“Seismic Analysis and Design of a four storey RC Building”

A PROJECT

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

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of
Prof. Poonam Dhiman

By

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to

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WAKNAGHAT SOLAN – 173 234

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CERTIFICATE

This is to certify that the work which is being presented in the project title **"Seismic Analysis and Design of RC building"** in partial fulfilment of the requirements for the award of the degree of Bachelor of technology and submitted to Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Dilpuneet Singh (121614)** during a period from July 2015 to December 2015 under the supervision of **Prof. Poonam Dhiman** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

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ABSTRACT

In this project first static and dynamic analysis of four storey reinforced concrete building was carried out. The calculations were carried out first manually for both the methods then same calculations were done using staad pro. The results of analysis in terms of member forces were compared and it was found that response spectrum dynamic analysis is more economical as compared to static. Design of beams and columns was carried out by preparing spread sheets for doubly reinforced beams and columns under biaxial bending. The seismic analysis and design is done which will ensure safe building during earthquake.

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Chapter 1
Introduction

1.Introduction

Earthquake-resistant structures are structures designed to withstand earthquakes. While no structure can be entirely immune to damage from earthquakes, the goal of earthquake-resistant construction is to erect structures that fare better during seismic activity than their conventional counterparts.

According to building codes, earthquake-resistant structures are intended to withstand the largest earthquake of a certain probability that is likely to occur at their location. This means the loss of life should be minimized by preventing collapse of the buildings for rare earthquakes while the loss of functionality should be limited for more frequent one.

Currently, there are several design philosophies in earthquake engineering, making use of experimental results, computer simulations and observations from past earthquakes to offer the required performance for the seismic threat at the site of interest. These range from appropriately sizing the structure to be strong and ductile enough to survive the shaking with an acceptable damage, to equipping it with base isolation or using structural vibration control technologies to minimize any forces and deformations. While the former is the method typically applied in most earthquake-resistant structures, important facilities, landmarks and cultural heritage buildings use the more advanced (and expensive) techniques of isolation or control to survive strong shaking with minimal damage. Examples of such applications are the Cathedral of Our Lady of the Angels and the Acropolis Museum.

In this project we will do seismic analysis of the four storey RC building with equivalent lateral force method and response spectrum method. In response spectrum we will first analysis without considering the infill wall and then with considering infill wall. We will calculate forces with both methods and then will compare the forces from the two methods. Our building is a plane frame model with four storeys.We will then do seismic analysis by staad pro and calculate results by both manually and staad pro. Then we will do design of beams and columns of the building.

Chapter 2

Methods of Seismic Analysis

2. Methods of seismic analysis

Seismic analysis is a subset of structural analysis and is the calculation of the response of a building (or nonbuilding) 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.

2.1 Equivalent Static Lateral Force Method

The concept employed in equivalent static lateral force procedures is to place static loads on a structure with magnitudes and direction that closely approximate the effects of dynamic loading caused by earthquakes. Concentrated lateral forces due to dynamic loading tend to occur at floor and ceiling/roof levels in buildings, where concentration of mass is the highest. Furthermore, concentrated lateral forces tend to be larger at higher elevations in a structure. Thus, the greatest lateral displacements and the largest lateral forces often occur at the top level of a structure (particularly for tall buildings).

2.2 Response Spectrum Method

With the advent of personal computers and improved structural analysis techniques, the use of more precise methods increased. One of the most popular was response spectrum analysis. The method requires the determination of a response spectrum from measured seismic activity. This data was then reduced into a spectrum of seismic action versus natural frequency. The seismic action could be displacement, velocity, or acceleration, although the typical value used was acceleration. Detailed information from the structural model was coupled with the corresponding spectral values for each specific mode of vibration. The independent results were then combined using an appropriate technique to determine the response of the overall structure.

2.3 Time History Method

Time-History analysis is a step-by-step procedure where the loading and the response history are evaluated at successive time increments, Δt – steps. During each step the response is evaluated from the initial conditions existing at the beginning of the step (displacements and velocities) and the loading history in the interval. With this method the non-linear behaviour may be easily considered by changing the structural properties (e.g. stiffness, k) from one step to the next. Therefore this method is one of the most effective for the solution of non-linear response, among the many methods available. Nevertheless, in the present text, a linear time history analysis is adopted i.e. the structural properties are assumed to remain constant during the entire loading history and further it is assumed that the structure behaves linearly.

Chapter 3

Design Example Problem

3. Design Example Problem

3.1 Introduction

A four storey RC building has been analysed by the equivalent static method, response spectrum method and time history method as per IS 1893 (Part 1): 2002. The example illustrates step by step procedure for determination of forces. One of the plane frame in transverse direction has been considered for the purpose of illustration by assuming that the building is symmetric in elevation and planned as shown in figure 3.1. The preliminary building data required for analysis are assumed in table 3.1.The building is evaluated by three methods i.e. Equivalent static lateral force method, response spectrum method and time history method.

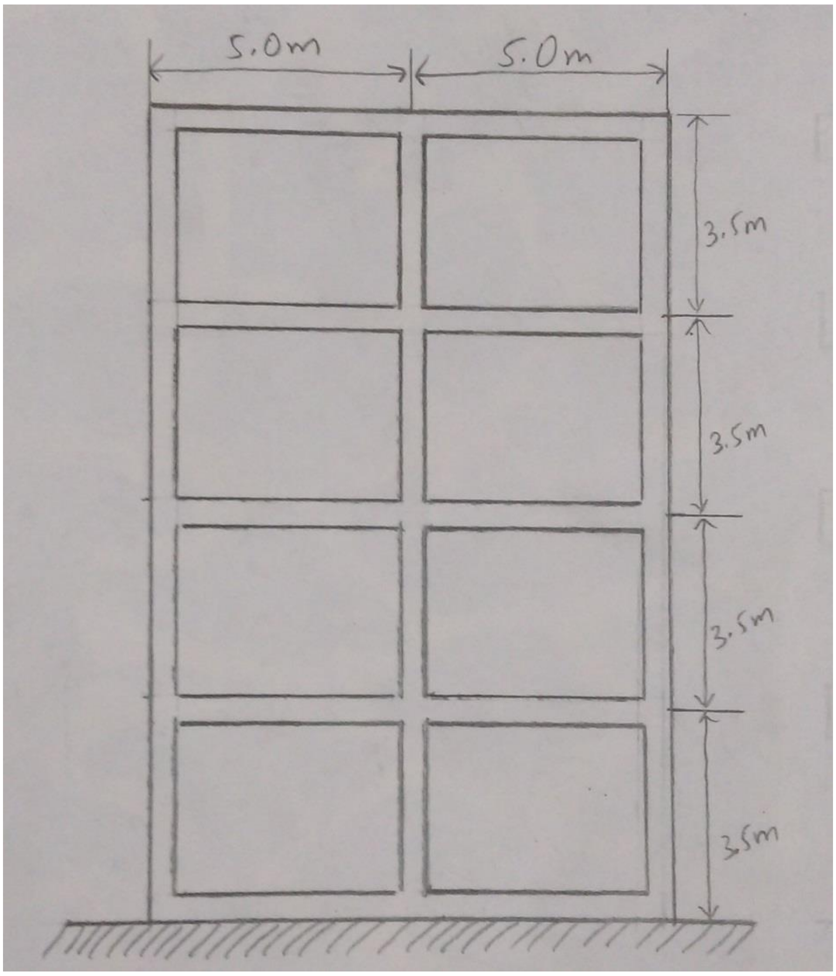

Fig. 3.1 Plane frame structure model

Table 3.1 Assumed preliminary data required for analysis of frame

1.	Type of structure	Multi storey rigid joined plane frame
2.	Seismic zone	V (table 2, IS 1893 (Part1): 2002)
3.	Number of storeys	Four (G+3)
4.	Floor height	3.5m
5.	Infill wall	300mm thick including plaster in longitudinal and 200mm in transverse direction
6.	Imposed load	3.5kN/m^2
7.	Materials	Concrete (M 20) and Reinforcement (Fe415)
8.	Size of columns	300mm × 500mm
9.	Size of beam	300mm × 450mm in longitudinal and 300mm × 400mm in transverse direction
10.	Depth of slab	150mm thick
11.	Specific weight of RCC	25kN/m^2
12.	Specific weight of infill	20kN/m^2
13.	Type of soil	Rock
14.	Response spectra	As per IS 1893 (Part 1): 2002
15.	Time History	Compatible to IS 1893 (Part 1): 2002 spectra at rocky site for 5% damping

3.2 Equivalent Static Lateral Force Method

A step by step procedure for analysis of the frame by equivalent static lateral force method is as follow:

3.2.1 Step1: Calculation of Lumped Masses to various floor levels

The earthquake forces shall be calculated for the full dead load plus the percentage of imposed load as given in Table 8 of IS 1893 (Part 1): 2002. The imposed load on roof is assumed to be zero. The lumped masses of each floor are worked out as follows:

Roof

Mass of infill + Mass of columns + Mass of beams in longitudinal and transverse direction of that floor + Mass of slab + Imposed load of that floor if permissible.

$= \{((0.3\times 10 \times (3.5/2) + 0.2 \times 15 \times (3.5/2))\ 20\} + \{(0.3 \times 10 \times 0.45 + 0.3 \times 15 \times 0.4)\ 25\} + \{0.15 \times 5 \times 10 \times 25\} + \{(0.3 \times 0.5 \times (3.5/2) \times 3) \times 25\} + 0^*$

= 495.9375kN (weight) = 50.57ton (mass)

3^{rd}, 2^{nd}, 1^{st} Floors

$= \{((0.3 \times 10 \times 3.5) + (0.2 \times 15 \times 3.5))\ 20\} + \{(0.3 \times 10 \times 0.45 + 0.3 \times 15 \times 0.4)\ 25\} + \{0.15 \times 5 \times 10 \times 25\} + \{0.3 \times 0.5 \times 3.5 \times 3 \times 25\} + \{5 \times 10 \times 3.5 \times 0.5^{**}\}$

= 813.125kN (weight) = 82.91ton (mass)

*imposed load on roof not considered.
**50% of imposed load, if imposed load is greater than $3kN/m^2$.

