VERIFICATION OF CODE RECOMMENDED FUNDAMENTAL TIME PERIOD FOR LOW AND MEDIUM RISE R.C.C BUILDINGS

(MAJOR PROJECT REPORT)

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

MASTER OF TECHNOLOGY IN CIVIL ENGINEERING
With Specialization in
STRUCTURAL ENGINEERING

Submitted By

MD ZAMEER ABID RAZI
ROLL NO. 2K11/STE/24

Under the guidance of

PROF. A. K. GUPTA

DEPARTMENT OF CIVIL ENGINEERING
DELHI TECHNOLOGICAL UNIVERSITY
SHAHBAD DAULATPUR, BAWANA ROAD
DELHI-110042, INDIA

July 2014

Department of Civil Engineering

Delhi Technological University

Delhi -110042 (India)

CANDIDATE'S DECLARATION

I do hereby certify that the work presented in this report entitled "**VERIFICATION OF CODE RECOMMENDED FUNDAMENTAL TIME PERIOD FOR LOW AND MEDIUM RISE R.C.C BUILDINGS**" in partial fulfillment of the curriculum of sixth semester of Master of Technology in Structural Engineering, submitted in the Department of Civil Engineering, DTU is an authentic record of my own work under the supervision of PROF. A. K. GUPTA, Professor Department of Civil Engineering.

I have not submitted this matter for the award of any other degree or diploma.

Date : July 10,2014

(Md Zameer Abid Razi)

2K11/STE/24

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

(PROF. A. K. GUPTA)

Professor

Department of Civil Engineering,

Delhi Technological University

Dedicated to My Parents

ACKNOWLEDGEMENT

I wish to express my sincere gratitude and indebtedness to my guide PROF A. K. GUPTA. It is due to his clarity of thinking and expression, this work has some substance and makes a better reading. The innumerable discussions that I had with him, has made a deep impact on my understandings. He is always ready to welcome his students with doubts in any sphere. Without his valuable suggestions, inspiration and guidance this report could not have been possible.

I express my sincere thanks to Sir Rabin Dasgupta and Er. Shahzeb Md. Danish who helped and guided me whenever I faced any roadblock in achieving my target of completing this thesis.

I would like to thank all my friends especially Shireen A Hossain, Noorul Islam , Sumit Maurya, Kuldeep Singh who have in their own way helped in keeping me energised and encouraged me whenever I felt low, to complete this thesis.

At last I thank Allah for giving me life & brain to admire his laws & get amazed.

Md Zameer Abid Razi

ABSTRACT

Fundamental time period of any structure is one of the most important aspect as it determines the amount of base shear and all other design forces that are to be considered in the analysis and design of the structure. If a structure has a higher value of time period making it fairly flexible, it will attract lesser forces compared to its stiffer counterpart with smaller time period.

The empirical formulae suggested by IS 1893 (2002) are based on observed natural period values on real buildings during the 1971 San Fernando earthquake in California which are very general in nature and does not incorporate the inherent variety of unsymmetry, irregularities existing in different buildings. The time period obtained using these formulae often gives large variations when compared with the fundamental mode time periods of dynamic analysis. As a result of this variation the base shear calculated using dynamic analysis is often lower than the static analysis. Due to this the code recommends to scale the dynamic analysis base shear, so that it matches with the static one. This approach however conservative may be, but is not accurate.

In the present study we are trying to find a rational approach by studying different models and investigate the variation in time period and forces between dynamic analysis results and code recommended empirical formulae results. An effort has been made to incorporate different kind of buildings along with some unsymmetry and irregularities; and investigate their vibrational behaviour. Regression analysis has also been carried out to generate empirical expressions from the dynamic analysis results and their variation with the codal formulae have been investigated.

After studying these variations it was realized that the basic issue with our code still remains in its empirical formula approach. However large be the sample size, there would always be buildings that are not part of that sample size. In fact, every other buildings may behave differently under dynamic loads. Thus a more rational approach would be to drop the empirical formula and analyse every building rigorously. A more rigorous dynamic analysis, pushover analysis or performance based analysis would be more suited for the purpose.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGUERS

LIST OF GRAPHS:

CHAPTER 1

Introduction

Clause 7.8.2 of IS1893:2002 [1] stipulates that dynamic analysis is to be carried out necessarily for some buildings, but the design forces shall be scaled up to match the forces calculated using the empirical formulas for time period of the building. This implies that empirical *T* may be more reliable than *T* computed by dynamic analysis. Dynamic analysis based on questionable assumptions may give an unduly large natural period, and hence, a much lower design seismic force. There are considerable uncertainties in modelling a building for dynamic analysis, such as:

- Stiffness contribution of non-structural elements;
- Stiffness contribution of masonry infill;
- Modulus of elasticity of concrete, masonry, and soil; and
- Moment of inertia of RC members.

Thus, there can be large variation in natural period, depending on how one models a building. For instance, ignoring the stiffness contribution of infill wall itself can result in a natural period several times higher.

As per NEHRP Commentary [FEMA 369, 2001] [3]:

"If one ignores the contribution of non-structural elements to the stiffness of the structure, the calculated period is lengthened, leading to a decrease in the design force. Non-structural elements do not know that they are non-structural. They participate in the behaviour of the structure even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the un-conservative side."

Even when the results of dynamic analysis are scaled up to design force based on empirical *T*, the load distribution with building height and to different elements is still based on the results of the dynamic analysis, and therein, lies the advantage of dynamic analysis. Hence this clause acts as a safeguard against improper assumptions in dynamic analysis.

Now the question that arises is, how reliable is our empirical formula for time period which is our key parameter? What if the empirical formula gives unnecessarily lower values of time period and thereby higher values of base shear? So, this formula needs to be investigated properly.

1.1 A case study of a medium rise residential building.

Project: The Sky Court, DLF

Location: Sector 86, Gurgaon

Building Height: 66.4 m (B+G+19)

Building Plan: 37.35mX13.9m (Refer Drawing 1)

Type: Framed RCC with Shear Walls

Software: ETABS 9.7.4

Seismic Zone: V (Considered)

HEIGHT 66.4 m

Width in X 37.35 m

Width in X 37.35 m

Time Period Dynamic		Time Period Empirical		
Mode 1	Mode 2	$0.075h^{0.75}$	$0.09h/\sqrt{d}$	
			in X	in Y
3.451	2.387	1.745	0.978	1.603

Base Shear Dynamic		Base Shear Static			
In X	In Y	0.075h.75		0.09h/√d	
		in X	in Y	in X	in Y
2799.71	4274.23	5710	5710	10188.46	6215.16
	Scale	2.039	1.336	3.639	1.454

Here we notice that the base shear is almost doubled in case of bare frames and four times in case of brick infill frames in X direction. We also observe that the base shear as calculated using the empirical formula (with infill) in X direction is more than the same in Y direction; but after dynamic analysis it just reverses i.e. the base shear calculated in X direction is less than that in the Y direction. From the attached framing plan we realize that, it is more logical to have higher base shear in the Y direction as given by dynamic analysis; since most of the columns and shear walls are oriented along Y direction. So, clearly there is a disagreement between the empirical approach and the dynamic analysis approach.

So the crux of the issue is, appropriate determination of time period is very important as it eventually determines how much design forces are to be applied to a building. Let's look at the two empirical formula suggested by IS1893-2002 [1].

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

T_a = 0.075 $h^{0.75}$ for RC frame building

= 0.085 $h^{0.75}$ for steel frame building

Where,

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

The two equations for frame buildings were taken from NEHRP's earlier provisions. These equations are based on observed natural period values on real buildings during the 1971 San Fernando earthquake in California. The basic issue with the above formulae is that they are completely not representative of our actual buildings. Neither do they accommodate the inherently present irregularities in stiffness present in our buildings. Several other parameters than height of the building are involved in the variation of time period of the structures.

7.6.2 The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

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

Where,

h= Height of building, in m as defined in 7.6.l; and

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

As per experimental studies (ambient vibration surveys) on Indian RC buildings with masonry infill, $T = 0.09h/(\sqrt{d})$ was found to give a good estimate. However, this formula does incorporate the effect of non-structural components on the stiffness and time period of buildings but sufficient literature on the modelling of buildings considering brick infill panels is not available. We often analyse a building using this formula for time period but do not consider the brick infill panels in our model. This further gives us erroneous analysis results.

1.2 Objective and Scope

1.2.1 Objective:

Verification of code recommended fundamental time period for low and medium rise R.C.C buildings

1.2.2 Scope:

Following is the scope of the present study to achieve the above objective:

Literature Review and Benchmarking Studies

1. Study the behaviour of low rise stiff buildings
    a. Modelling different buildings of low height (less than 10 storeys).
    b. Study their vibration behaviour and fundamental time period.
    c. Tabulate the results and generate graphs to establish key parameters that affect time period of the buildings.
    d. Carry out regression analysis to establish an empirical relationship between time period of the building and other key parameters.
    e. Check the validity of the established empirical formula and compare it with the existing Code recommended formula.
2. Study the behaviour of medium rise buildings
    a. Modelling different buildings of medium height (30 to 50 storeys).
    b. Study vibration behaviour and fundamental time period of buildings with different types of irregularities.
    c. Study vibration behaviour and fundamental time period variation of buildings with and without brick infill.
    d. Tabulate the results and generate graphs to establish key parameters that affect time period of the buildings.
    e. Carry out regression analysis to establish an empirical relationship between time period of the building and other key parameters.
    f. Check the validity of the established empirical formula and compare it with the existing Code recommended formula.

CHAPTER 2

Literature Review

Most of the codes refer to three types of analysis for earthquake forces, namely: (i) Response Spectrum Analysis (RSA), (ii) Response History Analysis (RHA), and (iii) Seismic Coefficient Method of Analysis. [2] Seismic Analysis of Structures, Prof. T K Datta The response spectrum method of analysis allows the designer to use a set of equivalent lateral forces for each mode of vibration and carry out a static analysis to obtain a good estimate of the mean peak response of the structure. The response history method of analysis provides the maximum response of the structure under any time history of loading. The term equivalent lateral load analysis of tall structures, such as buildings, chimneys, towers, and so on, is not only used for the response spectrum method of analysis of structures, but also for another very popular method of analysis called the seismic coefficient method prescribed in different codes. Out of these three methods the first two methods requires rigorous analysis and is calculation intensive, thereby making the Seismic Coefficient Method a popular choice.

2.1 Seismic Coefficient Method

The seismic coefficient method obtains a set of equivalent lateral forces for an earthquake using some empirical formulae for a ground supported structure and analyses it to find the seismic forces induced in the members of the structure. In this method, the total weight of the structure is multiplied by a coefficient, known as the seismic coefficient, to obtain the total base shear of the structure that is distributed as a set of lateral forces along the height of the structure. This distribution of lateral force bears a resemblance (but not the same) with that for the fundamental mode of the structure in RSA. It is obtained by an empirical formula that varies from code to code. The method of analysis consists of the following steps.

Maximum base shear is obtained as:

$$V_b = W \text{ X } C_h \tag{1}$$

in which, W is the total weight of the building; Ch is a seismic coefficient that depends on the fundamental time period of the structure.

1. The lateral load along the height of the structure is distributed such that the sum of the lateral loads is equal to the base shear V_b. Thus,

$$F_i = V_b \text{ X } f(h_i) \tag{2}$$

where, F_i is the lateral load corresponding to the *i*th lateral displacement degree of freedom of the structure and h_i is the height of the point of application of the lateral load above the ground.

2. Static analysis of the structure for the lateral forces F_i (1 to n) is carried out to find the response quantities of interest.

Different codes of practice have different provisions for the value of the seismic coefficient C_h and the distribution of the lateral load along the height of the structure. Furthermore, the fundamental time period of the structure, especially for buildings, is computed using some empirical formulae, which are derived from prototype measurements/experimental work/approximate analysis. Variation of the seismic coefficient C_h with the time period, T, follows a shape close to the design response spectrum prescribed in the codes.

Although a perfect inter-relationship does not exist between the earthquake structural dynamics and the development of the seismic coefficient method, an approximate relationship can be shown to exist between the two in terms of the computation of the fundamental time period, T, distribution of the lateral forces along the height of the structure, and the computation of the base shear.

2.2 Distribution of Lateral Forces

According to the response spectrum method of analysis, the lateral force, F_j, for the *j*th floor, for first mode of vibration is given by:

$$F_j = \lambda_1 X W_j X \phi_{j1} X \frac{Sa1}{g} \tag{3}$$

From Equation (3), it is possible to write:

$$\frac{Fj}{\Sigma Fj} = \frac{\mathrm{Wj\ X\ \phi j1}}{\mathrm{\Sigma Wj\ X\ \phi j1}} \tag{4}$$

As $\Sigma F_j = V_b$, F_j may be written as:

$$F_j = V_b X \frac{\mathrm{Wj\ X\ \phi j1}}{\mathrm{\Sigma Wj\ X\ \phi j1}} \tag{5}$$

If the fundamental mode shape of the building is assumed to be linear, the above equation simplifies to:

$$F_j = V_b X \frac{\mathrm{Wj\ X\ hj}}{\mathrm{\Sigma Wj\ X\ hj}} \tag{6}$$

in which h_j is the height of the *j*th floor.

Although the fundamental mode shape is not linear, the above equation may be modified to take the non-linearity into account by writing:

$$F_j = V_b X \frac{\mathrm{Wj\ X\ } hj^{\mathrm{k}}}{\mathrm{\Sigma Wj\ X\ } hj^{\mathrm{k}}} \tag{7}$$

in which $k > 1$, for $k = 2$, the fundamental mode shape varies quadratically along the height. Some codes prescribe the variation of the lateral force as a combination of the above two equations. Thus, it is apparent that the seismic co-efficient method of analysis considers only the contribution of the fundamental mode of vibration of the structure in an approximate way.

2.3 Computation of the Fundamental Time Period

Most of the codes provide an empirical formula for finding the fundamental time period of the buildings based on experimental and practical observations. However, some of the codes, such as the International Building Code, USA, and the National Building Code, Canada, allow calculation of the fundamental time period of buildings using a formula which is almost the same as that used for calculating the approximate fundamental time period of a building frame using Rayleigh's method [3].*Dynamics of Structures, A K Chopra.*

2.4 Computation of the Base Shear

According to the response spectrum method of analysis, the base shear in the *i*th mode is given by:

$$V_{bi} = \Sigma F_{ji} = \lambda_i \text{ X } \Sigma W_j \text{ X } \phi_{ji} \text{ X } \frac{Sai}{g} \tag{8}$$

As $W^e{}_i = \lambda_i$ X ΣW_j X ϕ_{ji}, the *i*th effective weight of the structure (that is, effective weight of the building in the *i*th mode), V_{bi} may be written as:

$$V_{bi} = W^e{}_i \text{ X } \frac{Sai}{g} \tag{9}$$

Instead of using the SRSS combination rule, if the absolute sum of the response is used to find an upper bound to the response quantity of interest, then:

$$V_b \leq \Sigma\ |(V_b)| \leq \Sigma \frac{Sai}{g} \text{ X } W^e{}_i \ (i = 1 \text{ to } n) \tag{10}$$

If it is assumed that S_{ai}/g for all modes are the same, and is equal to S_{a1}/g, then an upper bound estimate of the base shear is given as:

$$V_b = W \text{ X } \frac{Sa1}{g} \tag{11}$$

The base shear computed by the seismic coefficient method uses a similar formula, with S_{a1}/g replaced by C_h. Thus, the seismic coefficient method is expected to provide a conservative estimate of the base shear.

2.5 Overview of Design Philosophy adopted in IS1893:2002

IS1893:2002 clearly states its design philosophy in clause 6.1.3. Clause 6.1.3 The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (<DBE), which occur frequently, without damage; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur and aims that structures withstand a major earthquake (MCE) without collapse.

In order to achieve that goal, the Code primarily suggests response spectrum analysis. A simplified method based on the Seismic Coefficient Method have been given, popularly referred as Static Analysis Method to be carried out for regular and moderately irregular, low rise buildings. The Code also suggests a rigorous Dynamic Analysis Method based on response spectrum to be carried out for irregular and high rise structures. However the Code does suggest a Time History analysis method for Dynamic analysis, but it has not been covered in detail in the Code.

2.5.1 Static Analysis

The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{ZISa}{2Rg}$$

Where,

Z = Zone factor given in Table 2, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE).

I = Importance factor, depending upon the functional use of the structures, characterised by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance (Table 6).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterised by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0(Table 7). The values of R for buildings are given in Table 7.

S_a/g = Average response acceleration coefficient for rock or soil sites as given by Fig. 2 and Table 3 based on appropriate natural periods and damping of the structure. These curves represent free field ground motion.

The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression:

$V_B = A_h W$

For various loading classes as specified in IS 875(Part 2), the earthquake force shall be calculated for the full dead load plus the percentage of imposed load as given in Table 8. For calculating the design seismic forces of the structure, the imposed load on roof need not be considered. The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey. The seismic weight of the whole building is the sum of the seismic weights of all the floors.

The approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick in-filled panels may be estimated by the empirical expression:

T_a = $0.075\ h^{0.75}$ for RC frame building

= $0.085\ h^{0.75}$ for steel frame building

The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$T_a \quad = 0.09h/\sqrt{d}$

The design base shear (V_B) computed in 7.5.3 shall be distributed along the height of the building as per the following expression:

$Q_i \quad = V_B W_i h_i^2/\sum W_j h_j^2$

2.5.2 Dynamic Analysis

Dynamic analysis shall be performed to obtain the design seismic forces and its distribution to different levels along the height of building and to the various lateral load resisting elements in following cases:

a. Regular Building – Greater than 40 m height in zone IV and V and those greater than 90 m in height in zone II and III.

b. Irregular building – All framed buildings higher than 12 m in zone IV and V, and those greater than 40 m height in zone II and III.

c. For irregular building lesser than 40 m in height in zone II and III, dynamic analysis even though not mandatory, is recommended.

Method of Dynamic Analysis:

Buildings with regular, or nominally irregular plan configuration may be modelled as a system of masses lumped at floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration.

Un-damped free vibration analysis of entire building modelled as spring – mass model shall be performed using appropriate masses and elastic stiffness of the structural system to obtain natural periods (T) and mode shapes $\{\phi\}$ of those of its modes of vibration that needs to be considered. The number of modes to be used should be such that the sum of total of modal masses of all modes considered is at least 90% of total seismic mass.

