in 1970. The map has been revised again in 2002, and it now has only four seismic zones – II, III, IV and V. The areas falling in seismic zone I in the 1970 version of the map are merged with those of seismic zone II. Also, the seismic zone map in the peninsular region has been modified. Madras now comes in seismic zone III as against in zone II in the 1970 version of the map. This 2002 seismic zone map is not the final word on the seismic hazard of the country

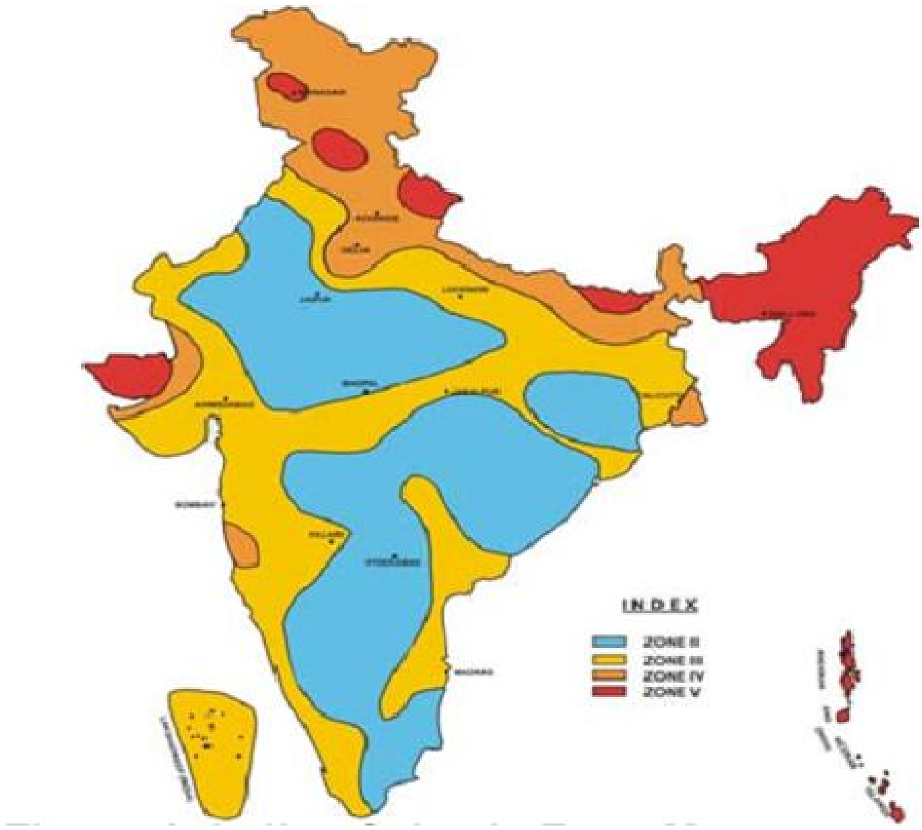


FIGURE 3.1: INDIAN SEISMIC ZONE MAP AS PER IS:1893 (PART 1)-2002

3.2|BASE SHEAR

Base shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. Calculations of base shear (**V**) depends on:

- soil conditions at the site
- proximity to potential sources of seismic activity (such as geological faults)
- probability of significant seismic ground motion
- the level of ductility and over strength associated with various structural configurations and the total weight of the structure
- the fundamental (natural) period of vibration of the structure when subjected to dynamic loading

3.3|STOREY DRIFT

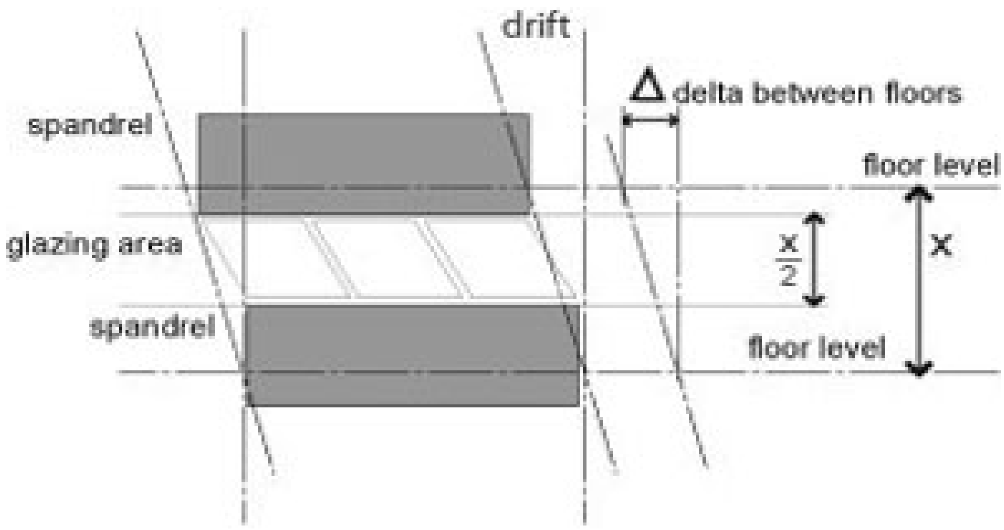


FIGURE 3.2: STOREY DRIFT

The storey drift in any storey due to the minimum specified design lateral force, with partial load factor of 1 should not exceed 0.004 times the storey height. For the purpose of displacement requirements only, it is permissible to use seismic force obtained from the computed fundamental period of the building without the lower bound limit on design seismic force specified.

There shall be no drift limit for single storey building which has been designed to accommodate storey drift. Drift in building frames is a result of flexural and shear mode contributions, due to the column axial deformations and to the diagonal and girder deformations, respectively. In low rise braced structures, the shear mode displacements are the most significant and, will largely determine the lateral stiffness of the structure. In medium to high rise structures, the higher axial forces and deformations in the columns, and the accumulation of their effects over a greater height, cause the flexural component of displacement to become dominant.

3.4|MODELLING OF INFILL WALLS

Different modelling proposal techniques that simulate the behavior of the infill panels can be found and are divided in two different groups, namely micro-models and simplified macro-models. The first of them involves models in which the panel is divided into numerous elements taking into account the local effects in detail, and the second includes simplified models based on the physical understanding of the behavior of the infills panels submitted to earthquakes loadings and past experimental tests. In the case of the last group, a few numbers of struts are used to represent the effect of this non-structural element on the structural response.

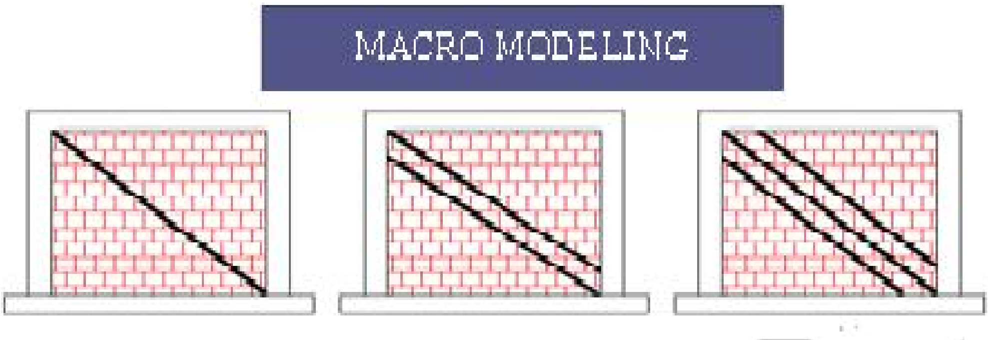
3.4.1|Micro-Modelling

The micro-modelling approach considers the effect of the mortar joints as discrete element in the model. Considering the fact of mortar joints are the weakest plane in a masonry infill wall, this approach can be considered the most exact. According to Lourenco (2002) and Asteris and Tzamtzis (2003), the micro-modelling procedures can be summarized in two different refinements for masonry walls: Simplified micro modelling where the expanded units are represent by continuum elements and the properties of the mortar and the brick mortar interface are lumped into a common element and detailed micro-modelling where brick units and the mortar are represented by continuum elements and the brick units–mortar interaction are represent by different continuum elements, which leads to accurate results and intensive computational requirement.

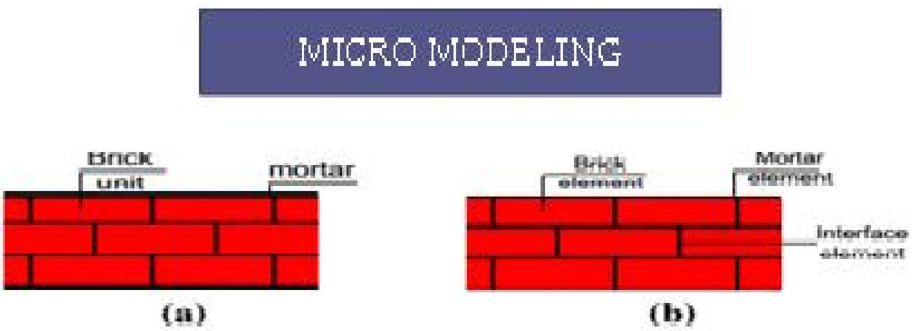
Depending on the composite characteristics of the infilled frames, different elements are required in the model as for example beam or continuum elements for the surrounding frame, continuous elements for the infill panels and interface elements for representing the interaction between the frame and the panel. The main advantage of the micro-modelling is that the infilled frames' in-plane behavior takes into account with the local effects related to cracking, crushing and contact interaction.

3.4.2|Macro-Modelling

The masonry infill walls can be analyzed through simplified macro models that use different strategies, ranging from very simple models such as the equivalent strut model to much more complex models like the double and triple strut model as illustrated in, respectively (Crisafulli 1997a). Polyakov (1960) suggested the possibility of considering the effect of modelling the infills as equivalent to one diagonal strut, which was later modified by Holmes (1961) that replaced the infill panel with an equivalent pin-jointed diagonal strut made of the same material and having the same thickness of the masonry infill wall. Later, Smith (1962) based on the experimental tests found the need of introducing new required parameters to modelling the in fills. Main stone and Weeks (1970) and Main stone (1974) proposed methods for calculating the effective diagonal strut width based on experimental tests. Klingler and Bertero (1978) considered the nonlinear behavior of the masonry infill wall when submitted to dynamic loadings, and Liauw and Kwan (1984) developed a semiempirical equation to compute the strut width as a function of other geometrical parameters of the panel. Zarnic and Tomazevic (1988) proposed a macro-model that takes into account the strength and stiffness of the infills. Saneinejad and Hobbs (1995) tried to predict their nonlinear behavior through a numerical model that represents the stiffness and strength degradation of one infill panel. Zarnic and Gostic (1997b) proposed an empirical equation, which was later modified by Dolsek and Fajfar (2002) to compute the shear ultimate strength of the masonry infill wall. Dolsek and Fajfar (2002) also defined a tri-linear response of the single strut model, including an elastic, hardening and post-capping branch. Flanagan and Bennet (1999) focused on the modelling of the corner crushing strength and stiffness of the infills. Later, through the obtained results it becomes clear that using only one single strut was insufficient to model the entire behavior of the infill panel. The shear forces and the bending moment in the frame members cannot be adequately given using one single strut connected to the two loaded corners. Different complex macro-models were proposed based on the number of diagonal struts which has the main advantage of representing the real behavior of the infill panel when submitted to seismic actions. Syrmakesis and Vratsanou (1986) changed this strut model to a five-diagonal strut model that can model the global force–displacement response but otherwise is not able to capture the interaction between infill panel and the surrounding frame. Schmidt (1989) proposed a double strut that takes into account with the frame–infill interaction and also the strength and stiffness of the panel. Chrystomou (1991) increases the number of struts in order to represent the infill panel response with three parallel struts in which direction.



a) Single strut model b) Double strut model c) Triple strut model



a) Masonry infill wall sample b) Detailed micro modelling strategy

FIGURE 3.3: DIFFERENT TYPES OF MODELING STRATEGIES

3.5.4 NUMERICAL MODELLING

Shear infill elements were used for modeling masonry infill walls. Masonry infill walls usually perform as a secondary bracing system for the building. Hence, the lateral rigidity of a masonry panel can be considered by assuming a compression strut with a width ‘‘α’’, and can be calculated as follows:

$$\propto = 0.175D(\lambda 1H)^{-0.4}$$

Where H is the height of the column and D is the diagonal length of the panel. Furthermore λ_1 can be calculated as follows:

$$\lambda 1 = \left(\frac{\mathbf{E}\mathrm{mt}\sin 2\theta}{4\mathbf{E}fe\mathbf{I}col\mathbf{h}}\right)^{\frac{1}{4}}$$

Where E_{mt} and E_{fe} denote elastic modulus of infill material and the frame materials, respectively, t is the thickness of infill wall, I_{col} is the column moment inertia and h is the infill height. FurthermoreΘ can be calculated as follows:.

$$\theta = \tan^{-1}\frac{h}{l}$$

Where *l* is the width of the infill panel

Finally the infill rigidity can be calculated as follows:

$$K = \boldsymbol{Em\alpha t}/\mathbf{D}$$

From the infill rigidity we can get area of steel since

$$\boldsymbol{k} = \boldsymbol{AE}/\boldsymbol{L}$$

And a suitable steel section can be chosen from the steel table.