Seismic weight of building

= Seismic weight of all floors = $M_1 + M_2 + M_3 + M_4$
= 82.91 + 82.91 + 82.91 + 50.57 = 299.3 ton

3.2.2 Step 2: Determination of Fundamental Natural Period

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

$T_a = 0.075 \times h^{0.75} = 0.075 \times 14^{0.75} = 0.5423$sec

where h is the height of the building, in meters.

3.2.3 Step 3: Determination of Design Base Shear

Design seismic base shear, $V_B = A_h \times W$

$A_h = (Z \times I \times S_a) / (2 \times R \times g) = (0.36 \times 1 \times 1.842) / (2 \times 5) = 0.066312$

For $T_a = 0.5423$

$S_a / g = 1/ T_a = 1.842$, for rock site from figure 2 of IS 1893 (Part 1): 2002

Design seismic base shear, $V_B = 0.066312 \times (299.3 \times 9.81) = 194.7$kN

3.2.4 Step 4: Vertical Distribution of Base Shear

The design base shear (V_B) computed shall be distributed along the height of the building as per the expression,

$$Q_i = V_B \times W_i \times h_i^2 \Big/ \sum_{i=1}^{n} W_i h_i^2 \qquad (3.1)$$

where,
Q_i = Design lateral forces at floor i,
W_i = Seismic weights of the floor i,
h_i = Height of the floor i, measured from base, and

n = Number of stories

Using the equation 3.1, base shear is distributed as follows:

$Q_1 = V_B (W_1h_1^2 / W_1h_1^2 + W_2h_2^2 + W_3h_3^2 + W_4h_4^2)$

$= 194.7 (813.125 \times 3.5^2 / 813.125 \times 3.5^2 + 813.125 \times 7^2 + 813.125 \times 10.5^2 + 495.9375 \times 14^2)$

= 8.19kN

Similarly,
$Q_2 = 0.1684 \times 194.7 = 32.78$kN
$Q_3 = 0.3788 \times 194.7 = 73.75$kN
$Q_4 = 0.4107 \times 194.7 = 79.96$kN

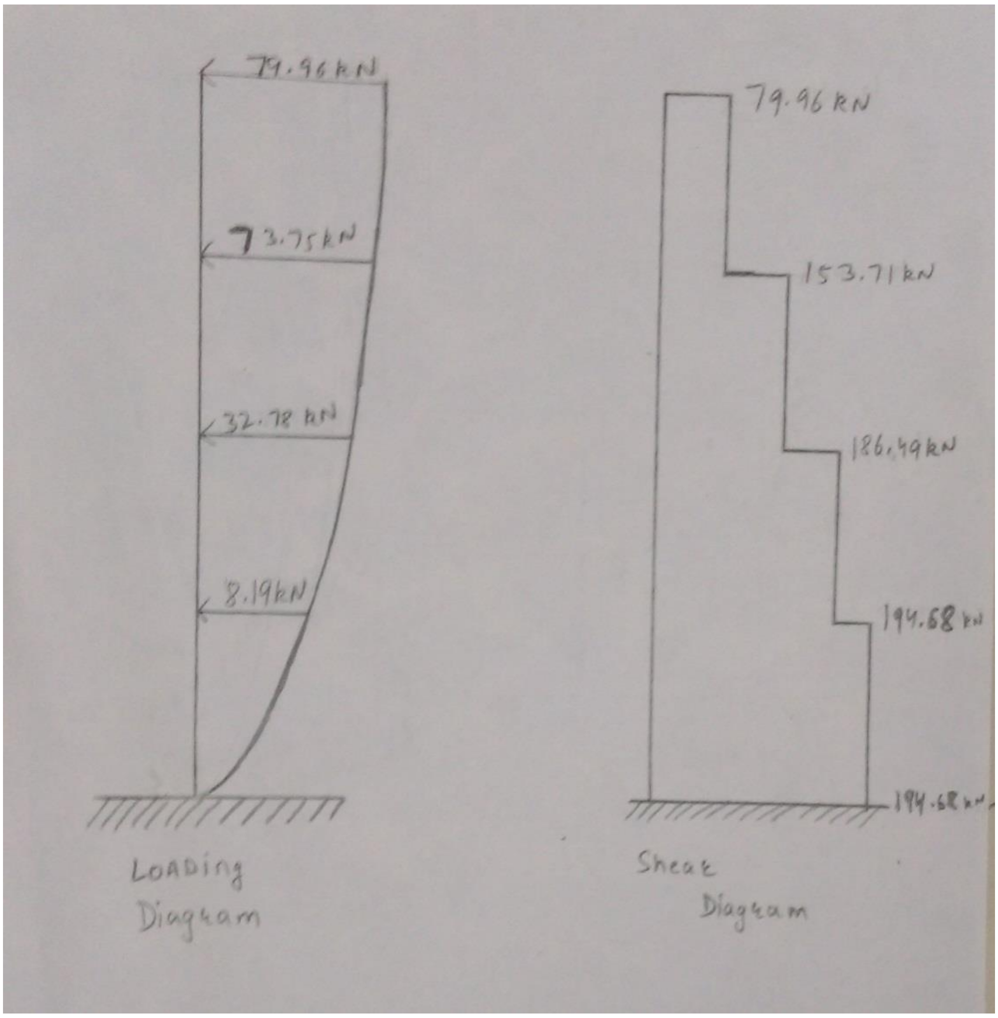

Fig. 3.2 Lateral force distribution at various floor level

3.3 Response Spectrum Method

A:Frame without considering the stiffness of infills

A step by step procedure for analysis of the frame by response spectrum method is as follows:

3.3.1 Step 1: Determination of Eigenvalues and Eigenvectors

Mass matrix, M and stiffness matrix, K of the plane frame lumped mass model are,

$$M = \begin{bmatrix} M_1 & 0 & 0 & 0 \\ 0 & M_2 & 0 & 0 \\ 0 & 0 & M_3 & 0 \\ 0 & 0 & 0 & M_4 \end{bmatrix} = \begin{bmatrix} 82.91 & 0 & 0 & 0 \\ 0 & 82.91 & 0 & 0 \\ 0 & 0 & 82.91 & 0 \\ 0 & 0 & 0 & 50.57 \end{bmatrix} \text{ ton}$$

Column stiffness of storey,

$k = 12EI / L^3 = (12 \times 22360 \times 10^3 \times (0.3 \times 0.5^3 / 12)) / 3.5^3 = 19556.85$kN/m

Total lateral stiffness of each storey,

$k_1 = k_2 = k_3 = k_4 = 3 \times 19556.85 = 58670.55$kN/m

Stiffness of lumped mass modified structure

$$K = \begin{bmatrix} k_1 + k_2 & -k_2 & 0 & 0 \\ -k_2 & k_2 + k_3 & -k_3 & 0 \\ 0 & -k_3 & k_3 + k_4 & -k_4 \\ 0 & 0 & -k_4 & k_4 \end{bmatrix}$$

$$= \begin{bmatrix} 117341.1 & -58670.55 & 0 & 0 \\ -58670.55 & 117341.1 & -58670.55 & 0 \\ 0 & -58670.55 & 117341.1 & -58670.55 \\ 0 & 0 & -58670.55 & 58670.55 \end{bmatrix} \text{ kN/m}$$

For the above stiffness and mass matrix, eigenvalues and eigenvectors are worked out as follows:

$$\left|K-\omega^2 m\right| = \begin{vmatrix} 2k-\omega^2 m & -k2 & 0 & 0 \\ -k2 & 2k-\omega^2 m & -k3 & 0 \\ 0 & -k3 & 2k-\omega^2 m & -k4 \\ 0 & 0 & -k4 & k-\omega^2 m \end{vmatrix} = 0$$

Taking $k/m = \omega_n^2$
Therefore,

$(\omega_n^2)^4 - 8.3(\omega_n^2)^3 (\omega^2) + 10.75(\omega^2_n)^2 (\omega^2)^2 - 4.45(\omega_n^2)(\omega^2)^3 + 0.575(\omega^2)^4 = 0$

By solving the above equation, natural frequencies (eigenvalues) of various modes are

Eigenvalues

$$[\omega^2] = \begin{bmatrix} 81 & & & \\ & 657 & & \\ & & 1475 & \\ & & & 2065 \end{bmatrix}$$

$\omega_1^2 = 81$, $\omega_2^2 = 657$, $\omega_3^2 = 1475$, $\omega_4^2 = 2065$

The quantity of ω_i^2, is called the i[th] eigenvalue of the matrix $\lfloor -M\omega i^2 + K \rfloor \Phi_i$. Each natural frequency (ω_i) of the system has a corresponding eigenvector (mode shape), which is denoted by Φ_i. The mode shape corresponding to each natural frequency is determined from the equations

$$\lfloor -M\omega 1^2 + K \rfloor \ \Phi_1 = 0$$

$$\lfloor -M\omega 2^2 + K \rfloor \ \Phi_2 = 0$$

$$\lfloor -M\omega 3^2 + K \rfloor \ \Phi_3 = 0$$

$$\lfloor -M\omega 4^2 + K \rfloor \ \Phi_4 = 0$$

Solving the above equation, modal vector (eigenvectors), mode shapes and natural periods under different modes are

Eigenvectors {Φ}

$$\{\Phi\} = \{\Phi_1\ \Phi_2\ \Phi_3\ \Phi_4\} = \begin{bmatrix} -0.0328 & 0.0795 & 0.0808 & -0.0397 \\ -0.0608 & 0.0644 & -0.0540 & 0.0690 \\ -0.0798 & -0.0273 & -0.0448 & -0.0799 \\ -0.0872 & -0.0865 & 0.0839 & 0.0696 \end{bmatrix}$$