In dynamic analysis following expressions shall be used for the computation of various quantities:

(a) Modal mass (M_k) – Modal mass of the structure subjected to horizontal or vertical as the case may be, ground motion is a part of the total seismic mass of the structure that is effective in mode *k* of vibration. The modal mass for a given mode has a unique value, irrespective of scaling of the mode shape.

$$M_k = (\sum W_i \phi_{ik})^2 / (g \sum W_i \phi_{ik}^2) \qquad (12)$$

Where,

g = acceleration due to gravity,

ϕ_{ik} = mode shape coefficient at floor *i* in mode *k*

W_i = Seismic weight of floor *i*.

(b) Modal Participation factor (Pk) – Modal participation factor of mode k of vibration is the amount by which mode k contributes to the overall vibration of the structure under horizontal or vertical earthquake ground motions. Since the amplitudes of 95 percent mode shape can be scaled arbitrarily, the value of this factor depends on the scaling used for the mode shape.

$$P_k = (\sum W_i \phi_{ik}) / (\sum W_i \phi_{ik}^2) \qquad (13)$$

(c) Design lateral force at each floor in each mode – The peak lateral force (Q_{ik}) at floor i in mode k is given by:

$$Q_{ik} = A_k \phi_{ik} P_k W_i \qquad (14)$$

Where,

A_k = Design horizontal spectrum value using natural period of vibration (T_k) of mode k

(d) Storey shear forces in each mode – The peak shear force (Vik) acting in storey i in mode k is given by:

$V_{ik} = \sum Q_{ik}$ (15)

(e) Storey shear force due to all modes considered – The peak storey shear force (Vi) in storey i due to all modes considered is obtained by combining those due to each mode as per following rules:

i. CQC method: The peak response quantities shall be combined as per Complete Quadratic Combination (CQC) method

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

Where,

r = Number of modes being considered,
ρ_{ij} = Cross-modal coefficient
λ_i = Response quantity in mode i including sign
λ_j = Response quantity in mode j including sign

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

ζ = Modal damping ratio (in fraction) 2% and 5% for steel and reinforced concrete building respectively

β = Frequency ratio = ω_i/ω_j

ω_i = Circular frequency in i^{th} mode and

ω_j = Circular frequency in j^{th} mode

ii. SRSS method : If the building does not have closely spaced modes, than the peak response quantity (λ) due to all modes considered shall be obtained as per Square Root of Sum of Square method.

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

Where

λ_k = Absolute value of quantity in mode k and

r = Number of modes being considered

Closely spaced modes of a structure are those of its natural modes of vibration whose natural frequencies differ from each other by 10 percent or less of the lower frequency.

iii. SAV: If the building has a few closely spaced modes, then the peak response quantity

(λ^*) due to these modes shall be obtained as

$$\lambda^* = \sum_{c}^{r} (\lambda_k)$$

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

The analytical model for dynamic analysis with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Building with plan irregularities like torsion irregularities, re-entrant corners, diaphragm discontinuity, out-of plane offset, non-parallel systems as defined in IS 1893 cannot be modelled for dynamic analysis as discussed above.

The design base shear (V_B) shall be compared with a base shear ($\tilde{V}_B$) calculated using a fundamental period T_a, where T_a, is as per 7.6. Where V_B is less than $\tilde{V}_B$, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $\tilde{V}_B/V_B$.

2.6 Comparison of Clauses of IS1893:2002, IBC 2012 and Euro Code 8

A detailed study was done for comparing the different provisions given in the above mentioned codes and it has been summarized in Annexure 1. Based on that analysis, following observations have been found, also pointed out by Professor Sudhir K Jain, IIT Roorkee *Review of Indian seismic code, IS 1893 (Part 1) : 2002* [4]

Risk level

Para 5 on page 3 of the code states

"The seismic hazard level with respect to ZPA at 50 percent risk level and 100 year service life goes on progressively increasing …"

This statement is made in the context of earthquake geology of the country. However, it may give a false impression that the values of ZPA (denoted by Z) given in the code are for 50 percent risk level and 100 year service life. Such a confusion needs to be avoided by modifying this statement as "The seismic hazard level goes on progressively increasing…."

2.6.1 Peak ground acceleration

Item (b) on page 2 of the code uses the term "Effective Peak Ground Acceleration" (EPGA). This term is also defined in Clause 3.11. For the purposes of the code it is not important to differentiate between EPGA and “Peak Ground Acceleration” (PGA). Similarly, the code also uses the term ”Zero Period Acceleration” (ZPA) at several places. Since the stiff structures (having natural period of zero) experience same acceleration as the ground acceleration, the ZPA value is same as PGA. To avoid confusion, it is best to just use the term “Peak Ground Acceleration” (PGA), and the terms ZPA and EPGA should be dropped from the code.

2.6.2 Service life of structure (Item (b) on page 2, Clause 3.33, and Clause 6.4.2)

Item (b) on page 2 states that the values of seismic zone factor reflect more realistic values of EPGA considering “Maximum Considered Earthquake” (MCE) and service life of structure in each seismic zone. A similar mention of the service life is made while defining Z in Clause 6.4.2. This confuses the user since he then asks questions such as:

1. What value of service life should be considered for his structure?
2. If he is willing to reduce the service life of his structure say from 100 years to 50 years, how much reduction in the seismic design force would be allowed by the code?

The fact remains that the values of Z specified in the code were arrived at empirically based on engineering judgment and no explicit calculations were done or envisaged for service life. Hence, it is best to drop the mention of .service life. This suggestion is consistent with the fact that in the definition of Z in Clause 3.33 also, the code makes no mention of service life.

2.6.3 Response spectrum (Clauses 3.5, 3.27, 3.30, 6.4)

In the code, different terms are used for response spectrum, for example, "Design Acceleration Spectrum" (Clause 3.5); “Response Spectrum" (Clause 3.27); "Acceleration Response Spectrum" (used in Clause 3.30); "Design Spectrum" (title of Clause 6.4); "Structural Response Factor"; "Average response acceleration coefficient" (see terminology of S_a/g on p. 11), etc. It is best to use one single term consistently to avoid confusion. It is suggested that the term be “Design

Acceleration Spectrum" for the plot of response spectrum with natural period, and the term be Response Acceleration Coefficient for the value of S_a/g for a given value of natural period.

2.6.4 Maximum considered earthquake (MCE) and design basis earthquake (DBE)

This edition of the code introduces two new terms:

"Maximum Considered Earthquake" (MCE): Defined in Clause 3.19 as "The most severe earthquake effects considered by this standard", and

"Design Basis Earthquake" (DBE): Defined in Clause 3.6 as "It is the earthquake which can reasonably be expected to occur at least once during the design life of the structure".

Both these definitions are quite incomplete and do not tell anything specific to the user. For instance, what is meant by "reasonable expectation"! Also, the design life of different structures may be different and yet the code specifies the same PGA value regardless of the design life of a structure.

Let us consider the use of these terms in the International Building Code (IBC). The IBC 2003 defines MCE as corresponding to 2 percent probability of being exceeded in 50 years (2,500 year return period), and the DBE as corresponding to 10 percent probability of being exceeded in 50 years (475 year return period). Clearly, there is no ambiguity in IBC on this account.

Since the seismic zone map in Indian code is not based on probabilistic hazard analysis, it is not possible to deduce the probability of occurrence of a certain level of shaking in a given zone based on this code. Therefore, use of terms such as MCE and DBE do not add any new information, and can sometimes cause confusion and disputes. For instance, someone may argue that the value of Z=0.36 for MCE in zone V of the code implies that the PGA value in zone V cannot exceed 0.36g, which is not the intention of the code. For instance, during 2001 Bhuj earthquake, ground acceleration ~0.6g has been recorded at Anjar located at 44 km from epicentre.

Clause 6.1.3 implies that DBE relates to the “moderate shaking” and MCE relates to the “strong shaking”. This is at variance with the definitions of MCE and DBE given in Clauses 3.19 and 3.6 as mentioned above. Again, it clearly shows that there is an element of confusion about the definition and implications of these two terms. Considering that these terms do not add any substantial value to the provisions, the two terms may be dropped from the code.

2.6.5 Centre of stiffness and centre of rigidity

In Clause 4.5, centre of stiffness is defined, but in Clause 4.21 while defining static eccentricity, the term centre of rigidity is used. Both centre of stiffness (CS) and centre of rigidity (CR) are the same terms for purposes of the code and hence to avoid confusion, it is best to use only one term consistently. It is proposed that centre of stiffness be replaced by the term centre of rigidity wherever it appears in the code.

Clause 4.5 defines centre of stiffness as “The point through which the resultant of the restoring forces of a system acts”. This definition is incomplete. For single storey buildings it may be defined as:

“If the building undergoes pure translation in the horizontal direction (that is, no rotation or twist or torsion about vertical axis), the point through which the resultant of the restoring forces acts is the centre of stiffness”.

For multi-storeyed buildings, centre of rigidity (stiffness) can be defined in two ways.

All floor definition of centre of rigidity: Centre of rigidities are the set of points located one on each floor, through which application of lateral load profile would cause no rotation in any floor, Fig 1(a). As per this definition, location of CR is dependent on building stiffness properties as well as on the applied lateral load profile.

Single floor definition of centre of rigidity: Centre of rigidity of a floor is defined as the point on the floor such that application of lateral load passing through that point does not cause any rotation of that particular floor, while the other floors may rotate Fig 1(b). This definition is independent of applied lateral load.

The two definitions for multi-storey buildings will give somewhat different values of design eccentricity but the difference is not very substantial. Hence, choice of the definition should be left to the designer and the above definitions should be added in the code.

2.6.6 Soft storey buildings

Clause 4.20 defines soft storey, while Table 5 of the code defines soft storey and extreme soft storey. Soft storey is defined as one with lateral stiffness less than 70 percent of that in the storey above, or less than 80 percent of the average lateral stiffness of the three storeys above. Extreme soft storey is defined when these numbers are 60 percent (in place of 70 percent) and 70 percent (in place of 80 percent), respectively.

This is in line with the US codes which separately define soft storey buildings and extreme soft storey buildings. However, in the US codes, extreme soft storey buildings require more stringent treatment in analysis and design as compared to soft storey buildings. In IS 1893, there is no difference between the treatment for soft and extreme soft storey buildings. Moreover, there is not much of a difference between soft storey and extreme soft storey buildings as defined in the code. Hence, it is suggested that the term "extreme soft storey" be dropped from Table 5.

2.6.7 Load combination 0.9DL ±1.5EL

Seismic loads are reversible in direction; in many cases, design is governed by effect of horizontal load minus the effect of gravity loads. In such situations, a load factor higher than 1.0 on gravity loads will be un-conservative, and hence, in Clause 6.3.1.2, a load factor of 0.9 is specified on gravity loads in the combination 4) for RC buildings. A similar load case (0.9DL ±1.7EL) should be added in Clause 6.3.1.1 for steel structures.

2.6.8 Seismic intensity (Table 2)

The seismic zone map in Indian code has been originally developed based on anticipated intensity of shaking. This is clearly outlined in the last para of page 3 of the code as: Zones II to V are associated with seismic intensity of VI (or less), VII, VIII, and IX (and above), respectively. However, Table 2 of the code gives "Seismic Intensity" as Low, Moderate, Severe and Very Severe for zones II to V, which is vague and contradicts a more specific mention of intensity on page 3. Hence, the row for seismic intensity in Table 2 should be removed.

2.6.9 Response reduction factor

Definition of R on page 14 contains the statement, "However, the ratio (I/R) shall not be greater than 1.0 (Table 7)". It is recommended to drop this statement. For buildings, I does not exceed 1.5 and the lowest value of R is 1.5 in Table 7 and therefore this statement does not become effective for buildings. For other structures, there could be situations where (I/R) will need to exceed 1.0, for instance, for bearings of important bridges.

2.6.10 Computation of the Fundamental Time Period

Most of the codes provide an empirical formula for finding the fundamental time period of the buildings based on experimental and practical observations. However, some of the codes, such as the International Building Code, USA, and the National Building Code, Canada, allow calculation of the fundamental time period of buildings using a formula which is almost the same as that used for calculating the approximate fundamental time period of a building frame using Rayleigh's method [2].

Comparison of Empirical Formula for Fundamental Time Period Prescribed by Different Earthquake Codes

Almost all countries have their own codes for seismic analysis and design of structures. It is difficult to compare all of them. In this section, a comparison between the code provisions for the calculation of the fundamental time period of buildings given by the following codes is made in order to demonstrate the type of variations that exist between the codes.

1. International Building Code, IBC – 2012 [5]

2. National Building Code of Canada, NBCC – 1995 [6]

3. Euro Code 8 – 2004 [7]

4. New Zealand Code, NZS 4203 – 1992 [8]

5. Indian Code, IS 1893 – 2002 [1]

However, it may be noted that, most of the Codes suggest the use of Rayleigh's method to calculate the fundamental time period.

$$T_1 = 2\pi \text{ X } \frac{\sum_{i=1}^{n} \mathrm{Wi\ X}\ ui^2}{g\sum_{i=1}^{n} \mathrm{Fi\ X\ ui}} \qquad (16)$$

Where,

W_i is the weight of the *i*th floor

u_i are the displacements due to static application of a set of lateral forces F_i at floor levels

N is the number of storey.

Euro Code 8 – 2004: Clause 4.3.3.2.2

For the determination of the fundamental period of vibration period T_1 of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used.

For buildings with heights of up to 40 m the value of T_1 (in s) may be approximated by the following expression:

$$T_1 = C_t H^{0.75} \qquad (17)$$

Where C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

Alternatively, for structures with concrete or masonry shear walls the value C_t in expression (13) may be taken as being:

$C_t = 0.075/\sqrt{Ac}$ (18)

Where,

$A_c = \Sigma[A_i (0.2 + (l_{wi}/H))^2]$ (19)

Where,

A_c is the total effective area of the shear walls in the first storey of the building, in m^2;

A_i is the effective cross-sectional area of the shear wall i in the first storey of the building, in m^2;

l_{wi} is the length of the shear wall i in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/H should not exceed 0.9.

Alternatively, the estimation of T_1 (in s) may be made by using the following expression:

$T_1 = 2\sqrt{d}$ (20)

Where, d is the lateral elastic displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction.

UBC 1997: Clause 30.4-6

Fundamental natural period:

$T = C_t*h^{0.75}$ (21)

Where C_t = 0.0853 for steel moment resisting frames

Where C_t = 0.0731 for reinforced concrete moment resisting frames and eccentrically braced frames

Where C_t = 0.0488 for all other buildings.

NZS 1170 -5: 2004 Clause 4.1.2.2

Fundamental natural period:

$T_1 = 1.25\ K_t\ h^{0.75}$ (22)

Where K_t = 0.075 - for moment resisting concrete frame

Where K_t = 0.050- for all other frames.

IS1893 – 2002: Clause 7.6

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

$T_a = 0.075\ h^{0.75}$ for RC frame building (23)

$= 0.085\ h^{0.75}$ for steel frame building

Where,

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

7.6.2 The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$T_a = \frac{0.09h}{\sqrt{d}}$ (24)

Where,

h= Height of building, in m as defined in 7.6.l; and

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

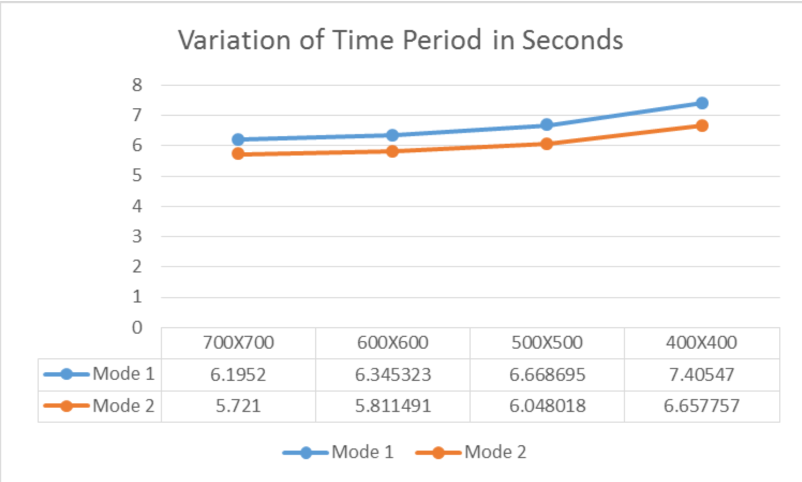
CHAPTER 3

EXPERIMENTAL INVESTIGATION FOR EFFECTS OF CHANGE IN GEOMETRIC AND MATERIAL PROPERTIES FOR BUILDINGS

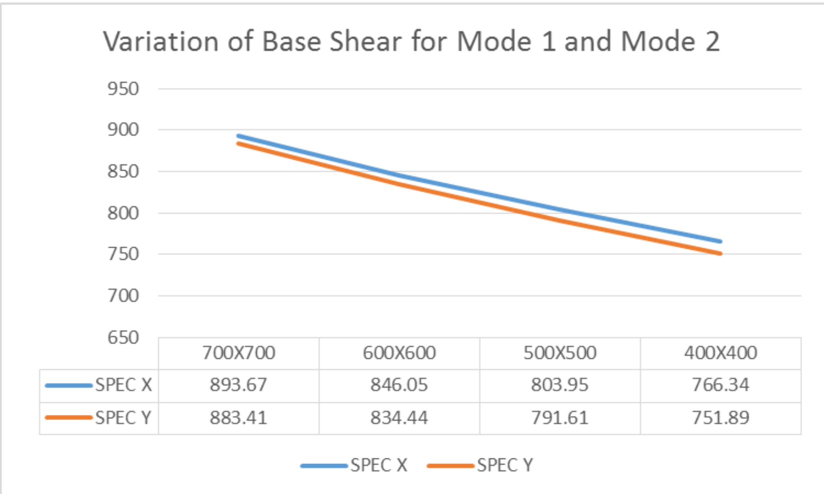
3.1 Variation of Time Period With Respect to Change in Column Size:

Table 3.1 Effect of Column Sizes

	Time Period		Base Shear	
Column Size	Mode 1	Mode 2	SPEC X	SPEC Y
700X700	6.1952	5.721	893.67	883.41
600X600	6.345323	5.811491	846.05	834.44
500X500	6.668695	6.048018	803.95	791.61
400X400	7.40547	6.657757	766.34	751.89



Graph 3.1 Effects of Column Sizes on Time Period



Graph 3.2 Effects of Column Sizes on Base Shear

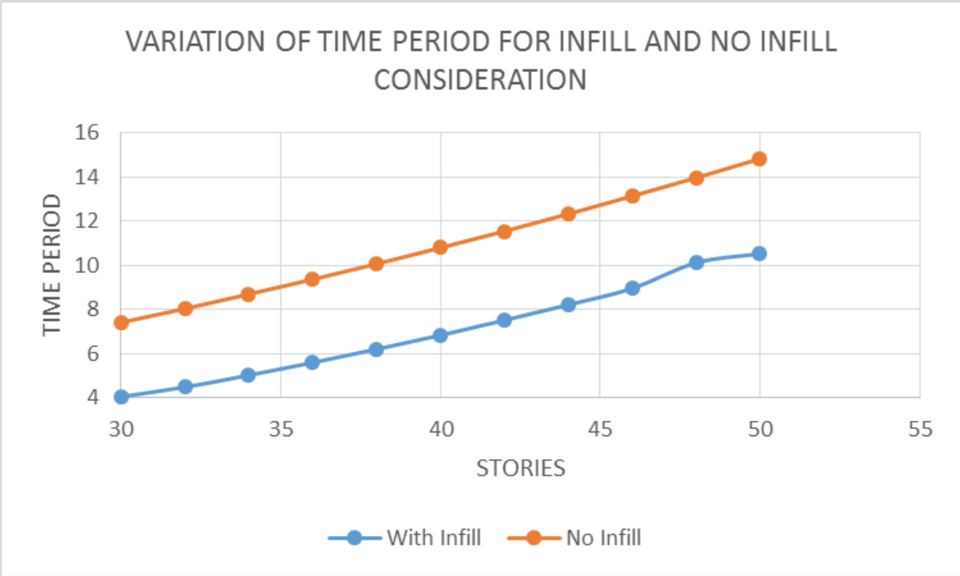
Here we see the structure same Plan area and height shows a change in time period. With the decrease in the column size, the stiffness of the structure decreases; hence the building becomes more flexible and the time period increases. And thus the base shear decreases.