3.5|OPEN GROUND STOREY EFFECT

Reinforced concrete (RC) frame buildings are becoming increasingly common in urban India. Many such buildings constructed in recent times have a special feature – the ground storey is left *open* for the purpose of parking , *i.e.* columns in the ground storey do not have any partition walls (of either masonry or RC) between them. Such buildings are often called open ground storey buildings or buildings on stilts An open ground storey building, having only columns in the ground storey and both partition wall sand columns in the upper storeys, 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.

3.5.1|Earthquake Behavior

Open ground storey buildings have consistently shown poor performance during past earthquakes across the world (for example during *1999 Turkey*, *1999Taiwan* and *2003 Algeria* earthquakes); a significant number of them have collapsed. A large number of buildings with open ground storey have been built in India in recent years. For instance, the city of Ahmedabad alone has about 25,000 *five-storey* buildings and about 1,500 *eleven-storey* buildings; majority of them have open ground storeys. Further, an huge number of similarly designed and constructed buildings exist in the various towns and cities situated in moderate to severe seismic zones (namely III, IV and V) of the country. The collapse of more than a hundred RC frame buildings with open ground storeys at Ahmedabad (~225km away from epicenter) during the 2001 Bhuj earthquake has emphasized that such buildings are *extremely* vulnerable under earthquake shaking. 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 common language, this type of buildings can be explained as a building on chopsticks. Thus, such buildings swing back-and-forth like inverted pendulums during earthquake shaking, and the columns in the open ground storey are severely stressed. If the columns are weak (do not have the required strength to resist these high stresses) or if they do not have adequate ductility, they may be severely damaged which may even lead to collapse of the building.

3.5.2|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.

3.5.3|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 infill). 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. 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. The owners should seek the services of qualified structural engineers who are able to suggest appropriate solutions to increased seismic safety of these buildings.

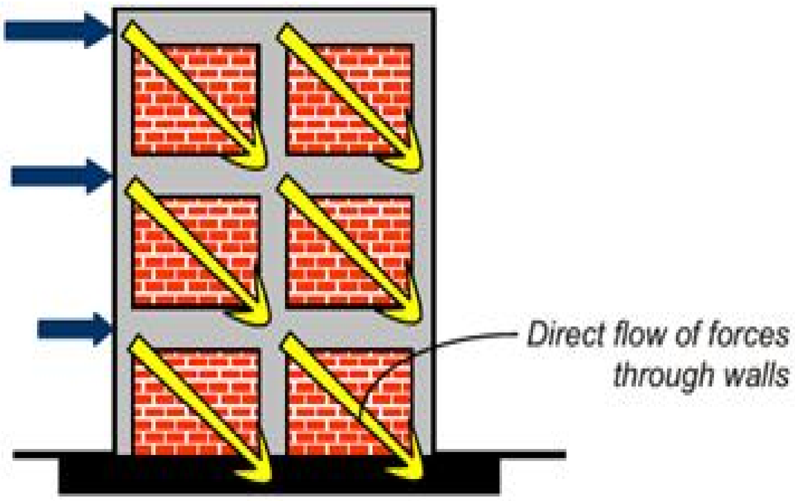
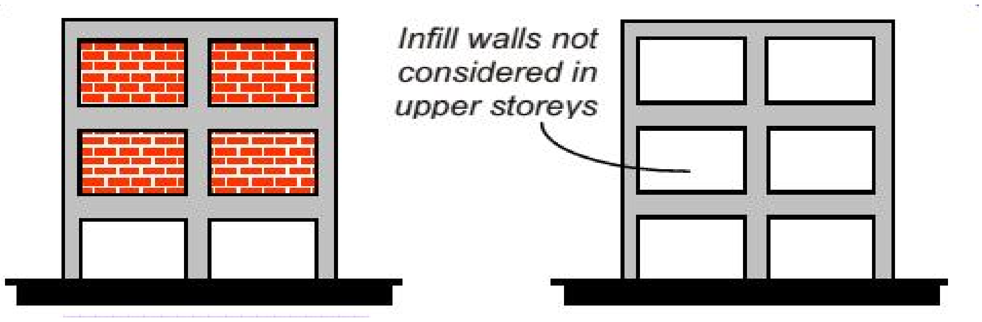


FIGURE 3.4: AVOIDING OPEN GROUND STOREY PROBLEM-CONTINUITY OF WALLS IN GROUND STOREY IS PREFERRED.



a) Actual Building b) Building being assumed in current design practice

FIGURE 3.5: OPEN GROUND STOREY BUILDING- ASSUMPTIONS MADE IN CURRENT DESIGN PRACTICE ARE NOT CONSISTENT WITH THE ACTUAL STRUCTURE

CHAPTER 4

MODELLING ON STAAD.Pro

4.1 DESCRIPTION OF THE STRUCTURE

Table 4.1: Details of the Structure

Type of building	**Commercial Office Building G+5** **RC moment resisting frame with infill walls.**
Location	Chandigarh
Soil type	Medium
Size of Beam Plinth beam	0.3m×0.4 m 0.25m*0.25m
Size of Column	0.4m*0.4m
Live load	4 kN/m^2 (As Per IS 875 PART 2)
	1.5kn/ m^2(on roof access provided) Table-2 IS 875 part 2
Dead Load	As per IS 875 Part 1
Wind Load	Not Considered. Since seismic load is considered.
Depth of slab	150mm
Steel section used for strut	ISMC 125
Grade of concrete	M35
Grade of steel	Fe 415

4.2 | SEISMIC CONSIDERATIONS

Table 4.2 Seismic Consideration

Zone factor (Z)	0.24 (From table 2 IS 1893 2002)
Response Reduction factor	5 (Since, moment resisting frame) (From, table 7 IS 1893 -2002)
Importance Factor	1.5 (Commercial Building Cosiderations) (From table 6 IS 1893 2002)
Damping Ratio	0.05
Time Period	X-Direction = 0.332
	Z-Direction = 0.332

4.2.1 | Calculation of Time Period

Since Staad. Pro by default uses the formula for frame structure as

$$T = (0.075H)^{.75}$$

But since our structure is provided with infill walls

The following empirical formulae is used for time period calculation

$$T_x = \frac{0.09H}{\sqrt{d}}$$

H = height of building (in m)

T = time period (in seconds)

d = length of building in direction of earthquake (in m)

4.2.2 | Calculations of Rigidity of Equivalent Strut

From 3.5.4 we get

$\alpha = 0.624$

D = 5.31m, $\mathbf{E_m}$ = 2200×10^{6}N/m^2 **,** **t** = 0.2m, $\mathbf{E_{fe}}$ = 29580×10^6 N/m^2 $\mathbf{I_{col}}$ = 0.00213m^4 **h** = 3.05m

Θ = 31.383 degrees

$\boldsymbol{\lambda_1}$ = 0.8446

From these we get,

K = 51706214.69 N/m

$\boldsymbol{K = AE/L}$

From this A = 1372.8mm^2

Section chosen **ISMC 125**

A = 1619mm^2

4.3 | PLAN OF THE STRUCTURE

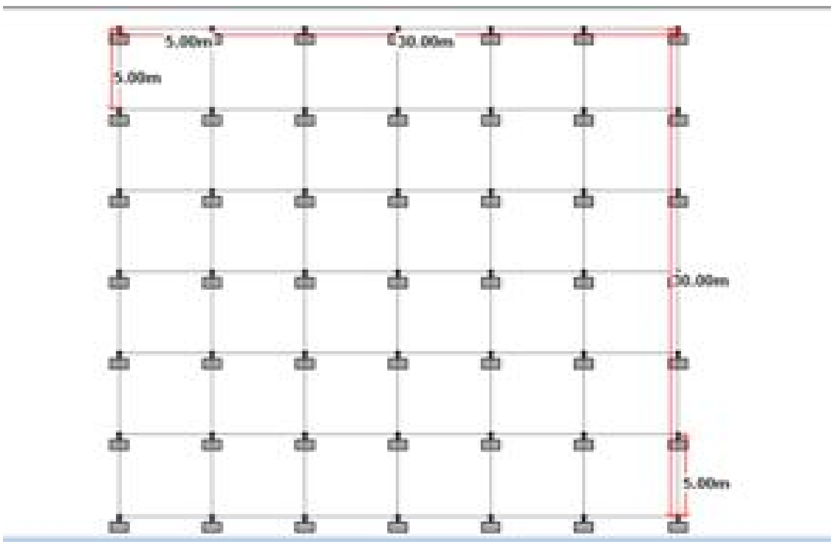


FIGURE 4.1: PLAN OF THE STRUCTURE (30m×30m)

4.4 |TYPES OF FRAMES CONSIDERED

FRAMES	DESCRIPTION
FRAME 1	Bare Frame W/O Infill Walls
FRAME 2	Infill Wall Only At Ground Floor
FRAME 3	No Infill Wall On Ground Storey
FRAME 4	Infill Walls On Every Floor

