iJRASET
International Journal For Research in Applied Science and Engineering Technology



INTERNATIONAL JOURNAL FOR RESEARCH

IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

Volume: 10 **Issue:** I **Month of publication:** January 2022

DOI: https://doi.org/10.22214/ijraset.2022.39972

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International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 10 Issue I Jan 2022- Available at www.ijraset.com

Study on Design of Highrise Steel Building Frames and Its Seismic Evaluation

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***Abstract:** Seismic Response Analysis of the given structural system is considered to be a subset of stability and configurationally analysis of the system. It is subjected under the calculations obtained in the form of structural response of the buildings produced by the earthquakes. It is found to be the essential and prime component of the designing phenomenon of the structures, or assessment of structural configuration and as a retrofit for the seismic prone regions of the India.*

The most severe types of earthquakes are essentially emerged closely nearer to the boundaries of the tectonic plates that are covered in the form of globe surface. These plates tries to move with respect one another in the form of relative motion but are resisted by doing so in terms of generation of friction until or unless the various stresses produced between the plates under epicenter reach to such a extent that the movement between the plates gets generated suddenly, which is termed as the earthquake. The local shock produces waves inside the ground which further propagates towards the surface of earth by the movement creation at the structural bases. The intensity of waves decreases with the propagation of distance as we move away from the point of epicenter. Hence, it results into the formation and existence of such regions on the earth surface having very high or less risk of seismic assessment, which indirectly depends upon the proximity to the main tectonic plates boundaries.

Apart from the major earthquakes that generally exists near the boundaries of tectonic plates, there are some other earthquakes, that may have their origin and formation at the interior parts of tectonic plates nearer to the fault lines. Such kind of earthquakes are Called as Intra-plate earthquakes. These earthquakes consist of lesser energy, but may be destructive to the area nearer to the point of epicenter

The earthquake imparts the action towards the structure in the form of shaking of ground and its motion in the form of vertical and horizontal components. The horizontal component consists of very high strength and the structures are usually designed to resist gravitational forces than horizontal forces, and as a results, it is the most severe component. The vertical components of the earthquake force are found to be 50% of its value in horizontal direction except in the region of point of epicenter because in the region of point of epicenter, both the components are of same orders.

I. INTRODUCTION

A study has been conducted based on the guidelines over the dowels of the epoxy grouted in earthquake strengthening projects [Wylli Jr. 1988]. Frequently, it is mandatory to strengthened existing concrete structures for improved seismic performance, either after a damaging earthquake or in preparation for a future extent. This work includes the attachment of new steel or concrete member to the pre-existing structures. The epoxy grouted dowels are found to be ideal for the above said task due to its high strength and installation ease. The short term load implementation on the dowels in the form of seismic loading concludes in the form of creep concern and as the dowels are grouted in the concrete, it is also required to impart an insulation to prevent the epoxy from the exposure heat such as fires.

As per Amr S Elnashai (2000), the past earthquakes have been resulted into a specific conclusion that the internal earthquake response and resistance to the steel structures must not be taken lightly. These structures require a special attention towards their design during earthquake. As per the study, it was earlier reported that the damage due to earthquake is strictly existed on the connections of the structures; related efforts and impacts in the many countries have resulted into some of the serious modifications in their designing and quality control measure practices. The other development that affected the steel structure designing is the performance evaluation in the displacement design method and the procedures to generalize the concepts of limit state methods into the framework of performance-based design method. This present paper presents the recent innovations; research, findings related to the material used, their section, connections between members, etc. are reviewed and analyzed with the various codal provisions of the different countries including India. It also deals with the implementations of investigations related to the various steel frames along-with the bare slabs and composite slabs.

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II. CONTENT

A. Lateral Force Method

The seismic resistant structure designs must follow the dynamic nature of the load as the seismic loads have the dynamic nature in themselves. But in case of simple regular structures, design by the use of equivalent linear static methods may also be followed. Th is has been permitted in almost all the codes of practice worldwide for regular structures having low to medium rise buildings.
For the analysis of such structures using linear static method, the structure is assumed to be treated as a discrete structure that consi sts of concentrated loads at the floor levels in the form of self-
weight of walls, columns, etc. These loads should be uniformly distributed to floors below and above the storey. Moreover, some a mple amount of imposed load is also considered at floor levels. This method includes the following two steps for load calculation a nd distributions.

1) Calculation of Base Shear and its distribution along the height of the structure.
2) Calculation of lateral forces and its distribution to the each floor of the building.

However, this method assumes that the response of the various building is always in the terms of its fundamental mode of frequenc y. The Linear Static method always ignores the non-linear behaviour of the structure and its corresponding dynamic effect.

B. Design Seismic Base Shear

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

$$V_B = A_h * W$$

Where, A_h= design horizontal acceleration spectrum.
W = seismic weight of all the floors.

C. Fundamental Natural Period

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

$$Ta = 0.075\ h^{0.75} \text{ for RC frame building}$$
$$Ta = 0.085\ h^{0.75} \text{ for steel frame building}$$

Where, h = Height of building, in m. This excludes the basement storey, where basement walls are connected with the ground floo r deck or fitted between the building columns. But it includes the basement storey, when they are not so connected.
The approximate fundamental natural period of vibration **(**T_a**),** in seconds, of all other buildings, including moment-
resisting frame buildings with brick lintel panels, may be estimated by the empirical Expression

$$T = .09H/\sqrt{D}$$

Where, H = Height of building in meter.
D = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

D. Design Spectrum

The purpose of determining seismic forces, our country has been classified into four seismic zones. The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression,

$$A_h = Z*I*S_a / 2*R*G$$

Provided that for any structure with T ≤0.1 s, the value of Ah will not be taken less than Z/2 whatever be the value of I/R.
Z= Zone factor given in Table 2, is defined for the Maximum Considered Earthquake (MCE) and service life of structure in that zo ne. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the fa ctor for Design Basis Earthquake (DBE)
I = Importance factor, depending upon the functional use of the buildings, which are characterized by hazardous consequences of it s 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, characterized 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 and Table 4.1 based on appropriate natural periods and damp ing of the structure. These curves Represent free field ground motion**.**

ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 10 Issue I Jan 2022- Available at www.ijraset.com

Table 1: Multiplying Factors for Obtaining Values for Other Damping (Clause 6.3.2)

Damping Present	0	2	5	7	10	15	20	25	30
Factor	3.2	1.4	1	0.9	0.8	0.7	0.6	0.55	0.5

The design acceleration spectrum for vertical motions, when required, may be taken as two thirds of the design horizontal accelerat ion spectrum specified in Clause 6.3.2.

E. For Medium Soil Sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15T & 0.00 \le T \le 0.10 \\ 2.5 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 3.00 \end{cases}$$

F. Seismic Weight

The seismic weight of the complete structure is the sum of seismic weights of all the different floors of the structure. As per Claus e 7.4.2 of IS: 1893 (Part 1)-
2002, the seismic load of each floor is addition of all the dead load and the appropriate amount of imposed load. The latter part is considered to be the part of the imposed loads that can be assumed to be attached to the structure at the time of earthquake shaking . It includes the weight of permanent and movable partitions, permanent equipment, a part of the live load, etc. While computing th e seismic weight of each floor, the weight of columns and walls in any storey should be equally distributed to the floors above and below the storey. Any weight supported in between storeys should be distributed to the floors above and below in inverse proporti on to its distance from the floors. As per IS: 1893 (Part 1)-
2002, the percentage of imposed load as given in code should be used. For calculating the design seismic forces of the structure, th e imposed load on the roof need not be considered.