3.2 Variation in time period when modelled with infill and without infill

Table 3.2 Variation of Time Period for Infill and No Infill Consideration

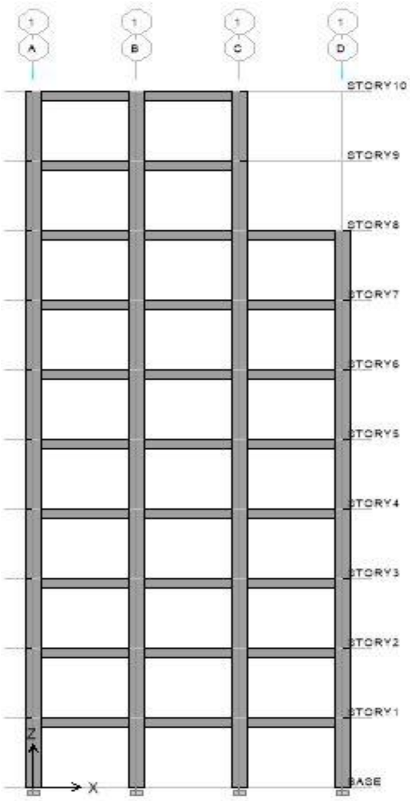
Type Z	With Infill				No Infill			
Storey	Time Period		Base Shear		Time Period		Base Shear	
	Mode 1	Mode 2	SPEC1	SPEC2	Mode 1	Mode 2	SPEC1	SPEC2
30	4.05	2.81	1540.01	1302.97	7.41	6.66	766.34	751.89
32	4.48	3.11	1538.02	1306.26	8.03	7.17	809.93	792.25
34	5.02	3.46	1534.59	1318.14	8.69	7.69	852.55	833.05
36	5.59	3.83	1521.58	1335.41	9.36	8.23	896.04	812.93
38	6.20	4.23	1519.35	1353.23	10.06	8.77	938.83	912.56
40	6.83	4.64	1546.34	1371.40	10.79	9.34	981.01	951.75
42	7.51	5.08	1565.08	1384.53	11.54	9.91	1023.38	990.20
44	8.21	5.54	1576.92	1398.97	12.32	10.50	1065.12	1028.77
46	8.95	6.02	1596.75	1409.15	13.13	11.10	1106.53	1067.39
48	10.11	6.78	1613.73	1420.55	13.96	11.72	1147.88	1106.95
50	10.52	7.05	1621.93	1429.71	14.82	12.35	1188.72	1146.99

The structure is modelled with and without considering infill in two different set of models and the results are tabulated. We observe here that the structure gets additional stiffness when infill is modelled hence the time period is much lower when modelled with infill than that with no infill. The base shear is higher for the model in which infill is considered, but this difference of base shear goes on decreasing with the change in height.

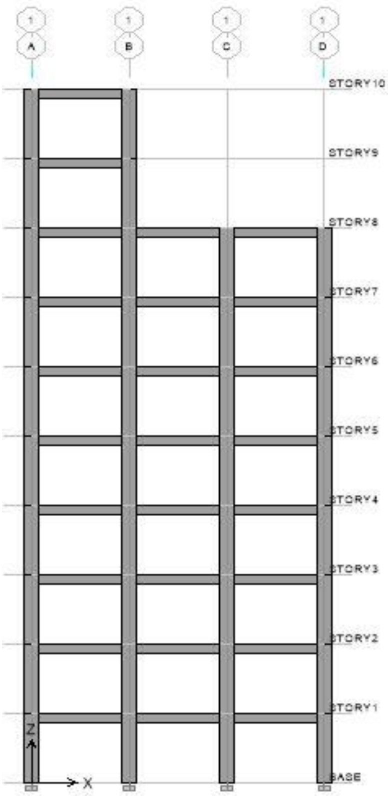


Graph 3.3 Variation of Time Period for Infill and No Infill Consideration

3.3 Geometric Irregularities:

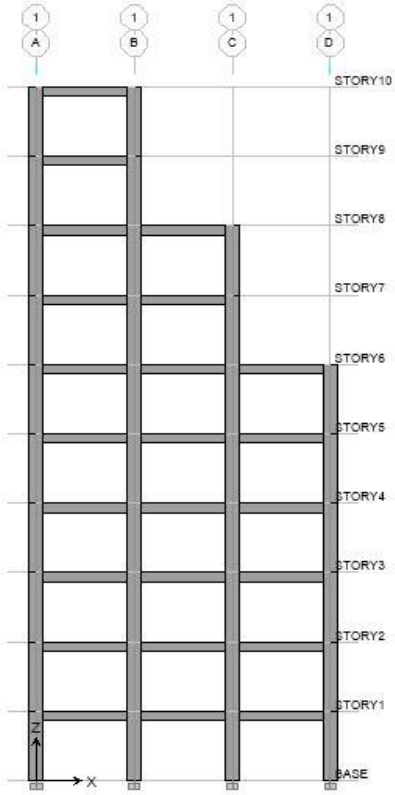


Irregularities 1

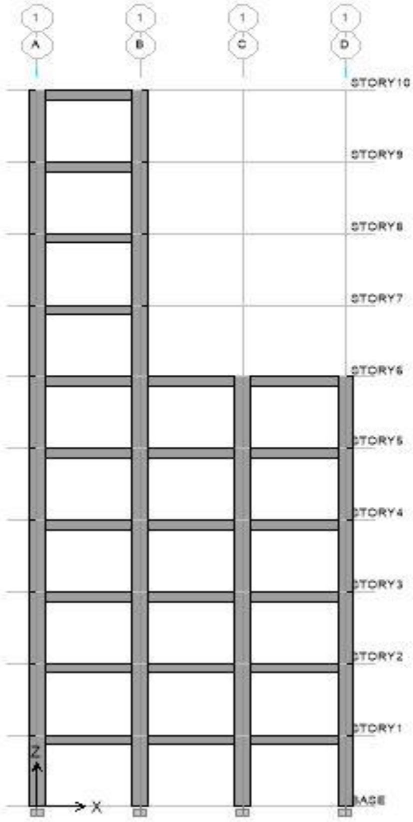


Irregularities 2

Fig 3.1(a) Different Types of Irregularities Considered.



Irregularities 3



Irregularities 4

Fig 3.1(b) Different Types of Irregularities Considered.

Table 3.3 showing the Change in Time Period with respect to geometric irregularities:

		Time Period		
Type	**Height**	**Modal Analysis**	**IS1893**	**UBC 97**
No Irregularities	30	1.782038	0.961396	0.93704
1	30	1.691982	0.961396	0.93704
2	30	1.589424	0.961396	0.93704
3	30	1.551101	0.961396	0.93704
4	30	1.490155	0.961396	0.93704

Symmetrical buildings with uniform mass and stiffness distribution behave in a fairly predictable manner, whereas buildings that are asymmetrical or with areas of discontinuity or irregularity do not. For such buildings, dynamic analysis is used to determine significant response characteristics such as

(1) The effect of the structure's dynamic characteristics on the vertical distribution of lateral forces;
(2) The increase in dynamic loads due to Torsional motions; and
(3) The influence of higher modes, resulting in an increase in story shears and deformations.

Static method specified in building codes are based on single-mode response with simple corrections for including higher mode effects. While appropriate for simple regular structures, the simplified procedures do not take into account the full range of seismic behaviour of complex structures. Therefore, dynamic analysis is the preferred method for the design of buildings with unusual or irregular geometry.

The fundamental periods for all the selected setback buildings as obtained from different methods available as tabulated above show that the buildings with same height and width may have different period depending on the amount of irregularity present in the setback buildings. Many empirical formulae suggested by codes do not take into account the irregularities present in buildings and thus they do not change for different type of irregularities. Consequently, here we observe that the ideology of static analysis is validated.

3.4 Effect of SSI Consideration on Time Period of Structure.

In the present study, two RCC framed structures are considered

a. Structure A - Fixed support at the base (not considering SSI)
b. Structure B - Flexible Support at the base (considering SSI)

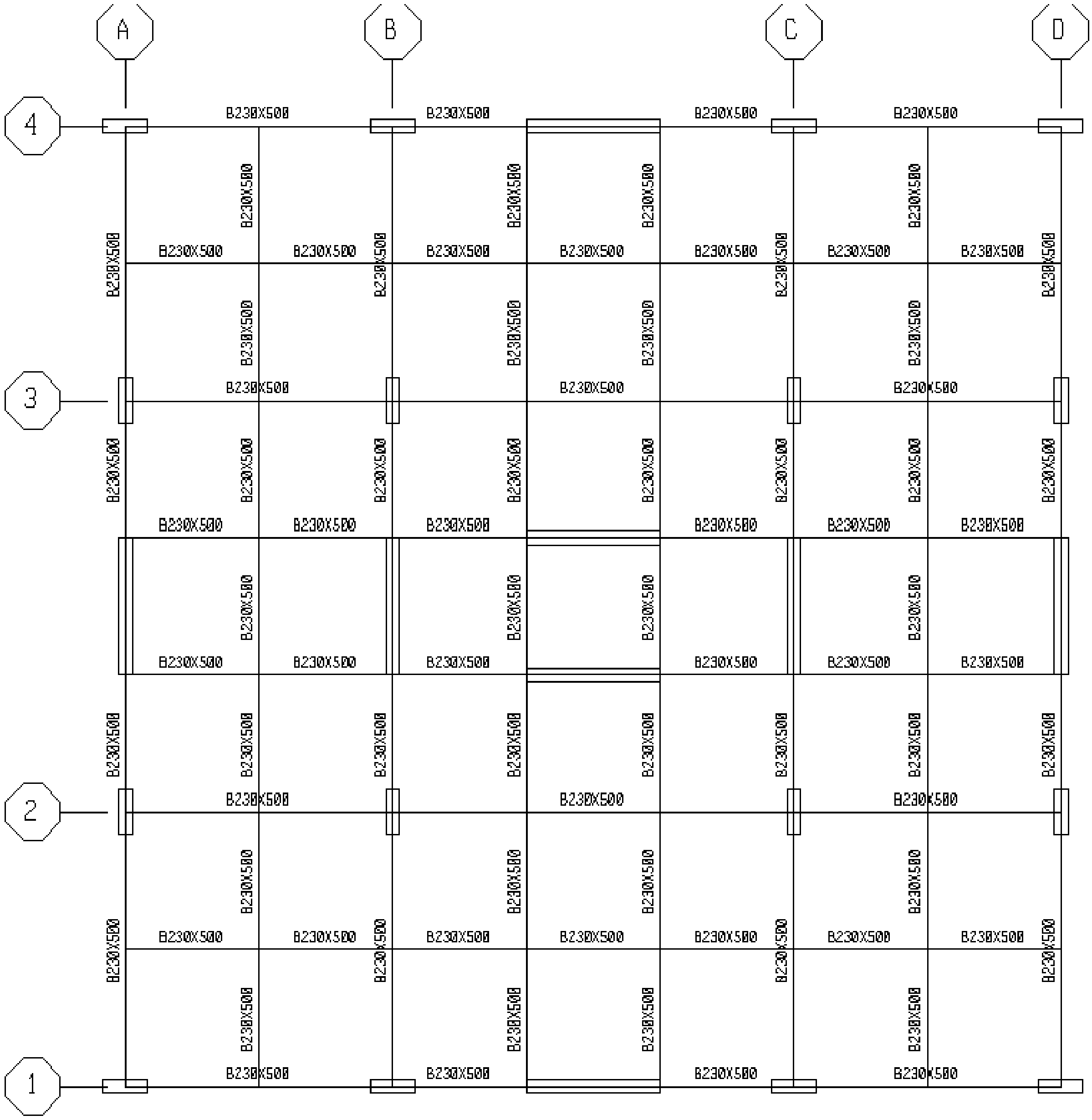


Fig 3.2 Typical Framing Plan of the Considered Structure

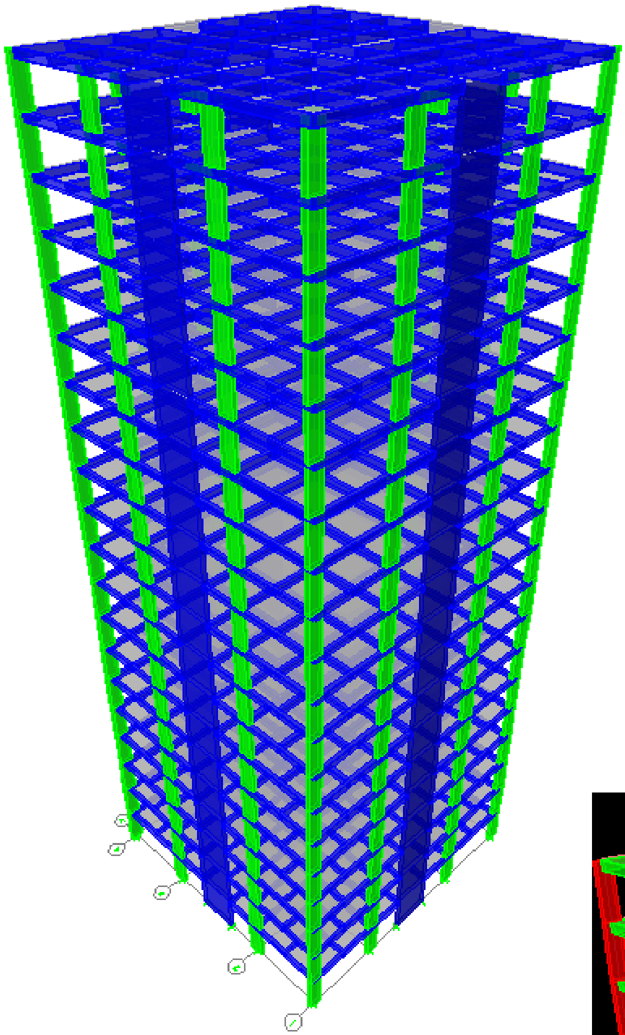


Fig 3.3 A 3D View of the "**Structure A**"

- The columns are fixed at the base

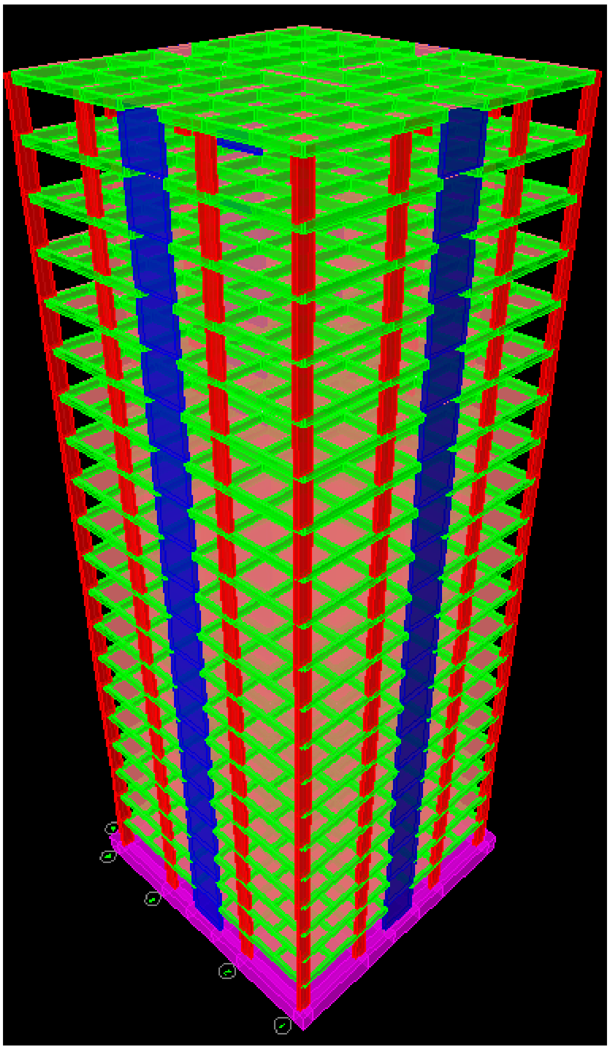


Fig 3.4 A 3D View of the "**Structure B**"

- Raft is modelled to support the structure

3.4.1 Forces acting on the Structure

The following loads have been considered for the analysis of the above structure.

a. Dead Load of the structure.
b. Live Load.
c. Earthquake forces (Response Spectrum)

3.4.2 Analysis of the Structures

Analysis is carried out by considering soil structure interaction / not considering soil structure interaction using ETABS.

Analysis of the above walls is carried out using following two methods.

3.4.3 Rigid Foundation

This is based on the assumption of liner distribution of contact pressure. The basic assumptions of this method are:

a. The foundation is rigid relative to the supporting soil and the compressive soil layer is relatively shallow.

b. The contact pressure variation is assumed as planar, such that the centroid of the contact pressure coincides with the line of action of the resultant force of all loads acting on the foundation.

