Comparative Study of Steel and RCC Structures Under Seismic Conditions

.S. Prathyusha, G. Ramesh*

Department of Civil Engineering, Raghu Institute of Technology, Visakhapatnam, India
Assistant Professor, Department of Civil Engineering, Raghu Institute of Technology, Visakhapatnam, India

ABSTRACT

In the world the most widely used material for building construction is Steel. Steel is playing an important role as a building material with combination of concrete as Reinforced cement concrete in the modern days. With the invention of steel as a building material revolution in high rise construction has been started. The genetic properties of steel like strength, toughness, ductility and other appropriate properties made the engineers to select as a building material and also made the characteristics that are useful for seisimic design. As a designer engineer he should know the application of the steel design rules which is given in code books for the seismic applications. The seismic design of building frame in this paper is based on IS 1893-2002 and codal provisions are considered for the design is IS 800- 2007 and STAAD Pro is used for analysis. The present work aims to analyze and design of a multi-bay and multi storied(G+5) steel structure for earthquake forces following IS 1893-2002 and design as per IS 800-2007.The selection of arbitrary sections have been done following a standard procedure and corrections are done accordingly for earthquake loads. The two methods that have been used for analysis are Equivalent static load method and Response Spectrum method. The frame has also been further checked for P-Δ analysis and required corrections has been done and final results are documented. Finally, the design of connection of an interior joint and an exterior joint of the frame have been done and the calculations have been shown and figures are drawn. The cost efficiency of both the methods have been compared.

KEYWORDS Seismic design, Equivalent static load method, Response Spectrum method and P-Δ analysis

INTRODUCTION

Many different types of loads such as dead, live, snow, wind, and seismic loads have been used in building codes for decades. Seismic loads are one of the most uncertain types of loads that have required engineers to consider in the design of buildings for many years. There have been a considerable amount of research work and study on different aspects of earthquakes and their consequent effects on the buildings in order to provide engineers with simple and practical instructions for performing a seismic design.

Severe earthquakes occur sparingly. Although it is possible to design and construct buildings for these earthquake events, it is generally considered uneconomical and un-necessary to do so. The seismic design is performed with the foresight that the earthquake which is severe would cause some damage and a seismic design philosophy on this basis has been developed over the years.

The aim of the seismic design is to cap the damage of the building to an acceptable level. The buildings designed with that goal in mind should be able to resist minor levels of earthquake ground motion without damage, resist medium levels of earthquake without structural damage, but possibly with some non-structural damage, and resist major levels of earthquake ground motion without collapse, but with more structural as well as non-structural damage. Steel structures are good at resisting earthquakes because of its ductility. The failures of many building may be explained by some of the specific features of steel structures. There are two ways by which the earthquake may be resisted:

a) structures which are made of sufficiently large sections subjected to only elastic stresses

b) structures which are made of smaller sections, designed to form numerous plastic zones.

A structure designed to the first way will be having large sections 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.

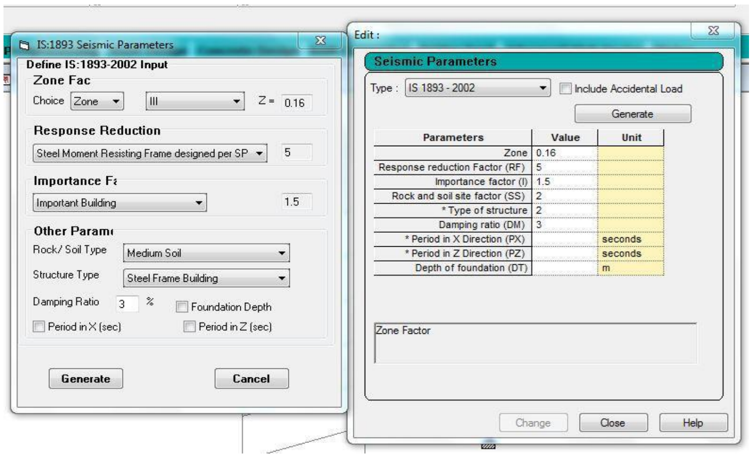
The structures global behavior is “ductile‟ and corresponds to concept

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.