G. Distribution of Design Forces

The vertical distribution of base shear at different floor level . The design base shear (V_B) calculated shall be distributed along the height of the building as

$$Q_i = \frac{V_B W_i h_i^2}{\sum W_i h_i^2}$$

Where, W_i = Seismic weight of floor i
Q_i = Design lateral force at floor i,
H_i = Height of floor i measured from base, and
N = Number of storey in the building is the number of levels at which the masses are located
In our case the value of T_a=0.09 X 18 / √24 = 0.33Hz.

After calculation of seismic forces that acts at various levels along the height of the structure, the various lateral forces and bending moments in all the different members may easily be calculated by the use of the standard computer application. The structure should be designed in such a way it must resist all the overturning impacts and lateral storey drifts due to lateral seismic forces. To ensure this type of consideration, the various forces and bending moments in the members are calculated by the use of P-Δ effect. The IS: 1893 – 2002 estimates the value of lateral storey drift in the structural storey due to specified lateral loads, having the 1.0 as the partial load factor and it must not be more than 0.4 % of the height of the storey.

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Table 2: Analysis of Frames by Lateral Force Method

Storey no.	Absolute displacement of storey D_i (m)	Design inter storey drift D_r (m)	Storey lateral force V_{tot} (KN)	Shear at storey P_{tot} (KN)
1	0.003728	0.003728	1.978	179.210
2	0.012569	0.008726	7.951	177.232
3	0.023865	0.011242	17.83	169.281
4	0.035892	0.012055	31.657	151.451
5	0.047566	0.011674	49.212	119.794
6	0.058123	0.010557	70.582	70.582

III. RESPONSE SPECTRUM ANALYSIS

Response spectrum analysis is one of the most popular methods in the field of seismic analysis. The diagram which is used to perform it is design spectrum. The ennoblement of multi storey buildings is used by response spectrum analysis by a basic assumption. The assumption used is that the mass is combined at the roof diaphragm levels and at the floor levels. The diaphragms are assumed as infinitely rigid and the column axially inextensible but laterally flexible. The dynamic response of the spectrum is determined in the form of lateral displacements of the combined mass with the degrees of dynamic freedom (or modes of vibration n) being equal to the number of masses. The un-damped analysis of the building can be done following standard methods of mechanics using appropriate masses and elastic stiffness of the structural system, and the natural period (T) and mode shapes of the modes in vibration can be obtained. The mode shapes can be determined by distribution of mass and the stiffness of buildings

Superposition of the vibrations of each individual combined mass gives the ground motion which is applied at the base of multi mass system, the deflected shape and combination of all mode shapes. Dynamic response of multi-degree of freedom system is determined by modal analysis procedure. IS 1893 recommends modal analysis is discussed herewith.

Each individual mode of vibration has its unique period of vibration (with its own shape called mode shape formed by locus of points of the deflected masses.)

Response is determined by using different modal combination methods such as square-root-of-sum- of-squares method (SRSS) or the complete quadratic method (CQC) which are used when natural periods of the different modes are well separated (when they differ by 10% of the lower frequency and the damping ratio does not exceed 5%. IS 1893 recommends the CQC method that can be considered as the modal coupling method of design and analysis of members.

Table 3: Analysis by Response Spectrum Method.

Storey no.	Absolute displacement of storey D_i(m)	Design inter storey drift D_r (m)	Storey lateral force V_{tot} (KN)	Shear at storey P_{tot} (KN)
1	0.00491	0.00491	1.877	120.981
2	0.0115	0.0066	6.112	119.104
3	0.0161	0.0046	10.651	112.992
4	0.0196	0.0035	17.331	102.341
5	0.0219	0.0023	29.98	85.01
6	0.0234	0.0015	55.03	55.03

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Table 4: Base Shear and Mass Participation Factor

MODE	BASE SHEAR(KN)	Mass participation factor
1	252.75	85.33
2	27.8	8.13
3	12.1	3.54
4	0	0
5	0.02	0.01
6	5.85	2.04

A. Analysis

It has been observed that the a rapid fall in both the values i.e., Base Shear and Mass Participation Factor takes place from Mode 1 to Mode 2. This sudden decrease then follows gradual decrease and then increase in both the values. It is interesting to note that the both values gets zero at mode 4. These variations have been shown in the fig. 1 and 2 for base shear and mass participation factor respectively.