3.4.4 Flexible foundation (Winkler's model)

In this method, it is assumed that the subgrade consists of an infinite array of individual elastic springs each is not affected by others. The spring constant is equal to the modulus of subgrade reaction (k). The contact pressure at any point under the raft is, therefore, linearly proportional to the settlement at the point.

3.4.5 Net SBC and Modulus of sub grade reactions

For the Net SBC and Modulus of sub grade reactions here reference has been adopted from Soil Investigation Report for a site DLF at Sector 86, New Gurgaon.

Table 3.4 Considered Safe Bearing Capacity and Modulus of Subgrade Reaction.

Foundation depth below NGL	Net SBC (T/Sqm) for 75mm settlement	Gross SBC (T/Sqm) for 75mm settlement	Modulus of sub grade reaction (Kg/Cu.Cm)
3.5m	19.8	25.4	1.5
4.5m	21.2	28.4	1.7
7.0m	28.0	39.2	2.3
8.0m	30.0	42.8	2.5
11.0m	35.8	53.4	3.0
12.0m	37.2	56.4 linearly	3.2

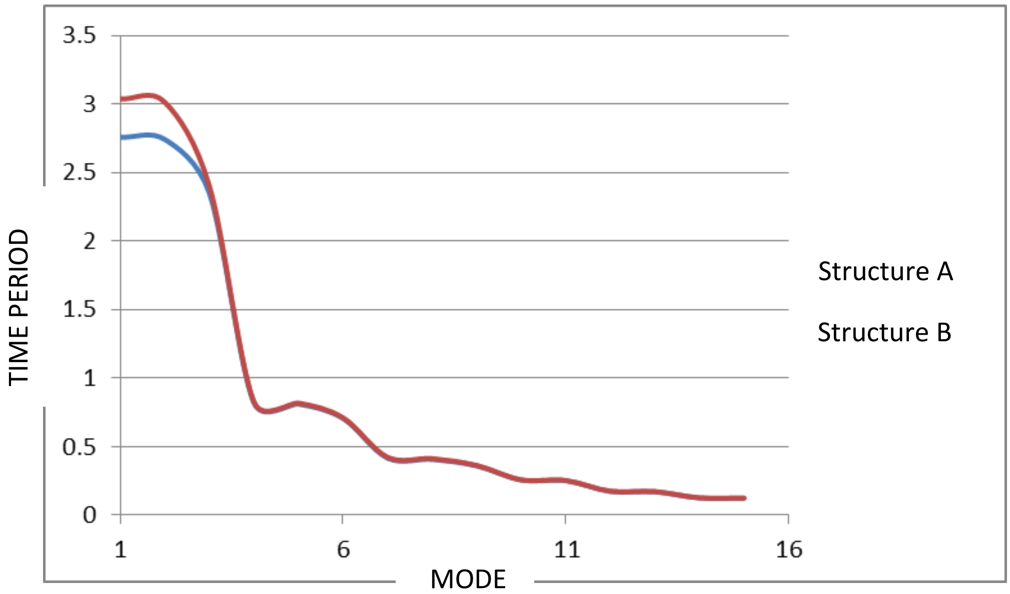
3.4.6 Natural Period of Vibration.

Natural Period of Vibration (for the first 15 modes) of the example structures were compared in the table below and the graph illustrates that the natural period of the Structure A is much less than that of the Structure B. And the difference in the natural period for lower modes was more significant compared to that of the higher modes. It can be explained that the rotation of column bases occurring in the first and second mode shape is larger than that of the higher modes.

Table 3.5 Natural Period of Vibration (for the first 15 modes) of the example structures.

Mode	Period	
	Structure A	Structure B
1	2.757667	3.03717
2	2.741171	3.012416
3	2.348174	2.399821
4	0.819207	0.824706
5	0.81089	0.81759
6	0.705391	0.71038
7	0.415837	0.422054

8	0.406543	0.412579
9	0.359882	0.362953
10	0.256115	0.257969
11	0.251073	0.252972
12	0.173075	0.174996
13	0.170261	0.172115
14	0.125319	0.12627
15	0.123667	0.124609



Graph 3.4 Natural Period of Vibration (for the first 15 modes) of the example structures.

3.4.7 Response Spectrum Analysis Results.

The difference in the natural periods will result in the difference in the seismic response. The accelerations of the structure with a raft tend to be smaller, because the natural periods of the structure B is longer than that of the structure A. Therefore if the raft is introduced in the analytical model, the seismic loads in the response spectrum analysis become relatively smaller. The base-shear force of the example structures from response spectrum analysis are listed below.

Table 3.6 Base Shear For Structure A (fixed base).

Structure A (Fixed Base)					
BASE SHEAR					
Story	Load	Loc	P	VX	VY
STORY1	SPECX	Top	0	**1364.05**	3.36
STORY1	SPECX	Bottom	0	**1364.05**	3.36
STORY1	SPECY	Top	0	3.35	**1373.3**
STORY1	SPECY	Bottom	0	3.35	**1373.3**

Table 3.7 Base Shear For Structure B (with Raft).

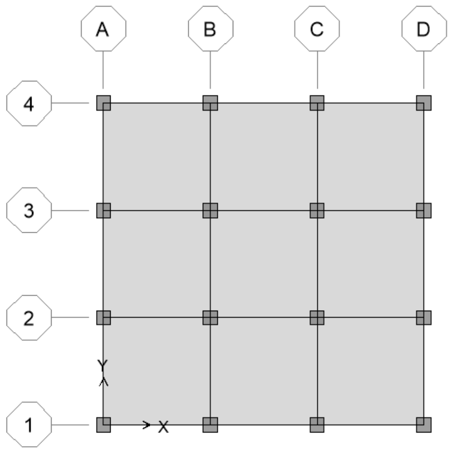
Structure B (With Raft)					
BASE SHEAR					
Story	Load	Loc	P	VX	VY
STORY1	SPECX	Top	0	**1277.1**	3.56
STORY1	SPECX	Bottom	0	**1277.1**	3.56
STORY1	SPECY	Top	0	3.58	**1281.32**
STORY1	SPECY	Bottom	0	3.58	**1281.32**

From the Above tables we can see that Base shear in Structure A is relatively more than the Base shear of structure B.

CHAPTER 4

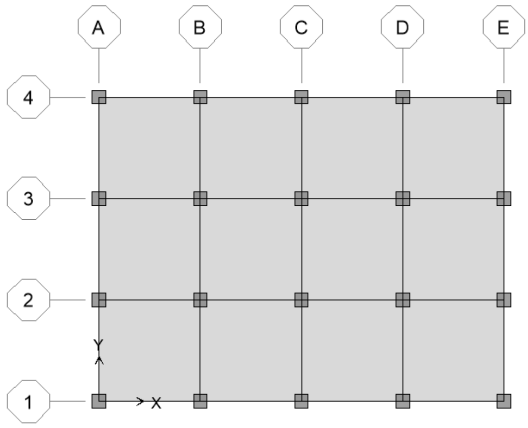
EXPERIMENTAL INVESTIGATION FOR LOW RISE BUILDINGS

In the present study, we considered 108 different models with different storey height, different spans and different number of storeys.



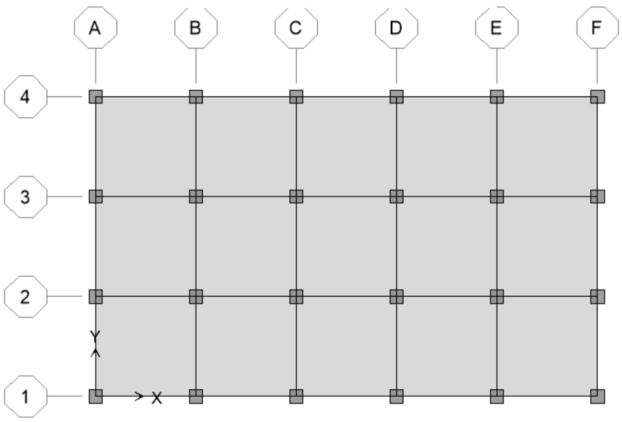
Type A

3/4/6 Storey
3m/4m/5m Span
3m/4m/5m Height



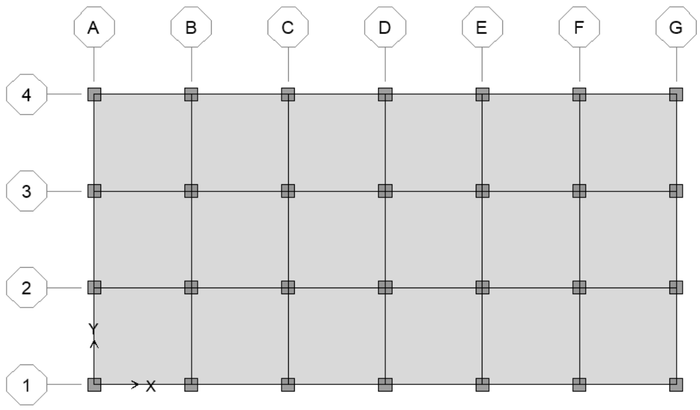
Type B

3/4/6 Storey
3m/4m/5m Span
3m/4m/5m Height



Type C

3/4/6 Storey
3m/4m/5m Span
3m/4m/5m Height



Type D

3/4/6 Storey
3m/4m/5m Span
3m/4m/5m Height

Fig 4.1 Different Plan Dimension of the structures modelled.

4.1 MATERIAL PROPERTIES

The material properties used in creating the model were as follows:

1. Grade of Concrete – M25,
2. Grade of Reinforcement – Fe500,
3. Poisson Ratio of Concrete – 0.2
4. Poisson Ratio of Reinforcement – 0.3
5. Density of Concrete – 25KN/m^3
6. Density of Reinforcement – 78.5KN/m^3
7. Young's Modulus of concrete – 25000000KN/m^2
8. Young's Modulus of reinforcement – $2.1X10^{10}$ KN/m^2
9. Damping Factor – 0.05 (As per Clause 7.8.2.1 of IS1893(Part 1):2002

4.2 GEOMETRIC PROPERTIES

The geometrical properties measured and used to create model were as follows:

1. The slab thickness – 100 mm
2. Beam cross sections on all floors – 0.23mX0.40m
3. Column cross section on all floors – 0.4mX0.4m
4. Storey Height – 3.0m / 4.0m / 5.0m
5. Spans – 3.0m / 4.0m / 5.0m

4.3 LOADING

1. Dead load due to self-weight of the structure.
2. Live load has been taken as 2KN/m^2.

4.4 RESULTS

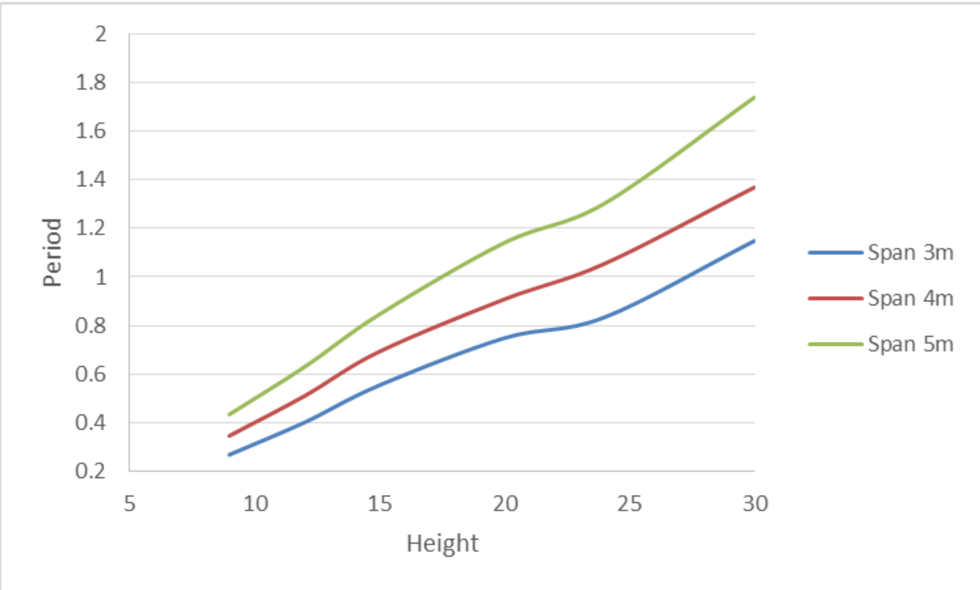
After study of the different models we tried to compile some variations or trends in the period of the structure with the alteration of the different geometric configuration.

4.4.1 Variation of the Period with respect to Height, Span and Plan Ratio.

The table 4.1 and Graph 4.1 shows that if the column beam configuration remains same and the span of the beam increased the natural period of the building increases. This is due to the reduction on the overall stiffness of the floor.

Table 4.1 Variation of period for different spans with respect to height

Height	Span 3m	Span 4m	Span 5m
9	0.269151	0.346875	0.43494
12	0.401661	0.510301	0.629719
15	0.554464	0.692894	0.845887
20	0.748753	0.908136	1.140691
24	0.833855	1.054185	1.301892
30	1.148323	1.368196	1.738035



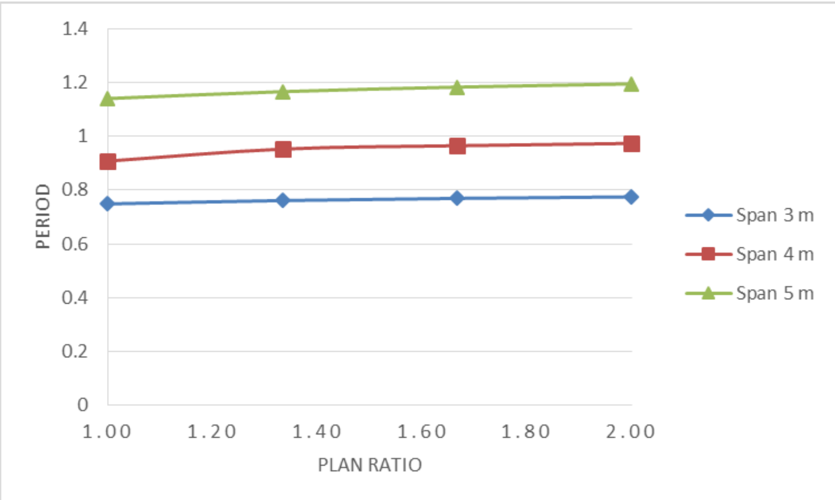
Graph 4.1 Variation of period for different spans with respect to height

The table 4.2 and Graph 4.2 shows that there is marginal variation in time periods if there is a difference in plan ratio.

From this we can infer that while establishing a generalized formula for the fundamental time period we can ignore this aspect of the structure and rather limit our self to the plan area and height of the structure.

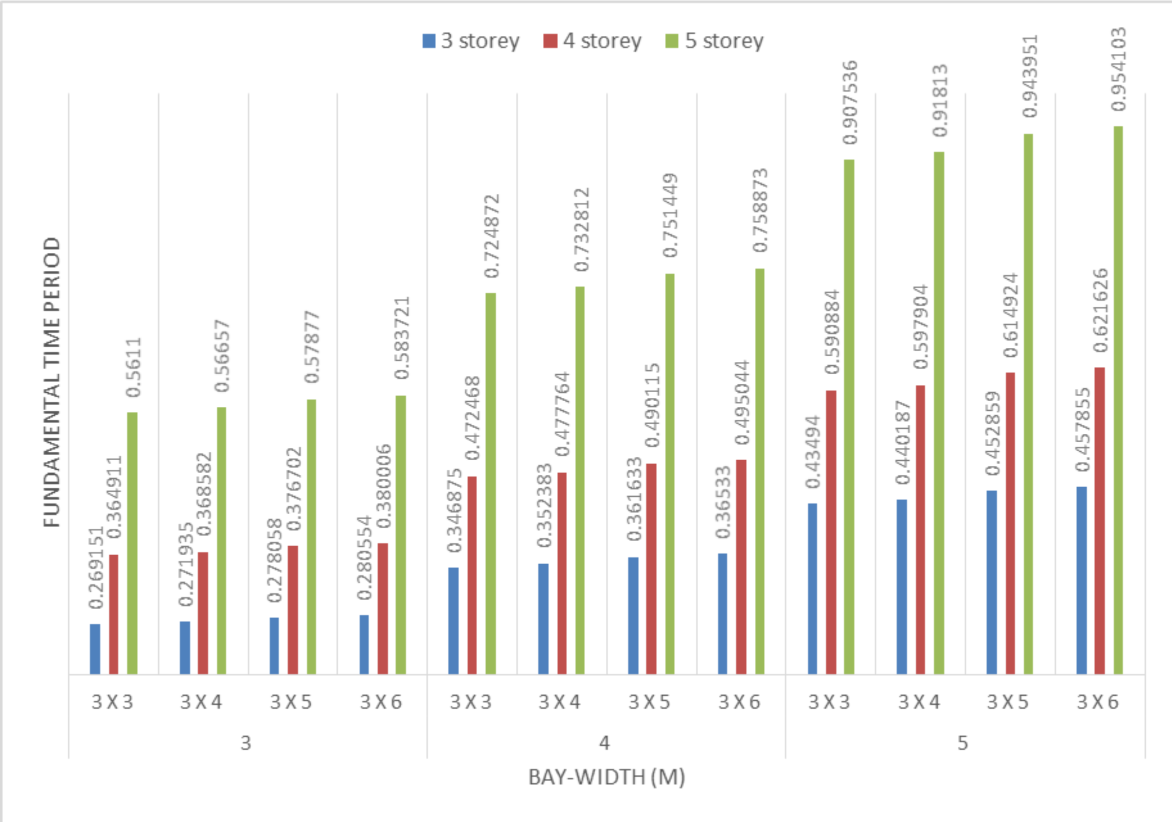
Table 4.2 Variation of period with respect to Plan Ratio

PLAN RATIO	Different Considered spans		
	Span 3 m	Span 4 m	Span 5 m
1.00	0.748753	0.908136	1.140691
1.33	0.761073	0.952781	1.166401
1.67	0.769178	0.965052	1.183233
2.00	0.774916	0.973724	1.195112



Graph 4.2 Variation of period with respect to Plan Ratio

The variation of fundamental time period is shown in a bar chart Graph 4.3. It shows that with increase in the bay width for the same number of stories there is an increase in the value of fundamental period. At the same time it shows that the increase in the plan ratio has only a marginal effect on the fundamental period.