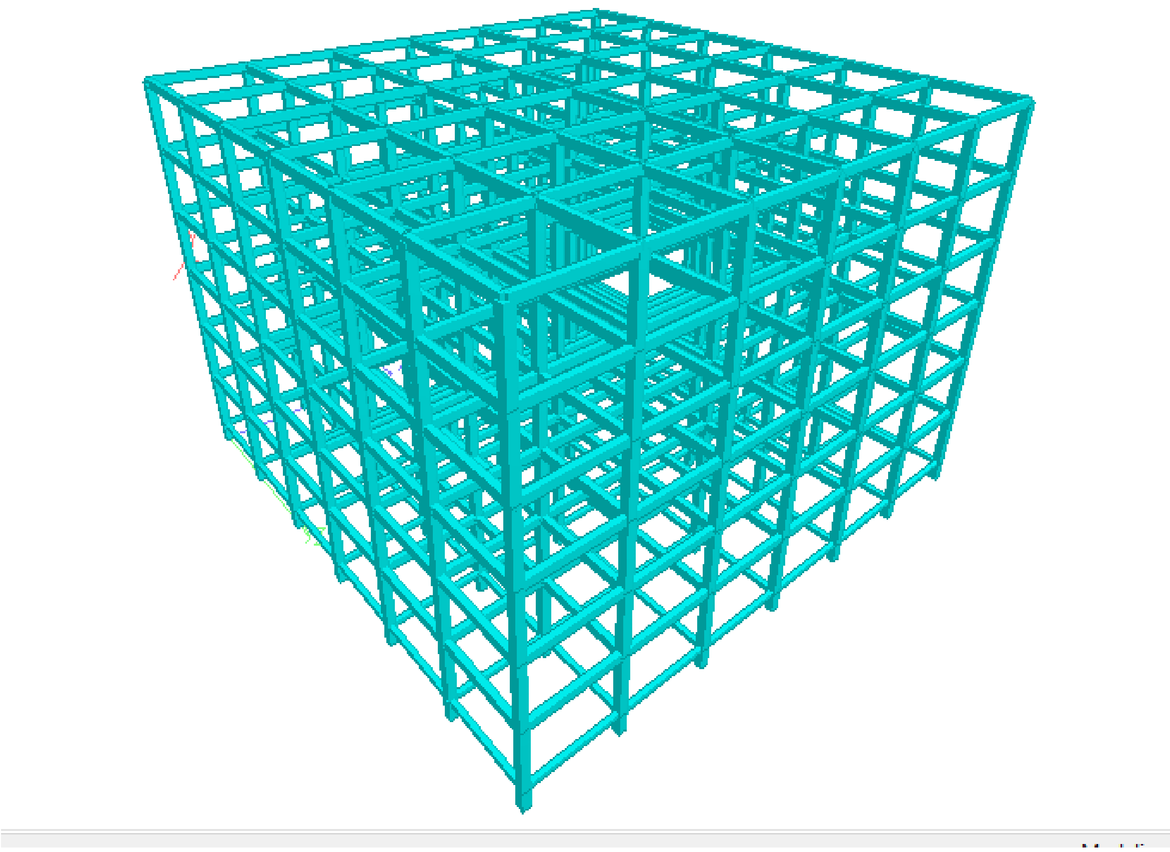


FIGURE 4.2: FRAME 1

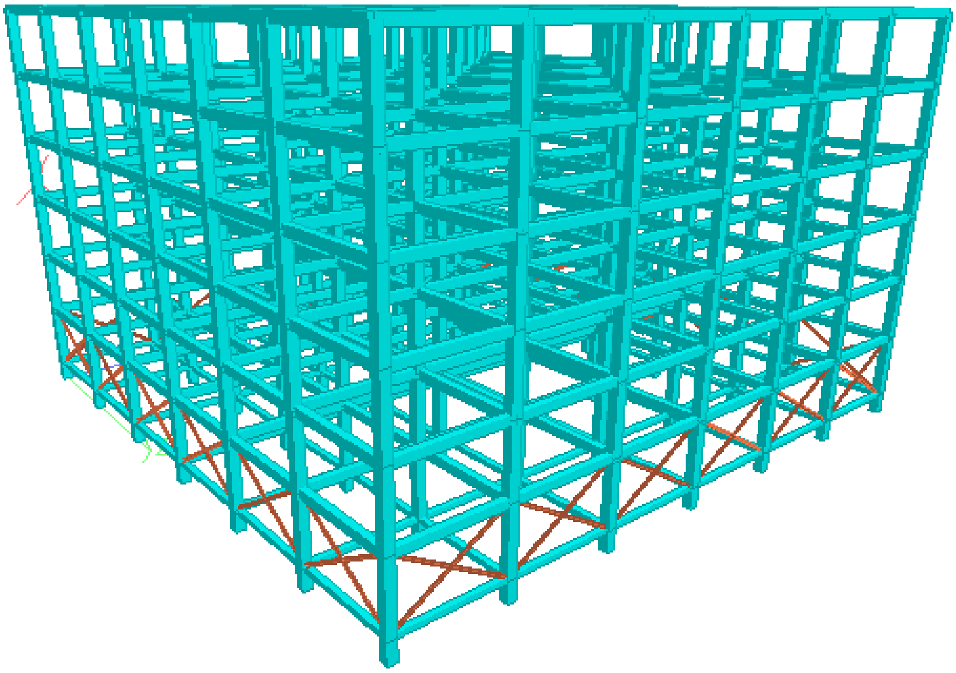


FIGURE 4.3: FRAME 2

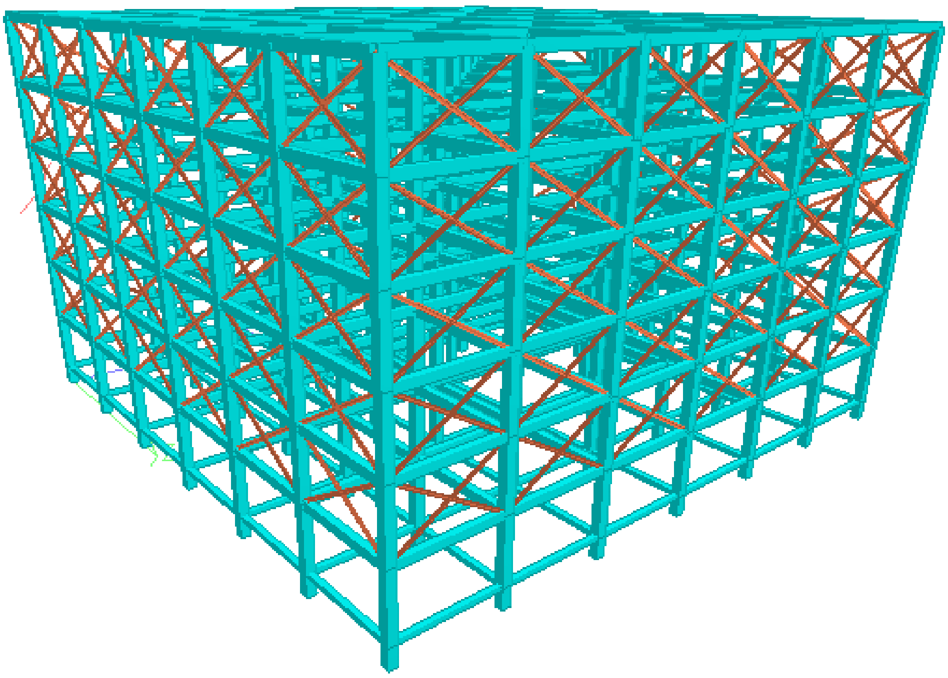


FIGURE 4.4:FRAME 3

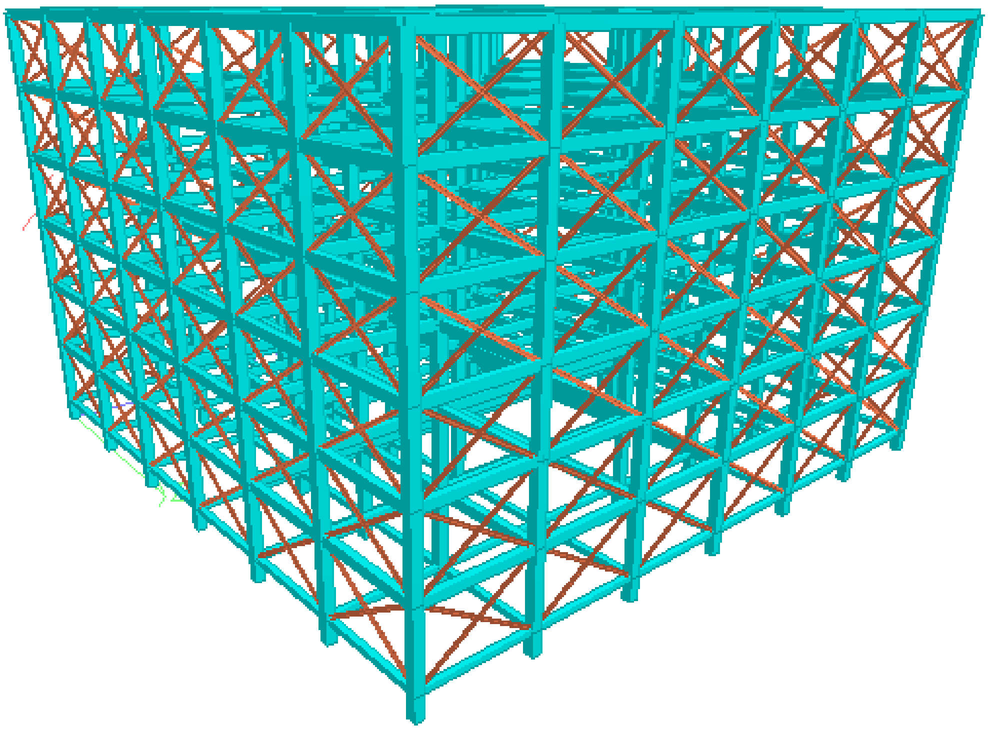


FIGURE 4.5 :FRAME 4

4.5|STEEL SECTION USED

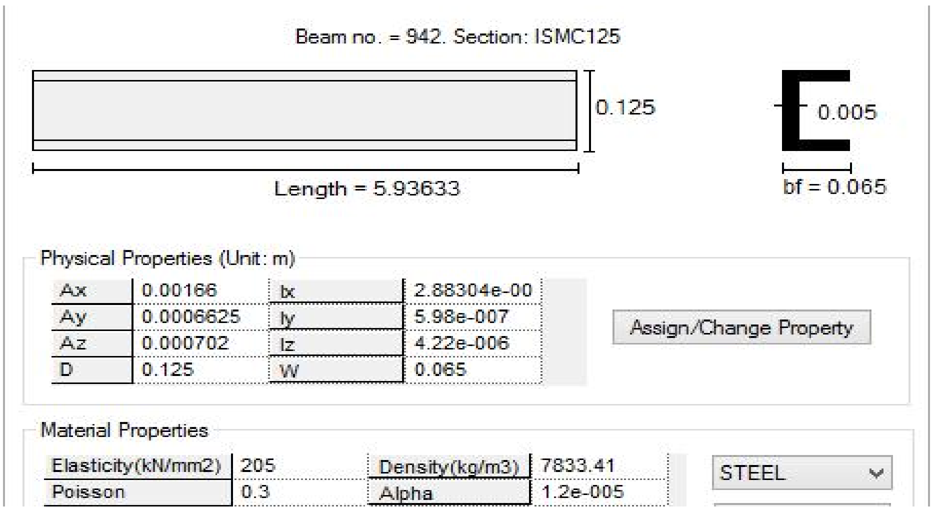


FIGURE 4.6 :DETAILS OF STEEL SECTION USED

CHAPTER 5

RESPONSE SPECTRUM ANALYSIS OF FRAMES

5.1| GENERAL

Response spectrum analysis of all the four frames is done. Four criteria are considered:

- Lateral displacement of the floors
- Storey drift
- Bending moment in the columns
- Peak storey shear

5.2|BARE FRAME W/O INFILL WALLS (FRAME 1)