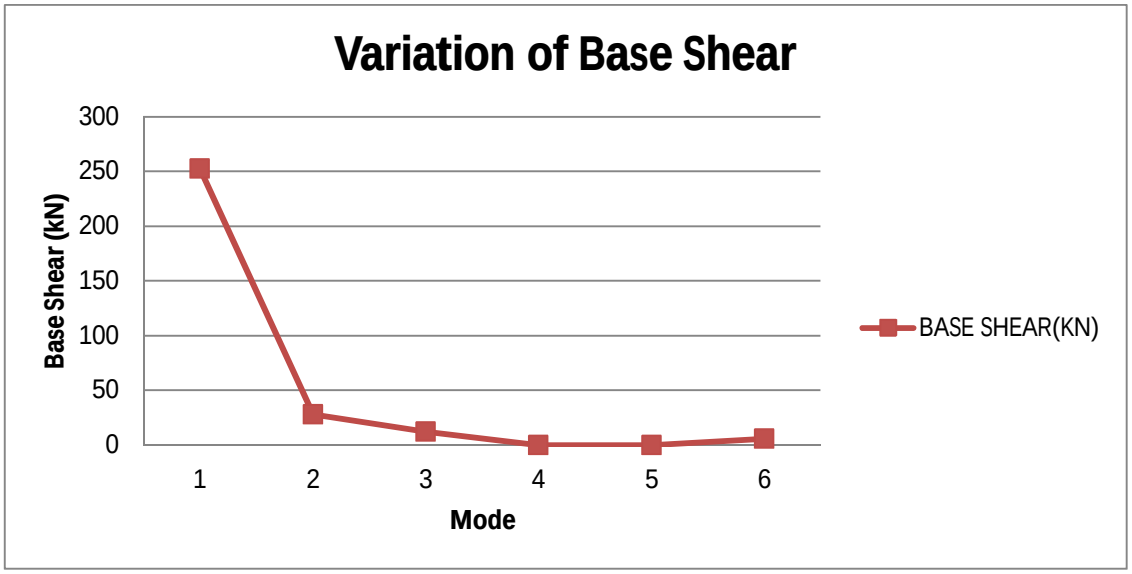


Fig. 1: Graphical Variation of Base Shear of Structure with respect to Mode

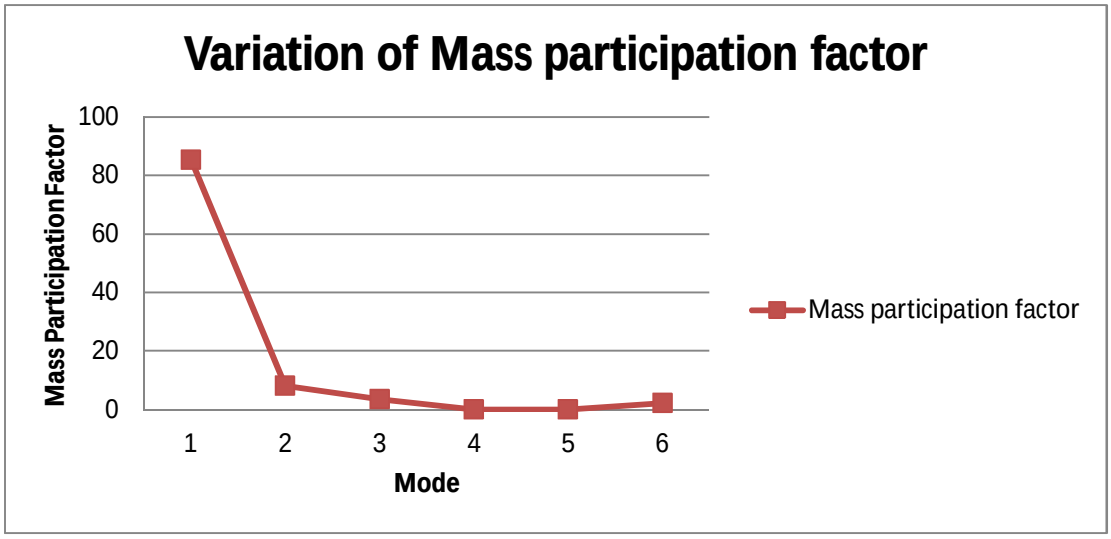


Fig. 2: Graphical Variation of Mass Participation Factor with respect to Mode

ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 10 Issue I Jan 2022- Available at www.ijraset.com

B. P - Δ Analysis

The P-Δ effect refers to the additional moment produced by the vertical loads and the lateral deflection of the column or other elements of the building resting lateral forces.

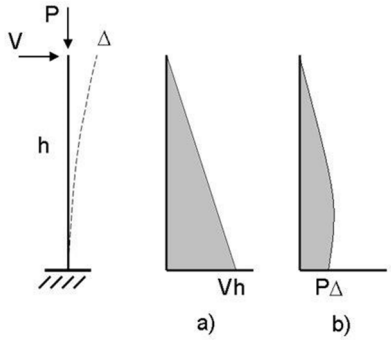


Fig. 3 P-Δ Variation of Column

Under the application of this load, the column is subjected to its corresponding drift or lateral displacement. In this present case of P - Δ effect, the secondary bending moment is calculated as $M_s = P\Delta$; which can be resisted by the additional shear in the column i.e., $\frac{P\Delta}{L}$. The above said secondary moment is used to impart the storey drift to the column and results in the increase of secondary moment and storey shear to the column.

Final drift represents the algebraic summation of the addtional drifts that are resulting from the rise of overturning moment to the already obtained primary drifts, which can be mathematically expressed as

$$\Delta_{tx} = \Delta_x \left(\frac{1}{1-\theta_x}\right) \text{ where, } \theta_x = \frac{M_{x`}}{M_x} = \frac{P_x \Delta_x}{V_x H_x}$$

Where, $M_{x`}$ represents the Secondary Bending Moment

P_x represents the Total weight of the structure (Combination of DL and LL) at any level X

H_x is the Storey Height X

V_x represents the shear force of the storey X

Δ_x is the Storey Drift X

The UBC Code typically specifies that if the ratio of secondary to primary bending moment of any storey is lower than unity, the P - Δ analysis cannot be implemented to the structure.

Table 5: Correction for P-Δ Effect (Lateral Force Method)

Storey no:	Absolute displacement of the storey D_i (m)	Design inter storey drift D_r(m)	Storey lateral forces	Shear at storey V_{tot}(KN)	Total cumulative gravity load at storey P_{tot} (KN)	Storey height: H_i(m)	Inter storey drift sensitivity coefficient: (θ)
1	0.003869	0.003869	1.969	179.201	7344	3	0.05285
2	0.012595	0.008726	7.951	177.232	6120	3	**0.10043***
3	0.023837	0.011242	17.83	169.281	4896	3	**0.10838***
4	0.035892	0.012055	31.657	151.451	3672	3	0.09742
5	0.047566	0.011674	49.212	119.794	2448	3	0.07951
6	0.058123	0.010557	70.582	70.582	1224	3	0.06102

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Table 6: Correction for P-Δ effect, (Response Spectrum Analysis)

Storey no:	Absolute displacement of the storey D_i (m)	Design inter storey drift D_r(m)	Storey lateral forces	Shear at storey V_{tot}(KN)	Total cumulative gravity load at storey P_{tot} (KN)	Storey height: H_i(m)	Inter storey drift sensitivity coefficient: (θ)
1	0.00491	0.00491	1.877	120.981	7344	3	0.09935
2	0.0115	0.0066	6.112	119.104	6120	3	**0.11304***
3	0.0161	0.0046	10.651	112.992	4896	3	0.06644
4	0.0196	0.0035	17.331	102.341	3672	3	0.04186
5	0.0219	0.0023	29.98	85.01	2448	3	0.02207
6	0.0234	0.0015	55.03	55.03	1224	3	0.01112

*In order to follow P - Δ Analysis for this present storey, the failure of beam takes place.

On comparison of the results from Table 5 and Table 6, it has been noted that the resultant value of Base Shear is found to be more when analyzed under Lateral Force Method as compared to that of in Response Spectrum Method. This value is 179.201 kN.

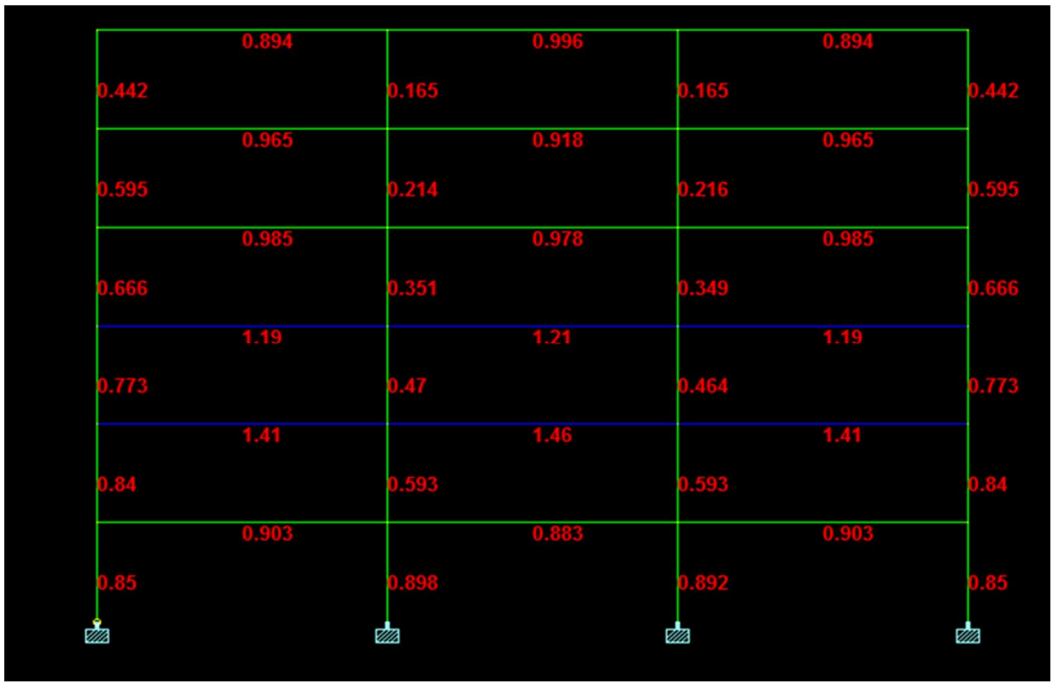


Fig. 4: Frame Showing Failed Members

IV. DESIGN OF MEMBERS

The Steel beams and columns are designed based on the IS: 800 – 2007. Based on the codes, the members can be designed for the following two criteria:

1) Design Strength Criteria (As per Clause 7.1.2 of IS: 800 – 2007)
2) Yield Strength Criteria (As per Clause 6.2 of IS: 800 – 2007)

A. Design Strength Criteria

All the compression members are designed on the basis of this criterion. The buckling strength of these members is mainly calculated which mainly gets affected by residual stresses, accidental load eccentricities, etc.