Graph 4.3 Variation of Fundamental period with respect to Bay width, storey and plan ratio

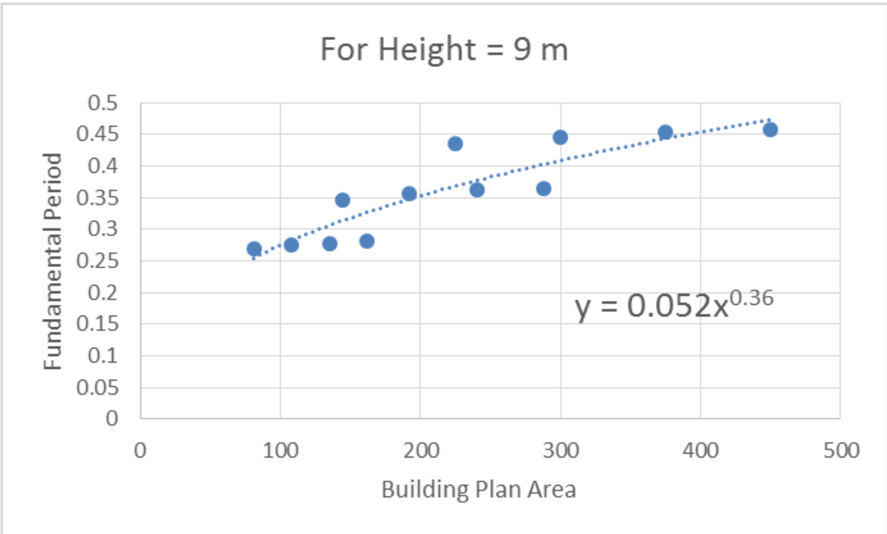
4.4.2 Best fit curve

The regular RCC moment resisting frames without infill are studied. Based on height and plan area, for 108 frame models regression analysis is carried out for finding their fundamental period.

Table 4.3 Fundamental period by Numerical Study

With respect to Plan Area at 9m height

Height	BD	Period
9	81	0.269151
9	108	0.274529
9	135	0.278058
9	144	0.346875
9	162	0.280554
9	192	0.356396
9	225	0.43494
9	240	0.361633
9	288	0.36533
9	300	0.445775
9	375	0.452859
9	450	0.457855



Graph 4.4 Fundamental period with respect to Plan Area at 9m height

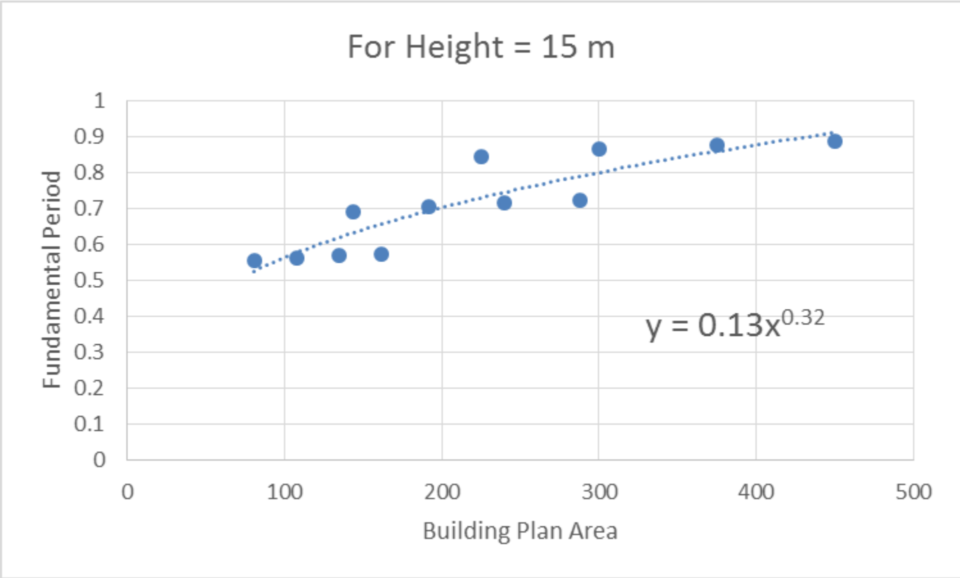
For a building of 9 m height, from the curve fit the fundamental natural period we can derive a power relationship as follows.

$$T = 0.052\,(BD)^{0.36} \qquad (25)$$

Table 4.4 Fundamental period with respect to Plan Area at 9m height

Height	BD	Numerical Study	**IS 1893 (Eq 19)**	% Variation	**Proposed (Eq 21)**	% Variation
9	81	0.269151	**0.389711**	45%	**0.252964**	-6%
9	108	0.274529	**0.389711**	42%	**0.282381**	3%
9	135	0.278058	**0.389711**	40%	**0.306152**	10%
9	144	0.346875	**0.389711**	12%	**0.313393**	-10%
9	162	0.280554	**0.389711**	39%	**0.327051**	17%
9	192	0.356396	**0.389711**	9%	**0.34781**	-2%
9	225	0.43494	**0.389711**	-10%	**0.368375**	-15%
9	240	0.361633	**0.389711**	8%	**0.377088**	4%
9	288	0.36533	**0.389711**	7%	**0.40283**	10%
9	300	0.445775	**0.389711**	-13%	**0.40883**	-8%
9	375	0.452859	**0.389711**	-14%	**0.443245**	-2%
9	450	0.457855	**0.389711**	-15%	**0.473504**	3%

This shows that the present study estimation of periods gives an error of about 0-17% (eq 25). Whereas the error is 0 to 45% as per the without infill time period formula of IS 1893-2002 (eq 23)



Graph 4.5 Fundamental period with respect to Plan Area at 9m height

For a building of 15 m height, from the curve fit the fundamental natural period we can derive a power relationship as follows.

$$T = 0.13\,(BD)^{0.32} \qquad (26)$$

Table 4.5 Fundamental period with respect to Plan Area at 15m height

Height	BD	Period	**IS 1893**	% Variation	**Proposed**	% Variation
15	81	0.554464	**0.571649**	3%	**0.530467**	-4%
15	144	0.692894	**0.571649**	-17%	**0.637704**	-8%
15	225	0.845887	**0.571649**	-32%	**0.7356**	-13%
15	108	0.563883	**0.571649**	1%	**0.581619**	3%
15	192	0.707104	**0.571649**	-19%	**0.699197**	-1%
15	300	0.865301	**0.571649**	-34%	**0.806533**	-7%
15	135	0.570078	**0.571649**	0%	**0.624669**	10%
15	240	0.716423	**0.571649**	-20%	**0.75095**	5%
15	375	0.878007	**0.571649**	-35%	**0.86623**	-1%
15	162	0.574462	**0.571649**	0%	**0.662198**	15%
15	288	0.723007	**0.571649**	-21%	**0.796066**	10%
15	450	0.886974	**0.571649**	-36%	**0.918272**	4%

This shows that the present study estimation of periods gives an error of about 0-13% (eq 26). Whereas the error is 0 to 36% as per the without infill time period formula of IS 1893-2002 (eq 23)

4.4.3 Regression Analysis

The findings of the regression analysis has been presented here.

SUMMARY OUTPUT								
Regression Statistics								
Multiple R	0.9643							
R Square	0.9299							
Adjusted R Square	0.9286							
Standard Error	0.0954							
Observations	108.0000							
ANOVA								
	df	*SS*	*MS*	*F*	*Significance F*			
Regression	2	12.6838	6.3419	696.8469	0.0000			
Residual	105	0.9556	0.0091					
Total	107	13.6393						
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-0.37070	0.03341	-11.09544	0.00000	-0.43695	-0.30445	-0.43695	-0.30445
Height	0.05143	0.00148	34.73593	0.00000	0.04849	0.05436	0.04849	0.05436
Area	0.00117	0.00009	13.67877	0.00000	0.00100	0.00135	0.00100	0.00135

Based on the above analysis, the formula for fundamental time period can be suggested as,

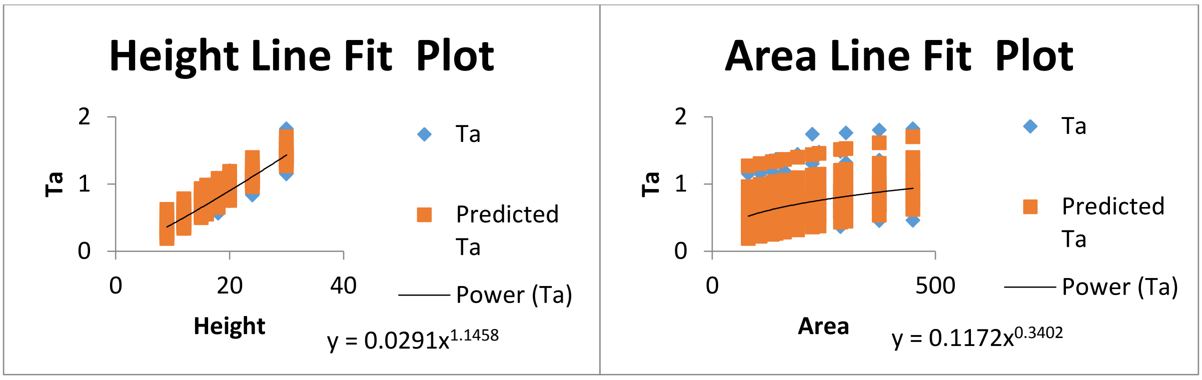
$$T_a = 0.05143h+0.00117A-0.3707 \quad (27)$$

Where,

h = Height of the building in meters.

A = Plan area of the building in square meters.

The above formula has an error range of -30% to 36% which is still better compared to -47% to 45% error given by the existing formula.



Graph 4.6 Fundamental period with respect to Plan Area and Height at 30 m height.

The formulae given by the plots in Graph 4.6 suggests the following:

1. Area Line Fit Plot has an error range of -57% – 136%.
2. Height Line Fit Plot has an error range of -27% - 42%.
3. Therefore, as the height of the building increases above a certain range, the fundamental time period depends more on the height and less on its plan area.

CHAPTER 5:

EXPERIMENTAL INVESTIGATION FOR MEDIUM RISE BUILDINGS

5.1 BASIS FOR SELECTION OF BUILDING PLANS

It is desired that we analyse sufficient number of models that would be completely representative of all types of buildings that are being constructed in the country. But preparation of an exhaustive list of such different types of buildings is very difficult. Thus we tried to choose four typical building plans that are representative in a small way to the actual building behaviour during earthquakes.

The storey heights considered here fall in the range of 30 to 50 storeys. In chapter 3, we have already covered low rise structures which are below 10 storeys. This was done, because the building behaviour remains nearly same for low rise stiff structures. Here, we lay our emphasis on the buildings where dynamic vibration of the building is of prime concern.

Type Z and Type Y are the most common type of buildings that we encounter. Often these buildings are stiffer along one axis than the other. And their vibration behaviour for earthquake along both axes is different. Similarly Type X has been considered just to see the behaviour when the building has similar stiffness along both the axes. Type T has been introduced to take into account minor plan irregularity and thereby causing drastic change in the vibration behaviour during shaking.

5.2 GEOMETRIC PROPERTIES

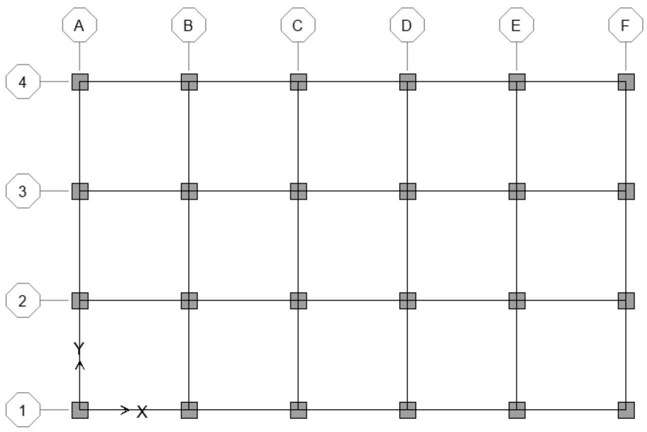
The geometrical properties measured and used to create model were as follows:

1. The slab thickness – 100 mm
2. Beam cross sections on all floors – 0.23mX0.40m

3. Column cross section on all floors – 0.4mX0.4m
4. Storey Height – 3.0m / 4.0m / 5.0m
5. Spans – 3.0m / 4.0m / 5.0m

5.3 LOADING

1. Dead load due to self-weight of the structure.
2. Live load has been taken as 2KN/m^2.



Type Z

Storey - 30 to 50
Span 4m
Height - 3m per storey

Type Y

Storey - 30 to 50
Span 4m
Height - 3m per storey

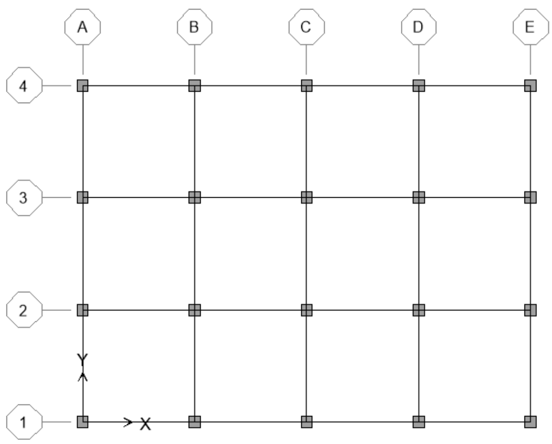
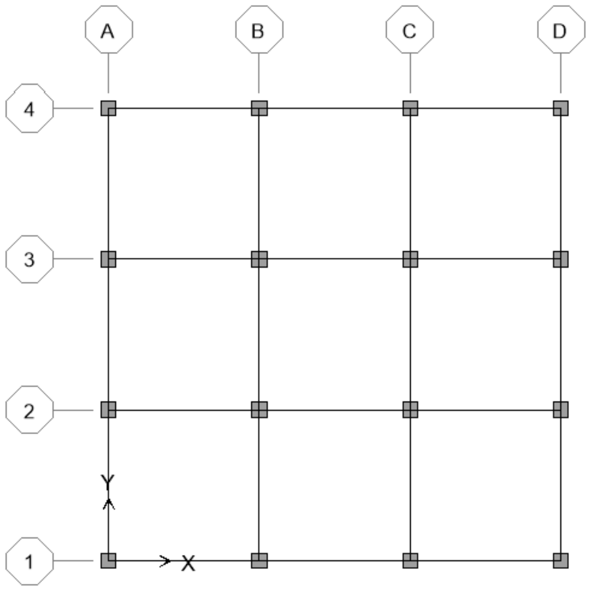


Fig. 5.1 (a) Different Plan Dimension of the structures modelled for Medium Rise Buildings for with and without in fills



Type X

Storey - 30 to 50
Span 4m
Height - 3m per storey

Type T

Storey - 30 to 50
Span 4m
Height - 3m per storey

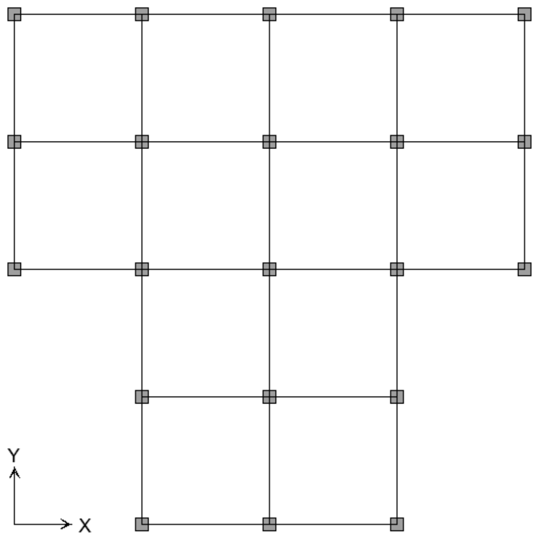
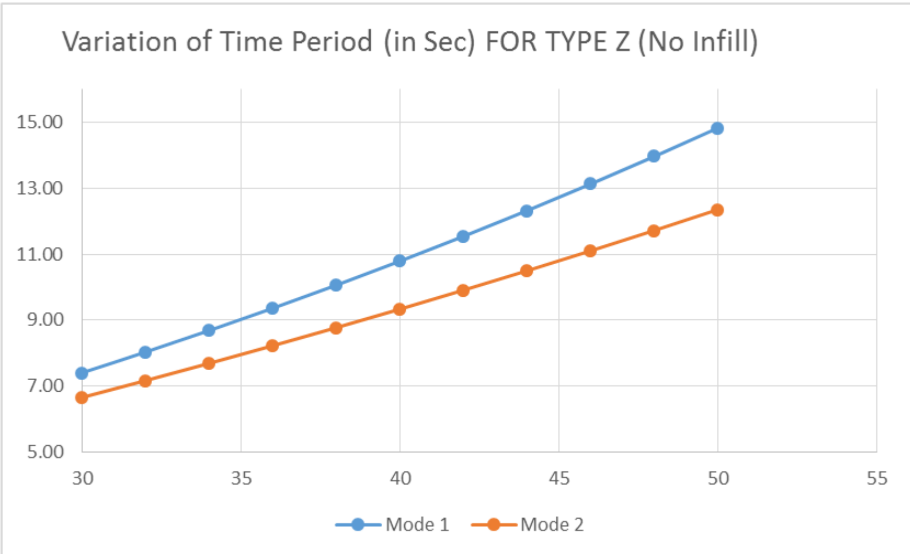


Fig. 5.1 (b) Different Plan Dimension of the structures modelled for Medium Rise Buildings for with and without in fills

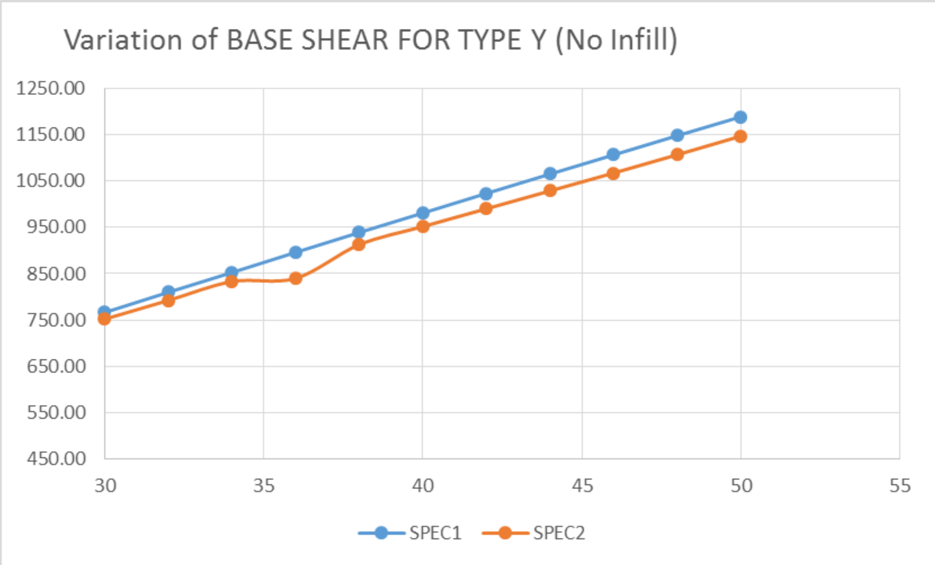
5.4 Results and Discussion

Table 5.1 Variation of Time Period (in Sec) For Type Z (No Infill)