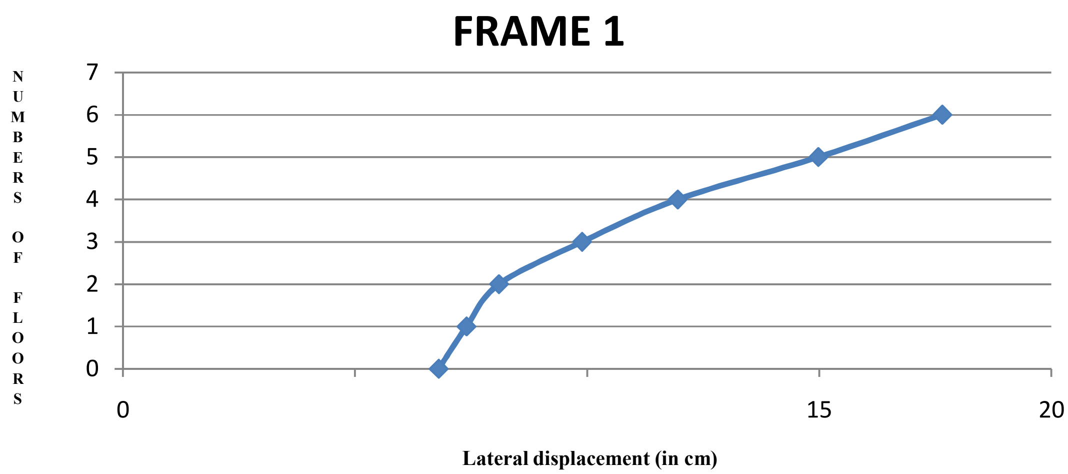


FIGURE 5.1 :LATERAL DISPLACEMENT

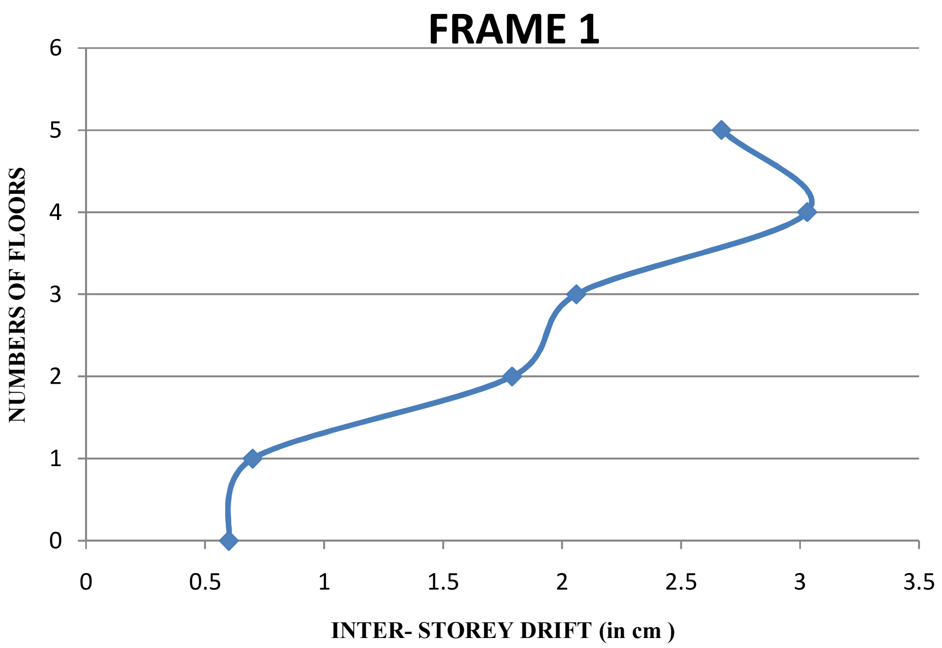


FIGURE 5.2 :INTER- STOREY DRIFT

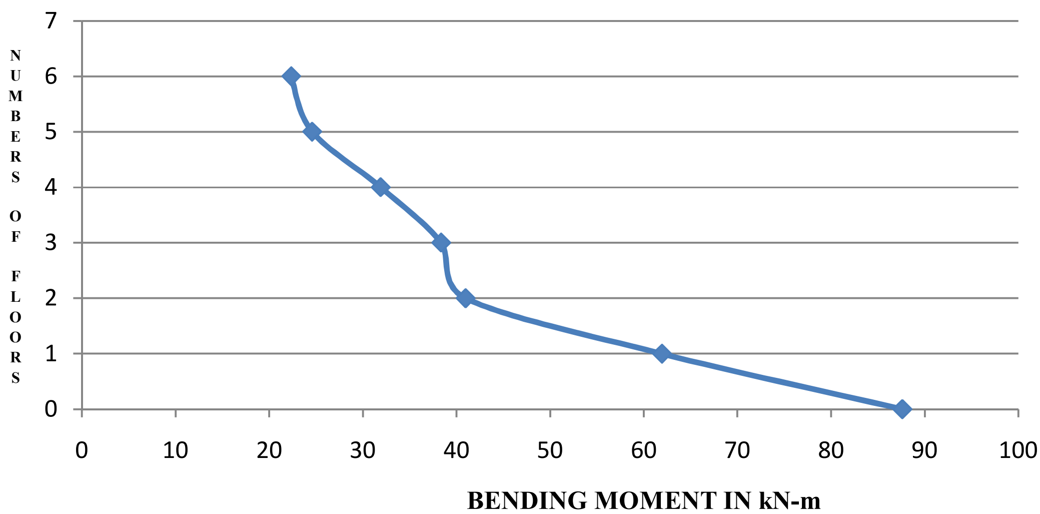


FIGURE 5.3 :BENDING MOMENT IN COLUMNS

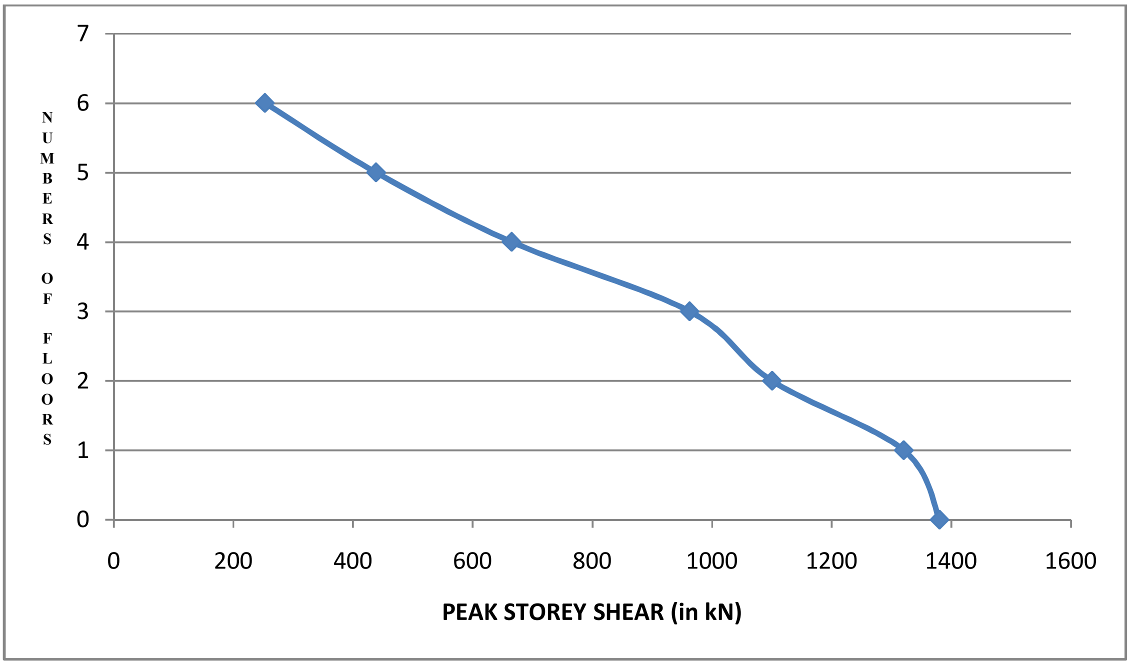


FIGURE 5.4 :PEAK STOREY SHEAR

5.3|INFILL WALL ONLY AT GROUND FLOOR

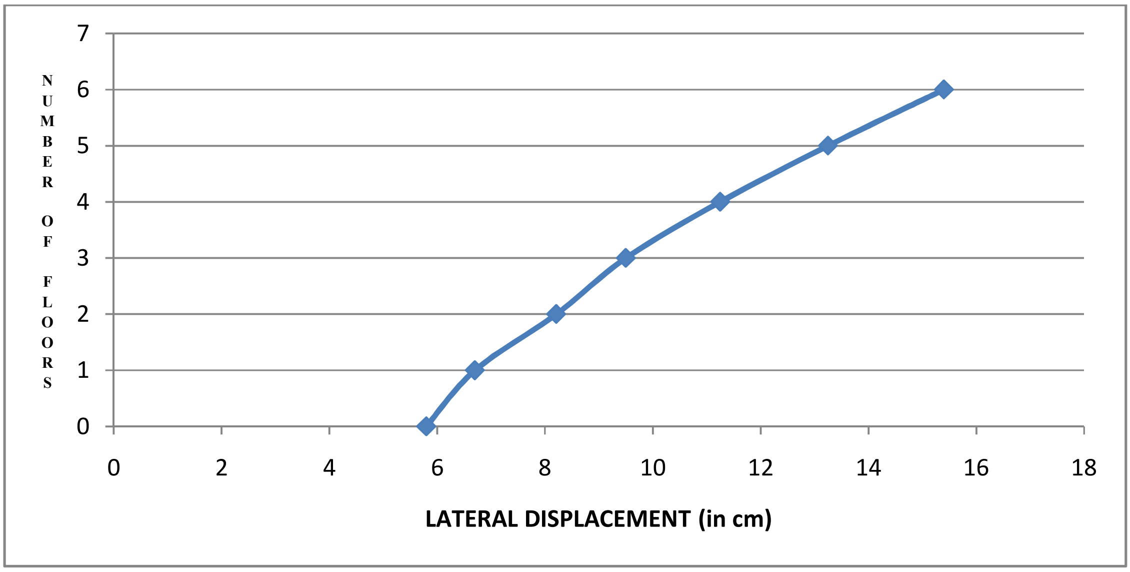


FIGURE 5.5 :LATERAL DISPACEMENT

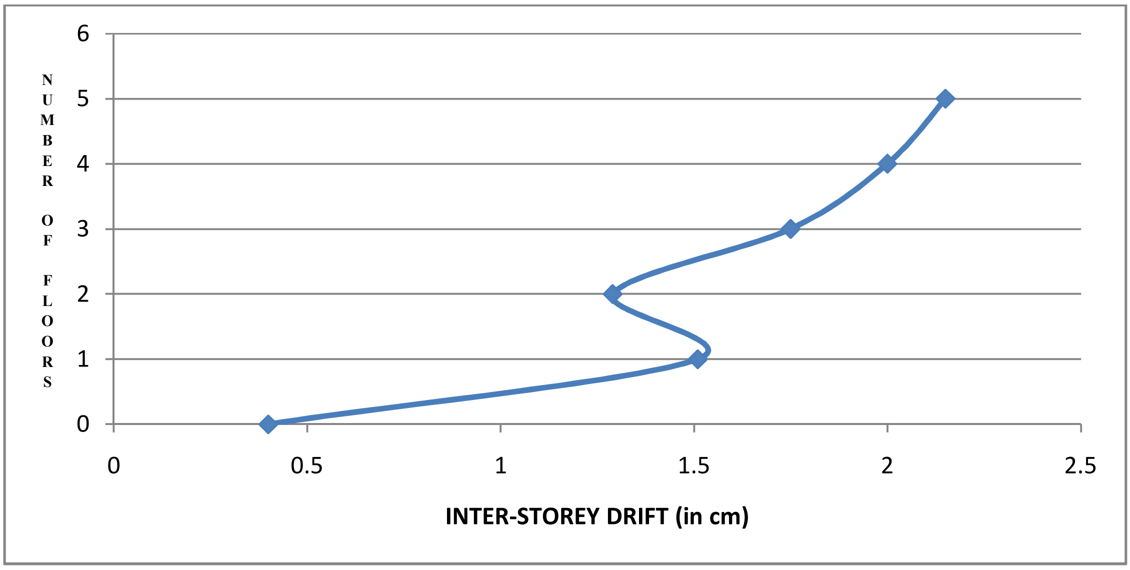


FIGURE 5.6 :INTER -STOREY DRIFT

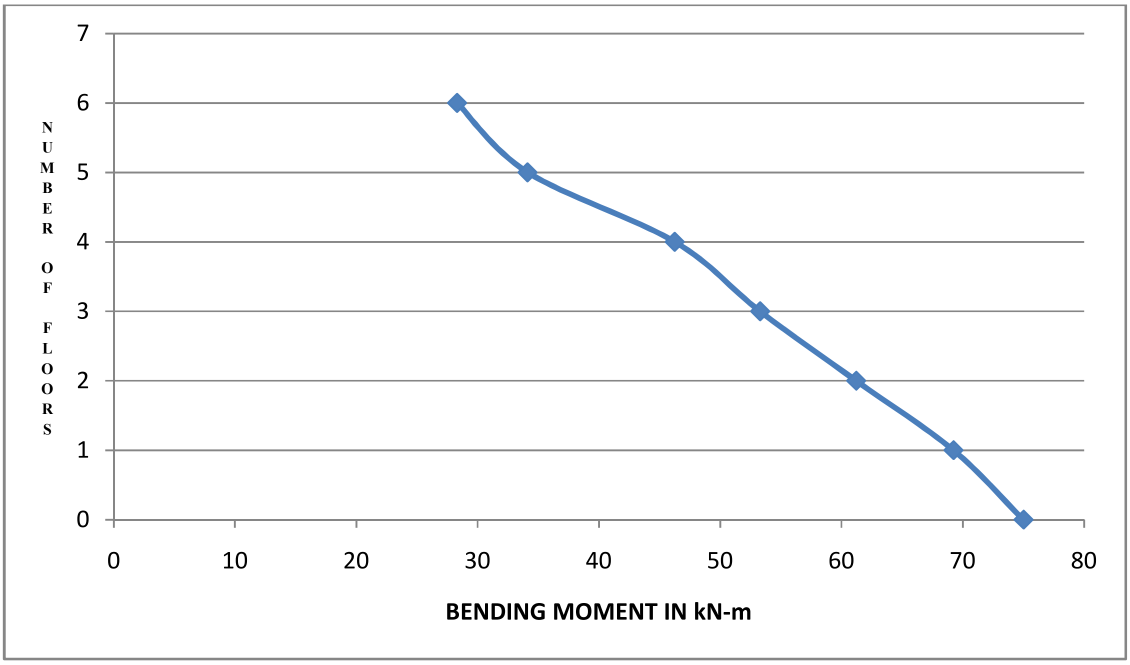


FIGURE 5.7 :BENDING MOMENT

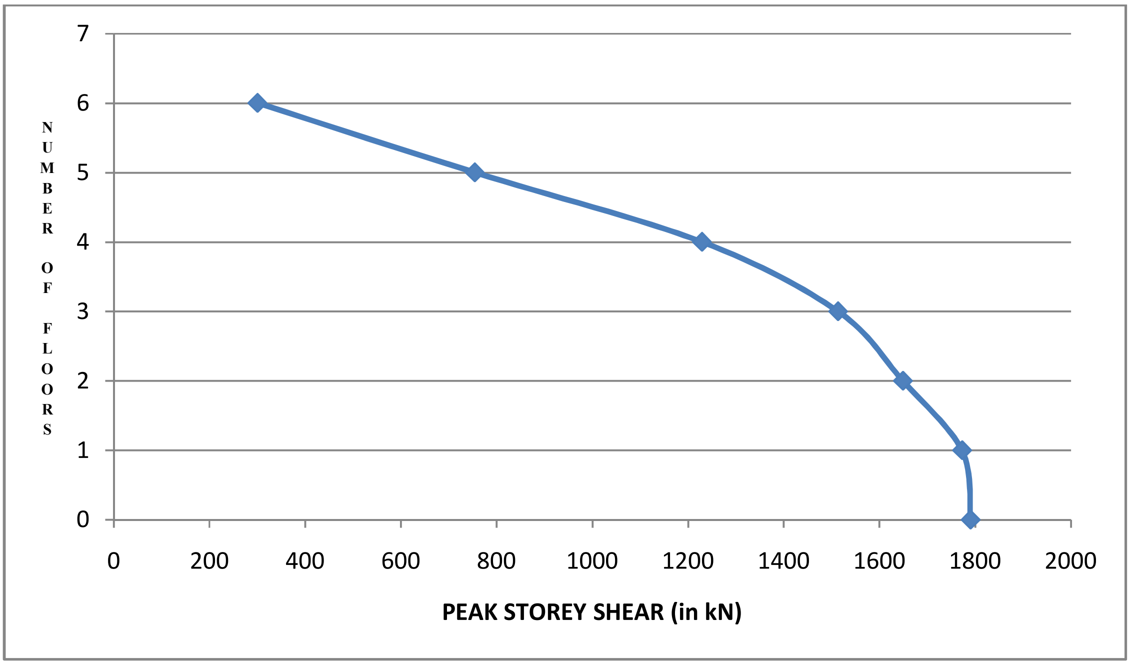


FIGURE 5.8 :PEAK STOREY SHEAR

5.4|NO INFILL WALL ON GROUND STOREY

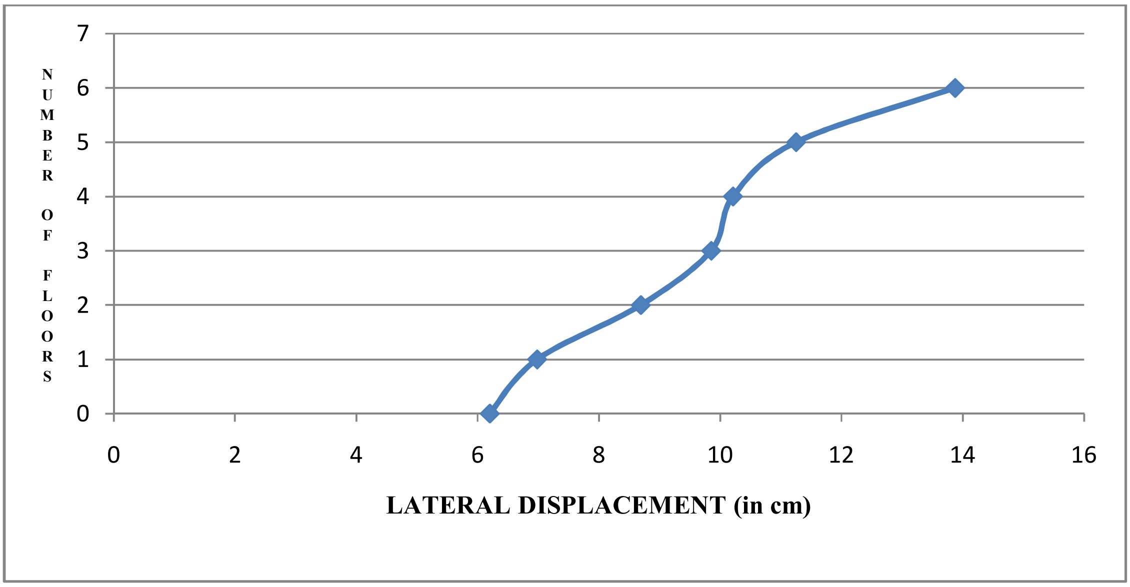


FIGURE 5.9 :LATERAL DISPLACEMENT

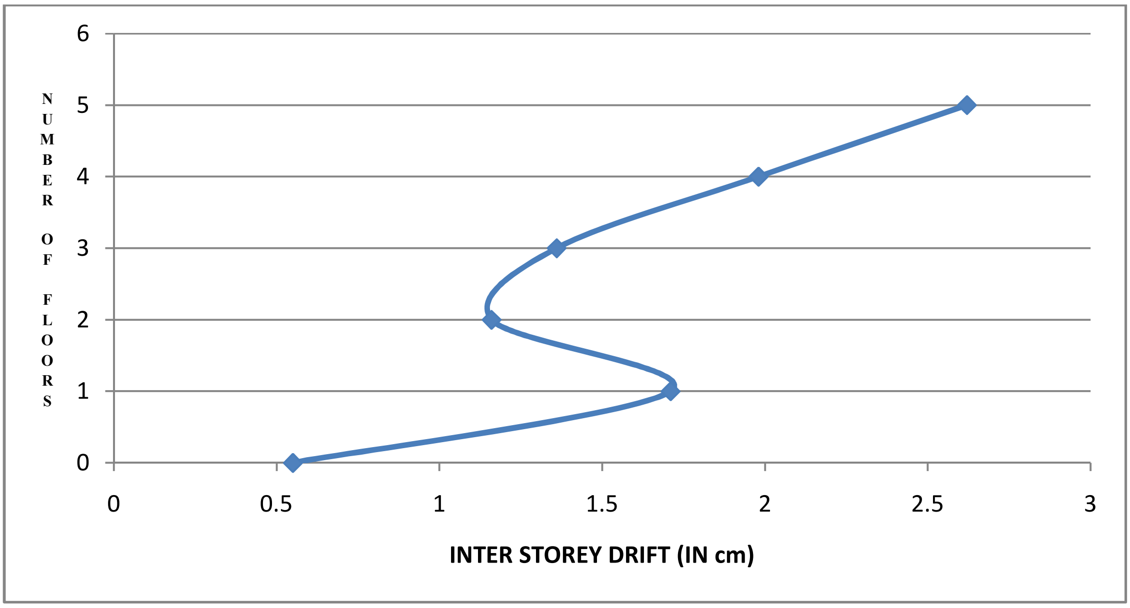


FIGURE 5.10 :INTER-STOREY DRIFT

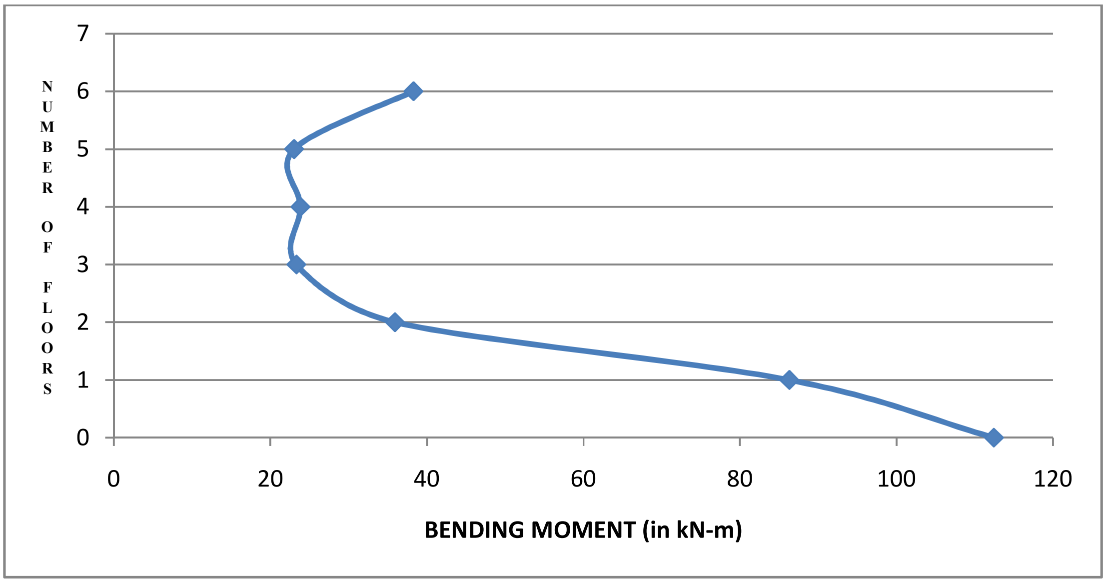