Mode Shapes

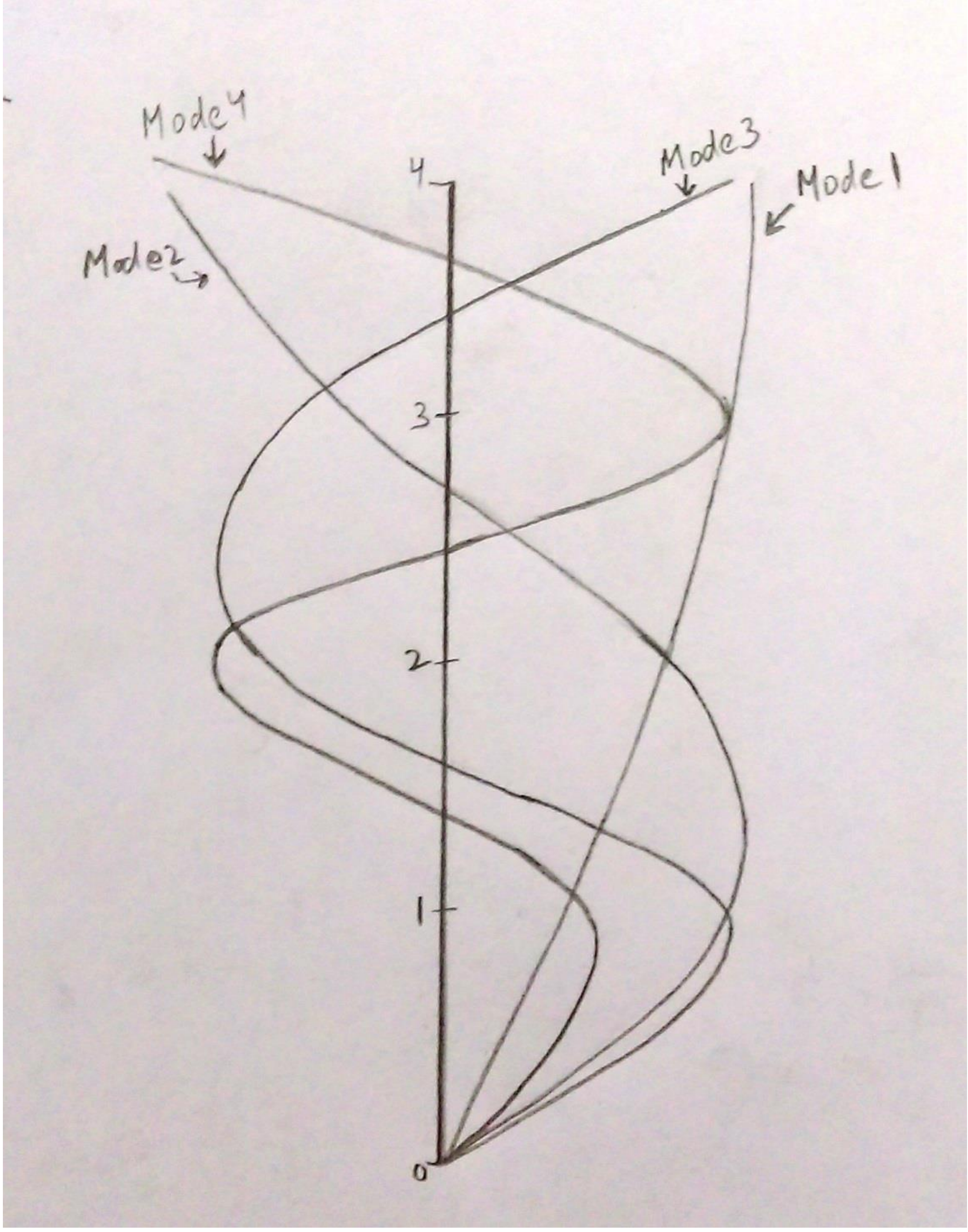

Fig. 3.3 Modal Shapes

Natural time period

$$T = \begin{bmatrix} 0.6977 & 0 & 0 & 0 \\ 0 & 0.2450 & 0 & 0 \\ 0 & 0 & 0.1636 & 0 \\ 0 & 0 & 0 & 0.1383 \end{bmatrix} s$$

3.3.2 Step 2: Determination of Modal Participation Factors

The modal participation factor (p_k) of mode k is,

$$p_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

$$p_1 = \frac{\sum_{i=1}^{4} W_i \phi_{i1}}{\sum_{i=1}^{4} W_i (\phi_{i1})^2} = \frac{(W_1\phi_{11} + W_2\phi_{21} + W_3\phi_{31} + W_4\phi_{41})}{(W_1(\phi_{11})^2 + W_2(\phi_{21})^2 + W_3(\phi_{31})^2 + W_4(\phi_{41})^2)} = -14.45$$

$$p_2 = \frac{\sum_{i=1}^{4} W_i \phi_{i2}}{\sum_{i=1}^{4} W_i (\phi_{i2})^2} = \frac{(W_1\phi_{12} + W_2\phi_{22} + W_3\phi_{32} + W_4\phi_{42})}{(W_1(\phi_{12})^2 + W_2(\phi_{22})^2 + W_3(\phi_{32})^2 + W_4(\phi_{42})^2)} = 4.06$$

Similarly,

$p_3 = 2.1$, $p_4 = -0.52$

3.3.3 Step 3: Determination of Modal Mass

The modal mass (M_k) of mode k is given by,

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

where,

g = Acceleration due to gravity,
Φ_{ik} = Mode shape coefficient at floor i in mode k, and
W_i = Seismic weight of floor i,

$$M_1 = \frac{\left[\sum_{i=1}^{4} W_i \phi_{i1}\right]^2}{g\left[\sum_{i=1}^{4} W_i (\phi_{i1})^2\right]}$$

$$M_1 = \frac{\left[9.81\left(82.91(-0.0328)+82.91(-0.0608)+82.91(-0.0798)+50.57(-0.0872)\right)\right]^2}{9.81\left[9.81\left(82.91(-0.0328)^2+82.91(-0.0608)^2+82.91(-0.0798)^2+50.57(-0.0872)^2\right)\right]}$$

= 269.85

$$M_2 = \frac{\left[\sum_{i=1}^{4} W_i \phi_{i2}\right]^2}{g\left[\sum_{i=1}^{4} W_i (\phi_{i2})^2\right]}$$, similarly, M_2 = 21.42, M_3 = 5.78, M_4 = 0.34

Modal contributions of various modes

For mode 1, $\frac{M_1}{M} = \frac{269.85}{299.3} = 0.90 = 90\%$

For mode 2, $\frac{M_2}{M} = \frac{21.42}{299.3} = 0.0715 = 7.15\%$

For mode 3, $\frac{M_3}{M} = \frac{5.78}{299.3} = 0.0193 = 1.93\%$

For mode 4, $\frac{M_4}{M} = \frac{0.34}{299.3} = 0.0011 = 0.11\%$

3.3.4 Step 4: Determination of lateral force at each floor in each mode

The design lateral force (Q_{ik}) at floor i in mode k is given by,

$Q_{ik} = A_k \, \Phi_{ik} \, P_k \, W_i$

where,

A_k = Design horizontal acceleration spectrum value as per clause 6.4.2 of IS 1893 (Part 1): 2002 using the natural period of vibration (T_k) of mode k,

The design horizontal seismic coefficient A_h for various modes are,

$$A_{hk} = \frac{Z}{2}\,\frac{I}{R}\,\frac{S_{ak}}{g}$$

$$A_{h1} = \frac{Z}{2}\,\frac{I}{R}\,\frac{S_{a1}}{g} = \frac{0.36}{2}\,\frac{1}{5}\;1.433 = 0.0515$$

$$A_{h2} = \frac{Z}{2}\,\frac{I}{R}\,\frac{S_{a2}}{g} = \frac{0.36}{2}\,\frac{1}{5}\;2.5 = 0.09$$

Similarly $A_{h3} = 0.09$, $A_{h4} = 0.09$

The average response acceleration coefficient for rock sites as per IS 1893 (Part 1): 2002 is calculated as follows:

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{Bmatrix} 1+1.5T; & 0.00 \le T \le 0.10 \\ 2.5; & 0.10 \le T \le 0.40 \\ 1.00/T; & 0.40 \le T \le 4.0 \end{Bmatrix}$$

For $T_1 = 0.6978 \Rightarrow \frac{S_{a1}}{g} = 1.433$

For $T_2 = 0.2450 \Rightarrow \frac{S_{a2}}{g} = 2.5$

For $T_3 = 0.1636 \Rightarrow \frac{S_{a3}}{g} = 2.5$

For $T_4 = 0.1382 \Rightarrow \frac{S_{a4}}{g} = 2.5$

Design lateral force in each mode

$Q_{i1} = (A_1\ P_1\ \Phi_{i1}\ W_i)$

$$[Q_{i1}] = \begin{bmatrix} (A_{h1} & P_1 & \Phi_{11} & W_1) \\ (A_{h1} & P_1 & \Phi_{21} & W_2) \\ (A_{h1} & P_1 & \Phi_{31} & W_3) \\ (A_{h1} & P_1 & \Phi_{41} & W_4) \end{bmatrix} =$$

$$\begin{bmatrix} ((0.0515) & (-14.45) & (-0.0328) & (82.91 \times 9.81)) \\ ((0.0515) & (-14.45) & (-0.0608) & (82.91 \times 9.81)) \\ ((0.0515) & (-14.45) & (-0.0798) & (82.91 \times 9.81)) \\ ((0.0515) & (-14.45) & (-0.0872) & (50.57 \times 9.81)) \end{bmatrix}$$

$$= \begin{bmatrix} (19.852) \\ (36.8) \\ (48.3) \\ (32.192) \end{bmatrix} \text{kN}$$

Similarly, $[Q_{i2}] = \begin{bmatrix} 23.62 \\ 19.13 \\ -8.11 \\ -15.68 \end{bmatrix}$, $[Q_{i3}] = \begin{bmatrix} 12.42 \\ -8.30 \\ -6.88 \\ 7.86 \end{bmatrix}$, $[Q_{i4}] = \begin{bmatrix} 1.511 \\ -2.62 \\ 3.04 \\ -1.615 \end{bmatrix}$

3.3.5 Step 5: Determination of storey shear forces in each mode

The peak shear force is given by,

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

The storey shear forces for the first mode is,

$$V_{i1} = \sum_{j=i+1}^{n} Q_{i1} = \begin{bmatrix} V_{11} \\ V_{21} \\ V_{31} \\ V_{41} \end{bmatrix} = \begin{bmatrix} (Q_{11} + Q_{21} + Q_{31} + Q_{41}) \\ (Q_{21} + Q_{31} + Q_{41}) \\ (Q_{31} + Q_{41}) \\ (Q_{41}) \end{bmatrix} = \begin{bmatrix} 137.144 \\ 117.292 \\ 80.492 \\ 32.192 \end{bmatrix} \text{kN}$$

Similarly,

$V_{i2} = \begin{bmatrix} 18.96 \\ -4.66 \\ -23.79 \\ -15.68 \end{bmatrix}$, $V_{i3} = \begin{bmatrix} 5.1 \\ -7.32 \\ 0.98 \\ 7.86 \end{bmatrix}$, $V_{i4} = \begin{bmatrix} 0.316 \\ -1.195 \\ 1.425 \\ -1.615 \end{bmatrix}$

3.3.6 Step 6: Determination of storey shear force due to all modes

The peak storey force (V_1) in storey i due to all modes considered is obtained by combining those due to each mode in accordance with modal combination i.e. SRSS (Square Root of Sum of Squares) or CQC (Complete Quadratic Combination) methods.