Type Z					No Infill	
			Time Period		Base Shear	
Storey	Height (in m)	LxB (in m)	Mode 1	Mode 2	SPEC1	SPEC2
30	90	20x12	7.41	6.66	766.34	751.89
32	96	20x12	8.03	7.17	809.93	792.25
34	102	20x12	8.69	7.69	852.55	833.05
36	108	20x12	9.36	8.23	896.04	812.93
38	114	20x12	10.06	8.77	938.83	912.56
40	120	20x12	10.79	9.34	981.01	951.75
42	126	20x12	11.54	9.91	1023.38	990.20
44	132	20x12	12.32	10.50	1065.12	1028.77
46	138	20x12	13.13	11.10	1106.53	1067.39
48	144	20x12	13.96	11.72	1147.88	1106.95
50	150	20x12	14.82	12.35	1188.72	1146.99



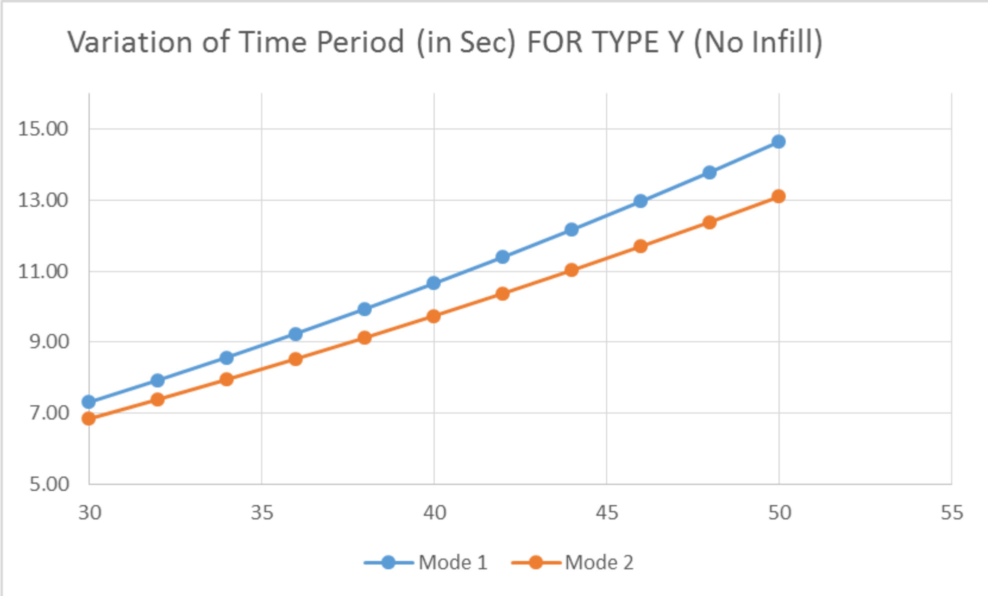
Graph 5.1 Variation of Time Period (in Sec) For Type Z (No Infill)



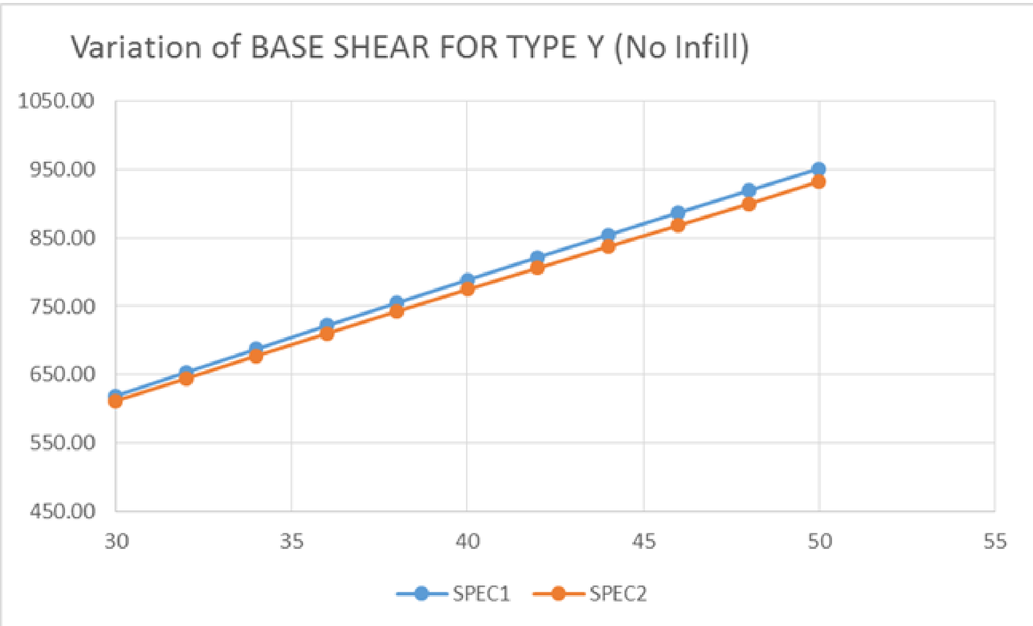
Graph 5.2 Variation of Base Shear For Type Z (No Infill)

Table 5.2 Variation of Time Period (in Sec) For Type Y (No Infill):

Type Y					**No Infill**	
			Time Period		Base Shear	
Storey	Height (in m)	LxB (in m)	Mode 1	Mode 2	SPEC1	SPEC2
30	90	16x12	7.31	6.84	619.15	611.91
32	96	16x12	7.93	7.39	653.55	644.58
34	102	16x12	8.58	7.95	687.45	677.75
36	108	16x12	9.24	8.53	721.71	710.23
38	114	16x12	9.94	9.13	755.25	742.77
40	120	16x12	10.65	9.74	788.59	774.69
42	126	16x12	11.39	10.37	821.72	805.92
44	132	16x12	12.16	11.02	854.28	837.25
46	138	16x12	12.96	11.70	886.84	868.13
48	144	16x12	13.78	12.38	919.07	899.73
50	150	16x12	14.64	13.09	951.15	932.27



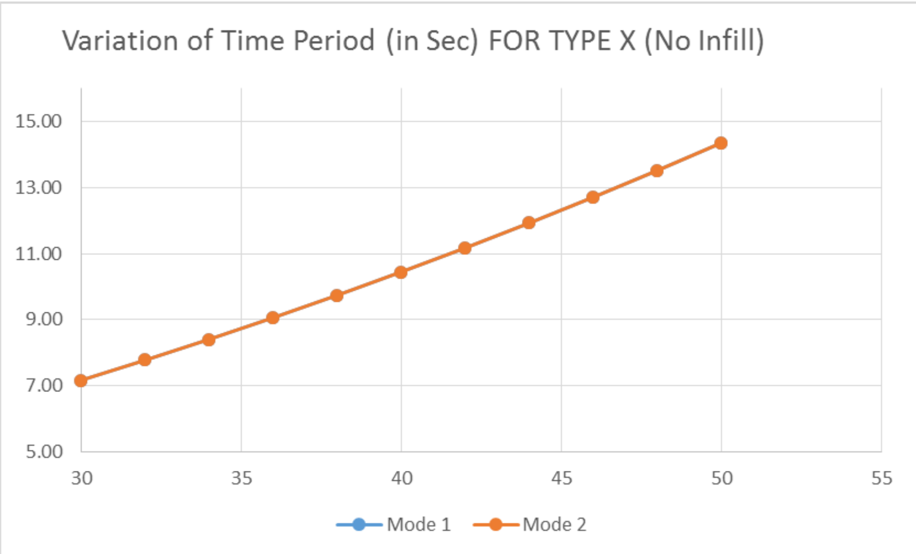
Graph 5.3 Variation of Time Period (in Sec) For Type Y (No Infill)



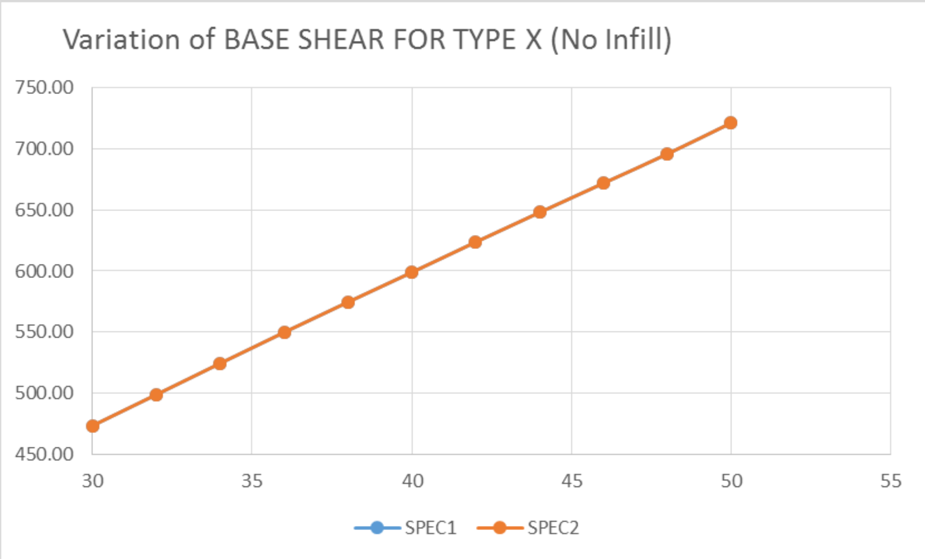
Graph 5.4 Variation of Base Shear For Type Y (No Infill)

Table 5.3 Variation of Time Period (in Sec) For Type X (No Infill) :

Type X					**No Infill**	
			Time Period		Base Shear	
Storey	Height (in m)	LxB (in m)	Mode 1	Mode 2	SPEC1	SPEC2
30	90	12x12	7.17	7.17	473.62	473.62
32	96	12x12	7.78	7.78	499.06	499.06
34	102	12x12	8.41	8.41	524.63	524.63
36	108	12x12	9.06	9.06	549.90	549.90
38	114	12x12	9.74	9.74	574.66	574.66
40	120	12x12	10.45	10.45	599.00	599.00
42	126	12x12	11.17	11.17	623.71	623.71
44	132	12x12	11.93	11.93	647.92	647.92
46	138	12x12	12.71	12.71	671.79	671.79
48	144	12x12	13.52	13.52	695.71	695.71
50	150	12x12	14.35	14.35	721.08	721.08



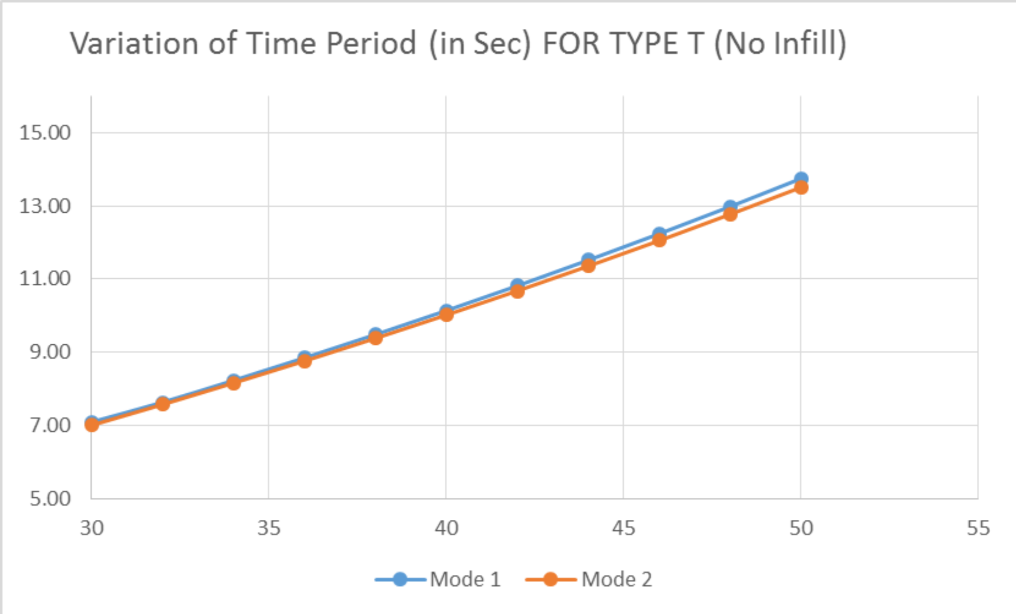
Graph 5.5 Variation of Time Period (in Sec) For Type X (No Infill



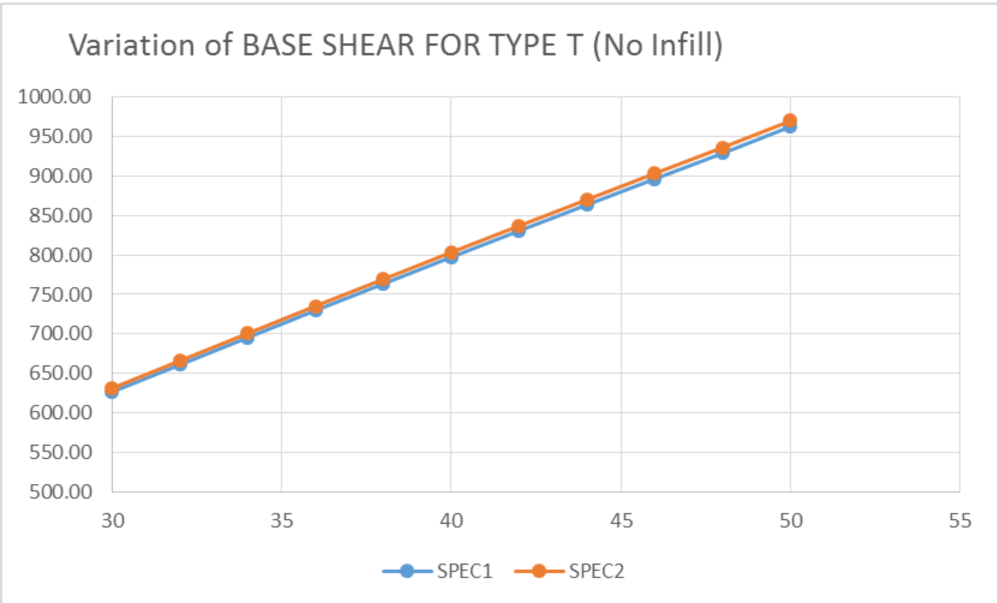
Graph 5.6 Variation of Base Shear For Type X (No Infill)

Table 5.4 Variation of Time Period (in Sec) For Type T (No Infill) :

Type T					**No Infill**	
			Time Period		Base Shear	
Storey	Height (in m)	LxB (in m)	Mode 1	Mode 2	SPEC1	SPEC2
30	90	16x16	7.09	7.01	626.82	631.51
32	96	16x16	7.63	7.57	661.19	666.21
34	102	16x16	8.23	8.16	695.59	700.80
36	108	16x16	8.85	8.76	729.96	735.47
38	114	16x16	9.48	9.38	763.54	769.37
40	120	16x16	10.14	10.02	797.32	803.37
42	126	16x16	10.82	10.68	830.56	836.93
44	132	16x16	11.51	11.35	863.43	870.05
46	138	16x16	12.24	12.05	896.24	903.20
48	144	16x16	12.98	12.77	928.65	935.95
50	150	16x16	13.74	13.50	962.30	969.51



Graph 5.7 Variation of Time Period (in Sec) For Type T (No Infill)



Graph 5.8 Variation of Base Shear For Type T (No Infill)

5.4.1 REGRESSION ANALYSIS FOR NO INFILL CONSIDERATION

SUMMARY OUTPUT								
Regression Statistics								
Multiple R	0.993292							
R Square	0.986629							
Adjusted R Square	0.985976							
Standard Error	0.272768							
Observations	44							
ANOVA								
	df	*SS*	*MS*	*F*	*Significance F*			
Regression	2	225.0878	112.544	1512.64	3.9E-39			
Residual	41	3.0505	0.0744					
Total	43	228.1383						
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	**-4.349**	0.351	-12.378	0.000	-5.058	-3.639	-5.058	-3.639
X Variable 1	**0.119**	0.002	54.921	0.000	0.115	0.123	0.115	0.123
X Variable 2	**0.004**	0.001	2.986	0.005	0.001	0.006	0.001	0.006

Based on the above analysis, the formula for fundamental time period can be suggested as,

$$T_a = 0.119030h + 0.003617A - 4.348864 \quad (28)$$

Where,

h = Height of the building in meters.

A = Plan area of the building in square meters.

The comperision in results is shown in Table No 5.5

This shows that the present study estimation of periods gives a variation of about 0-5% (eq 28). Whereas the variation is 0 to 70% as per the without infill time period formula of IS 1893-2002 (eq 19). This huge difference in the time period wrongly predict the building to be so much more stiff and results increase in Base shear of the structure to an unnecessary higher value.

