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NON LINIEAR ANALYSIS OF G+5BUILDING INCLUDING BASEMENT SITUATED AT KALIMAINDIR, AMBIKAPUR: CASE SYUDY

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Abstract

This project focuses on the seismic design of a multi-story, multi-bay moment-resisting building frame using IS 1893-2002 and IS 800 design provisions. The frame comprises six stories with three bays horizontally and five bays laterally. Two analysis methods, namely the Equivalent Static Load method and Response Spectrum method, are employed, and a comparative study of their results is conducted in terms of story displacement, inter-story drift, and base shear.

Furthermore, the frame undergoes P-Δ analysis and necessary moment corrections are made according to the IBC code. The design of the steel moment-resisting frame follows IS-800:2007, involving multiple iterations to satisfy the criteria specified in the code. The designed frame is re analyzed, and the results are compared based on the sections used. The cost efficiency of both analysis methods is also assessed.

Additionally, the design of connections for an interior joint and an exterior joint of the frame is carried out, and relevant calculations are provided. The foundation design, including the base plate, is performed according to IS 800:2007, with detailed calculations and accompanying figures.

The software used for both analysis and design in this project is STAAD PRO. Throughout the process, manual calculations have been performed and compared to ensure accuracy.

Keywords: Seismic design, Equivalent static load method, Response Spectrum method, Steel Frame, P-Δ analysis.

1.Introduction

Seismic Analysis is a subset of structural analysis and is the calculation of the response of a building structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofitting regions where earthquakes are prevalent.

The most important earthquakes are located close to the borders of the main tectonic plates which cover the surface of the globe. These plates tend to move relative to one another but are prevented by doing so by friction until the stresses between plates under the epicenter point become so high that a move suddenly takes place. This is an earthquake. The local shock generates waves in the ground which propagate over the earths surface, creating movement at the bases of structures. The importance of waves reduces with the distance from the epicenter. Therefore, there exists region of the world with more or less high seismic risk, depending on the proximity to the boundaries of the main tectonic plates

Besides the major earthquakes which take place at tectonic plate boundaries, others have their origin at the interior of the plates at fault lines. Called intra plates earthquakes, these less energy, but can still be destructive in the vicinity of the epicenter

The action applied to a structure by an earthquake is a ground movement with horizontal and vertical components. The horizontal movement is the most specific feature of earthquake action because of its strength and because structures are generally better designed to resist gravity than horizontal forces. The vertical component of the earthquake is usually about 50% of the horizontal component, except in the vicinity of the epicenter where it can be of the same order.

Steel structures are good at resisting earthquakes because of the property of ductility. Experienceshowsthatsteelstructuressubjectedtoearthquakesbehavewell.Globalfailuresandhugenumbers of casualties are mostly associated with structures made from other materials. This may be explained by some of the specific features of steel structures. There are two means by which the earthquake may be resisted:

Option 1 structures made of sufficiently large sections that they are subject to only elastic stresses

Option2 structures made of smaller sections, designed to form numerous plastic zones.

A structure designed to the first option will be heavier and may not provide a safety margin to cover earthquake actions that are higher than expected, as element failure is not ductile. In this case the structure's global behavior is brittle and corresponds for instance to concept a) in a Base Shear V- Top Displacement diagram. In a structure designed to the second option selected parts of the structure are intentionally designed to undergo cyclic plastic deformations without failure, and the structure as a whole is designed such that only those selected zones will be plastically deformed.

The structure's global behavior is „ductile‟ and corresponds to concept b) in the Base Shear V-Top Displacement d. The structure can dissipate a significant amount of energy in these plastic zones, this energy being represented by the area under the V-d curve. For this reason, the two design options are said to lead to dissipative and non-dissipative structures.

A ductile behavior, which provides extended deformation capacity is generally the better way to resist earthquakes. One reason for this is that because of the many uncertainties which characterize our knowledge of real seismic actions and of the analyses we make, it may be that the earthquake action and/ or its effects are greater than expected. By ensuring ductile behavior, any such excesses are easily absorbed simply by greater energy dissipation due to plastic deformations of structural components. The same components could not provide more strength(a greater elastic resistance) when option 1 is adopted. Furthermore, a reduction in base shear V (V reduced < V elastic) means an equal reduction in forces applied to the foundations, resulting in lower costs for the infrastructure of a building.

Steel structures are particularly good at providing an energy dissipation capability due to:

i) The ductility of steel as a material.

ii) The many possible ductile mechanisms in steel elements and their connections.
iii) The effective duplication of plastic mechanisms at a local level
iv) Reliable geometrical properties.
v) Relatively low sensitivity of the bending resistance of structural elements to the presence of coincident axial force.

Variety of possible energy dissipation mechanisms in steel structures, and the reliability of each of these possibilities, are the fundamental characteristics explaining the excellent seismic behavior of steel structures. Furthermore, steel structures tend to have more reliable seismic behavior than those using other materials, due to some of the other factors that characterize them:

- Guaranteed material strength, as result a of controlled production
- design sand constructions made by professional

2. PROBLEMSTATEMENT

The structure consisting of six stories with three bays in horizontal direction and six bays in lateral direction is taken and analyzed it by both equivalent static method and response spectrum analysis and designed.

The storey height is 3 meters and the horizontal spacing between bays is 8 meters and lateral spacing of bays is 6 meters

The seismic parameters of building site areas follows

- Seismiczone:3
- ZonefactorZ:0.16
- Building frame system: steel moment resisting frame designed as per SP 6
- Response reduction factor: 5
- Importance factor:1.5
- Damping ratio:3%