Square root of sum of squares (SRSS)

If the building does not have closely spaced modes, the peak response quality (λ) due to all modes considered shall be obtained as,

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

where,

λ_k = Absolute value of quantity in mode 'k', and r is the numbers of modes being considered

Using the above method, the storey shears are,

$V_1 = [(V_{11})^2 + (V_{12})^2 + (V_{13})^2 + (V_{14})^2]^{1/2}$

$= [(137.144)^2 + (18.96)^2 + (5.1)^2 + (0.316)^2]^{1\backslash 2} = 138.54$kN

$V_2 = [(V_{21})^2 + (V_{22})^2 + (V_{23})^2 + (V_{24})^2]^{1/2}$

$= [(117.292)^2 + (-4.66)^2 + (-7.32)^2 + (-1.195)^2]^{1\backslash 2} = 117.61$kN

$V_3 = [(V_{31})^2 + (V_{32})^2 + (V_{33})^2 + (V_{34})^2]^{1/2}$

$= [(80.492)^2 + (-23.79)^2 + (0.98)^2 + (1.425)^2]^{1\backslash 2} = 83.95$kN

$V_4 = [(V_{41})^2 + (V_{42})^2 + (V_{43})^2 + (V_{44})^2]^{1/2}$

$= [(32.192)^2 + (-15.68)^2 + (7.86)^2 + (-1.615)^2]^{1\backslash 2} = 36.69$kN

3.3.7 Step 7: Determination of lateral forces at each storey

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

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

Square root of sum of squares (SRSS)

$F_{roof} = F_4 = V_4 = 36.69$kN

$F_{floor3} = F_3 = V_3 - V_4 = 83.95-36.69 = 47.26$kN

$F_{floor2} = F_2 = V_2 - V_3 = 117.61-83.95 = 33.66$kN

$F_{floor1} = F_1 = V_1 - V_2 = 138.54-117.61 = 20.93$kN

B:Frame considering the stiffness of infills

The frame considered in previous section is again analysed by considering the stiffness of infill walls. The infill is modelled as equivalent diagonal strut.
The mass matrix [M] for the lumped plane frame model is

$$M = \begin{bmatrix} M_1 & 0 & 0 & 0 \\ 0 & M_2 & 0 & 0 \\ 0 & 0 & M_3 & 0 \\ 0 & 0 & 0 & M_4 \end{bmatrix} = \begin{bmatrix} 82.91 & 0 & 0 & 0 \\ 0 & 82.91 & 0 & 0 \\ 0 & 0 & 82.91 & 0 \\ 0 & 0 & 0 & 50.57 \end{bmatrix} \text{ ton}$$

Column stiffness of storey,

$k = 12EI / L^3 = (12 \times 22360 \times 10^3 \times (0.3 \times 0.5^3 / 12)) / 3.5^3 = 19556.85$kN/m

Stiffness of infill is determined by modelling the infill as an equivalent diagonal strut in which

Width of strut, $W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2}$

$$\alpha_h = \frac{\Pi}{2}\left[\frac{E_f I_c h}{2E_m t \sin 2\theta}\right]^{\frac{1}{4}}, \quad \alpha_l = \Pi\left[\frac{E_f I_b l}{E_m t \sin 2\theta}\right]^{\frac{1}{4}}, \quad \theta = \tan^{-1}\frac{h}{l}$$

Where,

E_f = Elastic modulus of frame material = 22360N/m^2
E_m= Elastic modulus of masonry wall = 13800N/m^2
t = Thickness of infill wall = 300mm
h = Height of infill wall = 3.5m
l = Length of infill wall = 5m

I_c = Moment of inertia of columns = $\frac{.3 \times .5^3}{12}$ = 0.003125 m^4

I_b = Moment of inertia of beams = $\frac{.3 \times .45^3}{12}$ = 0.002278 m^4

$$\alpha_h = \frac{\Pi}{2}\left[\frac{22360 \times 0.003125 \times 3.5}{2 \times 13800 \times 0.3 \times \sin 2(35)}\right]^{\frac{1}{4}} = 0.69 \text{ m}$$

$$\alpha_l = \Pi\left[\frac{22360 \times 0.002278 \times 5}{13800 \times 0.3 \times \sin 2(35)}\right]^{\frac{1}{4}} = 1.66 \text{ m}$$

$$W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2} = 0.8988 \text{ m}$$

A = Cross sectional area of diagonal stiffness = W×t = 0.8988×0.3 = 0.2696m^2

l_d = Diagonal length of strut = $\sqrt{h^2 + l^2}$ = 6.103 m

Therefore, stiffness of infill is

$$\frac{AE_m}{l_d}\cos^2\theta = \frac{0.2696 \times 13800 \times 10^6}{6.103} 0.819^2 = 408905.929\times10^3 \text{ N/m}$$

For the frame with two bays there are two struts participating in one direction, total lateral stiffness of each storey

$k_1 = k_2 = k_3 = k_4 = 3 \times 19556.85 + 2 \times 408905929 = 817870.5286$kN/m

Stiffness matrix [K] of lumped mass model is,

$$K = \begin{bmatrix} k_1 + k_2 & -k_2 & 0 & 0 \\ -k_2 & k_2 + k_3 & -k_3 & 0 \\ 0 & -k_3 & k_3 + k_4 & -k_4 \\ 0 & 0 & -k_4 & k_4 \end{bmatrix} =$$

$$\begin{bmatrix} 1.6357 & -0.8178 & 0 & 0 \\ -0.8178 & 1.6357 & -0.8178 & 0 \\ 0 & -0.8178 & 1.6357 & -0.8178 \\ 0 & 0 & -0.8178 & 0.8178 \end{bmatrix} \times 10^6 \text{kN/m}$$

For the above stiffness and mass matrices, eigenvalues and eigenvectors are,

$$\left|K - \omega^2 m\right| = \begin{vmatrix} 2k - \omega^2 m & -k_2 & 0 & 0 \\ -k_2 & 2k - \omega^2 m & -k_3 & 0 \\ 0 & -k_3 & 2k - \omega^2 m & -k_4 \\ 0 & 0 & -k_4 & k - \omega^2 m \end{vmatrix} = 0, \quad k/m = \omega_n^2$$

Therefore , quadratic equation is,

$$(\omega_n^2)^4 - 8.3(\omega_n^2)^3 (\omega^2) + 10.75(\omega^2{}_n)^2 (\omega^2)^2 - 4.45(\omega_n^2)(\omega^2)^3 + 0.575(\omega^2)^4 = 0$$

Eigenvalues

$$[\omega^2] = \begin{bmatrix} 1442 & & & \\ & 11698 & & \\ & & 26227 & \\ & & & 36719 \end{bmatrix}$$

$\omega_1^2 = 1442$, $\omega_2^2 = 11698$, $\omega_3^2 = 26227$, $\omega_4^2 = 36719$

Eigenvectors {Φ}

$$\{\Phi\} = \{\Phi_1\ \Phi_2\ \Phi_3\ \Phi_4\} = \begin{bmatrix} -0.0328 & 0.0795 & 0.0808 & -0.0397 \\ -0.0608 & 0.0644 & -0.0540 & 0.0690 \\ -0.0798 & -0.0273 & -0.0448 & -0.0799 \\ -0.0872 & -0.0865 & 0.0839 & 0.0696 \end{bmatrix}$$

Natural frequency in various modes

$$[\ \omega] = \begin{bmatrix} 37.975 & 0 & 0 & 0 \\ 0 & 108.157 & 0 & 0 \\ 0 & 0 & 161.947 & 0 \\ 0 & 0 & 0 & 191.621 \end{bmatrix} \text{rad/s}$$

Natural time period

$$T = \begin{bmatrix} 0.1655 & 0 & 0 & 0 \\ 0 & 0.0581 & 0 & 0 \\ 0 & 0 & 0.0388 & 0 \\ 0 & 0 & 0 & 0.0328 \end{bmatrix} \text{s}$$

Modal Participation Factors

$$p_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

$$p_1 = \frac{\sum_{i=1}^{4} W_i \phi_{i1}}{\sum_{i=1}^{4} W_i (\phi_{i1})^2} = \frac{(W_1\phi_{11} + W_2\phi_{21} + W_3\phi_{31} + W_4\phi_{41})}{(W_1(\phi_{11})^2 + W_2(\phi_{21})^2 + W_3(\phi_{31})^2 + W_4(\phi_{41})^2)} = -14.45$$

$$p_2 = \frac{\sum_{i=1}^{4} W_i \phi_{i2}}{\sum_{i=1}^{4} W_i (\phi_{i2})^2} = \frac{(W_1\phi_{12} + W_2\phi_{22} + W_3\phi_{32} + W_4\phi_{42})}{(W_1(\phi_{12})^2 + W_2(\phi_{22})^2 + W_3(\phi_{32})^2 + W_4(\phi_{42})^2)} = 4.06$$

Similarly,

$p_3 = 2.1$, $p_4 = -0.52$

Modal Mass

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

$$M_1 = \frac{\left[\sum_{i=1}^{4} W_i \phi_{i1}\right]^2}{g\left[\sum_{i=1}^{4} W_i (\phi_{i1})^2\right]}$$

$$M_1 = \frac{[9.81(82.91(-0.0328) + 82.91(-0.0608) + 82.91(-0.0798) + 50.57(-0.0872))]^2}{9.81[9.81(82.91(-0.0328)^2 + 82.91(-0.0608)^2 + 82.91(-0.0798)^2 + 50.57(-0.0872)^2)]}$$

$= 269.85$

$$M_2 = \frac{\left[\sum_{i=1}^{4} W_i \phi_{i2}\right]^2}{g\left[\sum_{i=1}^{4} W_i (\phi_{i2})^2\right]}$$, similarly, $M_2 = 21.42$, $M_3 = 5.78$, $M_4 = 0.34$