Table 5.5 Fundamental period for Medium Rise Buildings (No Infill)

Height (in m)	LxB (in m)	Time Period Mode 1	EXPESSION FROM REGRESSION	VARIATION (%)	IS 1893 (Eq 23)	VARIATION (%)
90	240	7.41	7.232	2.40%	2.192	70.40%
90	192	7.31	7.058	3.40%	2.192	70.00%
90	144	7.17	6.885	4.00%	2.192	69.40%
90	192	7.09	7.058	0.40%	2.192	69.10%
96	240	8.03	7.946	1.00%	2.3	71.40%
96	192	7.93	7.772	2.00%	2.3	71.00%
96	144	7.78	7.599	2.30%	2.3	70.40%
96	192	7.63	7.772	-1.90%	2.3	69.90%
102	240	8.69	8.66	0.30%	2.407	72.30%
102	192	8.58	8.487	1.10%	2.407	71.90%
102	144	8.41	8.313	1.20%	2.407	71.40%
102	192	8.23	8.487	-3.10%	2.407	70.80%
108	240	9.36	9.374	-0.20%	2.513	73.20%
108	192	9.24	9.201	0.40%	2.513	72.80%
108	144	9.06	9.027	0.40%	2.513	72.30%
108	192	8.85	9.201	-4.00%	2.513	71.60%
114	240	10.06	10.089	-0.30%	2.617	74.00%
114	192	9.94	9.915	0.30%	2.617	73.70%
114	144	9.74	9.741	0.00%	2.617	73.10%
114	192	9.48	9.915	-4.60%	2.617	72.40%

Table 5.5 Fundamental period for Medium Rise Buildings (No Infill) Continued

Height (in m)	LxB (in m)	Time Period Mode 1	EXPESSION FROM REGRESSION	VARIATION (%)	IS 1893 (Eq 23)	VARIATION (%)
120	240	10.79	10.803	-0.10%	2.719	74.80%
120	192	10.65	10.629	0.20%	2.719	74.50%
120	144	10.45	10.456	-0.10%	2.719	74.00%
120	192	10.14	10.629	-4.80%	2.719	73.20%
126	240	11.54	11.517	0.20%	2.821	75.60%
126	192	11.39	11.343	0.40%	2.821	75.20%
126	144	11.17	11.17	0.00%	2.821	74.70%
126	192	10.82	11.343	-4.80%	2.821	73.90%
132	240	12.32	12.231	0.70%	2.921	76.30%
132	192	12.16	12.058	0.80%	2.921	76.00%
132	144	11.93	11.884	0.40%	2.921	75.50%
132	192	11.51	12.058	-4.80%	2.921	74.60%
138	240	13.13	12.945	1.40%	3.02	77.00%
138	192	12.96	12.772	1.50%	3.02	76.70%
138	144	12.71	12.598	0.90%	3.02	76.20%
138	192	12.24	12.772	-4.30%	3.02	75.30%
144	240	13.96	13.66	2.20%	3.118	77.70%
144	192	13.78	13.486	2.10%	3.118	77.40%
144	144	13.52	13.312	1.50%	3.118	76.90%
144	192	12.98	13.486	-3.90%	3.118	76.00%
150	240	14.82	14.374	3.00%	3.215	78.30%
150	192	14.64	14.2	3.00%	3.215	78.00%
150	144	14.35	14.026	2.30%	3.215	77.60%

5.5 IN FILL CONSIDEARION

The same buildings are analysed for infill. For Infill Consideration, diagonal bracings are modelled as per Proposed Draft Provisions and Commentary on Indian Seismic Code IS 1893: 2002,

The modulus of elasticity (in MPa) of masonry, Em, may be taken as:

$$E_m = 550\ f_m$$

Where f_m is the compressive strength of masonry prism in MPa.

For the solid walls (without any openings) width equivalent diagonal strut (W_{ds}) shall be taken as one third of the diagonal length of the infill wall (d)

$$W_{ds} = d/3$$

Infilled frames with openings shall be modelled with reduced width of strut, which is given as: $W_{do} = \rho w\ W_{ds}$

Where Wds is the width of diagonal strut for infill walls without opennings and ρw is a reduction factor, which accounts for openings in infill, which is given by

$$Pw = 1 - 2.5Ar$$

A is the opening area ratio, which is the ratio of face area of opening to the face area of infill. If the opening area ratio is less than 0.05, i.e., the area of opening is less than 5% of the area of the infill panel, no reduction in the width of diagonal strut need to be made and the infill panel can be modelled as a solid panel. Whereas, if the opening area ratio is more than 0.4, i.e., the area of opening exceeds 40% of the area of the infill panel, the strut reduction factor shall be set to zero and the effect of infill shall be ignored in that panel.

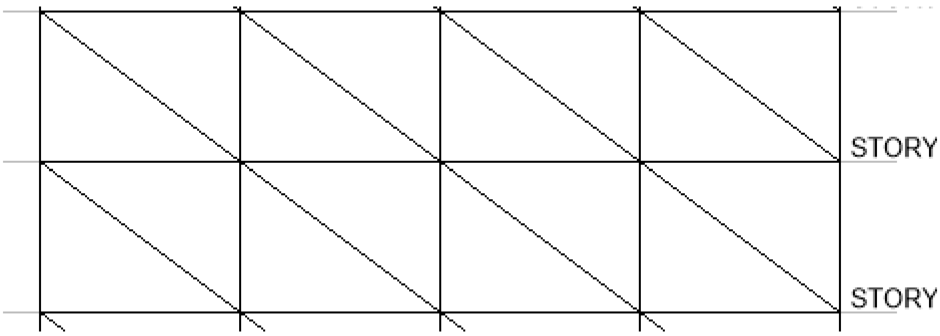


Fig 5.2 Infill Modelled in the Structure

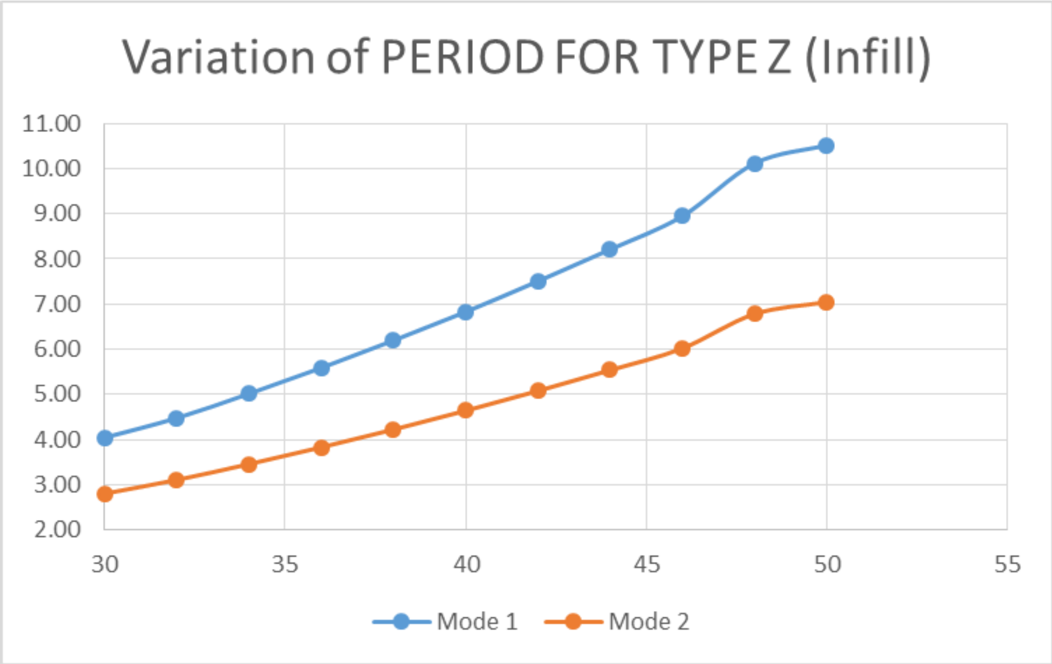
Infills are modelled as shown above with the following specifications.

1. Grade of Concrete – M25,
2. Modulus of Elasticity E = 6300000KN/m^2
3. Poisson Ratio of Brick – 0.15
4. W_{ds} = d/3 = 5/3 = 1.67m

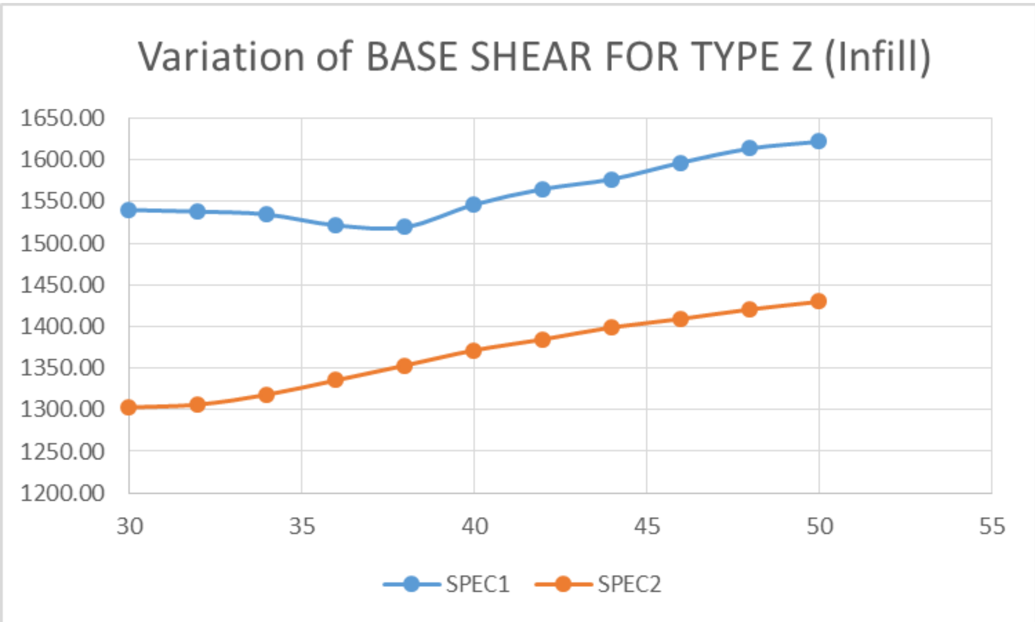
5.5.1 Observations for the different models it Infill consideration

Table 5.6 Variation of Time Period (in Sec) For Type Z (With Infill) :

Type Z					**With Infill**	
Storey	Height (in m)	LxB (in m)	Time Period		Base Shear	
			Mode 1	Mode 2	SPEC1	SPEC2
30	90	20x12	4.05	2.81	1540.01	1302.97
32	96	20x12	4.48	3.11	1538.02	1306.26
34	102	20x12	5.02	3.46	1534.59	1318.14
36	108	20x12	5.59	3.83	1521.58	1335.41
38	114	20x12	6.20	4.23	1519.35	1353.23
40	120	20x12	6.83	4.64	1546.34	1371.40
42	126	20x12	7.51	5.08	1565.08	1384.53
44	132	20x12	8.21	5.54	1576.92	1398.97
46	138	20x12	8.95	6.02	1596.75	1409.15
48	144	20x12	10.11	6.78	1613.73	1420.55
50	150	20x12	10.52	7.05	1621.93	1429.71



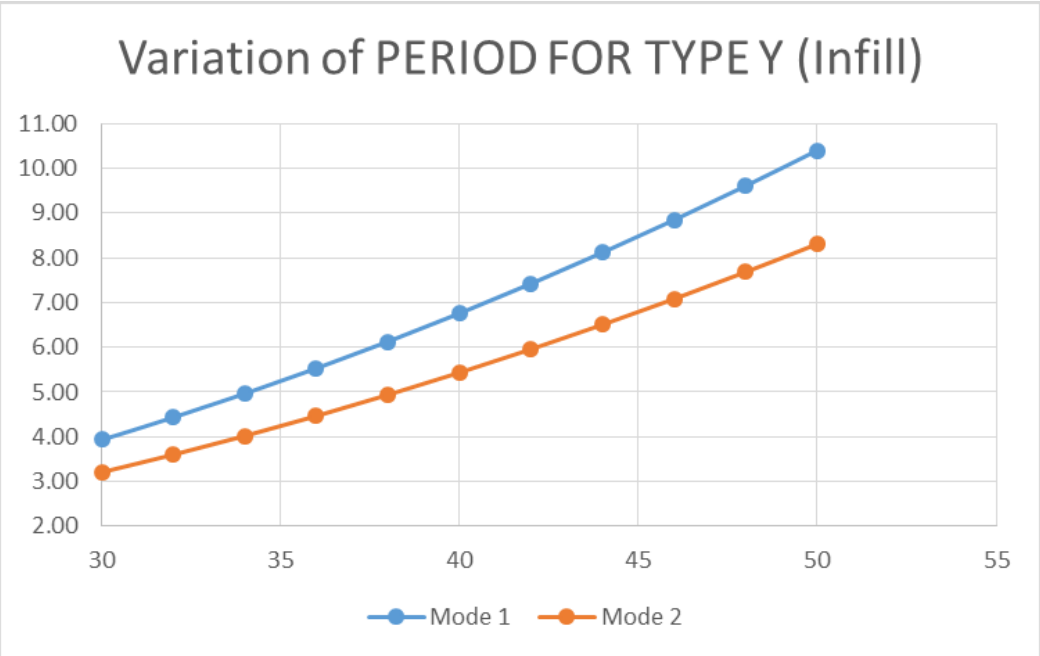
Graph 5.9 Variation of Time Period (in Sec) For Type Z (With Infill)



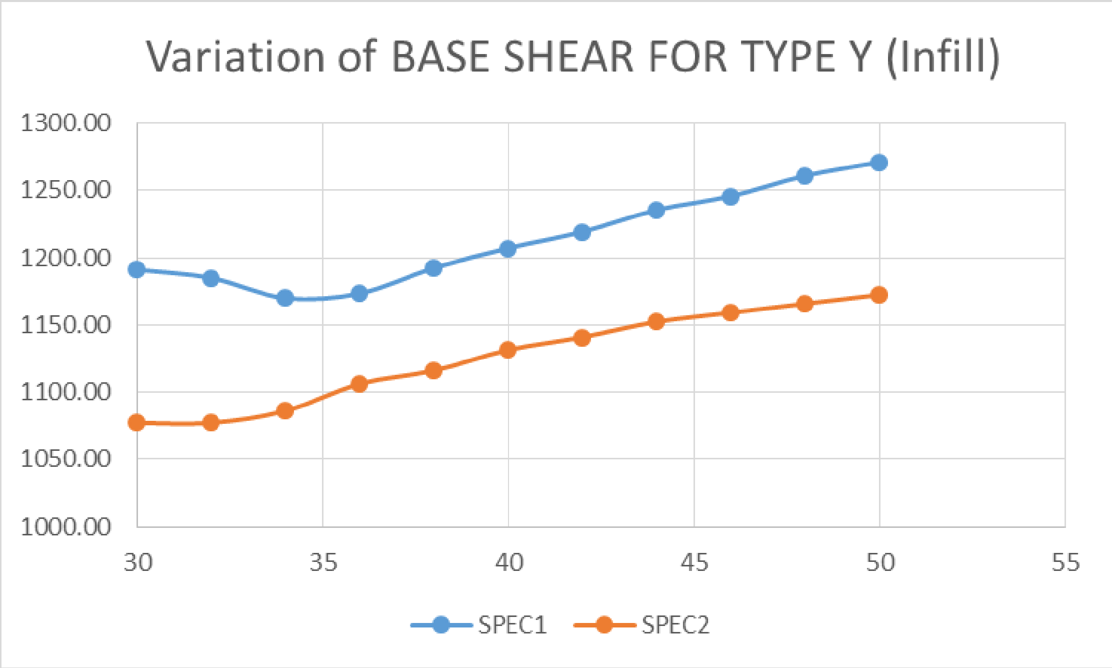
Graph 5.10 Variation of Base Shear For Type Z (With Infill)

Table 5.7 Variation of Time Period (in Sec) For Type Y (With Infill) :

Type Y					**With Infill**	
Storey	Height (in m)	LxB (in m)	Time Period		Base Shear	
			Mode 1	Mode 2	SPEC1	SPEC2
30	90	16x12	3.93	3.21	1191.12	1077.08
32	96	16x12	4.43	3.60	1184.99	1077.28
34	102	16x12	4.96	4.02	1169.93	1086.19
36	108	16x12	5.53	4.47	1173.45	1106.26
38	114	16x12	6.13	4.94	1192.38	1116.15
40	120	16x12	6.76	5.43	1207.20	1131.44
42	126	16x12	7.42	5.96	1219.31	1140.93
44	132	16x12	8.12	6.51	1235.50	1152.41
46	138	16x12	8.85	7.08	1245.77	1159.20
48	144	16x12	9.61	7.68	1260.92	1165.89
50	150	16x12	10.40	8.31	1271.03	1172.45



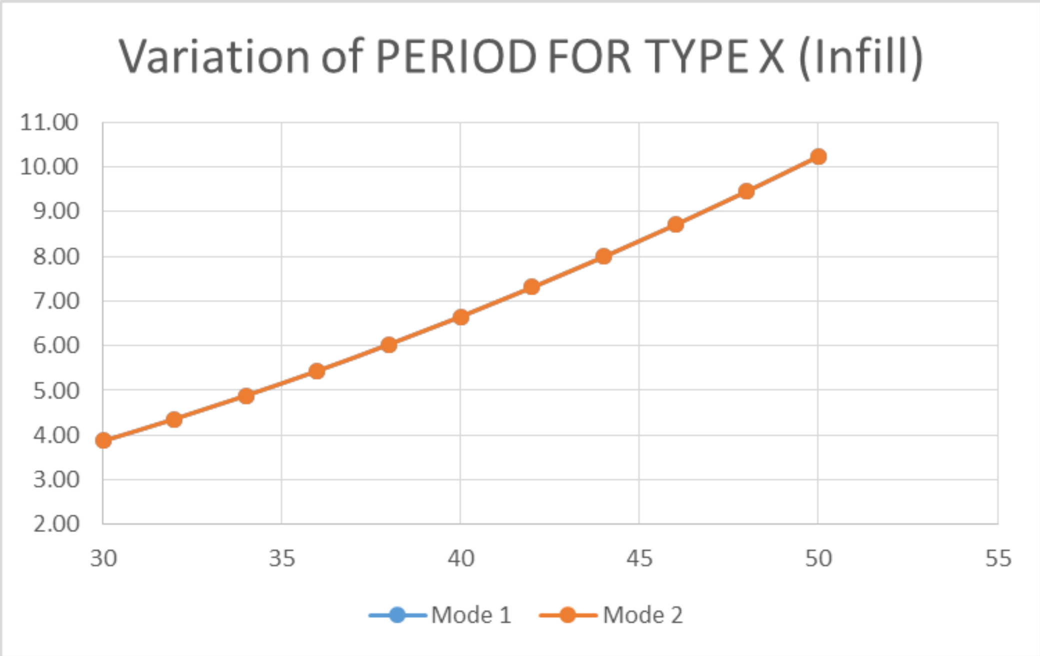
Graph 5.11 Variation of Time Period (in Sec) For Type Y (With Infill)