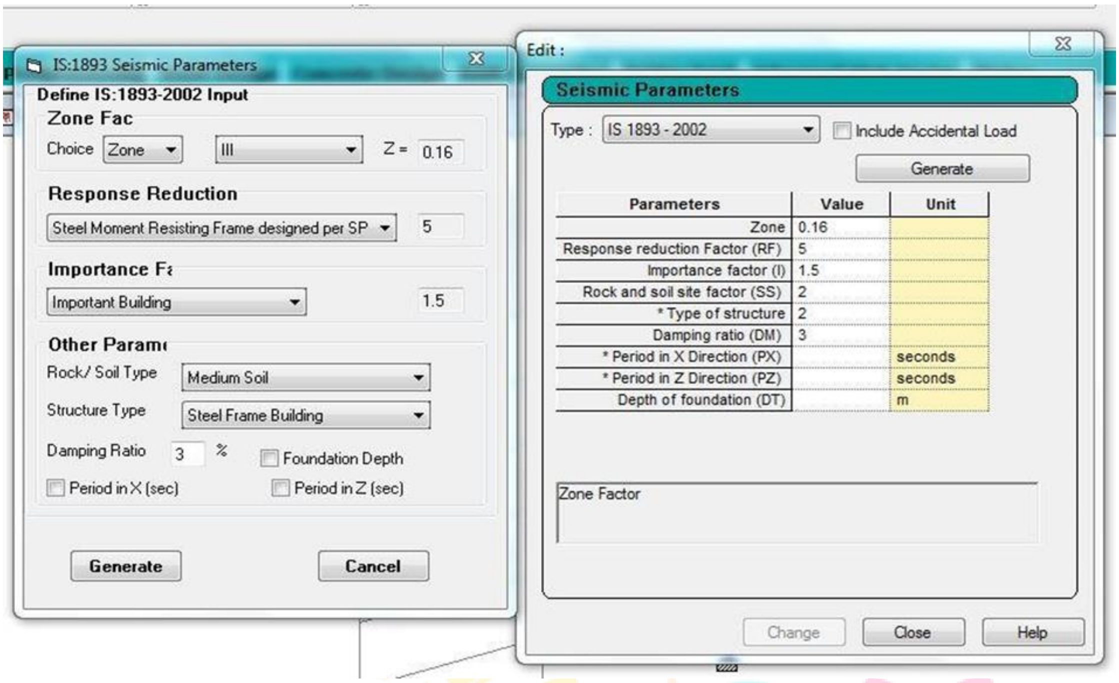


FIG2.1:STAADinputofseismicparameters

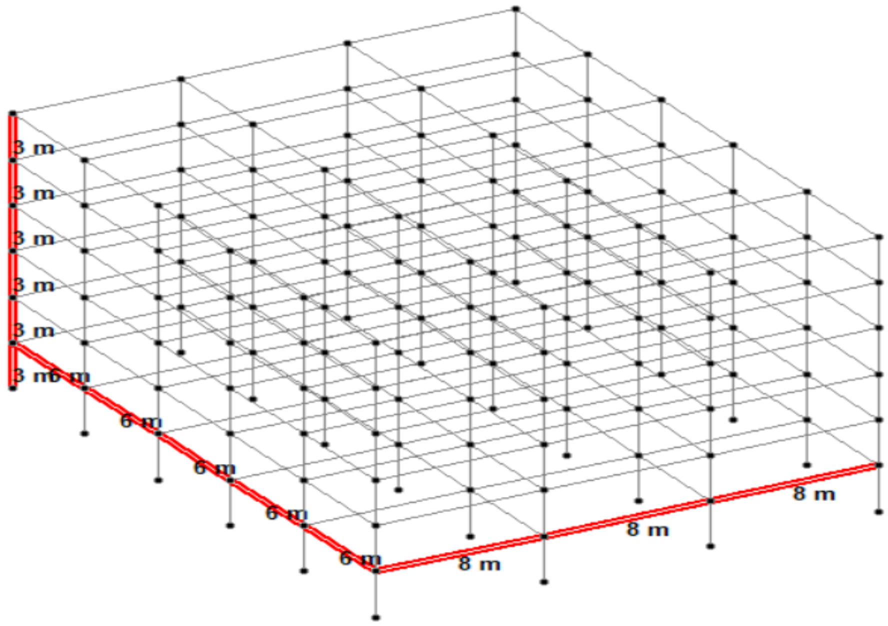
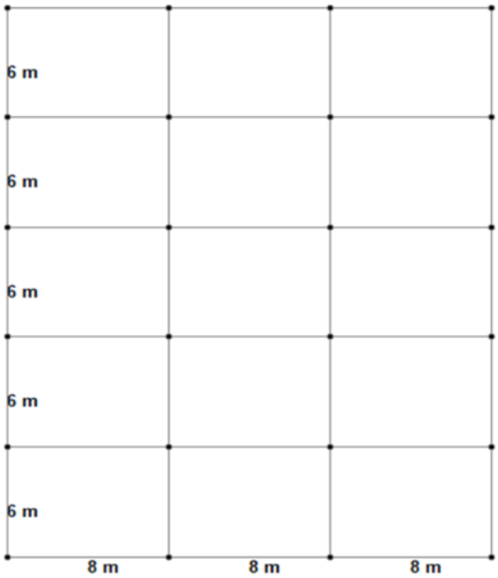