Modal contributions of various modes

For mode 1, $\dfrac{M_1}{M} = \dfrac{269.85}{299.3} = 0.90 = 90\%$

For mode 2, $\dfrac{M_2}{M} = \dfrac{21.42}{299.3} = 0.0715 = 7.15\%$

For mode 3, $\dfrac{M_3}{M} = \dfrac{5.78}{299.3} = 0.0193 = 1.93\%$

For mode 4, $\dfrac{M_4}{M} = \dfrac{0.34}{299.3} = 0.0011 = 0.11\%$

Design lateral force at each floor in each mode

The design lateral force (Q_{ik}) at floor i in mode k is given by,

$Q_{ik} = A_k \, \Phi_{ik} \, P_k \, W_i$

The design horizontal seismic coefficient A_h for various modes are,

$A_{hk} = \dfrac{Z}{2} \dfrac{I}{R} \dfrac{S_{ak}}{g}$

$A_{h1} = \dfrac{Z}{2} \dfrac{I}{R} \dfrac{S_{a1}}{g} = \dfrac{0.36}{2} \dfrac{1}{5} \; 2.5 = 0.090$

$A_{h2} = \dfrac{Z}{2} \dfrac{I}{R} \dfrac{S_{a2}}{g} = \dfrac{0.36}{2} \dfrac{1}{5} \; 1.871 = 0.067$

Similarly $A_{h3} = 0.056$, $A_{h4} = 0.053$

The average response acceleration coefficient for rock sites as per IS 1893 (Part 1): 2002 is calculated as follows:

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{Bmatrix} 1+1.5T; & 0.00 \le T \le 0.10 \\ 2.5; & 0.10 \le T \le 0.40 \\ 1.00/T; & 0.40 \le T \le 4.0 \end{Bmatrix}$$

For $T_1 = 0.1655 \Rightarrow \frac{S_{a1}}{g} = 2.5$

For $T_2 = 0.0581 \Rightarrow \frac{S_{a2}}{g} = 1+15T = 1.871$

For $T_3 = 0.0388 \Rightarrow \frac{S_{a3}}{g} = 1+15T = 1.582$

For $T_4 = 0.1382 \Rightarrow \frac{S_{a4}}{g} = 1+15T = 1.49$

Design lateral force in each mode

$Q_{i1} = (A_1\ P_1\ \Phi_{i1}\ W_i)$

$$[Q_{i1}] = \begin{bmatrix} (A_{h1} & P_1 & \Phi_{11} & W_1) \\ (A_{h1} & P_1 & \Phi_{21} & W_2) \\ (A_{h1} & P_1 & \Phi_{31} & W_3) \\ (A_{h1} & P_1 & \Phi_{41} & W_4) \end{bmatrix} =$$

$$\begin{bmatrix} ((0.090) & (-14.45) & (-0.0328) & (82.91\times 9.81)) \\ ((0.090) & (-14.45) & (-0.0608) & (82.91\times 9.81)) \\ ((0.090) & (-14.45) & (-0.0798) & (82.91\times 9.81)) \\ ((0.090) & (-14.45) & (-0.0872) & (50.57\times 9.81)) \end{bmatrix}$$

$$= \begin{bmatrix} (34.69) \\ (64.31) \\ (84.40) \\ (56.25) \end{bmatrix} \text{kN}$$

Similarly, $[Q_{i2}] = \begin{bmatrix} 17.58 \\ 14.24 \\ -6.04 \\ -11.67 \end{bmatrix}$, $[Q_{i3}] = \begin{bmatrix} 7.728 \\ -5.16 \\ -4.28 \\ 4.89 \end{bmatrix}$, $[Q_{i4}] = \begin{bmatrix} 0.88 \\ -1.54 \\ 1.79 \\ -0.95 \end{bmatrix}$

Storey shear forces in each mode

The peak shear force is given by,

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

The storey shear forces for the first mode is,

$$V_{i1} = \sum_{j=i+1}^{n} Q_{i1} = \begin{bmatrix} V_{11} \\ V_{21} \\ V_{31} \\ V_{41} \end{bmatrix} = \begin{bmatrix} (Q_{11}+Q_{21}+Q_{31}+Q_{41}) \\ (Q_{21}+Q_{31}+Q_{41}) \\ (Q_{31}+Q_{41}) \\ (Q_{41}) \end{bmatrix} = \begin{bmatrix} 239.65 \\ 204.96 \\ 140.65 \\ 56.25 \end{bmatrix} \text{kN}$$

Similarly,

$$V_{i2} = \begin{bmatrix} 14.11 \\ -3.47 \\ -17.71 \\ -11.67 \end{bmatrix}, V_{i3} = \begin{bmatrix} 3.178 \\ -4.55 \\ 0.61 \\ 4.89 \end{bmatrix}, V_{i4} = \begin{bmatrix} 0.18 \\ -0.7 \\ 0.84 \\ -0.95 \end{bmatrix}$$

Storey shear force due to all modes

Square root of sum of squares (SRSS)

$V_1 = [(V_{11})^2 + (V_{12})^2 + (V_{13})^2 + (V_{14})^2]^{1/2}$

$= [(239.65)^2 + (14.11)^2 + (3.178)^2 + (0.18)^2]^{1\backslash 2} = 240.08\text{kN}$

$V_2 = [(V_{21})^2 + (V_{22})^2 + (V_{23})^2 + (V_{24})^2]^{1/2}$

$= [(204.96)^2 + (-3.47)^2 + (-4.55)^2 + (-0.7)^2]^{1\backslash 2} = 205.04\text{kN}$

$V_3 = [(V_{31})^2 + (V_{32})^2 + (V_{33})^2 + (V_{34})^2]^{1/2}$

$= [(140.65)^2 + (-17.71)^2 + (0.61)^2 + (0.84)^2]^{1\backslash 2} = 141.76\text{kN}$

$V_4 = [(V_{41})^2 + (V_{42})^2 + (V_{43})^2 + (V_{44})^2]^{1/2}$

$= [(56.25)^2 + (-11.67)^2 + (4.89)^2 + (-0.95)^2]^{1\backslash 2} = 57.66\text{kN}$

Lateral forces at each storey due to all modes

Square root of sum of squares (SRSS)

$F_{roof} = F_4 = V_4 = 57.66kN$

$F_{floor3} = F_3 = V_3 - V_4 = 141.76\text{-}57.66 = 84.1kN$

$F_{floor2} = F_2 = V_2 - V_3 = 205.04\text{-}141.76 = 63.28kN$

$F_{floor1} = F_1 = V_1 - V_2 = 240.08\text{-}205.04 = 35.04kN$

Table 3.2: Comparison of lateral forces by static and dynamic analysis

	By Equivalent Static Lateral Force Method (kN)	By Response Spectrum Method	
		By not considering the stiffness of infills (kN)	By considering the stiffness of infills (kN)
Roof	79.96	36.69	57.66
3rd Floor	73.75	47.26	84.1
2nd Floor	32.78	33.66	63.28
1st Floor	8.19	20.93	35.04

From the above results it was conclude that the value of forces that building can take are coming less in dynamic analysis as compared to static analysis. This shows that if we design building using dynamic analysis though we have to perform more calculations but it give more precise results as compared. So we can say that dynamic analysis is economic

Chapter 4
Seismic Analysis by Staad Pro

4.1 About 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 nonlinear 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 an VB macro system included in the application or other by including OpenSTAAD functionality in applications that themselves include suitable programmable macro systems. Additionally STAAD.Pro has added direct links to applications such as RAM Connection and STAAD.Foundation to provide engineers working with those applications which handle design post processing not handled by STAAD.Pro itself. Another form of integration supported by STAAD.Pro is the analysis schema of the CIMsteel Integration Standard, version 2 commonly known as CIS/2 and used by a number modelling and analysis applications.

4.2 Static Analysis

The above analysis done manually is done by staad pro below and results are obtained.

4.2.1 Source code

```
Staad Space
Start Job Information
Engineer Date 04-Apr-16
End Job Information
Input Width 79
Unit Meter Kn
Joint Coordinates
1 0 0 0; 2 5 0 0; 3 10 0 0; 4 0 3.5 0; 5 5 3.5 0; 6 10 3.5 0; 7 0 7 0; 8 5 7 0;
9 10 7 0; 10 0 10.5 0; 11 5 10.5 0; 12 10 10.5 0; 13 0 14 0; 14 5 14 0;
15 10 14 0; 16 0 0 2.5; 17 5 0 2.5; 18 10 0 2.5; 19 0 3.5 2.5; 20 5 3.5 2.5;
21 10 3.5 2.5; 22 0 7 2.5; 23 5 7 2.5; 24 10 7 2.5; 25 0 10.5 2.5;
26 5 10.5 2.5; 27 10 10.5 2.5; 28 0 14 2.5; 29 5 14 2.5; 30 10 14 2.5;
31 0 0 5; 32 5 0 5; 33 10 0 5; 34 0 3.5 5; 35 5 3.5 5; 36 10 3.5 5; 37 0 7 5;
38 5 7 5; 39 10 7 5; 40 0 10.5 5; 41 5 10.5 5; 42 10 10.5 5; 43 0 14 5;
44 5 14 5; 45 10 14 5;
Member Incidences
1 4 5; 2 5 6; 3 7 8; 4 8 9; 5 10 11; 6 11 12; 7 13 14; 8 14 15; 9 1 4; 10 2 5;
11 3 6; 12 4 7; 13 5 8; 14 6 9; 15 7 10; 16 8 11; 17 9 12; 18 10 13; 19 11 14;
20 12 15; 21 19 20; 22 20 21; 23 22 23; 24 23 24; 25 25 26; 26 26 27; 27 28 29;
28 29 30; 29 16 19; 30 17 20; 31 18 21; 32 19 22; 33 20 23; 34 21 24; 35 22 25;
36 23 26; 37 24 27; 38 25 28; 39 26 29; 40 27 30; 41 34 35; 42 35 36; 43 37 38;
```

```
44 38 39; 45 40 41; 46 41 42; 47 43 44; 48 44 45; 49 31 34; 50 32 35; 51 33 36;
52 34 37; 53 35 38; 54 36 39; 55 37 40; 56 38 41; 57 39 42; 58 40 43; 59 41 44;
60 42 45; 61 4 19; 62 5 20; 63 6 21; 64 7 22; 65 8 23; 66 9 24; 67 10 25;
68 11 26; 69 12 27; 70 13 28; 71 14 29; 72 15 30; 73 19 34; 74 20 35; 75 21 36;
76 22 37; 77 23 38; 78 24 39; 79 25 40; 80 26 41; 81 27 42; 82 28 43; 83 29 44;
84 30 45;
Element Incidences Shell
85 13 14 29 28; 86 28 43 44 29; 87 29 30 45 44; 88 10 11 26 25; 89 11 12 27 26;
90 25 26 41 40; 91 26 27 42 41; 92 7 8 23 22; 93 22 37 38 23; 94 23 24 39 38;
95 8 9 24 23; 96 4 5 20 19; 97 5 6 21 20; 98 19 20 35 34; 99 20 21 36 35;
100 14 15 30 29;
Element Property
85 To 100 Thicknesses 0.15
Define Material Start
Isotropic Concrete
E 2.17185e+007
Poisson 0.17
Density 23.5616
Alpha 1e-005
Damp 0.05
Type Concrete
Strength Fcu 27579
End Define Material
Member Property American
9 To 20 29 To 40 49 To 60 Pris Yd 0.5 Zd 0.3
61 To 84 Pris Yd 0.45 Zd 0.3
1 To 8 21 To 28 41 To 48 Pris Yd 0.4 Zd 0.3
Constants
Material Concrete All
Supports
1 To 3 16 To 18 31 To 33 Fixed
Define 1893 Load
Zone 0.36 Rf 5 I 1 Ss 1 St 1
Selfweight 1
Floor Weight
Yrange 0 0 Fload 1.75 Xrange 3.5 10 Zrange 0 0
Load 1 Loadtype Seismic Title El X +Ve
1893 Load X 1
Load 2 Loadtype Seismic Title El X -Ve
1893 Load X -1
Load 3 Loadtype Seismic Title El Z+Ve
1893 Load Z 1
Load 4 Loadtype Seismic Title El Z-Ve
1893 Load Z -1
Load 5 Loadtype Dead Title Dl
Selfweight Y -1
Member Load
1 To 8 21 To 28 41 To 48 61 To 84 Uni Gy -56
Load 6 Loadtype Live Title Ll
Floor Load
Yrange 3.5 10.5 Fload -1.75 Gy
Load Comb 7 1.5(Dl + Ll +El X+Ve)
1 1.5 5 1.5 6 1.5
Load Comb 8 1.5(Dl + Ll +El Z+Ve)
```