B. Yield Strength Criteria

All the members are designed on the basis of axial tension, T_d is given by

$T_{dg} = A_g f_y / \Upsilon_{mo}$

Where, f_y= Yield stress of the material, A_g= Gross area of cross-section, and

Υ_{mo} = Partial Safety Factor for Failure in Tension by Yielding

Table 7: Table of Members Failed and Modified Sections
(By Lateral Force Method)

S. No.	Failed member no:	Failed section	Critical condition	Staad design section(passed)
1	1	ISMB350	IS 6.2	ISWB500
2	3,8,11,14,15	ISMB350	IS 6.2	ISLB550
3	10,12,17	ISMB350	IS 7.1.2	ISWB600
4	13	ISMB350	IS 6.2	ISHB450A
5	4,5,6,7,9,16,18	ISMB350	IS 7.1.2	ISWB600A
6	2	ISMB350	IS 6.2	ISHB450

Table 8: Members failed and New Modified Sections
(By Response Spectrum Analysis)

Sl no.	Failed member no:	Failed section	Critical condition	Staad design section(passed)
1	1,13	I80012B50012	IS 7.1.2	I80012B50016
2	2,14	I80012B50012	IS 7.1.2	I80012B55012
3	3,15	I80012B50012	IS 7.1.2	ISWB550
4	7,8,9,40,42	ISMB350	IS 6.2	I100012B50012
5	21	I80012B50012	IS 7.1.2	I100012B50012
6	27	I80012B50012	IS 7.1.2	ISWB600A
7	41	ISMB350	IS 6.2	ISMB600

V. DESIGN OF CONNECTIONS

If we consider node 16, the connection is to be designed between the sections ISWB600A, ISHB 350 and I80012B55012. The design process has been explained as below:

A. Connection of Flange To Steel Plate

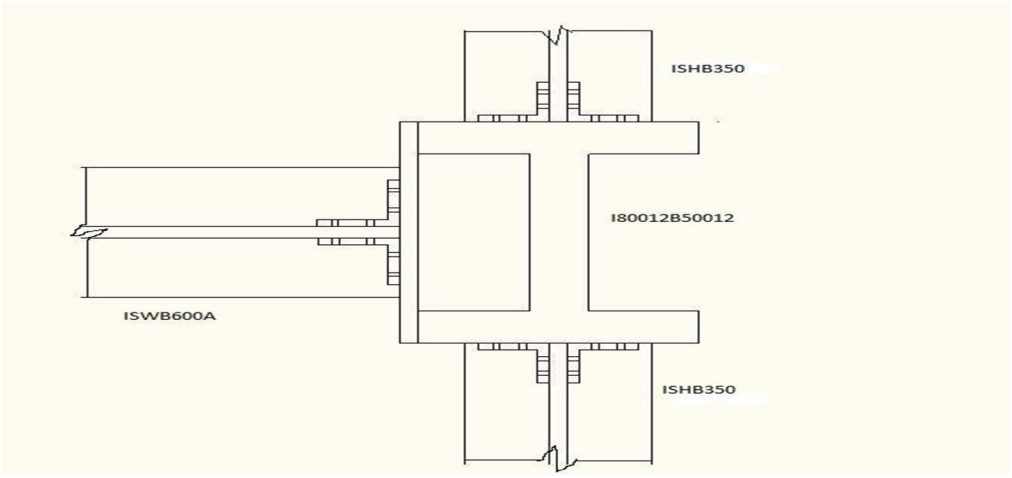


Fig 5: Top View of Exterior Connection Joint

Let the welded connection is provided between the members and the type of weld be Butt Weld having full penetration. The weld thickness is 10 mm and the permissible stress due to bending is assumed to be 150 N/mm^2.

The Length of weld Required, L = $\sqrt{\frac{6\ X\ M}{t\ X\ P_b}}$

Where, M = Bending Moment of the member,
t = thickness of the butt weld,
P_b = Permissible bending stress in the weld = 150 N/mm^2

Therefore, L = $\sqrt{\frac{6\ X\ 281.52\ X\ 1000\ X\ 1000}{10\ X\ 150}}$ = 1061 mm

Max. Stress in the weld due to Bending, $P_b = \frac{M\ X\ y}{I} = \frac{6\ X\ M}{t\ X\ b^2} = \frac{6\ X\ 281.52\ X\ 1000\ X\ 1000}{850\ X\ 300^2}$

$$\boldsymbol{P_b = 21.85\ N/mm^2}$$

Stress due to Shear in the Weld, $P_s = \frac{W}{d\ X\ t} = \frac{120\ X\ 1000}{10\ X\ 1295}$ = 9.273 N/mm^2

Resultant Stress, $P_e = \sqrt{P_b^2 + P_s^2} = \sqrt{21.85^2 + 9.273^2}$

$P_e = 27.121 \frac{N}{mm^2} < 225 \frac{N}{mm^2}$ **(Design is Safe)**

Hence, the plate size is 825 mm X 850 mm which may be connected to the flange of the section.

B. Connection Of Beam To Steel Plate

Let us provide two equal angle sections **2 X ISA 100 X 100 X 8** which are connected by the use of 20 mm diameter closed turned bolt.