FIG2.2:3-dimensionalview ofthesteelbuildingframe



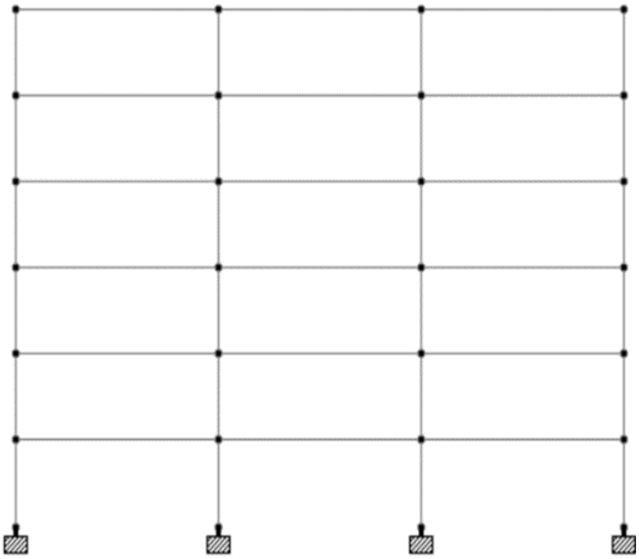


FIG 2.3: Plan of the building frame **FIG 2.4: Elevation of the building frame**

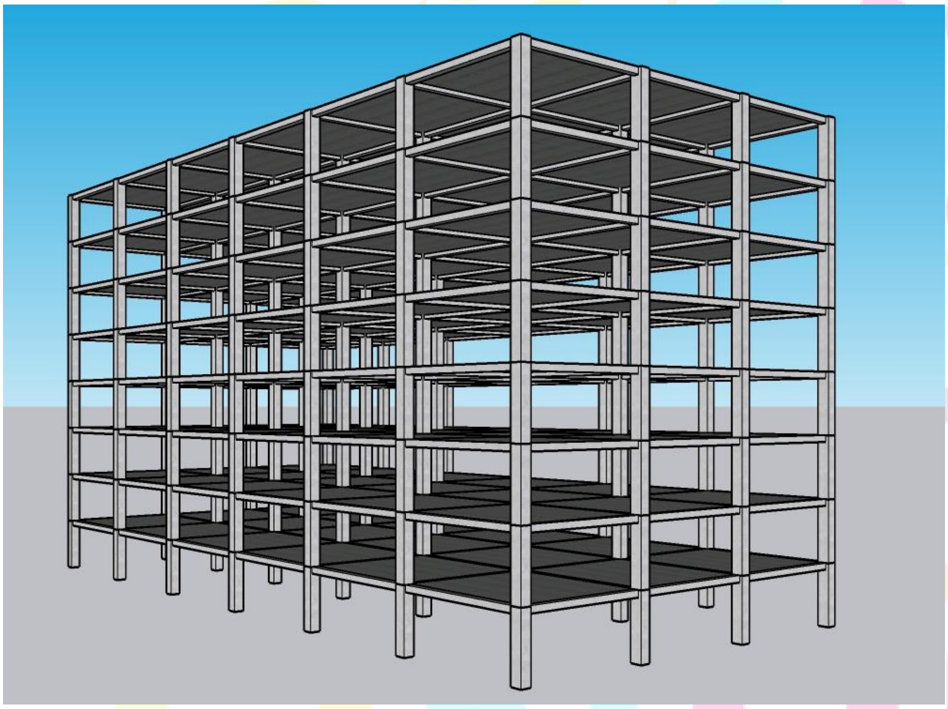


FIG 2.5: 3D building frame

3 LOAD PARAMETERS:

Dead Load Is Taken As = 5 Kn/m^2 and Live Load Is Taken As 3 Kn/m^2 Load Calculation

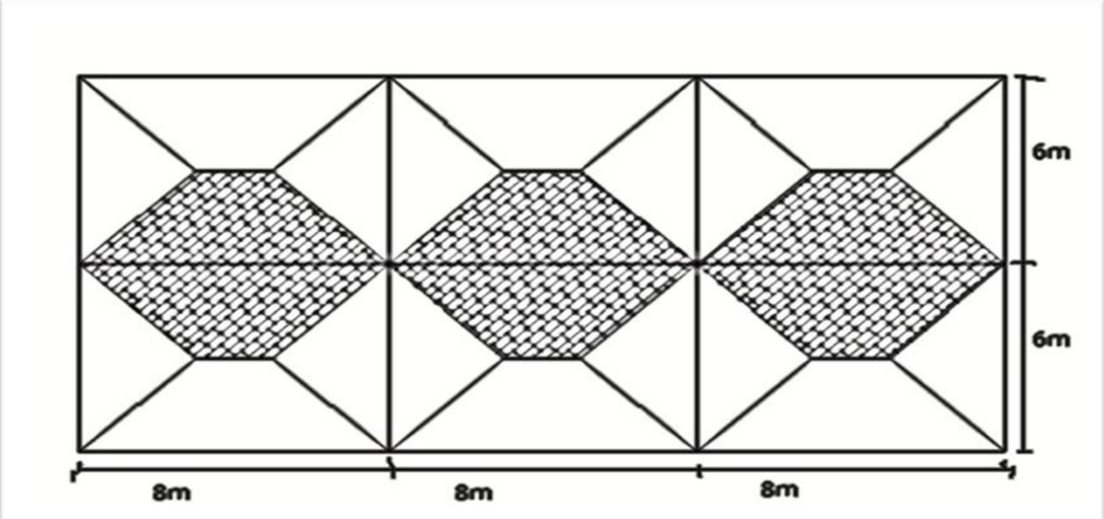


FIG 3.1: Load distribution diagram

Load on beam along horizontal direction

•	1 Dead Load	=	30 m2 x 5 KN/M2	=	150KN
•	Uniformly Distributed Load	=	150/8	=	18.75 KN/m
•	2. Live Load	=	30x3	=	90KN
•	Uniformly Distributed Load	=	90/8	=	11.75KN/m

Load combinations as per IS1893-2002:

- 1.7 (DL+LL)
- 1.7(DL+EQ)
- 1.7(DL-EQ)
- 1.3(DL+LL+EQ)
- 1.3(DL+LL-EQ)

4 METHODOLOGIES:

The initial step is preliminary design of building frame. The procedure involved are selection of sections of members of the frame. Since the dynamic action effects are a function of member stiffness, the process un avoidably involves much iteration.

The example considered here involves a building in which seismic resistance is provide by moment resisting frames (MRF), in both x and y directions. Moment resisting frames (MRF) are known to be flexible structures. Thus, their design is often governed by the need to satisfy deformation criteria under service earthquake loading, or limitation of P-Δ effects under design earthquake loading. For this reason, rigid connections are preferred. The Preliminary design consists of following steps:

- Defining beam sections, checking deflection and resistance criteria under gravity loading.
- Following an iterative process, going through the following steps until all design criteria are fulfilled.

The iterative process can make use either of lateral force method or the spectral response modal superposition method.

1. Selection of Beam Sections.
2. Definition of Column Sections checking the „weak beam strong column criteria".
3. Check compression /buckling at ground floor level under gravity loading.
4. Calculation of seismic mass.

5. Static analysis of one plane frame under lateral loads.
6. Static analysis under gravity loading.
7. Stability check using P-Δ effects (parameter ϴ) in the seismic loading situation.
8. Deflection check under earthquake loading.
9. For Response spectrum analysis step 5 is replaced by response spectrum analysis of one plane frame to evaluate earthquake action effects.