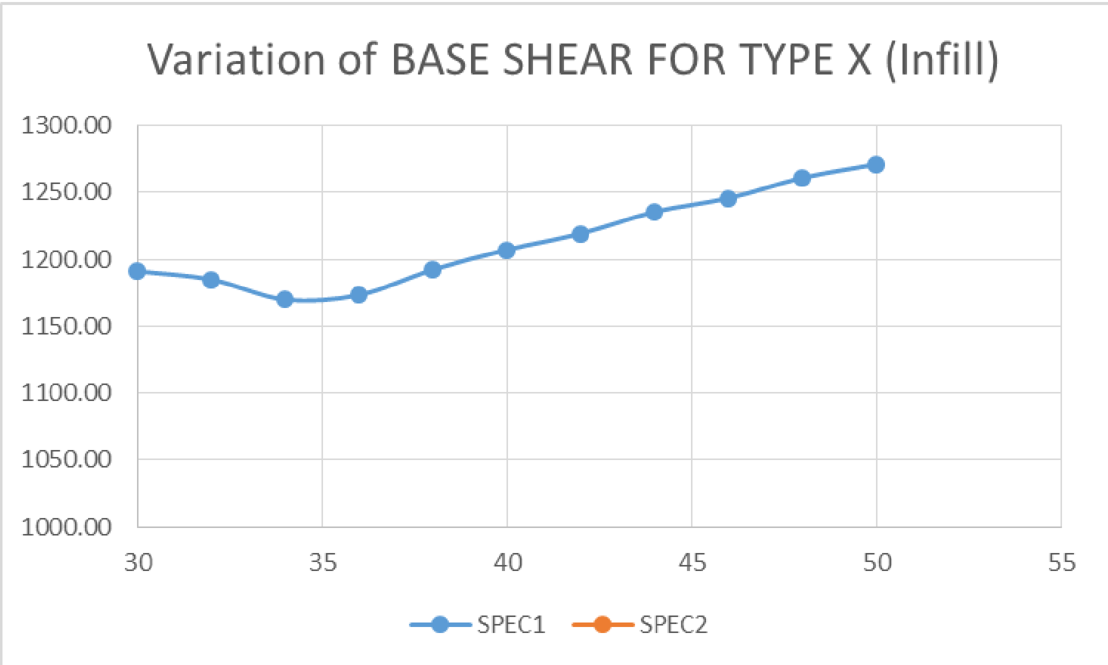
Graph 5.12 Variation of Base Shear For Type Y (With Infill)

Table 5.8 Variation of Time Period (in Sec) For Type X (With Infill) :

Type X					**With Infill**	
Storey	Height (in m)	LxB (in m)	Time Period		Base Shear	
			Mode 1	Mode 2	SPEC1	SPEC2
30	90	12x12	3.87	3.87	843.95	843.95
32	96	12x12	4.36	4.36	842.95	842.95
34	102	12x12	4.89	4.89	846.46	846.46
36	108	12x12	5.44	5.44	860.26	860.26
38	114	12x12	6.03	6.03	871.93	871.93
40	120	12x12	6.65	6.65	884.03	884.03
42	126	12x12	7.30	7.30	891.40	891.40
44	132	12x12	7.99	7.99	900.16	900.16
46	138	12x12	8.70	8.70	905.42	905.42
48	144	12x12	9.45	9.45	911.21	911.21
50	150	12x12	10.23	10.23	915.97	915.97



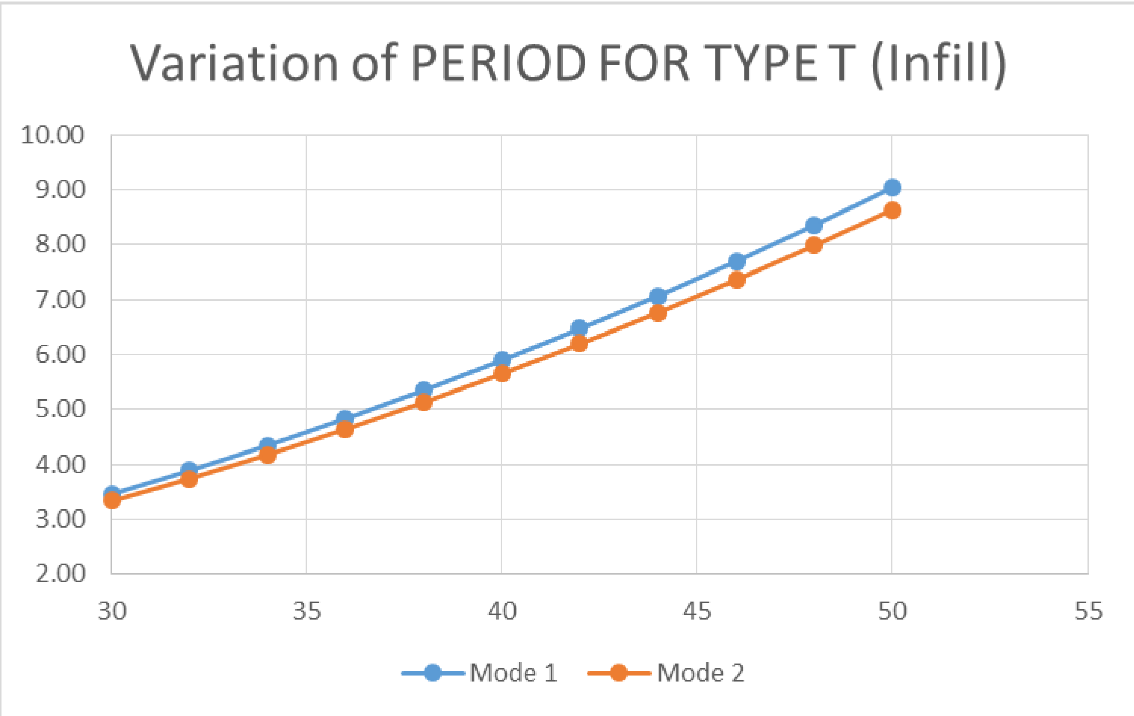
Graph 5.13 Variation of Time Period (in Sec) For Type X (With Infill)



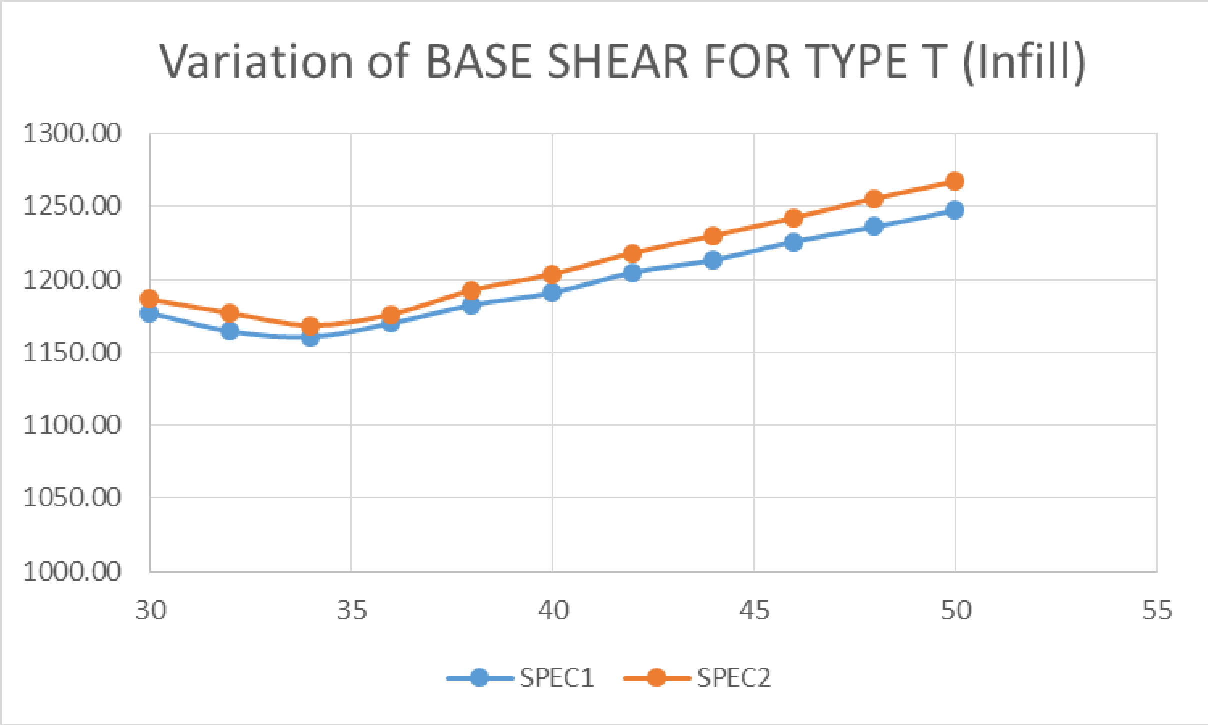
Graph 5.14 Variation of Base Shear For Type X (With Infill)

Table 5.9 Variation of Time Period (in Sec) For Type T (With Infill) :

Type T					**With Infill**	
Storey	Height (in m)	LxB (in m)	Time Period		Base Shear	
			Mode 1	Mode 2	SPEC1	SPEC2
30	90	16x16	3.46	3.33	1177.12	1186.35
32	96	16x16	3.89	3.74	1164.46	1177.01
34	102	16x16	4.35	4.18	1160.61	1168.59
36	108	16x16	4.83	4.64	1169.96	1176.11
38	114	16x16	5.35	5.13	1182.49	1192.62
40	120	16x16	5.90	5.65	1190.84	1203.67
42	126	16x16	6.47	6.19	1204.80	1218.21
44	132	16x16	7.07	6.76	1213.30	1230.14
46	138	16x16	7.70	7.36	1225.96	1242.17
48	144	16x16	8.36	7.99	1235.97	1255.53
50	150	16x16	9.04	8.64	1247.13	1267.07



Graph 5.15 Variation of Time Period (in Sec) For Type T (With Infill))



Graph 5.16 Variation of Base Shear For Type T (With Infill)

5.5.2 REGRESSION ANALYSIS FOR With INFILL CONSIDERATION

SUMMARY OUTPUT								
Regression Statistics								
Multiple R	0.98752							
R Square	0.97520							
Adjusted R Square	0.97399							
Standard Error	0.33130							
Observations	44							
ANOVA								
	df	*SS*	*MS*	*F*	*Significance F*			
Regression	2	176.951	88.475	806.1	1.22E-33			
Residual	41	4.50004	0.1097					
Total	43	181.451						
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-7.0619745	0.37752	-18.706	0	-7.8244	-6.2990	-7.8244	-6.2995
X Variable 1	0.1044527	0.00263	39.68	0	0.0991	0.1090	0.0991	0.1097
X Variable 2	0.006993	0.00114	6.1351	0	0.0047	0.0090	0.0046	0.0092

Based on the above analysis, the formula for fundamental time period can be suggested as,

$$T_a = 0.1044527h+0.006993A-7.0619745 \qquad (29)$$

Where,

h = Height of the building in meters.

A = Plan area of the building in square meters.

The comperision in results is shown in Table No 5.10

This shows that the present study estimation of periods gives a variation of about 0 to 15% (eq 29). Whereas the variation is 40 to 70% as per the without infill time period formula of IS 1893-2002 (eq 19). This huge difference in the time period wrongly predict the building to be so much more stiff and results increase in Base shear of the structure to an unnecessary higher value.

Table 5.10 Fundamental period for Medium Rise Buildings (With Infill)

Height (in m)	L (in m)	LxB (in m)	Time Period Mode 1	EXPRESSION FROM REGRESSION	VARIATION (%)	IS 1893 (Eq. 24)	VARIATION (%)
90	**16**	**192**	**3.46**	3.581	-3.60%	2.025	41.40%
90	12	144	3.87	3.346	13.50%	2.338	39.60%
90	16	192	3.93	3.681	6.40%	2.025	48.50%
90	20	240	4.05	4.017	0.80%	1.811	55.30%
96	**16**	**192**	**3.89**	4.308	-10.80%	2.16	44.40%
96	12	144	4.36	3.973	9.00%	2.494	42.80%
96	16	192	4.43	4.308	2.80%	2.16	51.30%
96	20	240	4.48	4.644	-3.60%	1.932	56.90%
102	**16**	**192**	**4.35**	4.935	-13.50%	2.295	47.20%
102	12	144	4.89	4.599	5.90%	2.65	45.80%
102	16	192	4.96	4.935	0.60%	2.295	53.80%
102	20	240	5.02	5.271	-5.00%	2.053	59.10%

Table 5.10 Fundamental period for Medium Rise Buildings (With Infill) Continued

Height (in m)	L (in m)	LxB (in m)	Time Period Mode 1	EXPRESSION FROM REGRESSION	VARIATION (%)	IS 1893 (Eq. 24)	VARIATION (%)
108	**16**	**192**	**4.83**	5.562	-15.00%	2.43	49.70%
108	12	144	5.44	5.226	3.90%	2.806	48.40%
108	16	192	5.53	5.562	-0.60%	2.43	56.10%
108	20	240	5.59	5.897	-5.50%	2.173	61.10%
114	**16**	**192**	**5.35**	6.188	-15.60%	2.565	52.10%
114	12	144	6.03	5.853	3.00%	2.962	50.90%
114	16	192	6.13	6.188	-1.00%	2.565	58.10%
114	20	240	6.2	6.524	-5.30%	2.294	63.00%
120	**16**	**192**	**5.9**	6.815	-15.60%	2.7	54.20%
120	12	144	6.65	6.479	2.60%	3.118	53.10%
120	16	192	6.76	6.815	-0.80%	2.7	60.10%
120	20	240	6.83	7.151	-4.70%	2.415	64.60%
126	**16**	**192**	**6.47**	7.442	-15.10%	2.835	56.20%
126	12	144	7.3	7.106	2.70%	3.274	55.20%
126	16	192	7.42	7.442	-0.30%	2.835	61.80%
126	20	240	7.51	7.777	-3.60%	2.536	66.20%
132	**16**	**192**	**7.07**	8.068	-14.10%	2.97	58.00%
132	12	144	7.99	7.733	3.20%	3.429	57.10%
132	16	192	8.12	8.068	0.60%	2.97	63.40%
132	20	240	8.21	8.404	-2.40%	2.656	67.60%
138	**16**	**192**	**7.7**	8.695	-12.90%	3.105	59.70%
138	12	144	8.7	8.36	4.00%	3.585	58.80%
138	16	192	8.85	8.695	1.70%	3.105	64.90%
138	20	240	8.95	9.031	-0.90%	2.777	69.00%
144	**16**	**192**	**8.36**	9.322	-11.60%	3.24	61.20%
144	12	144	9.45	8.986	4.90%	3.741	60.40%
144	16	192	9.61	9.322	3.00%	3.24	66.30%
144	20	240	10.11	9.658	4.50%	2.898	71.30%
150	**16**	**192**	**9.04**	9.949	-10.00%	3.375	62.70%
150	12	144	10.23	9.613	6.10%	3.897	61.90%
150	16	192	10.4	9.949	4.30%	3.375	67.50%
150	20	240	10.52	10.284	2.20%	3.019	71.30%

CONCLUSIONS:

The issue of fundamental time period and its impact on low and medium rise buildings was studied in detail. However the sample of buildings investigated was still not sufficient to justify any conclusive statement, but a few points have clearly come out of this analysis.

#The formula given in the code for bare frames is of very general nature and does not categorically apply to all kinds of buildings. An approach more suited for this purpose would be to use different formulae for different buildings.

Low rise buildings which vibrate with high frequency behave in an entirely different way than medium or high rise buildings. Here height does not play a vital role in the determination of fundamental time period of the building, but the plan dimension does. Hence a formula of time period depending on the plan area than the height would be more suited. Here, two empirical formulae have been generated that applies to buildings under ten and fifteen meters. They seem to be more suited than the formula given in the code.

Medium or high rise buildings which vibrate with long time periods are critical structures that needs to be analysed properly. While height does play an important role in the determination of fundamental time period, but other factors like irregularities, unsymmetry come into play when the height of the building increases beyond a certain level. Certain assumptions have been made in order to incorporate these complications and empirical relationship has been developed.

Regular buildings vibrate in a fairly predictable way, but it is the irregular and unsymmetric ones that sometimes behave in an erratic manner. And that is where dynamic modal analysis comes to action, providing a rational distribution of lateral

forces. Now, the modal analysis forces are often lesser compared to their static analysis counterparts, thereby, raising the need for scaling the forces as per static analysis based on fundamental time period. Here the need for a justifiable empirical formula is towered. An effort have been made here to find an empirical formula that somehow represents and includes those irregularities.

Non structural components like brick infill walls contribute to the stiffness of the buildings. Their contribution affect the time period of the building. Though it is difficult to estimate the contribution of the non structural components to the stiffness and thereby the time period of the buildings, but certain assumptions have to be made to incorporate these effects. Here we considered buildings with infil making some reasonable assumptions and found expression that seeminly works better than our codal expression.

The basic issue with our code still remains in its empirical formula approach. However large be the sample size, there would always be buildings that are not part of that sample size. In fact, every other buildings may behave differently under dynamic loads. Thus a more rational approach would be to drop the empirical formula and analyse every building rigorously. A more rigorous dynamic analysis, pushover analysis or performance based analysis would be more suited for the purpose.

Recommendations:

Modelling of a structure to be more precise, non structural elements like brick walls to be modelled for more exact results.

For irrigularities in structure we need to model them more exactly to get the actual behaviour.

Dynamic analysis to be more frequently used for structures with irrigularities and for any height except for certain very low rise buildings.

For medium and high rise buildings instead of scaling the forces to an unnecessarily higher value, a more rigorous dynamic analysis, pushover analysis or performance based analysis would be more suited for the purpose.

Future scope :

A bigger sample of buildings to be investigated with large number of irrigularities, different plan area and plan dimensions to get a more generalized esmtimation of time period.

The same buildings to be analysed by pushover analysis or performance based analysis to get more the exact behaviour.

REFRENCES :

[1] **IS 1893:2002** *Criteria for earthquake resistant design of structures (part1) general provisions and buildings , 2002*

[2] **DATTA.T.K** *Seismic analysis of structure , John Wiley & Sons(Asia) Pte ltd*, 2010

[3] **Chopra. A K**. *Dynamics of structures, Theory and application to earthquake engineering,* 3rd Edition Pearson India , 2007

[4] **Jain. S K**, IIT Roorkee *Review of Indian seismic code, IS 1893 (Part 1) : 2002*

[5] **IBC – 2012** International Building Code,

[6] **NBCC – 1995** National Building Code of Canada,

[7] **Euro Code 8 – 2004**

[8] **NZS 4203 – 1992** New Zealand Code,