No. of bolts calculated, n = $\sqrt{\frac{6\ X\ M}{R\ X\ P\ X\ m}} = \sqrt{\frac{6\ X\ 281.52}{108.92\ X\ 0.06\ X\ 4}}$ = 8 bolts

Where, R = Bolt Value of the connection, p = pitch between two bolts
M = no. of rows of the bolt

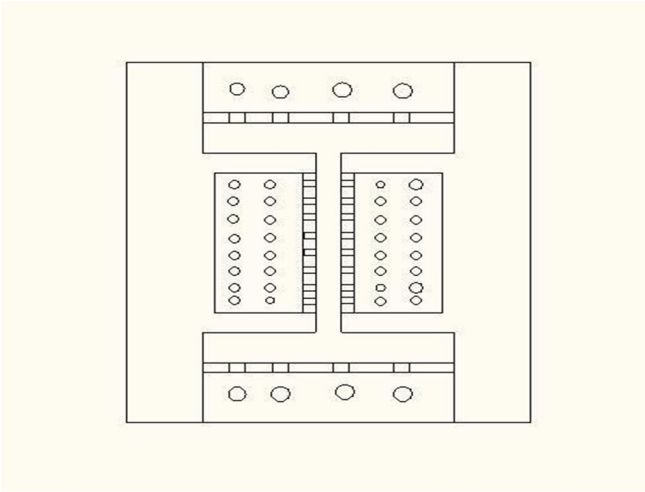


Fig 6: Front View of Connection between Plate and Beam using Angle Sections

Application of Check for Stresses

$\sigma_{tf\ Cal} = \frac{6\ M}{m\ X\ p\ X\ n^2\ X\ A_b} = \frac{6\ X\ 281.52\ X\ 1000}{4\ X\ 6\ X\ 8^2\ X \frac{\pi}{4} X\ 21.5^2}$ = 298.9 N/mm^2

Shear Stress at each bolt, $\tau_{vf\ cal} = \frac{W}{m\ X\ n\ X\ A_b} = \frac{120\ X\ 1000}{4\ X\ 8\ X \frac{\pi}{4} X\ 21.5^2}$ = 1.03 N/mm^2

Permissible Combined Shear Stress and Bending Stress

$$\frac{\sigma_{tf\ Cal}}{\sigma_{tf}} + \frac{\tau_{vf\ cal}}{\tau_{vf}} \leq 1.4$$

$$= \frac{298.9}{300} + \frac{1.03}{100} = 1.0097 \qquad (\leq 1.4)$$

Hence, The Design is OK.

As per the seismic resistant design philosophy, the various structural members should not possess any damage during the minor ground shaking, while repairable and even irreparable damage can be possible in case of moderate and severe shaking respectively. To fulfil these conditions, sometimes steel structures may be used in the seismic prone area. This chapter deals with the steel frame analysis by the use of methods namely Lateral Force Method and Response Spectrum (RSA) Method (A type of Dynamic Method).

From the analysis, it is observed that the most critical load combination is 1.7(EQ+DL) and its corresponding drift variation and maximum moment diagrams are obtained from analysis which is shown in the figure 7 and figure 8 respectively. The analysis also provides the value of lateral storey drift at each storey. It is also noted that all the values of drifts lie within permissible limits of deflection as per IS: 800 – 2007.

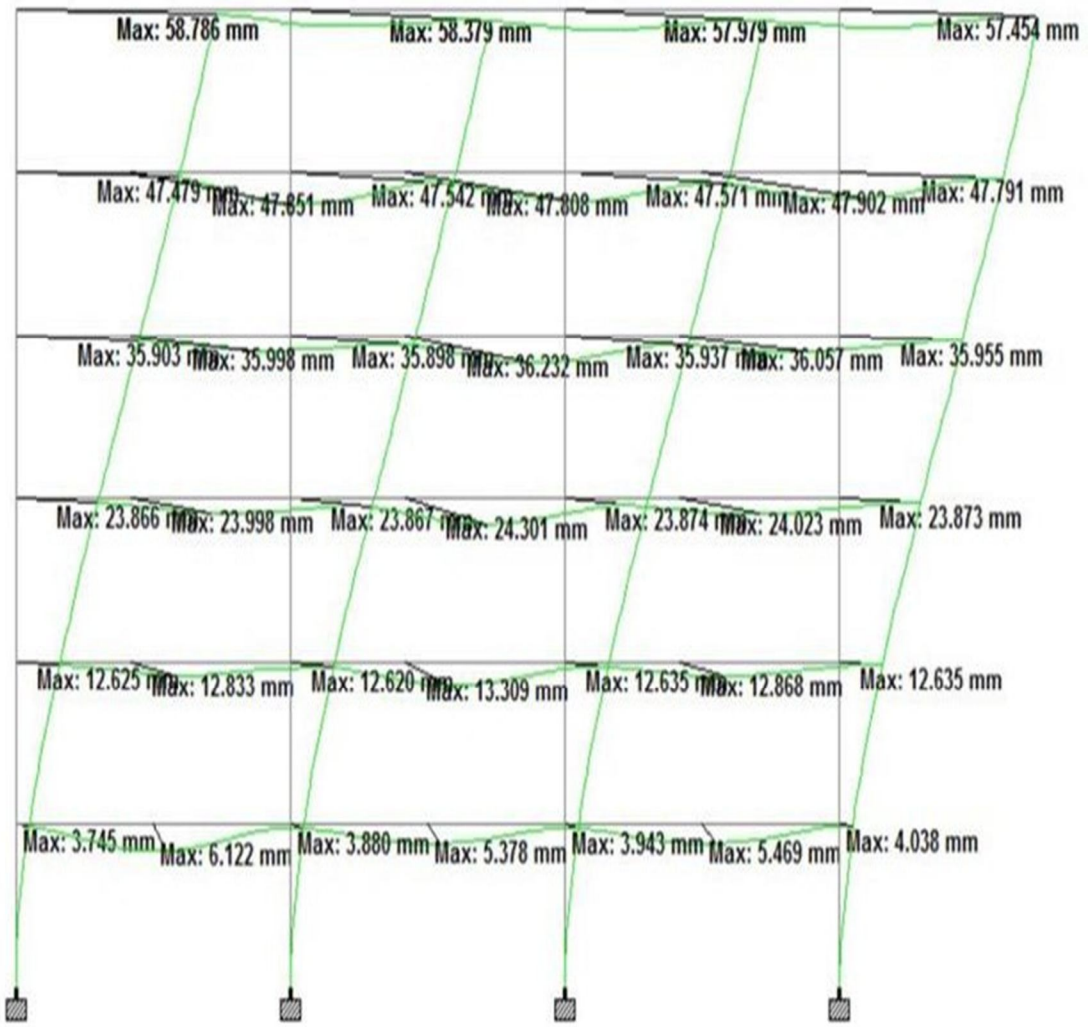


Fig. 7: Displacement Diagram for Load Combination 1.7(EQ+DL)