5 CALCULATIONS:

Moment Resistance check and Deflection criteria.
Checking deflection limits of Beams in x- direction.
Total Dead load + Live load =51 KN/m = gravity load
$\delta = PL^4/384EI$
$L/300 = PL^4/384EI$
$I_{Required} = 9714.3 cm^4$
Selection of beam ISMB350
Definition of Column Sections checking the 'weak beam strong column criteria'
ΣMc = Mc1 +Mc2
$\Sigma Mg = Mg1 + Mg2$
$\Sigma Mc \geqslant 1.2 Z Mg$ (as per IS 800:2007)
$\Sigma fyc \times Zcolumn > 1.2 \Sigma fyb \times Zbeam$
$Z_{req.} = 656.88\ cm^3$
the section selected is I80012B50012.

5.1 CALCULATIONS OF SEISMIC MASS:

Dead load = $5KN/m^2$, live load= $3KN/m^2$
Area load contributed to each beam is $30m^2$
DL+LL= 3× 30 × (5 + 3) =720KN
Nodal loads of 144KN is put on both interior nodes and a nodal load of 72KN is applied on the exterior nodes. There fore, total nodal load contribution for the seismic mass is: =144× 2 + 72 × 2 = 432Kn
Weight of wall is also contributing to the seismic mass. Weight of the wall is 3KN/m. therefore total wall weight per storey= 3×24 =72KN
Therefore, total seismic mass per storey is given by= 720+432+72 = 1224KN

6 ANALYSIS PROCEDURE:

6.1LATERAL FORCE METHOD:

In the equivalent static method, which accounts for the dynamics of the buildings in approximate manner, the design seismic base shear is determined by VB=Ah × W
The following assumptions are involved in the equivalent static method procedure

- Fundamental mode of building makes the most significant contribution to the base shear
- The total building mass is considered against the modal mass that would be used in dynamic procedure. And both of these assumptions are valid for low and medium rise buildings which are regular

It should be distributed along the height of the building using the following expression.
$Qi = Vb(W^i h^k / ZW^j h^j)$

Where V_b is total design lateral force.
Wi is the seismic weight of floor i
H_i is the height of floor measured from base
The approximate fundamental natural period of vibration in seconds, for a moment resisting frame without brick infill panels is given by:
Ta=0.085h0.75
for all other buildings including moment resisting frame buildings with brick infill, $T = 0.09h/\sqrt{d}$
d'' is base dimension of the building at the plinth level
for buildings with concrete and masonry shear walls,

$$Ta=\frac{0.075h^{0.75}}{\sqrt{A_w}}$$

Aw is the total effective area of the walls in the first storey of the building in square meters in our case the value of Ta=0.09×18/ √24=0.33 Hz.
The structure must also be designed to resist the overturning effects caused by seismic forces. And also storey drifts, member forces and moment due to P- delta effect must be determined. IS 1893 stipulates that the storey drift in any storey due to the minimum specified lateral loads, with a partial load factor of 1.0 should not exceed 0.004 times the storey height.

Table6.1:Analysisbylateral 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	0.003869	0.003869	1.969	179.201
2	0.012595	0.008726	7.951	177.232
3	0.023837	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
7	0.06914	0.0140	77.93	71.266

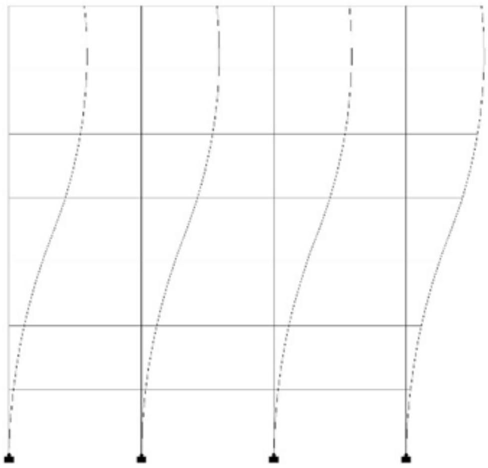
6.1 RESPONSE SPECTRUM ANALYSIS:

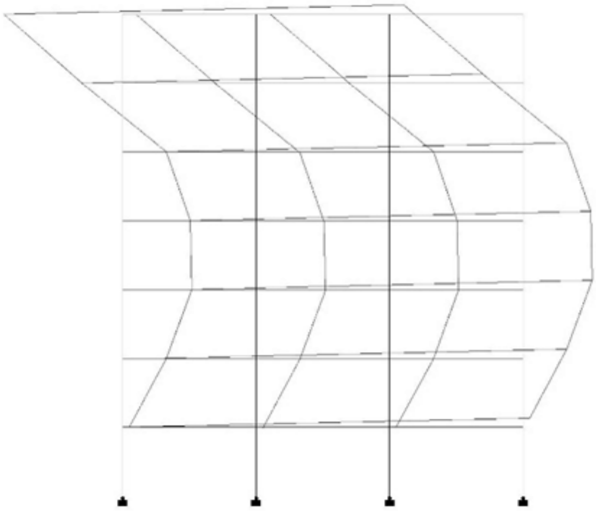
Table6.2:Analysisbyresponsespectrummethod.

Storey no.	Absolute dis-placement of Storey D_i(m)	Design inter sto-rey drift D_r (m)	Storey lateral-forceV_{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
7	0.02895	0.0008	59.60	54.97

MODE SHAPES: fig (6.1)

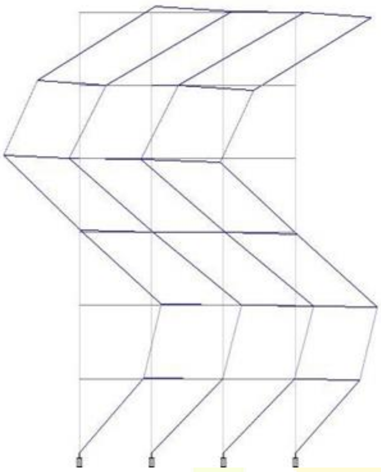


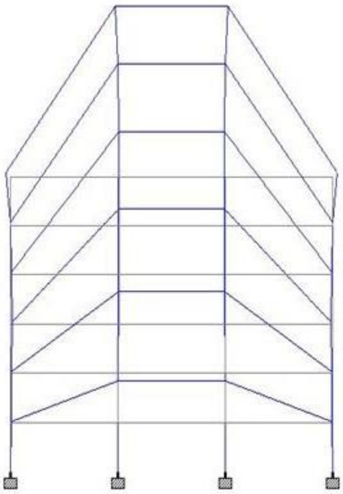


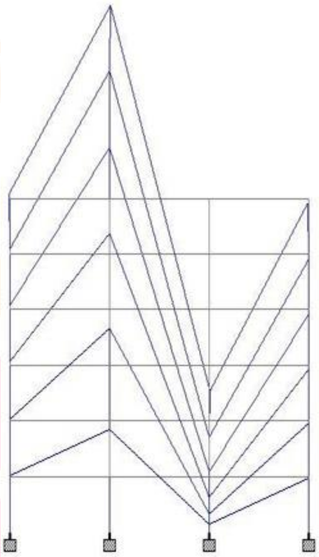
Mode 1 (1.592hz),
Modal Participation factor
MPF= 85.33