3 1.5 5 1.5 6 1.5
Load Comb 9 1.2(Dl + Ll -El X-Ve)
5 1.2 6 1.2 2 -1.2
Load Comb 10 1.2(Dl + Ll -El Z-Ve)
5 1.2 6 1.2 4 -1.2
Perform Analysis
Perform Analysis Print Statics Check
Load List 7 To 10
Print Joint Displacements List All
Print Member Forces List All
Print Support Reaction List 1 To 3 16 To 18 31 To 33
Print Story Drift
Print Analysis Results
Finish

4.2.2 Results

4.2.2.1 **Table 4.1: Beam member forces**

Beam no.	Max. moment in Z direction (M_z) (kNm)	Max. shear force (kN)
1	215.2	239.92
2	198.23	232.11
3	208.33	236.89
4	207.71	235.74
5	196.88	231.85
6	209.1	236.12
7	192.8	229.21
8	184.43	225.53
21	217.87	244.79
22	201.99	237.28
23	210.68	241.59
24	212.18	241.14
25	199.96	236.87
26	213.94	241.71
27	189.03	227.46
28	182.26	224.42
41	215.2	239.92
42	198.23	232.11
43	208.33	236.89
44	207.71	235.74
45	196.88	231.85
46	209.1	236.12
47	192.8	229.21
48	184.43	225.63
61	252.33	141.5
62	242.73	137.78
63	252.33	141.5
64	233.64	132.34
65	222.78	128
66	233.64	132.34
67	223.605	123.37

68	228.36	127.82
69	221.85	123.34
70	205.335	115.86
71	205.14	117.08
72	204.33	116.14
73	264.105	137.92
74	255.345	135.27
75	264.105	137.92
76	270.69	143.7
77	265.14	142.53
78	270.69	143.7
79	263.22	141.83
80	261.765	142.66
81	263.22	141.83
82	221.775	123.54
83	218.775	122.99
84	221.75	123.54

4.2.2.2 **Table 4.2: Column member forces**

Column no.	**Max. moment in Z direction (M_z) (kNm)**	**Max. moment in Y direction (M_y) (kNm)**	**Max. shear force (P_u) (kN)**
9	65.72	27.71	1461.79
10	47.53	28.18	2528.01
11	83.82	27.71	1551.8
12	93.2	19.04	1109.05
13	42.51	21.57	1897.6
14	120.4	21.74	1171.93
15	85.38	22.19	741.4
16	37.21	24.27	1263.47
17	107.21	24.69	775.48
18	148.25	29.22	359.68
19	27.07	32.12	628.44
20	165.48	31.42	371.26
29	68.91	37.76	1960.17
30	50.32	38.4	3033.68
31	87.99	37.76	2022.94
32	97.35	38.75	1453.15
33	51.02	40.57	2231.64
34	126.12	38.75	1495.34
35	87.59	32.64	959.09
36	44.04	34.02	1474.06
37	110.88	32.64	980.68
38	143.39	20.36	472.36
39	25.35	20.92	747.39
40	161.61	20.36	479.12
49	57.34	39.36	1610.4
50	47.53	41.02	2641.98
51	83.82	39.36	1610.4
52	101.21	47.18	1208.6
53	42.51	50.69	1972.25

54	120.4	47.18	1208.6
55	91.77	47.05	793.17
56	37.21	49.66	1301.25
57	107.21	47.05	793.17
58	152.16	45.41	375.5
59	27.07	48.13	639.19
60	165.48	45.41	375.5

4.3 Response Spectrum Analysis

The above analysis done manually is done by staad pro below and results are obtained.

4.3.1 Source code

```
Staad Space
Start Job Information
Engineer Date 04-Apr-16
End Job Information
Input Width 79
Unit Meter Kn
Joint Coordinates
1 0 0 0; 2 5 0 0; 3 10 0 0; 4 0 3.5 0; 5 5 3.5 0; 6 10 3.5 0; 7 0 7 0; 8 5 7 0;
9 10 7 0; 10 0 10.5 0; 11 5 10.5 0; 12 10 10.5 0; 13 0 14 0; 14 5 14 0;
15 10 14 0; 16 0 0 2.5; 17 5 0 2.5; 18 10 0 2.5; 19 0 3.5 2.5; 20 5 3.5 2.5;
21 10 3.5 2.5; 22 0 7 2.5; 23 5 7 2.5; 24 10 7 2.5; 25 0 10.5 2.5;
26 5 10.5 2.5; 27 10 10.5 2.5; 28 0 14 2.5; 29 5 14 2.5; 30 10 14 2.5;
31 0 0 5; 32 5 0 5; 33 10 0 5; 34 0 3.5 5; 35 5 3.5 5; 36 10 3.5 5; 37 0 7 5;
38 5 7 5; 39 10 7 5; 40 0 10.5 5; 41 5 10.5 5; 42 10 10.5 5; 43 0 14 5;
44 5 14 5; 45 10 14 5;
Member Incidences
1 4 5; 2 5 6; 3 7 8; 4 8 9; 5 10 11; 6 11 12; 7 13 14; 8 14 15; 9 1 4; 10 2 5;
11 3 6; 12 4 7; 13 5 8; 14 6 9; 15 7 10; 16 8 11; 17 9 12; 18 10 13; 19 11 14;
20 12 15; 21 19 20; 22 20 21; 23 22 23; 24 23 24; 25 25 26; 26 26 27; 27 28 29;
28 29 30; 29 16 19; 30 17 20; 31 18 21; 32 19 22; 33 20 23; 34 21 24; 35 22 25;
36 23 26; 37 24 27; 38 25 28; 39 26 29; 40 27 30; 41 34 35; 42 35 36; 43 37 38;
44 38 39; 45 40 41; 46 41 42; 47 43 44; 48 44 45; 49 31 34; 50 32 35; 51 33 36;
52 34 37; 53 35 38; 54 36 39; 55 37 40; 56 38 41; 57 39 42; 58 40 43; 59 41 44;
60 42 45; 61 4 19; 62 5 20; 63 6 21; 64 7 22; 65 8 23; 66 9 24; 67 10 25;
68 11 26; 69 12 27; 70 13 28; 71 14 29; 72 15 30; 73 19 34; 74 20 35; 75 21 36;
76 22 37; 77 23 38; 78 24 39; 79 25 40; 80 26 41; 81 27 42; 82 28 43; 83 29 44;
84 30 45;
Element Incidences Shell
85 13 14 29 28; 86 28 43 44 29; 87 29 30 45 44; 88 10 11 26 25; 89 11 12 27 26;
90 25 26 41 40; 91 26 27 42 41; 92 7 8 23 22; 93 22 37 38 23; 94 23 24 39 38;
95 8 9 24 23; 96 4 5 20 19; 97 5 6 21 20; 98 19 20 35 34; 99 20 21 36 35;
100 14 15 30 29;
Element Property
85 To 100 Thicknesses 0.15
Define Material Start
Isotropic Concrete
E 2.17185e+007
Poisson 0.17
```

```
Density 23.5616
Alpha 1e-005
Damp 0.05
Type Concrete
Strength Fcu 27579
End Define Material
Member Property American
9 To 20 29 To 40 49 To 60 Pris Yd 0.5 Zd 0.3
61 To 84 Pris Yd 0.45 Zd 0.3
1 To 8 21 To 28 41 To 48 Pris Yd 0.4 Zd 0.3
Constants
Material Concrete All
Supports
1 To 3 16 To 18 31 To 33 Fixed
Define 1893 Load
Zone 0.36 Rf 5 I 1 Ss 1 St 1
Selfweight 1
Floor Weight
Yrange 3.5 10.5 Fload 1.75
Load 1 Loadtype Live   Title Load Case 1 Live
Floor Load
Yrange 3.5 10.5 Fload -1.75 Gy
Load 2 Loadtype Seismic   Title Load Case 3 Rs
Self-weight X 1
Selfweight Y 1
Selfweight Z 1
Member Load
1 To 84 Uni Gx 56
1 To 84 Uni Gz 56
Floor Load
Yrange 0 0 Fload 1.75 Gx
Yrange 0 0 Fload 1.75 Gy
Yrange 0 0 Fload 1.75 Gz
Spectrum Srss 1893 X 0.036 Acc Damp 0.05
Soil Type 1
Load 3 Loadtype Dead   Title Load Case 2 Dead
Selfweight Y -1
Member Load
1 To 84 Uni Gy -56
Perform Analysis
Perform Analysis Print Mode Shapes
Print Analysis Results
Finish
```