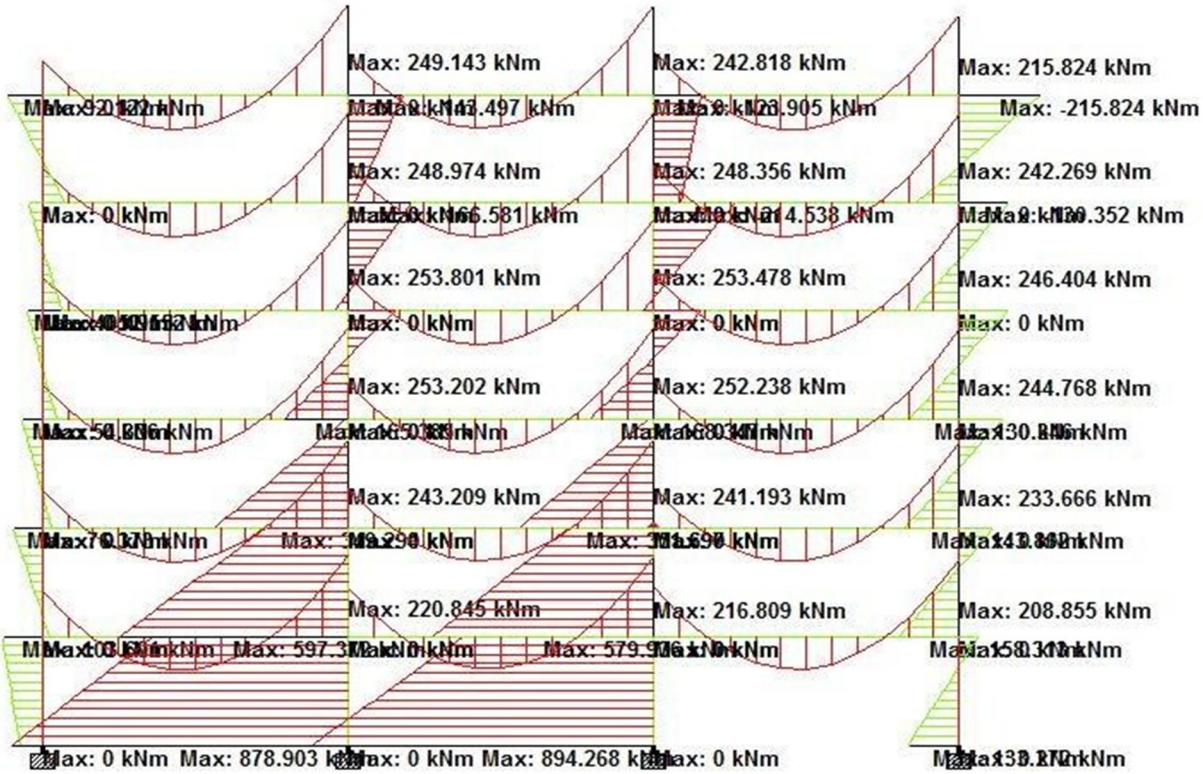


Fig. 8: Bending Moment Diagram for Load Combination 1.7(EQ+DL)

VI. RESULTS OF RESPONSE SPECTRUM ANALYSIS (RSA)

From the analysis, it is observed that the most critical load combination is 1.3(DL+LL+EQ) and its corresponding maximum moment and shear variation diagrams are obtained from analysis which is shown in the figure 9. The analysis also provides the value of lateral storey drift at each storey. It is also noted that all the values of drifts lie within permissible limits of deflection as per IS: 800 – 2007.

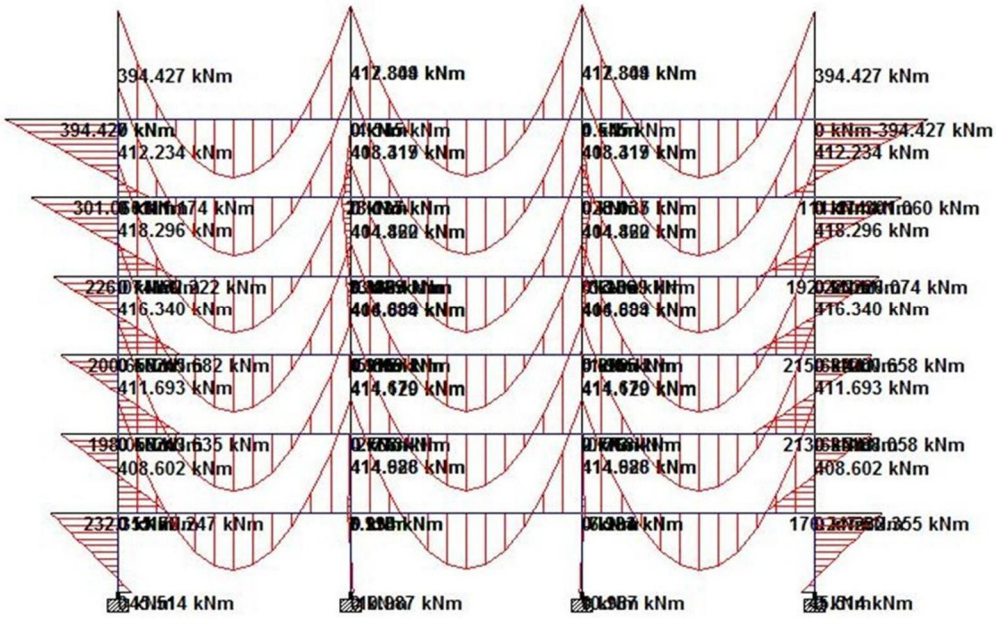


Fig.9 : Bending moment diagram for load combination 1.3(DL+LL+EQ)

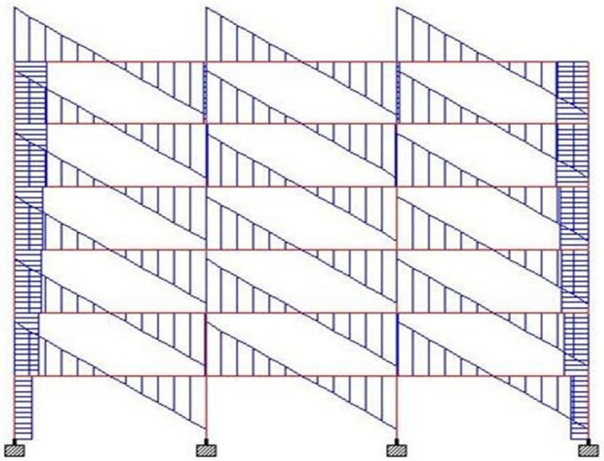


Fig. 10: Shear Force Diagram in X-axis for load combination 1.3(DL+LL+EQ)

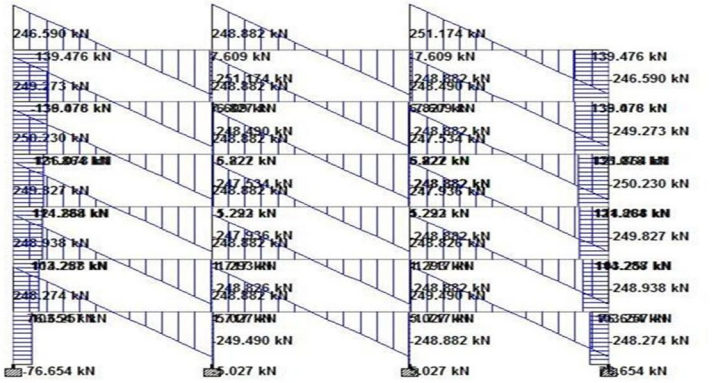


Fig. 11: Shear Force Diagram in Y-axis for Load Combination 1.3(DL+LL+EQ)

A. Comparison Of Absolute Storey Drift In Both Methods

The absolute storey drift is determined for each storey using both the methods. Table 9 shows all the values of storey drift at each storey and figure 12 indicates its variation. It is very interesting to note that the storey drift at first storey i.e., at 3m height of storey is almost same but the variation in storey drift is observed to be higher in Lateral force method as compared to that in response spectrum method of analysis. The storey drift value at last storey i.e., at 18m storey height, the response spectrum method provides comparatively 60% lesser drift as compared with that of the linear response method of analysis.