mode 2 (5.224hz),

MPF=8.13







Mode 3 (9.525hz)
MPF 0

mode 4 (12.796hz)
MPF 0.01

mode 5 (13.294hz)
MPF 2.04

Table6.3: Base shear and mass participation factor

MODE	BASESHEAR(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

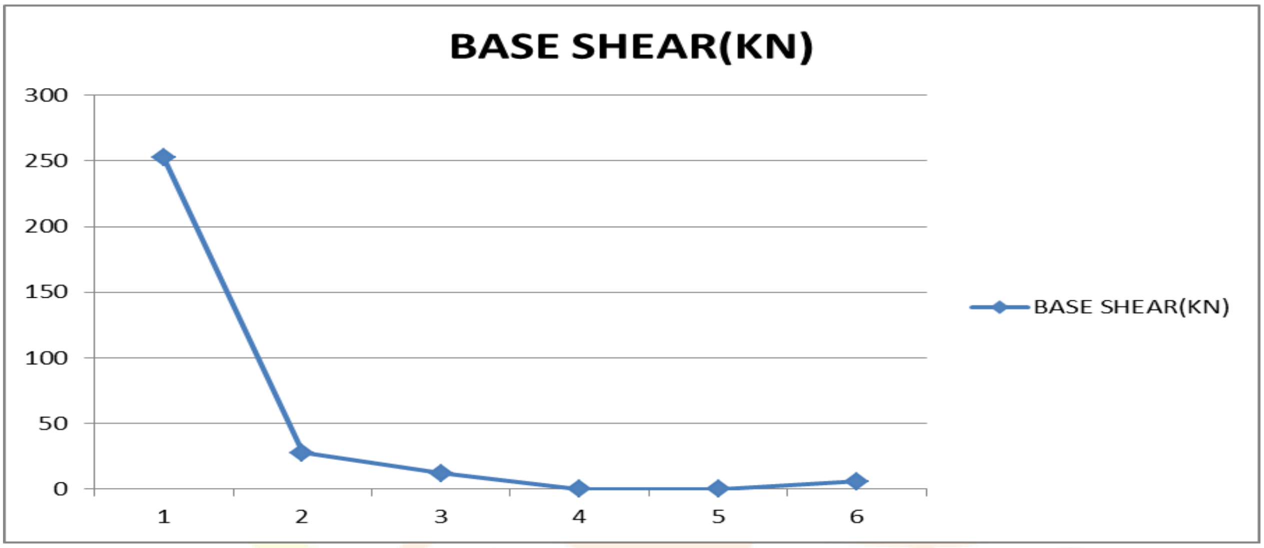


Fig (6.2) graph of modes Vs base shear

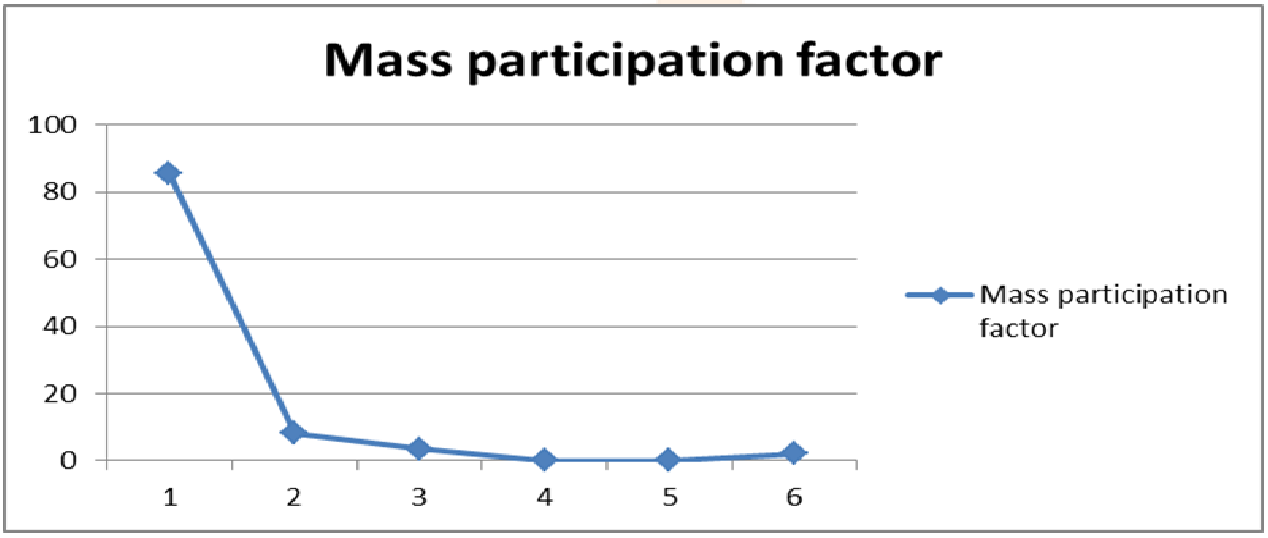


Fig (6.3) Graph of mass participation factor

6.3 P-Δ ANALYSIS:

Table6.5: Correction for P-Δeffect (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 coeffi-cient:(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

Table6.6:CorrectionforP-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 sto-rey P_{tot} (KN)	Storey height: H_i(m)	Inter Storey drift Sensitivity coeffi-cient:(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

7 DESIGNS:

BEAM AND COLUMN DESIGN

Staad pro is used for designing all members of frame following IS 800- 2007

IS 800:2007 CLAUSE 7.1.2

Design strength"