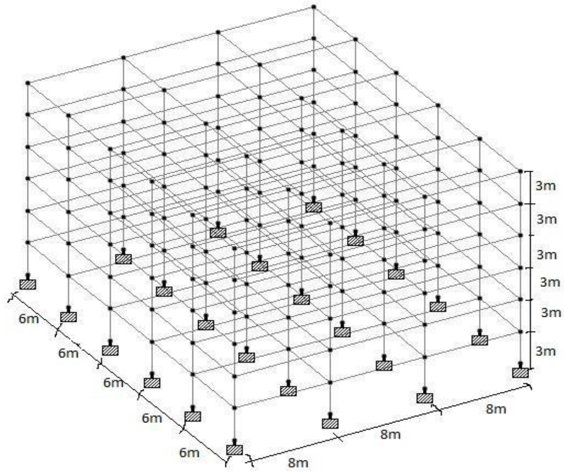
A ductile behavior, which provides extended deformation capacity, is generally the better way to resist earthquakes. One reason for this is that because of the many uncertainties which characterize our knowledge of real seismic actions and of the analyses we make, it may be that the earthquake action 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:
.. The ductility of steel as a material
.. The many possible ductile mechanisms in steel elements and their connections
.. The effective duplication of plastic mechanisms at a local level
.. Reliable geometrical properties

.. 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
 designs and constructions made by professional
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 are as follows
• Seismic zone: 3
• Zone factor “Z‟: 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%

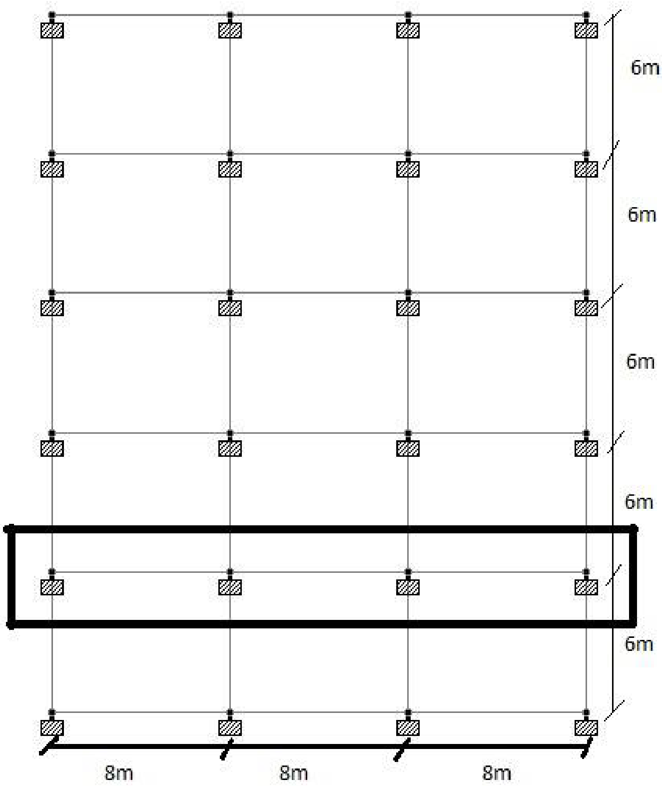
ANALYSIS AND DESIGN:



STAAD input of siesmic parameters

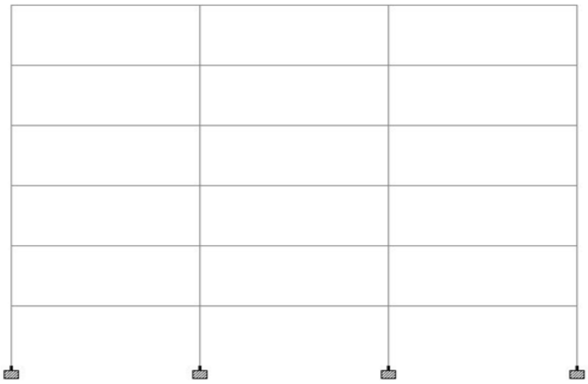


3-dimensional view of the steel building frame

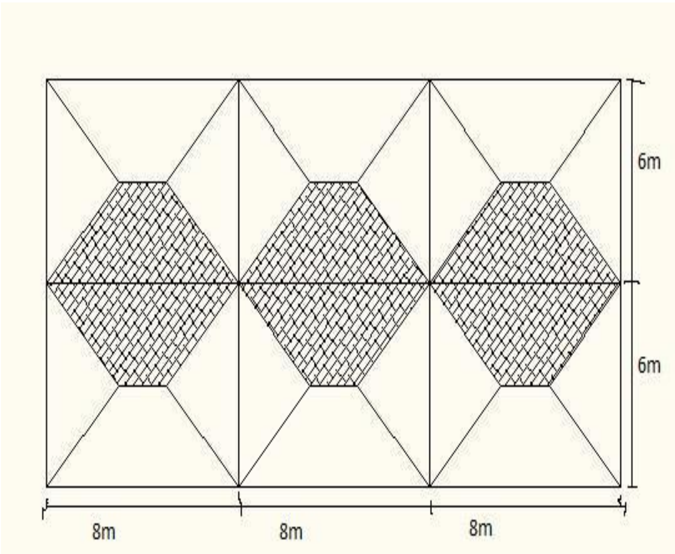


Plan of the building frame

LOAD PARAMETERS:
Dead load is taken as = 5KN/m2 and live load is taken as 3 KN/m2
Load Calculation:



Elevation of the building frame



Load distribution diagram

Load on beam along horizontal direction
1. Dead Load = 30m2 5KN/m2 = 150KN

Uniformly Distributed Load = 150/8 = 18.75KN/m
2. Live Load = 30 3 = 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)

METHODOLOGY:
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 unavoidably 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.

LATERAL FORCE METHOD:
The seismic load of each floor is calculated at its full dead load and imposed load. The weight of columns and walls in any storey should be appropriately divided to the floors above and below the storey. Buildings designed for the storage purposes are likely to have large percentages of service load present at the time of the earthquake. The imposed load on the roof is not considered.

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

After the base shear is determined, it should be distributed along the height of the building using the following expression

$$Qi = V_b\left(\frac{w_i h_i^k}{\Sigma w_j h_j^k}\right)$$

Where Vb is total design lateral force.
Wi is the seismic weight of floor i
Hi 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, Ta= 0.09h/√d
where „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.

After obtaining the seismic forces acting at different levels, the forces and moments in different members can be obtained by using any standard computer program for various load combinations specified in the code. 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.

Analysis 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	0.003869	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

RESPONSE SPECTRUM ANALYSIS:

In the field of seismic analysis this is one of the most popular methods. The design spectrum diagram is used to perform it. The response spectrum method uses the idealization of a multi storey shear building by a basic assumption. The assumption used is that the mass is lumped at the roof diaphragm levels and at the floor levels. The diaphragrams are assumed as infinitely rigid and the column axially inextensible but laterally flexible. The dynamic response of the spectrum is represented in the form of lateral displacements of the lumped mass with the degrees of dynamic freedom (or modes of vibration n) being equal to the number of masses. The undamped 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 (Φ) of the modes in vibration can be obtained. The distribution of mass and the stiffness of the building determine the mode shapes.

As the ground motion is applied at the base of the multi mass system, the deflected shape is but a combination of all mode shapes, which otherwise can be obtained by superposition of the vibrations of each individual lumped mass. A modal analysis procedure is utilized in determining the dynamic response of multi-degree-of-freedom system. Modal analysis as suggested by IS 1893 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.)

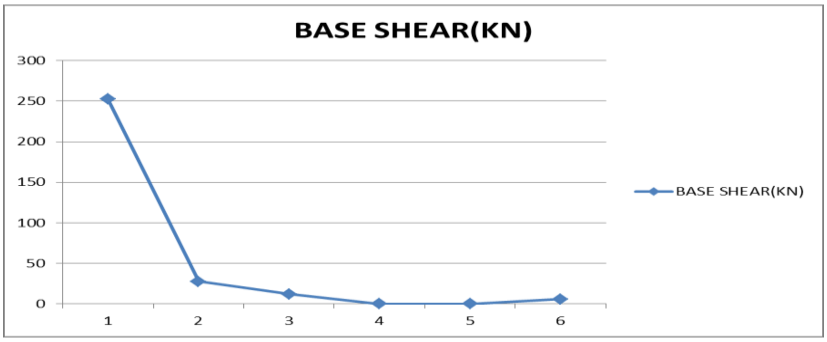
Response is obtained by using different modal combination methods such as square-root-of-sum-of-squares method(SRSS)or the complete quadratic method (CQC) which are used when natural periods of the different modes are well separated (when they differ by 10% of the lower frequency and the damping ratio does not exceed 5%.The CQC is a method which can account for modal coupling methods suggested by IS 1893.