Table 9 Values of Storey Drift by Lateral force Method and Response Spectrum Method

Storey no.	Storey Height	Lateral Force Method (cm)	Response Spectrum Method (cm)
1	3	0.3869	0.491
2	6	1.2595	1.15
3	9	2.3837	1.61
4	12	3.5892	1.96
5	15	4.7566	2.19
6	18	5.8123	2.34

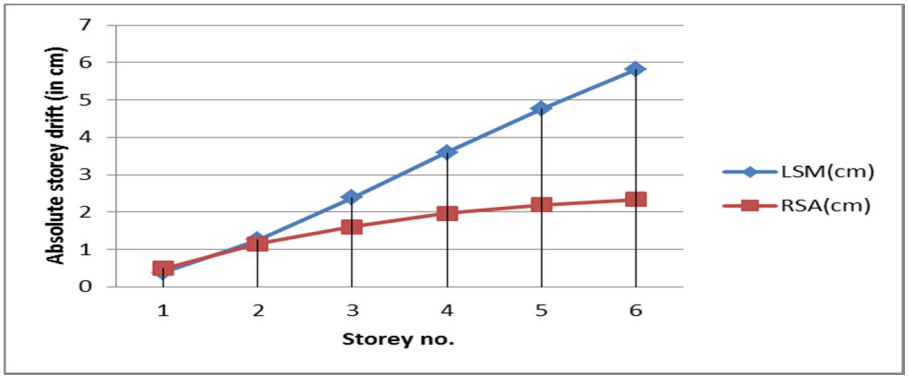


Fig. 12: Comparison of Absolute Storey Drift

B. Comparison of Storey Shear In Both Methods

The storey shear is determined for each storey using both the methods. Table 10 shows all the values of storey shear at each storey and figure 13 indicates its variation. It is very interesting to note that the difference in observed values of storey shear between both the methods is 28.91%. However, the observed value is higher is Lateral force method as compared to response spectrum method. An increase in difference between shear values is observed with increase in storey height. Moreover, there is a slight decrease in the value from third storey to fourth storey. This difference in the values is maximum in fifth and sixth storey. The average difference in storey shear is around 29.7% in each storey.

Table 10: Comparison of Storey Shear (Using Both LSM and RSA)

Storey no.	Storey height	Lateral Force Method (kN)	Response Spectrum Method (kN)	Difference in %
1	3	179.201	120.981	28.91
2	6	177.232	119.104	32.79
3	9	169.281	112.992	33.25
4	12	151.451	102.341	32.42
5	15	119.794	85.01	28.99
6	18	70.582	55.03	22.033

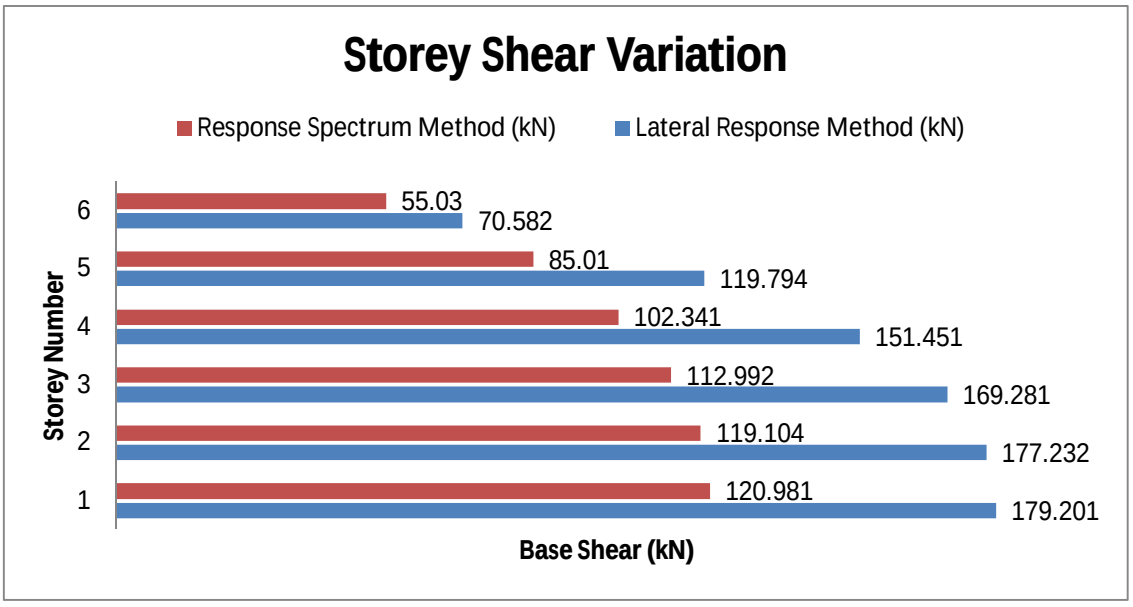


Fig. 13: Storey Shear Variation as per Response Spectrum and Lateral force Method

C. Comparison Between Pre-Design Drift And Post-Design Drift

Table 11 shows the drift values in frame before the design of members and after the design of members. The average difference between drift values for pre-design and post-design condition is 62.1%. In addition to this, it is observed that the drift values for pre-design conditions are very high as compared to post design condition in Lateral force Method. It can also be said that drift in the storey can be easily controlled by proper designing of the members as per the forces applied. Fig. 14 shows the variation between the values of drift pre-design and post-design members.

Table 11: Variation of Storey Drift by Lateral Force Method

Storey no.	Pre design drift (cm)	Post design drift(cm)	% Difference
1	0.3869	0.2056	46.85
2	1.2595	0.5472	56.55
3	2.3837	0.9052	68.11
4	3.5892	1.2561	65
5	4.7566	1.5729	66.93
6	5.8123	1.8012	69.05

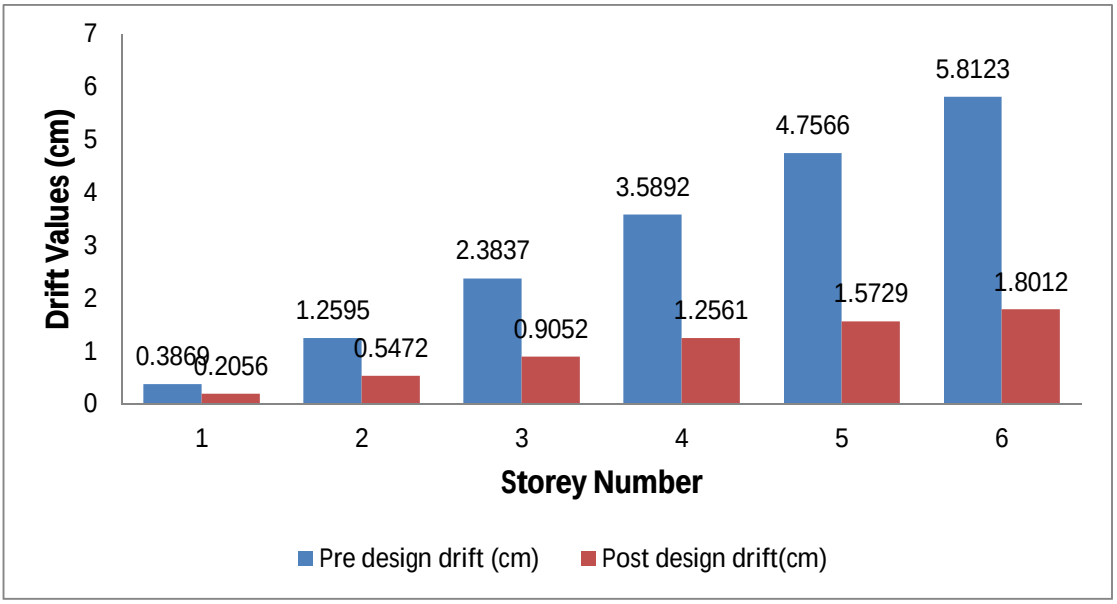


Fig. 14: Graphical Variation of Pre-Design Drift and Post-Design Drift Values with Storey Number