4.3.2 Results

4.3.2.1 Table 4.3: Beam member forces

Beam no.	Max. moment in Z direction (M_z) (kNm)	Max. shear force (kN)
1	124.7	149.57
2	124.7	149.57
3	119.69	147.42
4	119.69	147.42
5	123.07	148.55
6	123.07	148.55
7	123.3	150.52
8	123.3	150.52
21	124.34	149.39
22	124.34	149.39
23	119.15	147.15
24	119.15	147.15
25	123.79	148.79
26	123.79	148.79
27	122.04	149.85
28	122.04	149.85
41	124.7	149.57
42	124.7	149.57
43	119.69	147.42
44	119.69	147.42
45	123.07	148.55
46	123.07	148.55
47	123.3	150.52
48	123.3	150.52
61	26.87	75.22
62	26.22	74.9
63	26.87	75.22
64	28.55	77.55
65	29.44	78.39
66	28.55	77.55
67	31.53	80.61
68	33.28	82
69	31.53	80.61
70	24.15	77.01
71	23.89	77.32
72	24.15	77.01
73	26.22	75.22
74	26.87	74.9
75	26.87	75.22
76	28.55	77.55
77	29.44	78.39
78	28.55	77.55
79	31.53	80.61
80	33.28	82

81	31.53	80.61
82	24.15	77.01
83	23.89	77.32
84	24.15	77.01

4.3.2.2 **Table 4.4: Column member forces**

Column no.	Max. moment in Z direction (M_z) (kNm)	Max. moment in Y direction (M_y) (kNm)	Max. shear force (P_u) (kN)
9	91.05	7.65	1770.69
10	104.96	7.99	2433.1
11	91.05	7.65	1770.69
12	62.67	13.16	1335.94
13	83.71	13.55	1828.45
14	62.67	13.16	1335.94
15	56.97	15.13	891.06
16	69.37	15.23	1221.59
17	56.97	15.13	891.06
18	99.39	19.77	438.92
19	52.18	20.68	614.33
20	99.39	19.77	438.92
29	95.08	37.76	2061.2
30	110.78	38.4	2746.93
31	95.08	37.76	2061.2
32	61.23	38.75	1537.44
33	99.49	40.57	2038.18
34	61.23	38.75	1537.44
35	54.9	32.64	1022.10
36	81.87	34.02	1357.55
37	54.9	32.64	1022.1
38	94.63	20.36	511.39
39	60.97	20.92	687.46
40	94.63	20.36	511.39
49	44.25	39.36	149.57
50	104.96	41.02	2433.1
51	91.05	39.36	1770.69
52	62.67	47.18	1335.94
53	83.71	50.69	1828.45
54	62.67	47.18	1335.94
55	56.97	47.05	891.06
56	69.37	49.66	1221.59
57	56.97	47.05	891.06
58	99.39	45.41	438.92
59	52.18	48.13	614.33
60	99.39	45.41	438.92

From the above results it was concluded that the member forces of building are coming less in dynamic analysis as compared to static analysis. This shows that if design of building is done using dynamic analysis though to perform more calculations but it give more precise results as compared. So it can be said that dynamic analysis is economic.

Chapter 5

Design of beams and columns by spreadsheet

5.1 Design of beams

Doubly reinforced beams are generally resorted to in situations where the cross-sectional dimensions of the beam are restricted and where singly reinforced sections are not adequate in terms of moment resisting capacity. Doubly reinforced beams are also used in situations where reversal of moments is likely. The presence of compression reinforcement reduces long term deflections due to shrinkage and creep. All compression reinforcement must be enclosed by closed stirrups in order to prevent their possible buckling and to provide some ductility by confinement of concrete.

5.2 Spreadsheet of design of doubly reinforced beam

List of symbols

b = breadth of beam (mm)
D = depth of beam (mm)
d’ = effective depth of beam (mm)
M_u = moment (kNm)
A_{st} = area of steel in tension (mm^2)
n_t = number of bars in tension
$Ø_t$ = diameter of bars in tension (mm)
A_{sc} = area of steel in compression (mm^2)
n_c = number of bars in compression
$Ø_c$ = diameter of bars in compression (mm)
$Ø_\tau$ = diameter of stirrups (mm)
n_{sl} = number of ties
S = spacing of stirrups (mm/cc)
D_x = breadth of column (mm)
D_y = depth of column (mm)
M_{ux} = moment in X direction (kNm)
M_{uy} = moment in Y direction (kNm)
P_u = axial load on compression member (kN)

Table 5.1: Design of beam

	value	**units**	
breadth of beam (b)	300	mm	
depth of beam (D)	400	mm	
characteristic compressive strength of concrete(f_{ck})	25	MPa	
characteristic strength of steel(f_y)	415	MPa	
effective depth (d)	350	mm	
effective span(l)	6000	mm	
factored moment(M_u)	215.2	kNm	
$M_{u,lim}$	127.62	kNm	
check for singly or double reinforced beam	Mu > Mu,lim	Doubly reinforced	
$x_{u,max}/d$	0.48		
$p_{t,lim}$	1.2	%	

Determining A_{st}			
balanced section where x_u=$x_{u,max}$			
$A_{st,lim}$	1260	mm^2	
diameter of bars for tension steel(Ø)	20	mm	
diameter of stirrups($Ø_\tau$)	6	mm	
clear cover provided to the reinforcement	30	mm	
effective cover to reinforcement(d')	46	mm	
$(\Delta A_{st})_{reqd}$	798	mm^2	
$(A_{st})_{reqd}$	2058	mm^2	
number of bars	3		
$Ø_{reqd}$	30	mm	
A_{st}	2121	mm^2	
actual d	349	mm	
as actual d is less than the d assumed earlier so revising the above calculations by d=349mm			
$M_{u,lim}$	126.89	kNm	
$A_{st,lim}$	1256.4	mm^2	
$(\Delta A_{st})_{reqd}$	801	mm^2	
$(A_{st})_{reqd}$	2057.4	mm^2	
Actual $(\Delta A_{st})_{provided}$	864.6	mm^2	
Determining A_{sc}			
assuming x_u=$x_{u,max}$			
d'/d	0.132		
from table A			
f_{sc}	345.82	MPa	
$(A_{sc})_{reqd}$	933	mm^2	
number of bars	5		
$Ø_{reqd}$	16	mm	
A_{sc}	1006	mm^2	
Design Check			
assuming $x_u \leq x_{u,max}$,it suffices to establish $p_c \geq p_c$*			
actual d provided	349	mm	
d'	44	mm	
d'/d	0.127		
from table A			
f_{sc}	346.77	MPa	
actual p_t	2.026	%	
actual p_c	0.961	%	

p_c*	0.889	%	
as p_c > p_c* , Hence OK			
Check for deflection control			
p_t	2.026	%	
f_s	234	MPa	
k_t (from fig. 4 of code IS 456:2000)	0.8468		
p_c	0.961	%	
k_c (from fig.5 of code IS 456:2000)	1.2		
$(l/d)_{max}$	20.33		
$(l/d)_{provided}$	17.2		
as $(l/d)_{max}$ > $(l/d)_{provided}$, Hence OK			
Shear Design			
max. shear stress (τ_{cmax})	3.1	MPa	
factored shear force (V_u)	230.37	kN	
nominal shear stress (τ_v)	2.21	MPa	
β	1.44		
design shear strength of concrete (τ_c)	0.82	MPa	
check whether $\tau_{c,max}$ > τ_c	Yes		
shear force resisted by concrete (V_{uc})	85.86	kN	
check whether V_u > V_{uc}	Yes	Provide 'calculated shear rft.'	
diameter of stirrups($Ø_\tau$)	6	mm	
number of stirrup legs	2		
Minimum shear reinforcement			
stirrup spacing along beam length	171	mm/cc	
check whether stirrup spacing < 300mm	Yes		
provide stirrups of diameter	6 mm	2 legged	171 mm c/c
Calculated shear reinforcement			
shear force taken up by stirrups (V_{us})	144.51	kN	
stirrup spacing along beam length	50	mm/cc	
check whether stirrup spacing < 300mm	Yes		
provide stirrups of diameter	6 mm	2 legged	50 mm c/c

5.3 Design of beams of building

5.3.1 Table 5.2: Beams in transverse direction

Beam No.	bxD mm	d mm	M_u kNm	$M_{u,lim}$ kNm	A_{st} mm^2	n_t	$Ø_t$ mm	A_{sc} mm^2	n_c	$Ø_c$ mm	Shear reinforcement		
											$Ø_τ$ mm	n_{sl}	S mm/cc
1	300x400	349	215.2	127.62	2121	3	30	1006	5	16	6	2	50
2	300x400	349.5	198.23	127.62	1982	3	29	884	5	15	6	2	49
3	300x400	349	208.33	127.62	2121	3	30	1006	5	16	6	2	48
4	300x400	349	207.71	127.62	2121	3	30	1006	5	16	6	2	48
5	300x400	349.5	196.88	127.62	1982	3	29	884	5	15	6	2	49
6	300x400	349	209.1	127.62	2121	3	30	1006	5	16	6	2	48
7	300x400	349.5	192.8	127.62	1982	3	29	884	5	15	6	2	50
8	300x400	350	184.43	127.62	1848	3	28	664	5	13	6	2	50
21	300x400	349	217.87	127.62	2121	3	30	1006	5	16	6	2	45
22	300x400	349.5	201.99	127.62	1982	3	29	884	5	15	6	2	47
23	300x400	349	210.68	127.62	2121	3	30	1006	5	16	6	2	46
24	300x400	349	212.18	127.62	2121	3	30	1006	5	16	6	2	46
25	300x400	349.5	199.96	127.62	1982	3	29	884	5	15	6	2	47
26	300x400	349	213.94	127.62	2121	3	30	1006	5	16	6	2	46
27	300x400	350	189.03	127.62	1848	3	28	664	5	13	6	2	50
28	300x400	350	182.26	127.62	1848	3	28	664	5	13	6	2	51
41	300x400	349	215.2	127.62	2121	3	30	1006	5	16	6	2	47
42	300x400	349.5	198.23	127.62	1982	3	29	884	5	15	6	2	49
43	300x400	349	208.33	127.62	2121	3	30	1006	5	16	6	2	48