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

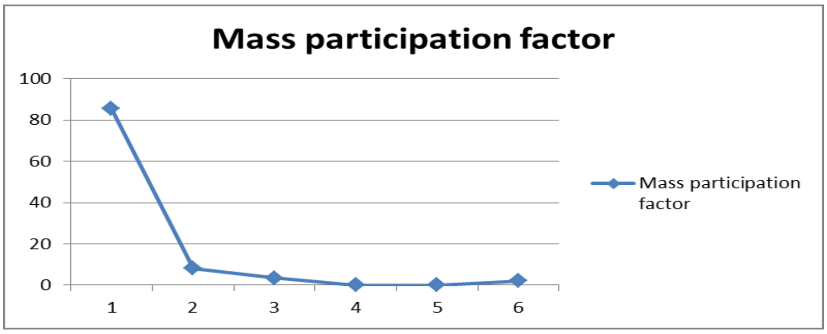
Analysis by response spectrum method.

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

Base shear and mass participation factor



Graph of modes Vs base shear



Graph of mass participation factor

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.

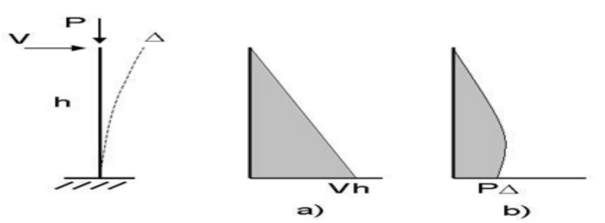


Fig 3.4: figure showing P-Δ effect

• Due to this load, the column undergoes a relative displacement or drift Δ. In this case P-Δ effect results in a secondary moment Ms=PΔ. Which is resisted by an additional shear force PΔ/ L in the column. This secondary moment Ms=PΔ will further increase the drift in the column and consequently will produce an increment of the secondary moment and shear force in the column.

• To calculate the final drift Δtx add additional drift resulting from the incremental overturning moment i.e. Mxθx, (Mx θx)θx, ((Mxθx)θx)θx . . . . . to the primary drift Δx i.e.
• Δtx=Δx+ Δxθx + Δxθx2 + ...
• Which is equal to = Δx(where θx= =
• Mx‟= secondary moment
• Px=total weight (DL+LL) at level X & above
• Δx= drift of storey X
• Vx= shear force of storey X
• Hx=height of storey X
• The code (uniform building code UBC) stipulates that the P-Δ need not be evaluated when the ratio of the secondary moment Mx‟ to the primary moment Mx at each level of the building is less than 0.1

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

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

*Beams in this storey failed to satisfy P-Δ effect

From the above table checks are made on the limitation of P-Δ effects with the results from the **lateral force method**.

The value of resultant base shear is: 179.201KN
ϴ <0.1 at storeys 1,4,5,6. bending moment and other action effects found from the analysis at storeys 2 and 3 have to be increased by 1/(1-ϴ (1.11at storey 2 and 1.12 at storey 3)
The maximum bending moment is at storey 2 : 230.172KNm
With the 1/(1-ϴ) increase: 1.11164×230.172=255.868KNm
Beams are ISMB350: M=Zx × fy=877×250=219.25KNm And 219.25<230.172
(**So beams are failing**)