VII. CONCLUSION

1) The inter-storey drift is determined by the use of the both the methods i.e., the lateral force method and the response spectrum method. It is observed that the lateral displacement of the structure analyzed by the response spectrum method is comparatively lesser than the lateral displacement of the structure analyzed by the lateral force method.
2) The shear force distribution along the height of structure from the use of response spectrum method of analysis is observed very less as compared to the distribution obtained from the use of lateral force method of analysis.
3) The numerical difference between the results obtained from both the methods is always analyzed based on some assumptions which are prevalent for the lateral force method of analysis which are as follows:

a) The mode natural fundamental frequency of the structures imparts the most reliable and significant role to the distribution of base shear completely throughout the height of the structure.

b) The whole mass of structure is assumed to be used in the dynamic procedure. The above assumptions are completely valid to the low to medium rise structures in which mass distribution is completely uniform along the height.

4) From the above results, it is concluded that the results obtained from the dynamic methods of analysis are comparatively lesser than the results obtained from the lateral force method of analysis. The reason behind this result lies in terms of its fundamental time period. The fundamental mode of time period is 0.62913 from dynamic analysis method which is more than the fundamental mode of time period from the analysis by the use of lateral force method which is approximately in terms of 0.33.

5) The both the comparative analysis indicates that the weight of the first model is nearly 86 % of its complete seismic weight based on IS: 1893 – 2002. The weight of second modal is only 8.24% of its complete seismic weight and the fundamental natural time period of the structure is in the range of 0.20s.

6) During the post design analysis of the models, the decrease in the value of storey drift and distribution of base shear along the height of structure is observed significantly for the high weighted structure. As a result, the provision of heavier structural members results in the safe design. E.g., **ISMB 350** section members had used for designing but these sections have found to be failed and when the section is redesigned in the platform STAAD. Pro V8i, the higher section, **ISWB 600 A** is concluded.

7) The cost of steel sections is directly proportional to the quantity of the steel used and it is found to be lesser for lateral force method of analysis when compared with response spectrum method of analysis. It is due to the fact that the response spectrum method of analysis is a type of dynamic analysis, is very effectively accurate method which is taken into the considerations with so many parameters such as the shape of the member, mode of frequency, factors of mass participation to determine the fundamental natural time period and vibrations. The response spectrum method of analysis is highly realistic in nature for the design & analysis of steel framed structures and this present work concluded that the linear method of analysis results into the very high cost effectiveness for the seismic design and analysis of steel structure.

8) The requirement of quantity of the steel for the design and analysis of steel structure by the use of lateral force method, a type of static method is observed to be 19.92% lesser than the quantity of steel required by the use of analysis of response spectrum method.

9) The response spectrum method includes the use of heavier section of member, the value of lateral displacement and the value of storey drift are comparatively less than sections designed by the use of lateral force method

10) It is also observed that the coefficient of sensitivity of the inter storey drift does not necessarily vary by the considerable amount for the both methods of analysis.

11) The base shear value of structure frame designed by the use of lateral force method is 48.83 % base shear value of structure frame designed by the use of response spectrum method.

12) The quantity of steel required for sections of members and their connections is higher for design and analysis by Response Spectrum Method as compared to that of Lateral force Method.

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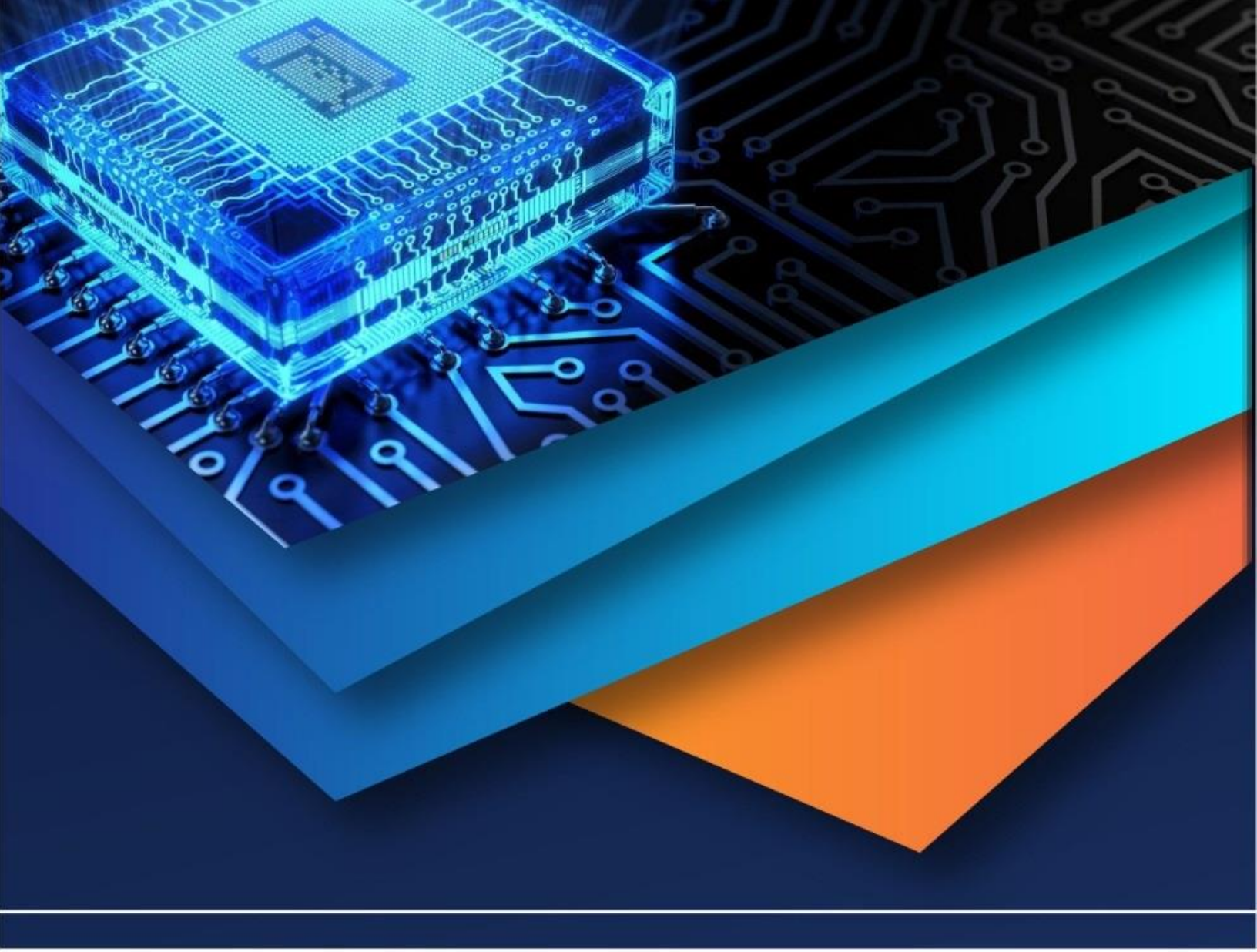
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