44	300x400	349	207.71	127.62	2121	3	30	1006	5	16	6	2	48
45	300x400	349.5	196.88	127.62	1982	3	29	884	5	15	6	2	49
46	300x400	349	209.1	127.62	2121	3	30	1006	5	16	6	2	48
47	300x400	349.5	192.8	127.62	1982	3	29	884	5	15	6	2	50
48	300x400	350	184.43	127.62	1848	3	28	664	5	13	6	2	50

5.3.2 Table 5.3: Beams in longitudinal direction

Beam No.	**bxD** mm	**d** mm	**M_u** kNm	**$M_{u,lim}$** kNm	**A_{st}** mm^2	**n_t**	**$Ø_t$** mm	**A_{sc}** mm^2	**n_c**	**$Ø_c$** mm	**Shear reinforcement**		
											$Ø_\tau$ mm	**n_{sl}**	**S** mm/cc
61	300x450	399	252.33	166.68	2121	3	30	764	3	18	6	2	56
62	300x450	399	242.73	166.68	2121	3	30	764	3	18	6	2	57
63	300x450	399	252.33	166.68	2121	3	30	764	3	18	6	2	56
64	300x450	399.5	233.64	166.68	1982	3	29	604	3	16	6	2	59
65	300x450	399.5	222.78	166.68	1982	3	29	604	3	16	6	2	60
66	300x450	399.5	233.64	166.68	1982	3	29	604	3	16	6	2	59
67	300x450	399.5	223.605	166.68	1982	3	29	604	3	16	6	2	63
68	300x450	399.5	228.36	166.68	1982	3	29	604	3	16	6	2	61
69	300x450	399.5	221.85	166.68	1982	3	29	604	3	16	6	2	63
70	300x450	400	205.335	166.68	1848	3	28	462	3	14	6	2	65
71	300x450	400	205.14	166.68	1848	3	28	462	3	14	6	2	65
72	300x450	400	204.33	166.68	1848	3	28	462	3	14	6	2	65
73	300x450	398.5	264.105	166.68	2265	3	31	943	3	20	6	2	58
74	300x450	398.5	255.345	166.68	2265	3	31	943	3	20	6	2	59

75	300x450	398.5	264.105	166.68	2265	3	31	943	3	20	6	2	58
76	300x450	398.5	270.69	166.68	2265	3	31	943	3	20	6	2	56
77	300x450	398.5	265.14	166.68	2265	3	31	943	3	20	6	2	56
78	300x450	398.5	270.69	166.68	2265	3	31	943	3	20	6	2	56
79	300x450	398.5	263.22	166.68	2265	3	31	943	3	20	6	2	56
80	300x450	398.5	261.765	166.68	2265	3	31	943	3	20	6	2	56
81	300x450	398.5	263.22	166.68	2265	3	31	943	3	20	6	2	56
82	300x450	399.5	221.775	166.68	1982	3	29	604	3	16	6	2	63
83	300x450	399.5	218.775	166.68	1982	3	29	604	3	16	6	2	63
84	300x450	399.5	221.75	166.68	1982	3	29	604	3	16	6	2	63

5.4 Design of columns

A compression member is a structural element which is subjected to axial compressive forces. Compression members are most commonly encountered in reinforced concrete buildings as columns forming part of vertical framing system. Whether the structure is man made or created by nature the key element in resisting collapse under gravity load is the column. The column is representative of all types of compression members and hence sometimes the terms column and compression member are used interchangeably.

5.5 Spreadsheet of design of short columns under axial compression with biaxial bending

Table 5.4 Design of Column		
	values	**units**
breadth of column (b) (D_x)	300	mm
depth of column (D) (D_y)	500	mm
unsupported length of column (l)	3500	mm
P_u	1461.79	kN
M_{ux}	65.72	kNm
M_{uy}	27.71	kNm
characteristic strength of steel(f_y)	415	MPa
characteristic compressive strength of concrete(f_{ck})	25	MPa
Slenderness ratio		

effective length ratio	0.85	
effective length about x-x axis (l_{ex})	2975	mm
effective length about y-y axis (l_{ey})	2975	mm
l_{ex}/D_x	9.92	
l_{ey}/D_y	5.95	
check for l_{ey}/D_y	Yes short column	
check for l_{ex}/D_x	Yes short column	
Check minimum eccentricities		
Applied eccentricities		
e_x	44.96	mm
e_y	18.96	mm
Minimum eccentricities		
$e_{x,min}$	20	mm
$e_{y,min}$	23.67	mm
Longitudinal reinforcement		
M_u	83	kNm
depth of compression reinforcement (d')	50	mm
d'/D	0.1	
p_u	0.39	
m_u	0.03	
from chart 44 of SP16, p/f_{ck}	0.02	
p_{reqd}	0.5	
$A_{s,reqd}$	750	mm^2
number of bars	6	
Ø	18	mm
A_s	1527	mm^2
provide 6 bars of 18mm dia		
$p_{provided}$	1.02	%
p/f_{ck}	0.05	
along x-x axis		
d'/D	0.17	
from SP16 value of m_{ux}	0.06	
M_{ux1}	112.5	kNm
along y-y axis		
d'/D	0.1	

from SP16 value of m_{uy}	0.08	
M_{uy1}	150	kNm
P_{uz}	2145.6	kN
P_u/P_{uz}	0.69	
α_n	1.82	
check safety under biaxial loading	0.43	trial section is safe
Transverse reinforcement		
tie diameter ($Ø_t$)	6	mm
tie spacing (s_t)	288	mm
provide 6 Ø ties @ 288c/c		

5.6 Table 5.5: Design of columns of building

Column No.	**D_x x D_y mm**	**l mm**	**P_u kN**	**M_{ux} kNm**	**M_{uy} kNm**	**M_u kNm**	**Longitudinal reinforcement**			**Transverse reinforcement**	
							A_s mm^2	**Ø mm**	**No.of bars**	**$Ø_t$ mm**	**S mm/cc**
9	300 x 500	3500	1461.79	65.72	27.71	83	1527	18	6	6	288
10	300 x 500	3500	2528.01	47.53	28.18	64	3927	25	8	8	300
11	300 x 500	3500	1551.8	83.82	27.71	102	1527	18	6	6	288
12	300 x 500	3500	1109.05	93.2	19.04	110	1527	18	6	6	288
13	300 x 500	3500	1897.6	42.51	21.57	55	1885	20	6	6	300
14	300 x 500	3500	1171.93	120.4	21.74	141	1527	18	6	6	288
15	300 x 500	3500	741.4	85.38	22.19	102	1527	18	6	6	288

16	300 x 500	3500	1263.47	37.21	24.27	52	1527	18	6	6	288
17	300 x 500	3500	775.48	107.21	24.69	127	1527	18	6	6	288
18	300 x 500	3500	359.68	148.25	29.22	174	1527	18	6	6	288
19	300 x 500	3500	628.44	27.07	32.12	49	1527	18	6	6	288
20	300 x 500	3500	371.26	165.48	31.42	194	1527	18	6	6	288
29	300 x 500	3500	1960.17	68.91	37.76	91	2946	25	6	6	300
30	300 x 500	3500	3033.68	50.32	38.4	73	4909	25	10	6.25	300
31	300 x 500	3500	2022.94	87.99	37.76	111	2946	25	6	6.25	300
32	300 x 500	3500	1453.15	97.35	38.75	121	1527	18	6	6	288
33	300 x 500	3500	2231.64	51.02	40.57	75	2946	25	6	6.25	300
34	300 x 500	3500	1495.34	126.12	38.75	152	1527	18	6	6	288
35	300 x 500	3500	959.09	87.59	32.64	108	1527	18	6	6	288
36	300 x 500	3500	1474.06	44.04	34.02	64	1527	18	6	6	288
37	300 x 500	3500	980.68	110.88	32.64	133	1527	18	6	6	288
38	300 x 500	3500	472.36	143.39	20.36	167	1527	18	6	6	288
39	300 x 500	3500	747.39	25.35	20.92	38	1527	18	6	6	288
40	300 x 500	3500	479.12	161.61	20.36	187	1527	18	6	6	288

49	300 x 500	3500	1610.4	57.34	39.36	80	1527	18	6	6	288
50	300 x 500	3500	2641.98	47.53	41.02	73	7069	30	10	7.5	300
51	300 x 500	3500	1610.4	83.82	39.36	107	1527	18	6	6	288
52	300 x 500	3500	1208.6	101.21	47.18	129	1527	18	6	6	288
53	300 x 500	3500	1972.25	42.51	50.69	77	2946	25	6	6.25	300
54	300 x 500	3500	1208.6	120.4	47.18	149	1527	18	6	6	288
55	300 x 500	3500	793.17	91.77	47.05	119	1527	18	6	6	288
56	300 x 500	3500	1301.25	37.21	49.66	72	1527	18	6	6	288
57	300 x 500	3500	793.17	107.21	47.05	135	1527	18	6	6	288
58	300 x 500	3500	375.5	152.16	45.41	183	1527	18	6	6	288
59	300 x 500	3500	639.19	27.07	48.13	64	1527	18	6	6	288
60	300 x 500	3500	375.5	165.48	45.41	198	1527	18	6	6	288

Conclusions

From the work presented above the following conclusions can be made :

1. From the Table 3.2 it was concluded that the value of forces that building can take are coming less in dynamic analysis as compared to static analysis. This shows that if design of building is done using dynamic analysis though we have to perform more calculations but it give more precise results as compared.

2. While carrying out response spectrum analysis consideration of infill wall in stiffness leads to better estimation of earthquake forces.

3. Software analysis and design using staad pro saves a lot of time spent in computation of results.

4. From the Table 4.1, Table 4.2, Table 4.3 and Table 4.4 it was concluded that response spectrum analysis is more economical.

   In work presented above design, spread sheets for doubly reinforced beam and biaxial column were prepared. Their reinforcement detail is calculated from spread sheet and the results were tabulated.

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