Common hot rolled and built-up steel members (section: I80012B50012, member 17) used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected toaxial compression is defined by buckling class a, b, c, or d as given Table 7

Table7.1: Table of members failed and modified sections(by lateral force method)

Sl no.	Failed member no:	Failed section	Critical condition	Staad design section(passed)
1	1	ISMB350	IS6.2	ISWB500
2	3,8,11,14,15	ISMB350	IS6.2	ISLB550
3	10,12,17	ISMB350	IS7.1.2	ISWB600
4	13	ISMB350	IS6.2	ISHB450A

5	4,5,6,7,9,16,18	ISMB350	IS7.1.2	ISWB600A
6	2	ISMB350	IS6.2	ISHB450

Table 7.2: Table of 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	IS7.1.2	I80012B50016
2	2,14	I80012B50012	IS7.1.2	I0012B55012
3	3,15	I80012B50012	IS7.1.2	ISWB550
4	7,8,9,40,42	ISMB350	IS6.2	I100012B50012
5	21	I80012B50012	IS7.1.2	I100012B50012
6	27	I80012B50012	IS7.1.2	ISWB600A
7	41	ISMB350	IS6.2	ISMB600

8 RESULTS AND DISCUSSIONS:

8.1RESULTS OF LATERAL FORCE METHOD:

Maximum bending moment, shear force etc. are obtained for load combination 1.7(EQ+DL)

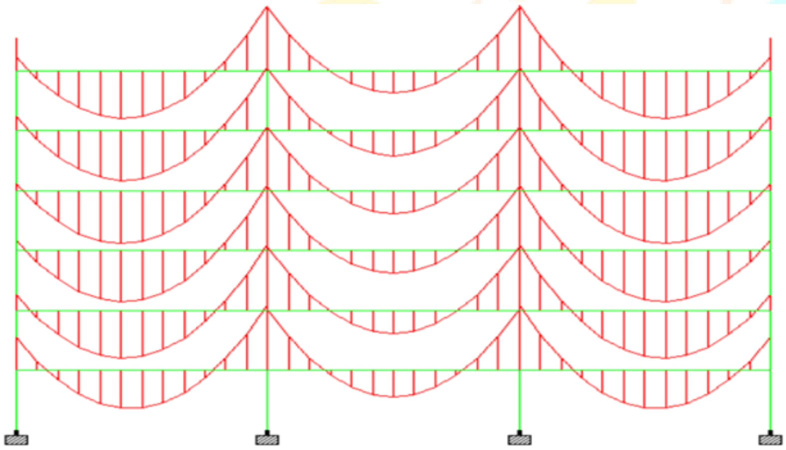


FIG (8.1) Displacement diagram for load combination 1.7(EQ+DL)

The inter storey drift as seen from above diagram is with in the limits of deflection of the code i.e, it is within .004 of storey height=0.004X3000=12mm.

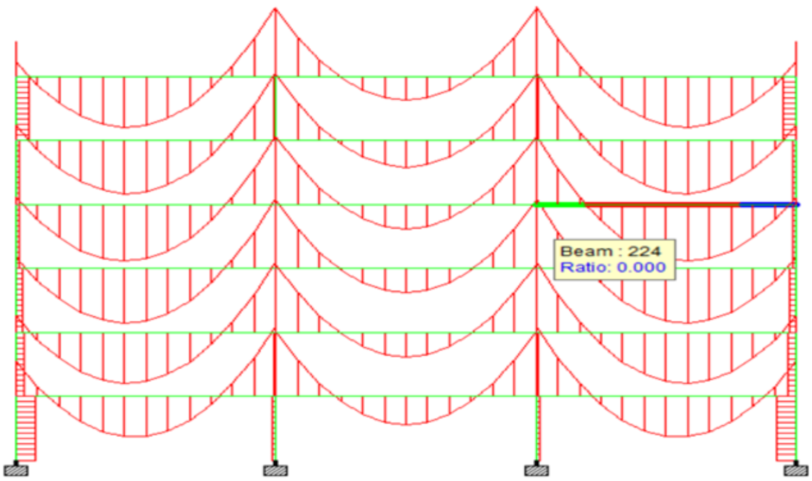


FIG (6.2) Bending moment diagram for load combination 1.7(EQ+DL)

8.3RESULTS OF RESPONSE SPECTRUM ANALYSIS:

Maximum bending moment, shear force etc. are obtained for load combination 1.3(DL+LL+EQ)

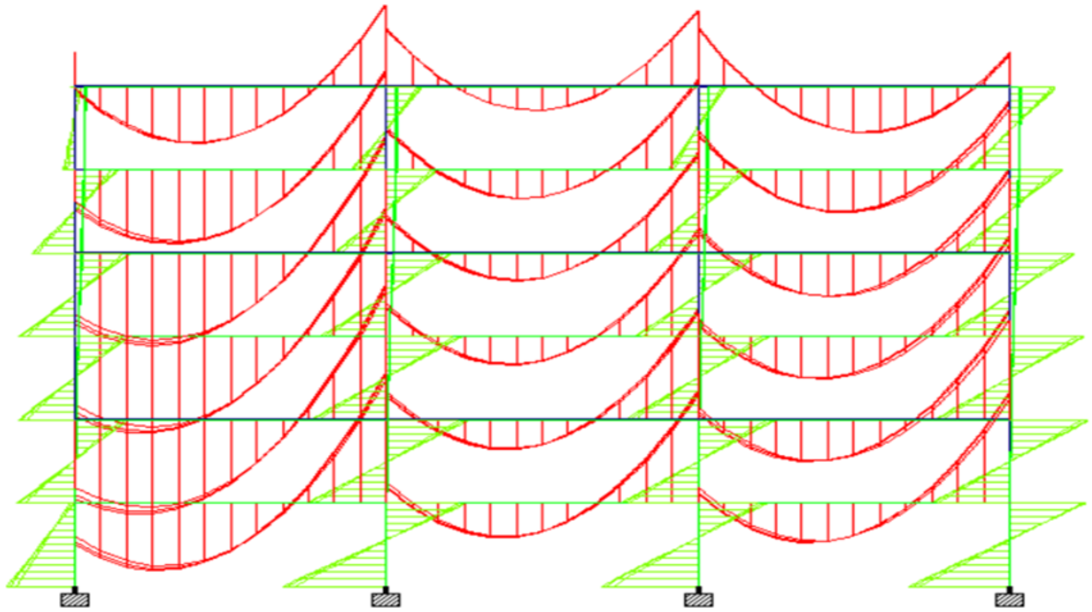


Fig (8.3) Bending moment diagram for load combination 1.3(DL+LL+EQ)