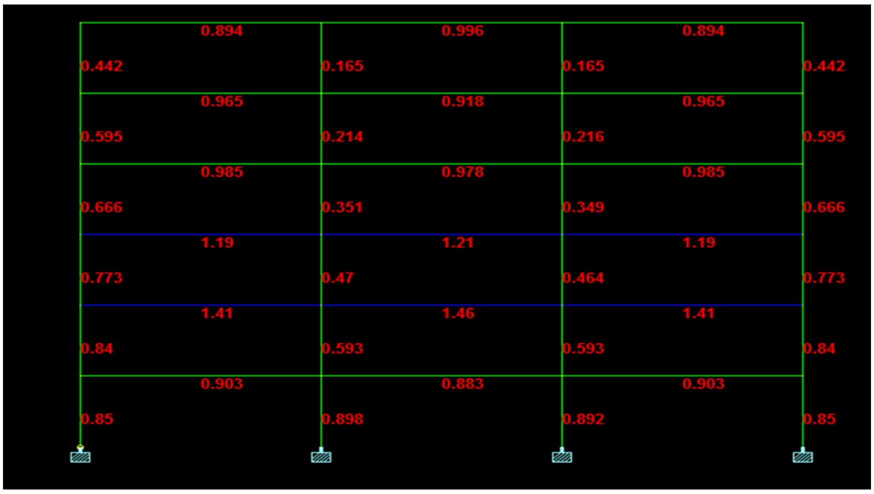
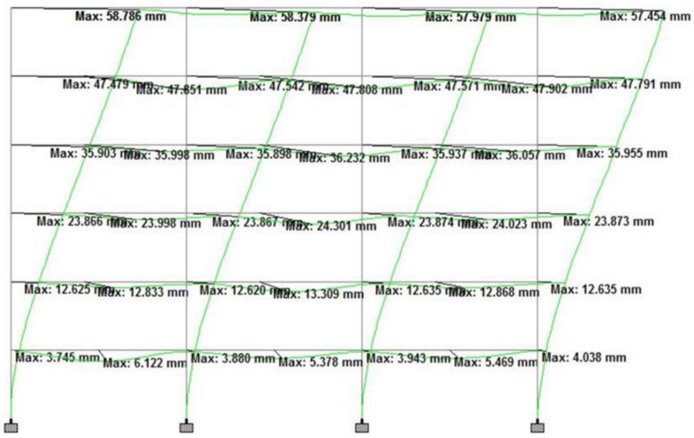
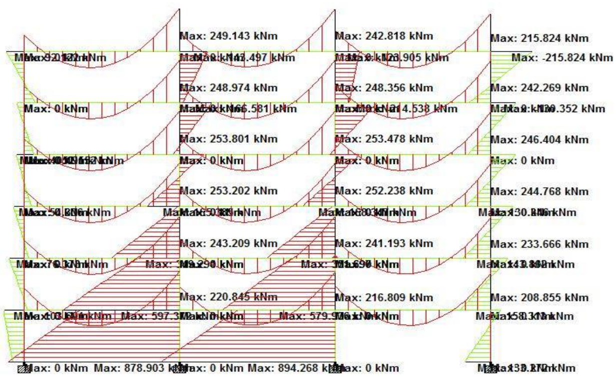


Diagram showing failed members

RESULTS
RESULTS OF LATERAL FORCE METHOD:
Maximum bending moment, shear force etc. are obtained for load combination 1.7(EQ+DL)

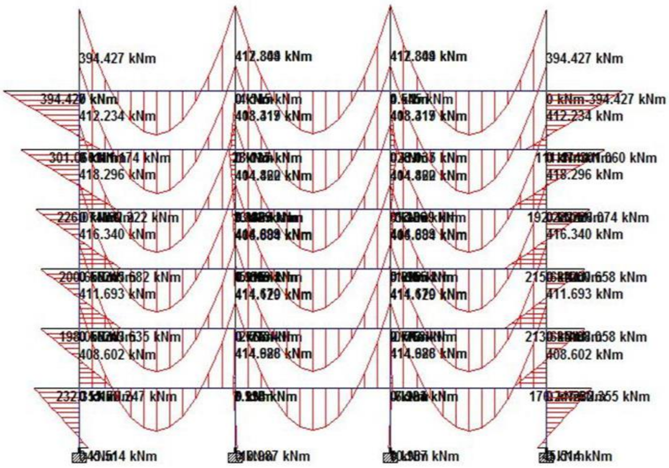


Displacement diagram for load combination 1.7(EQ+DL)
The inter storey drift as seen from above diagram is within the limits of deflection of the code i.e. it is within .004 of storey height= 0.004X3000= 12mm.