FIGURE: 5.11 :BENDING MOMENT

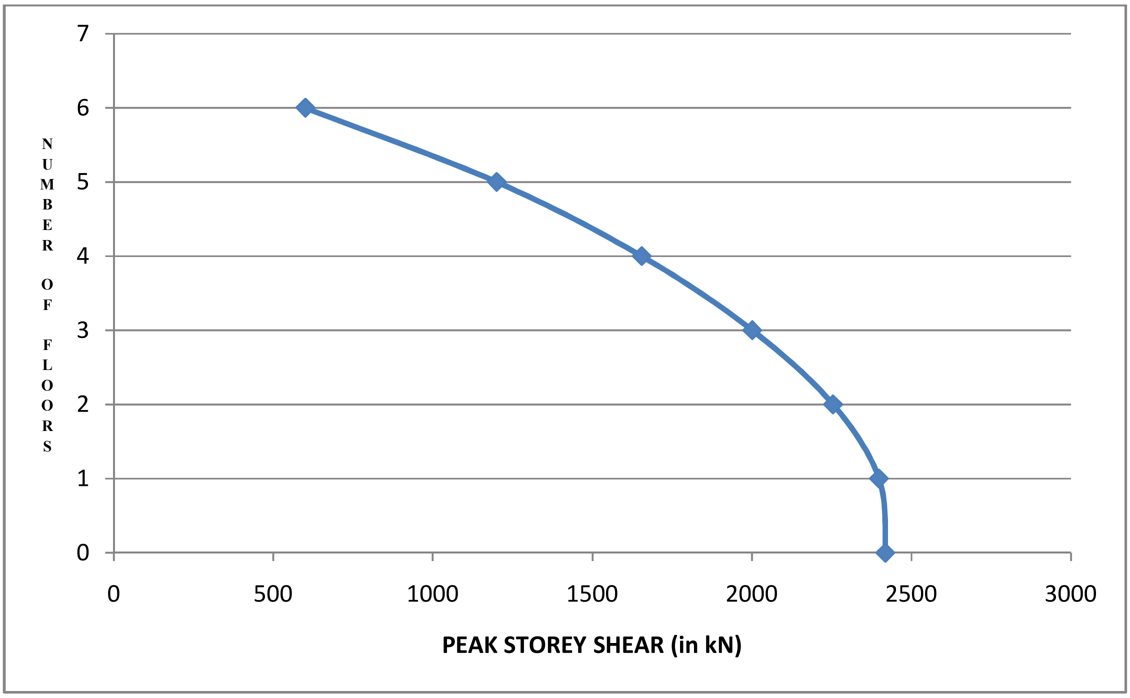


FIGURE: 5.12 :PEAK STOREY SHEAR

5.5 INFILL WALLS ON EVERY FLOOR

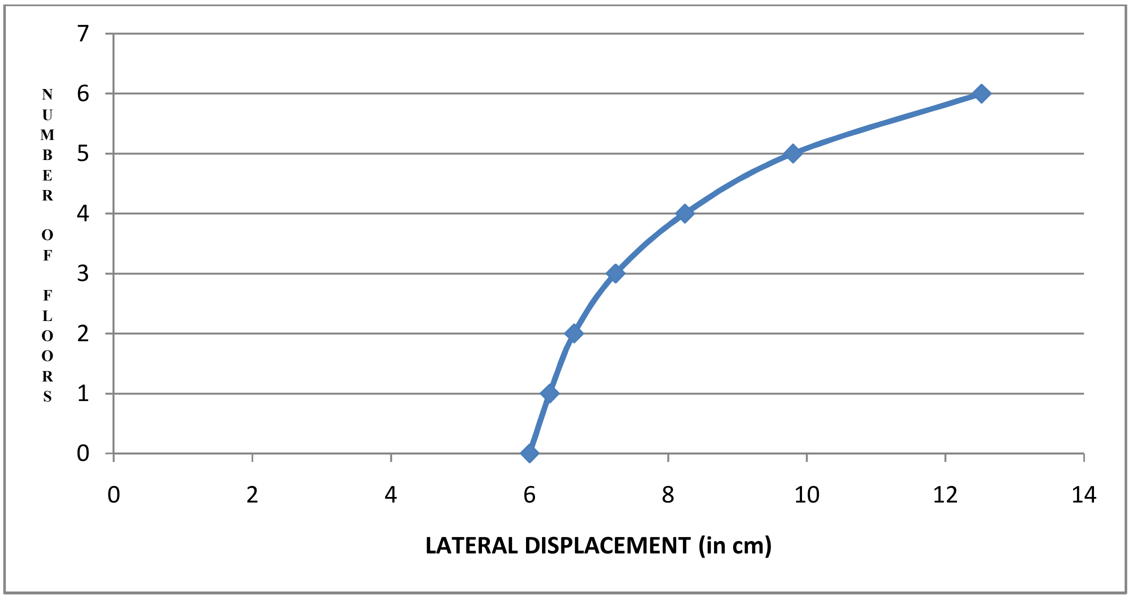


FIGURE: 5.13 :LATERAL DISPLACEMENT

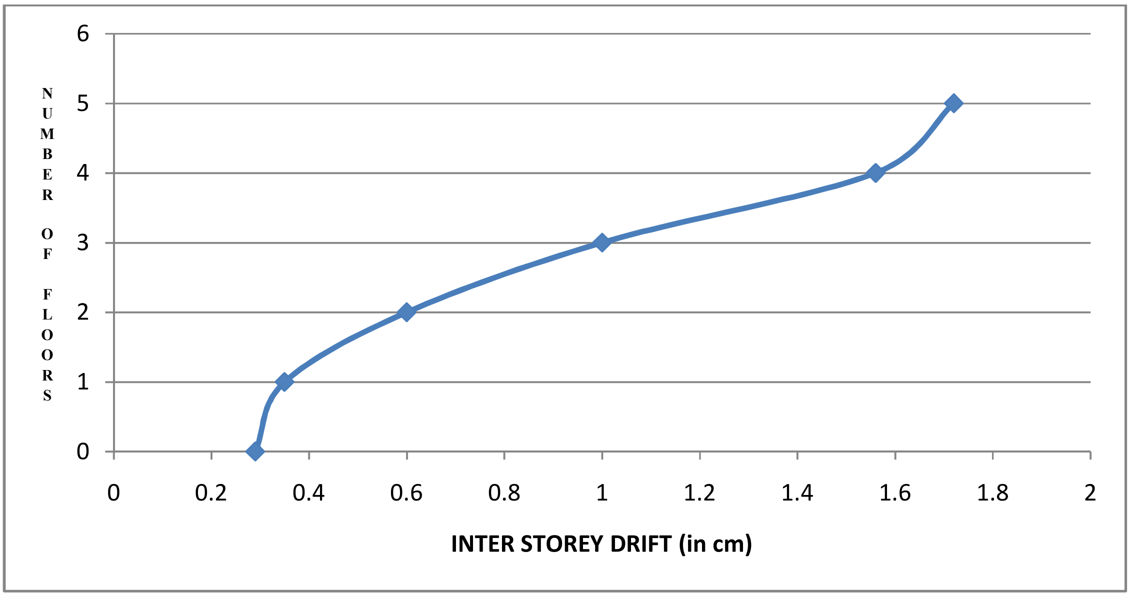


FIGURE 5.14 :STOREY DRIFT

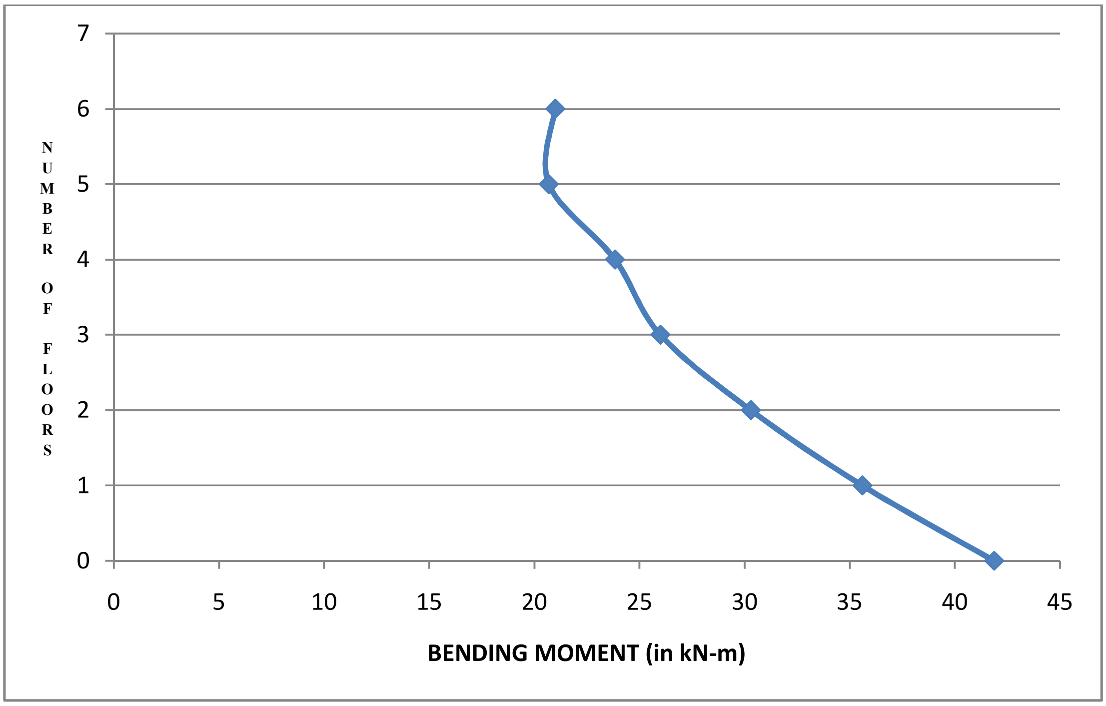


FIGURE 5.15 :BENDING MOMENT

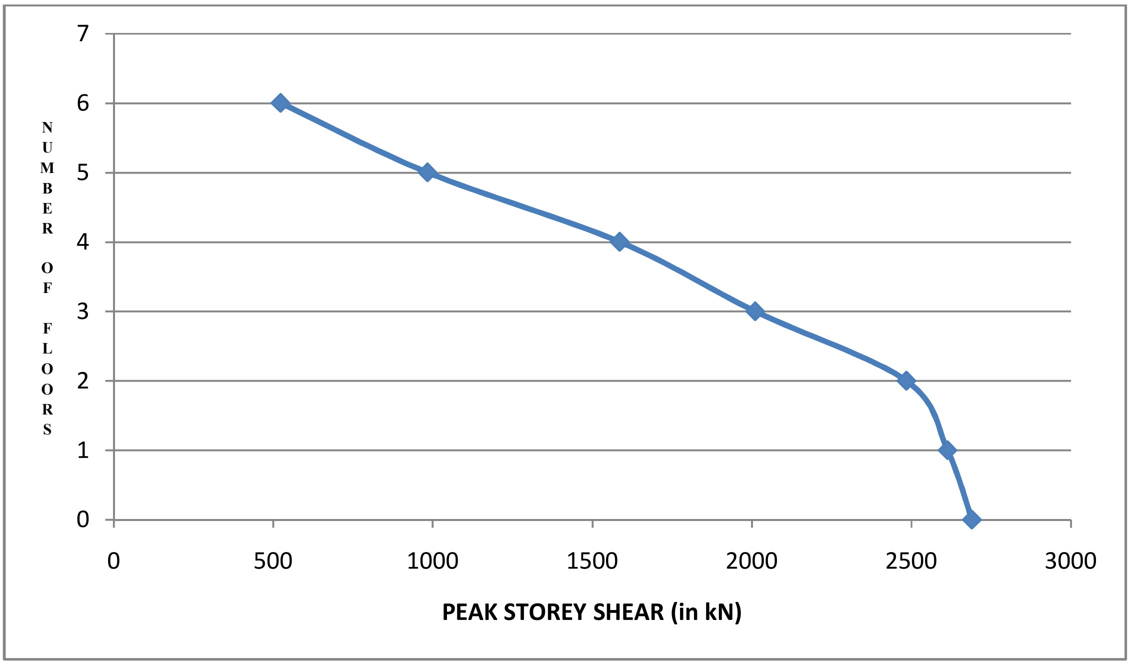


FIGURE 5.16 :PEAK STOREY SHEAR

5.6 BASE SHEAR COMPARISON

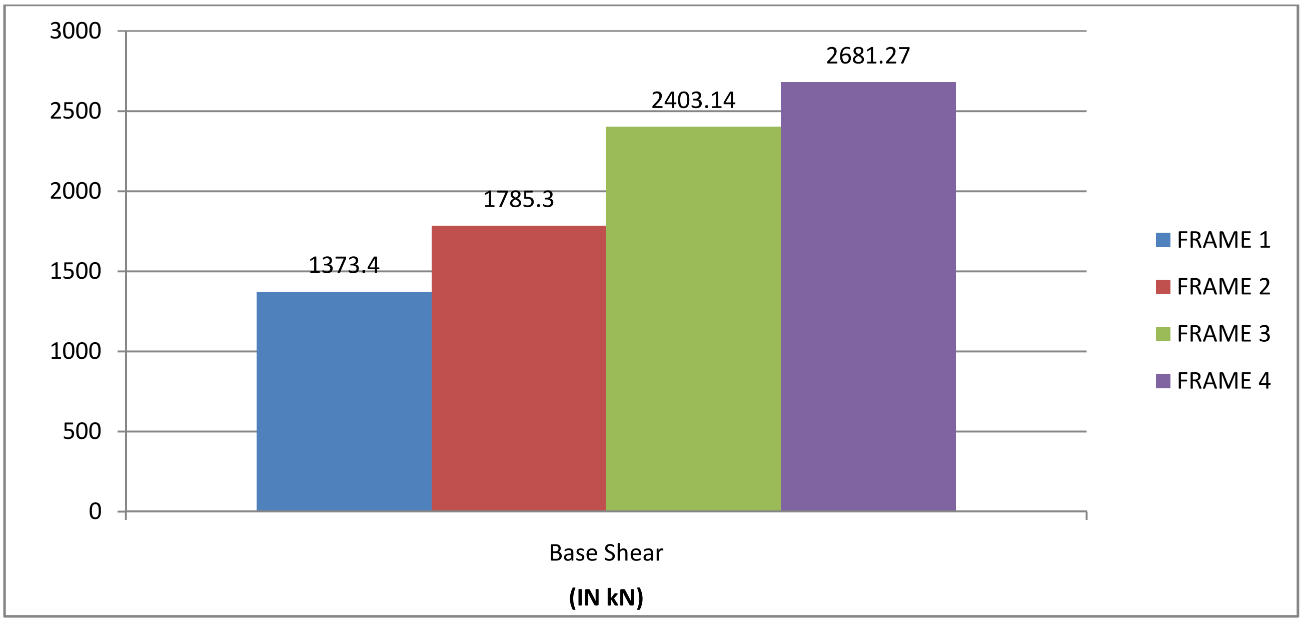


FIGURE 5.17 :BASE SHEAR COMPARISON

5.7 DISCUSSION OF RESULTS

From the results presented in this chapter, following observations are made:

- From the results obtained from the analysis it is found that base shear increased by 95.2% from frame 1 to frame 4.
- Storey shear increased on each floor while considering the effect of infill walls.