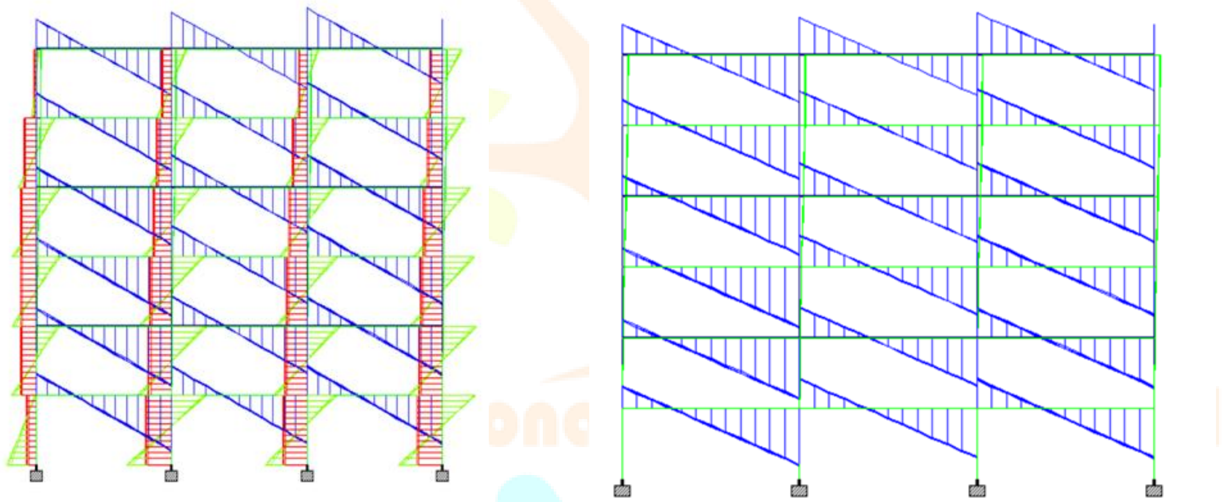


Fig (8.4) shear force diag.in X-axis shear force diag. in Y-axis Load combination is same in both cases-Load case 1.3(DL+LL+EQ).

Comparison of absolute storey drift in both methods: (table 8.1)

Storey no.	Storey height	LSM (cm)	RSA (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

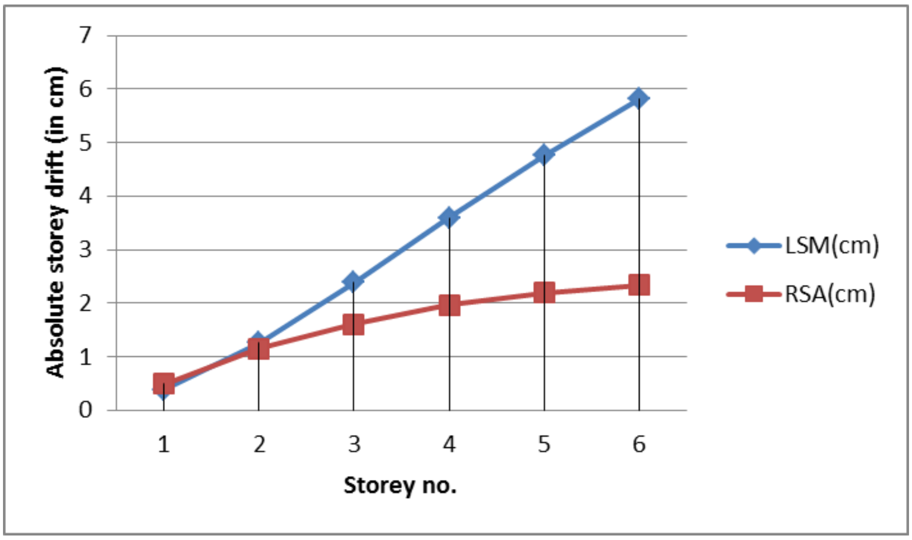


Fig (8.5) Graph of comparison of absolute storey drift

Table (8.2) Comparison of storey shear: (using both LSM and RSA)

Storey no.	Storey height	LSM(KN)	RSA(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

It is found that the difference storey shear by both these methods are about 29.73 % at an average per storey.