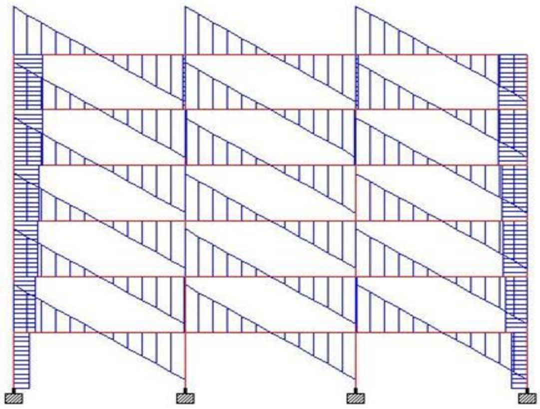


Bending moment diagram for load combination 1.7(EQ+DL)

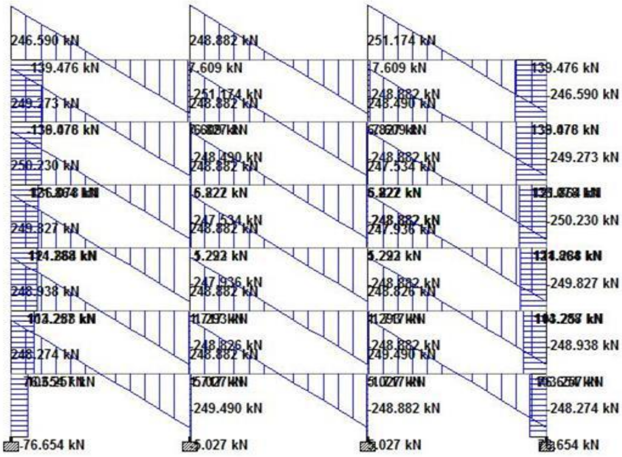
RESULTS OF RESPONSE SPECTRUM ANALYSIS:
Maximum bending moment, shear force etc. are obtained for load combination 1.3(DL+LL+EQ)



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



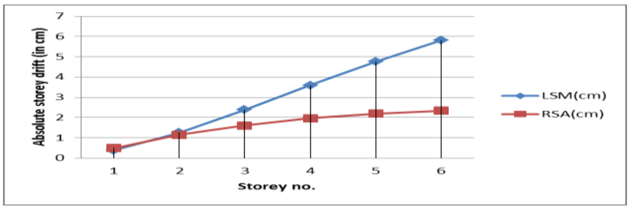
Shear force diag.in X-axis



Axis shear force diag. in Y-axis

Comparison of absolute storey drift in both methods

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



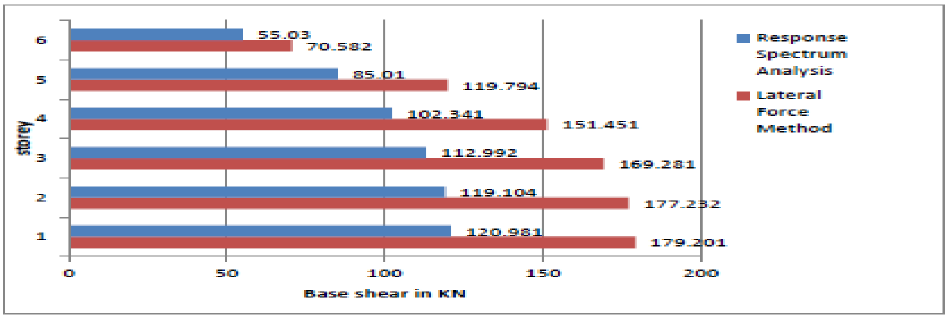
Graph of comparison of absolute storey drift

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.

Graph of comparison of storey shear



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RESULTS AND DISCUSSION

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 was 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. They are:
a. The fundamental mode of the building makes most significant contribution to the base shear.
b. 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.
4. 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.

5. 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.
6. 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)**
7. The steel take 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.
8. 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
9. Because of the heavier sections used in response spectrum method the absolute displacement, storey drift are less than lateral force method
10. It is found that the inter storey drift sensitivity coefficient θ does not differ much in both the methods of analysis
11. The values of resultant base shear in lateral force method is 49.33 % more than that of response spectrum method