- Storey drift of the ground floor decreased significantly by about 50% while the storey drifts of other floors do not have any major changes.
- Bending moment decreased significantly for the infilled structure.
- The code suggests that the forces in the columns , beams and shear walls 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 bare frame analysis.

CHAPTER 6

TIME HISTORY ANALYSIS OF FRAMES

6.1 | TIME HISTORY ANALYSIS

It is the analysis of a structure applying data over increment time step as a function of:

- Acceleration
- Force
- Moment
- Displacement

It provides the response of structure over time during and after the application of a load .The time history response of a structure is simply the response (motion or force) of the structure evaluated as a function of time

6.1.1 | Time Step

- Time increment used for analysis.
- Output time step size should be small enough to provide sufficient resolution for analysis.
- The required resolution is dependent upon the characteristics of applied loading and structural properties like natural period.
- Time step used in the analysis is equal to 0.00139 sec.

6.1.2 | Arrival Time

- It is the time at which load assignment begins.
- Can be positive,negative or zero.
- Begins at time zero by default.
- Portion of the input which occurs before time zero is ignored for negative arrival time.
- Arrival time is used in analysis is equal to 2 sec.

- Number of modes contribution is 25.
- Duration = 43.9 seconds

6.2|INTER-STOREY DRIFT

6.2.1| No Infill Wall on Ground Storey (FRAME 3)

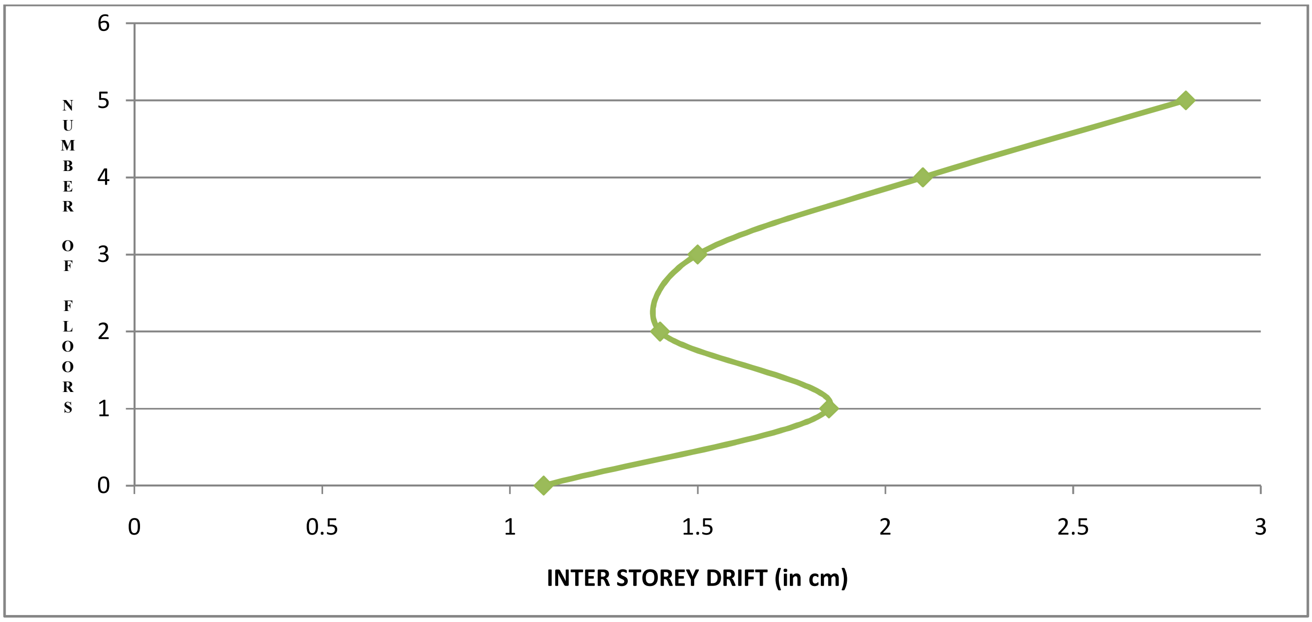


FIGURE 6.1 :INTER STOREY DRIFT

6.2.2|Infill Wall Only at Ground Floor (FRAME 2)

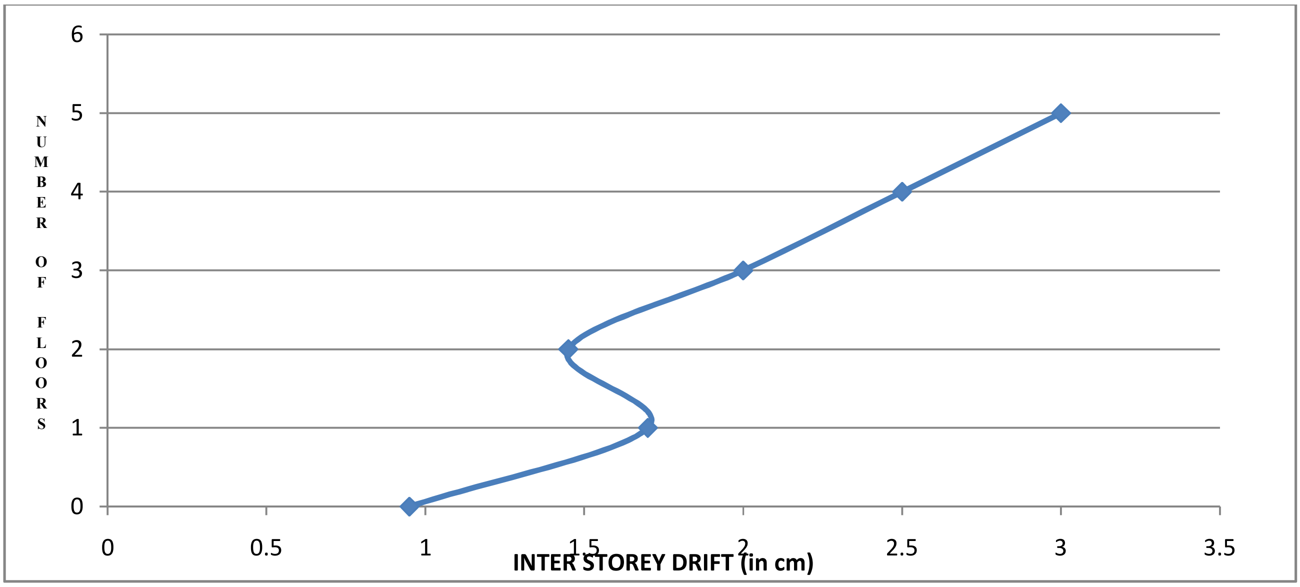


FIGURE 6.2 :INTER STOREY DRIFT

6.2.3|Infill Walls on Every Floor (FRAME 4)