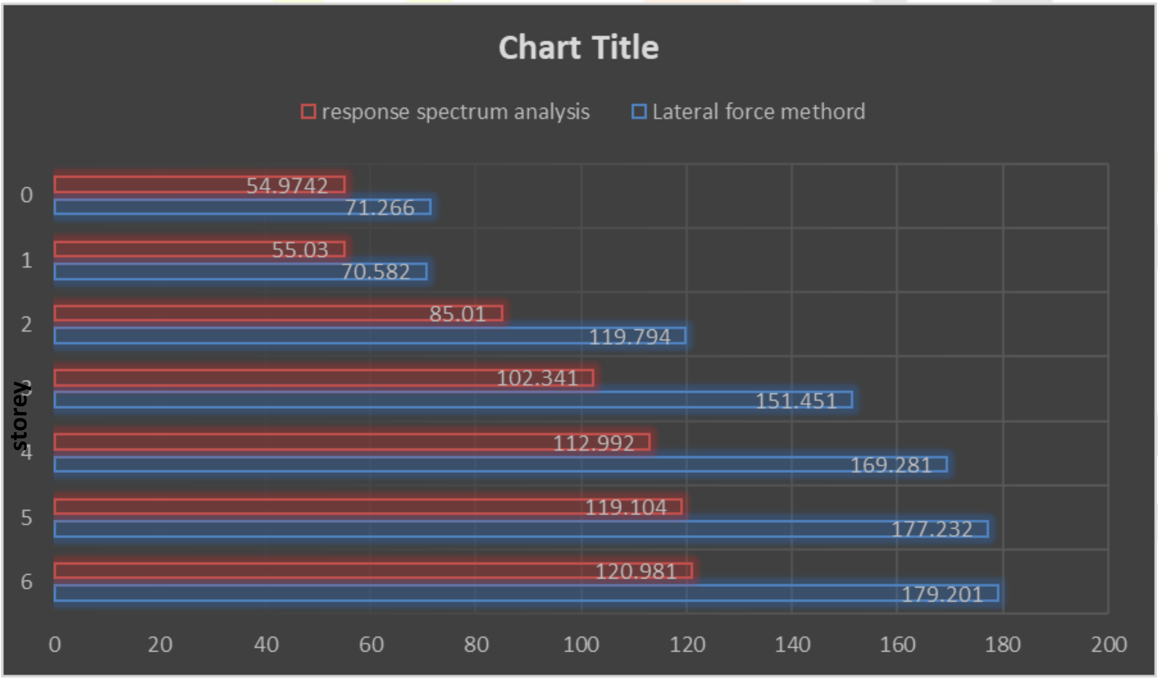


Fig (8.6)Graph of comparison of storey shear

Final results compared with initial design result:

Table(8.3)Drift:ByLateralForceMethod

Storey no.	Pre design drift(cm)	Post design drift(cm)	Difference in%
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

It is observed that the difference in drift in post and pre design is almost as high as 62.08% at an average per storey.

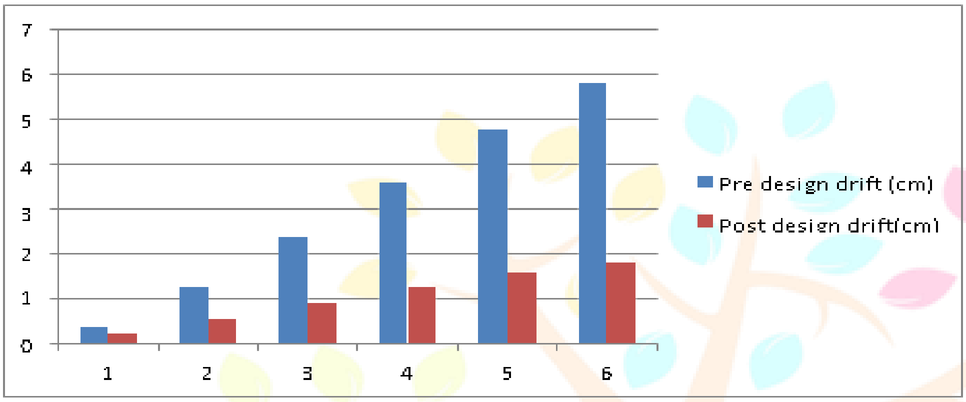


Fig (8.7) Graph of storey drift for final and initial design results

Response Spectrum Method:

Participation factor:

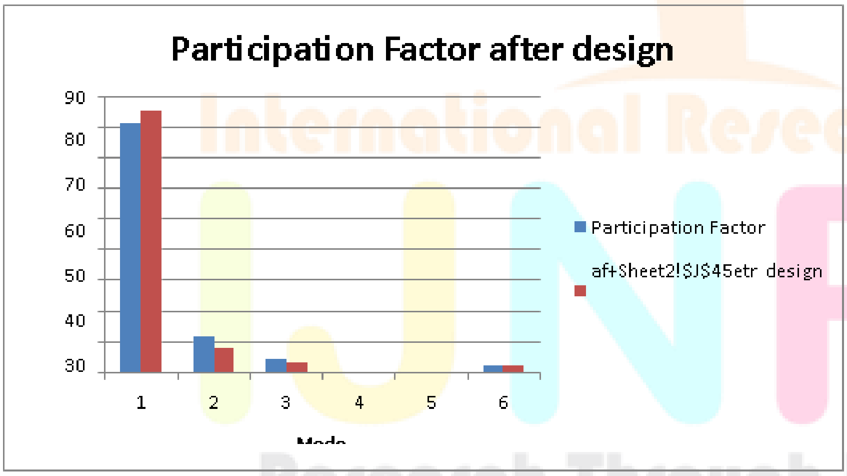


Fig.(8.8)graph of mode participation for final and initial design results

Total amount of steel required in the form of connection sand member sections are more for analysis and design based on response spectrum method than lateral force method

9 CONCLUSIONS:

1.) Inter storey drift was found out using lateral force method and response spectrum method and it was found that the displacements of response spectrum method were less than that of lateral force method.
2.) Storey shear found by response spectrum method is less than that found by lateral force method.
3.) The difference in results of response spectrum and lateral force method are attributed to certain assumptions prevalent in the lateral force method.
4.) The fundamental mode of the building makes most significant contribution to the base shear.
5.) The total building mass is considered as against the modal mass that is used in dynamic procedure. Both the assumptions are valid for low and medium rise buildings which are regular.
6.) As observed in the above results the values obtained by following dynamic analysis are smaller than those of lateral force method. This is so because the first mode period by dynamic analysis is 0.62803 is greater than the estimated 0.33 s of lateral force method.
7.) The analysis also shows that the first modal mass is 85.33% of total seismic mass. The second modal mass is 8.13% of the total seismic mass m and the time period is 0.19s.
8.) In the post design analysis, the inter storey drift and base shear both have decreased significantly owing to heavier member sections leading to safe design. For example the initially used sections (eg:-ISMB 350) have failed and Staad Pro has redesigned and adopted higher section (eg:-ISWB 600 A)
9.) The steel takes off or the cost of steel used (which is directly proportional to the amount of steel used) is less in lateral force method as compared to the response spectrum method. This is so because the response spectrum method, being dynamic in nature, is a more accurate method taking into account many more parameters like mode shape, mass participation factors to calculate the seismic vibration results. Response spectrum method is more realistic method of analysis and design of steel building frame and from the present work it is found that lateral force method leads to more cost effective of seismic design of steel frame.
10.) The amount of steel required for seismic design by using lateral force method is found to be 19.73% less than that by using response spectrum analysis
11.) Because of the heavier sections used in response spectrum method the absolute displacement, storey drift are less than lateral force method
12.) It is found that the inter storey drift sensitivity coefficient θ does not differ much in both the methods of analysis
13.) The values of resultant base shear in lateral force method is 49.33 % more than that of response spectrum method

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