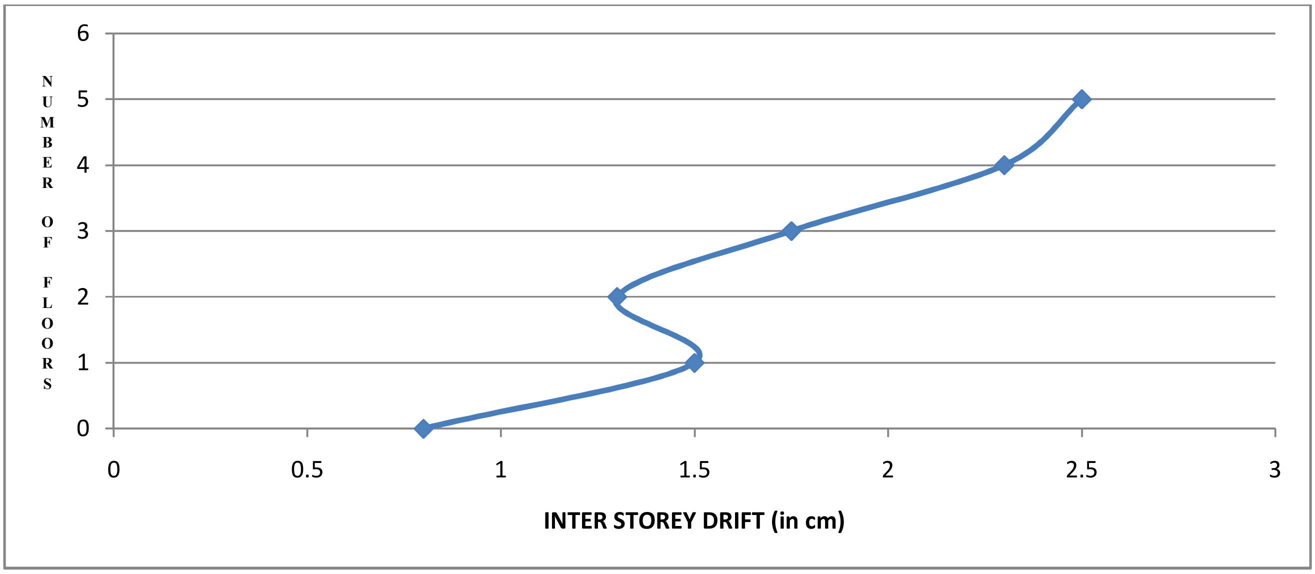


FIGURE 6.3 :INTER STOREY DRIFT

6.3|TIME DISPLACEMENT GRAPH

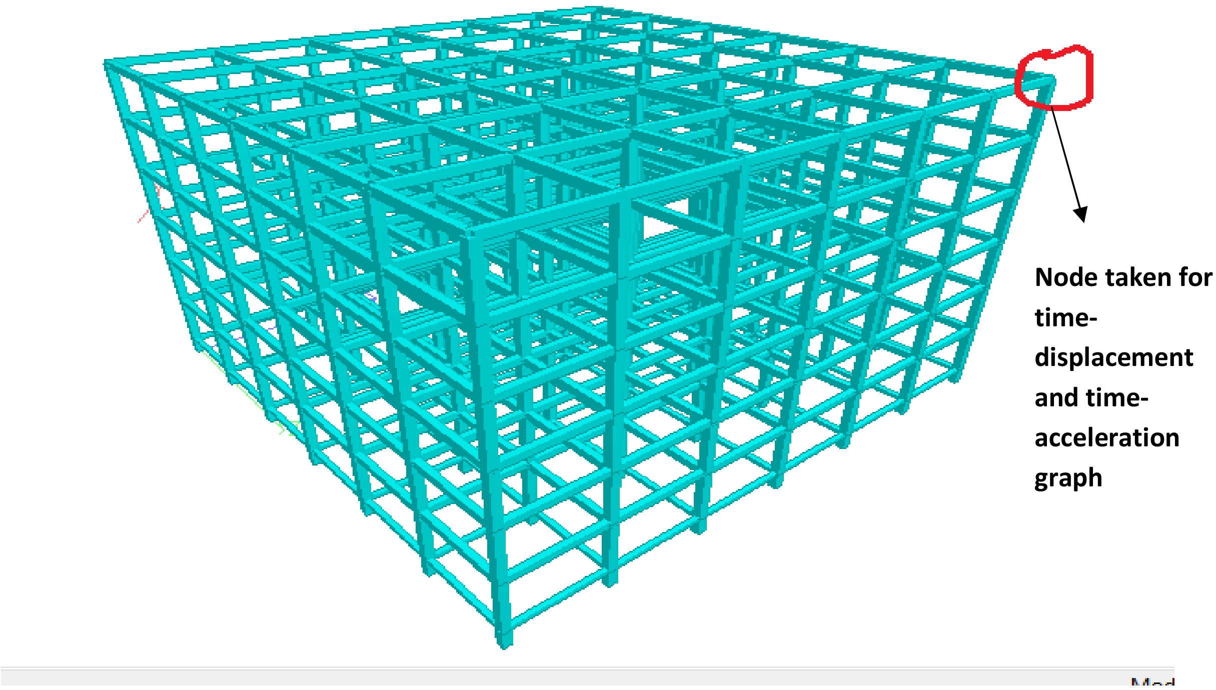
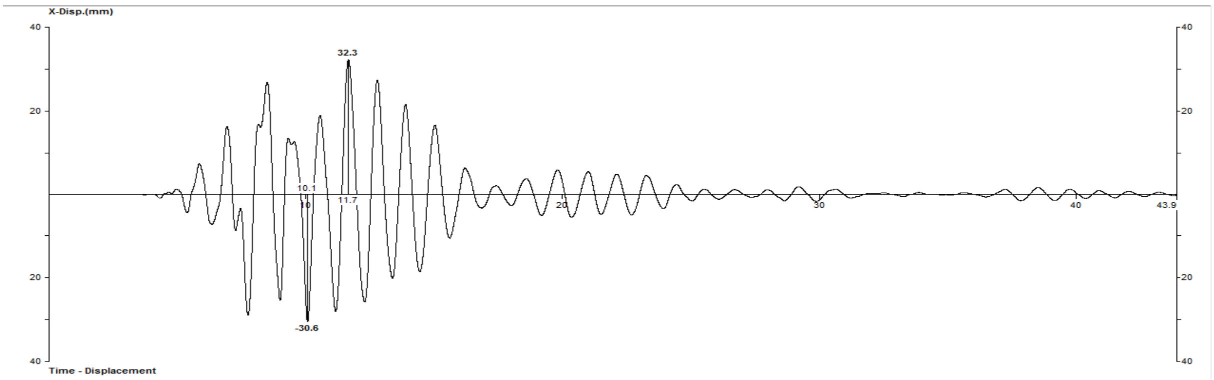


FIGURE 6.4 :NODE TAKEN FOR TIME-DISPLACEMENT AND TIME-ACCELERATION GRAPH

6.3.1| No Infill Wall on Ground Storey (FRAME 3)



Time (in seconds)

FIGURE 6.5 :TIME DISPLACEMENT GRAPH

6.3.2|Infill Wall Only at Ground Floor (FRAME 2)

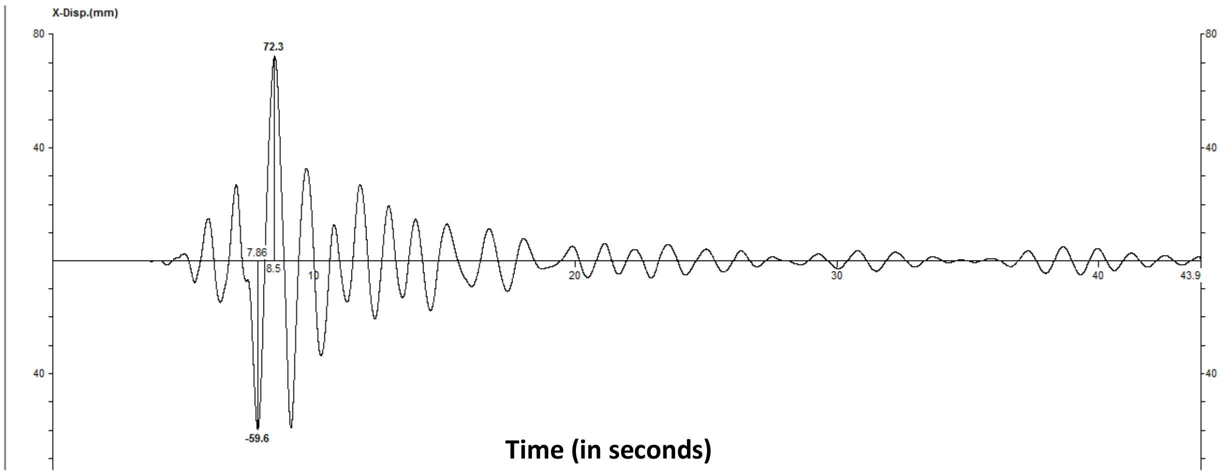


FIGURE 6.6 :TIME DISPLACEMENT GRAPH

6.4|TIME ACCLERATION GRAPH

6.4.1| No Infill Wall on Ground Storey (FRAME 3)

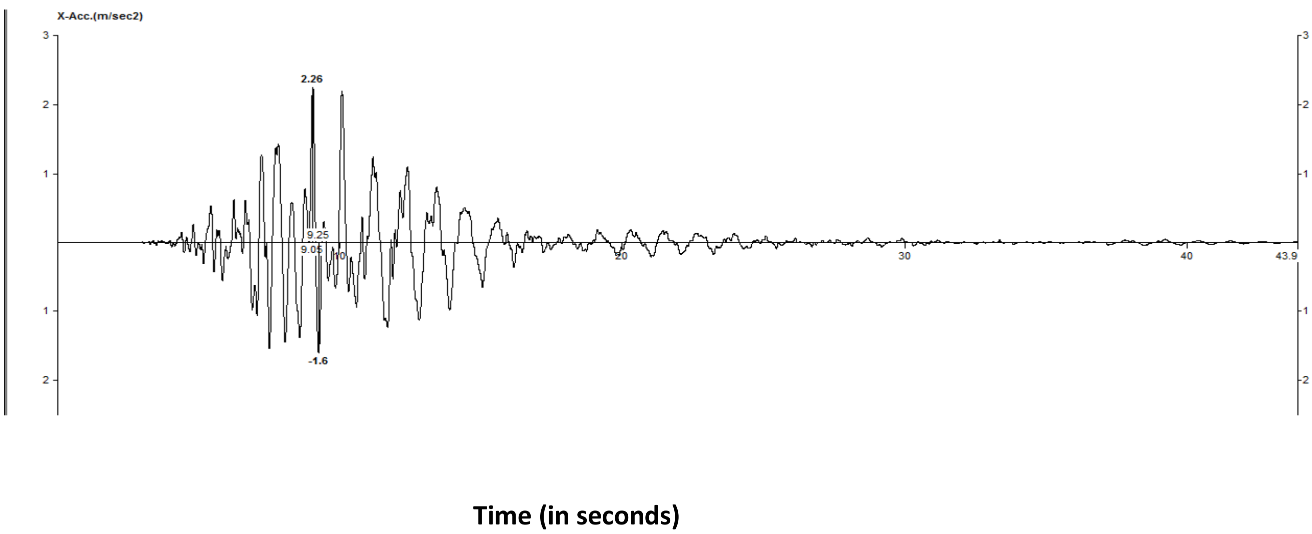


FIGURE 6.7: TIME ACCLERATION GRAPH

6.4.2|Infill Wall Only at Ground Floor (FRAME 2)

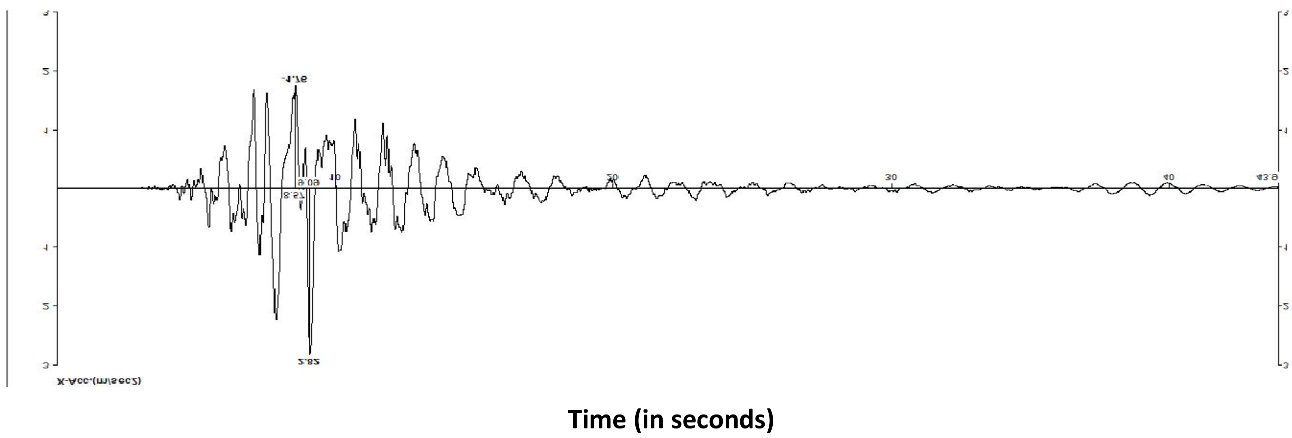


FIGURE 6.8: TIME ACCLERATION GRAPH

CHAPTER 7

CONCLUSIONS

7.1 CONCLUSIONS

From the study carried out in the present work, the following conclusions have been drawn.

1. The results of both the methods of analysis (response spectrum and time-history analyses) used were approximately same.
2. Due to inclusion of infill walls, behaviour and failure modes of buildings have changed. The results show the importance of considering infill walls in modelling, to get the real scenario of damages.
3. The code suggests that the forces in the columns , beams and shear walls 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 bare frame analysis.
4. Infill walls increases the stiffness of the structure and attracts large force. Bare frame (without infill walls) are less stiff as compared to with infill frames and are flexible. Hence, bare frame structure attracts less force and value of base shear is less. This may lead to under-estimation of base shear and other demands causing a different design of frames compared to the case when infill walls beheviour is considered.

7.2 SCOPE FOR FUTURE WORK

The work presented in this report may be extended or improved by taking into consideration the following technical points.

- More number of different arrangements of infill walls could have been taken, especially for the interior infill walls.
- Micro modelling of infill walls using FEM could have been done to better simulate the behaviour of the frames.

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