SEISMIC EVALUATION AND DESIGN OF HIGH RISE STEEL BUILDING FRAMES

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Abstract - *Steel is one of the most frequently used construction material all over the world. The major characteristics of steel are its toughness, strength and higher value of ductility which are considered to be very ideal for seismic analysis and design. To use these merits during seismic design of structure, various codes related to steel design have been used so far. In this present work, a high rise structure G + 5 is considered in the form of moment-resistant frame. These frames are multi-bay type having total five bays in transverse direction and three bays in longitudinal direction. The selection of particular section is done as per IS : 800 – 2007 by the standard procedure. The frame considered is analyzed in STAAD. Pro software. It is subjected to Response Spectrum Analysis (RSA) Method and Equivalent Static load method. Based on both the types of analysis, storey displacement and base shear are calculated and compared for both the methods.*

***Key Words*:** Ductility, Storey Drift, Lateral Displacement, Base Shear

1. INTRODUCTION

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.

Due to the higher ductile properties of steel, the steel structures show the very good performance against the seismic resistance. The past studies and field experience represents that the behavior of the steel structures is very well and defined observed when these are subjected to earthquakes. At the world wide scale, the structural failures and large numbers of casualties are mostly associated with structures which are composed of other materials such as RCC, Brick Masonry. It can be typically explained by some prime features of the steel structures. The earthquake may be resisted by two means:

Mode 1: The structures composed of larger sections or spans, are subjected to the elastic stresses only.

Mode 2: The structures composed of the smaller spans or sections, are designed towards the formation of various zones of plastic hinges.

The structures which are designed for Mode 1 are supposed to be heavier and do not provide the marginal safety against the seismic actions that are observed to be in more numbers than their expected values, as the failure of element is not proved to be of ductile nature. In the present case, the global behavior of the structures is found to be brittle and relates to the concept of Base Shear Displacement diagram.

The structures which are designed for Mode 2, the selected structural components of are designed intentionally to form the deformations in the cyclic plastic form without any failure to the structure, and the complete structure is designed as a whole in such a way that the selected zones are deformed plastically. In such case, behavior of structure is considered to be ductile and can be related to the concept of Base Shear Displacement diagram. The structural system may dissipate the sufficient amount of energy in the plastic zones of the structures, which is represented by the area covered in the Base shear – displacement curve. As a result, these two modes of designs are resulted into the form of dissipative and non-dissipative structures.

1.1 DUCTILITY BEHAVIOUR

The ductility behavior, that extends the deformation capability, is found to be the better approach against the earthquake forces. This may be due to the reason of its uncertainties that characterize the knowledge of the designer for the actions against the real seismic forces. Any excess in the values of earthquake forces than the designed values, the ductile behavior may be ensured, and these are easily counteracted by the dissipation of seismic energy due to the formation of the plastic deformations of the various structural components. The same structural components are not able to impart more strength in the form of the elastic resistance if mode 1 is adopted for designs. Moreover, the decrease in the value of base shear results into the uniform decrease in applied forces towards the foundations, that leads to the decrease in the lower costs for the construction of the structures.

The various steel structures are observed to be very good at imparting the capability of the energy dissipation. The followings may be the reasons for the capability of the energy dissipation:

- Higher ductile value of the steel
- Reliable geometrical configurations and their properties
- Possibility of the formation of the ductile mechanisms in components of steel structures along with their connections
- The net effective plastic mechanism formation at the local level of structure
- Comparatively low sensitiveness towards the resistance of bending moment in the elements of structure due to existence of the coincident axial forces

The Various types and forms of mechanisms of the energy dissipation to the components of steel structures, and their dependency on each of them, are the basic features that elaborate the excellent seismic behavior tendency of the structures. In addition to this, the steel structures try to exhibit the more reliability to the earthquake behavior as compared to other structures which are composed of other materials like RCC, stone masonry and brick masonry, etc. due to numerous factors which characterize them in terms of their strength of material guaranteed, their designs aspects.

2. RESEARCH OBJECTIVE

- To design the Steel framed high-rise structure with the STAAD.Pro
- To check the designed members as per IS: 800 – 2007.
- To analyze the designed frame on the basis of Lateral Force Method and Response Spectrum Method of Analysis.
- To analysis and compare the results of story drift and story shear for high rise steel framed structure.

3. DETAILS OF THE CONSIDERED FRAME

The considered frame consists of total six stories having three bays along longitudinal axis and five bays along lateral axis. The height of each storey is 3.2 meters and the centre to centre spacing between bays along longitudinal direction is 7.5 meters and the centre to centre spacing between bays along lateral direction is 5.0 meters. The following earthquake parameters during the earthquake of structural site are given as below:

- Zone of Seismicity : IV
- Zone factor, Z (based on the Clause 6.4.2 from IS: 1893 – 2002) : 0.24
- Damping ratio: 2.5 %
- Type of Structural frame System: Steel moment resisting frame designed as per SP 6
- Response reduction factor (based on the Clause 6.4.2 from IS: 1893 – 2002) : 5
- Importance factor (based on the Clause 6.4.2 from IS: 1893 – 2002) :1.5

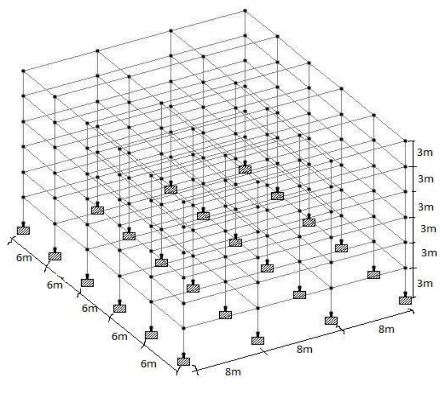

Fig. 1: 3-Dimensional View of the Steel Building Frame

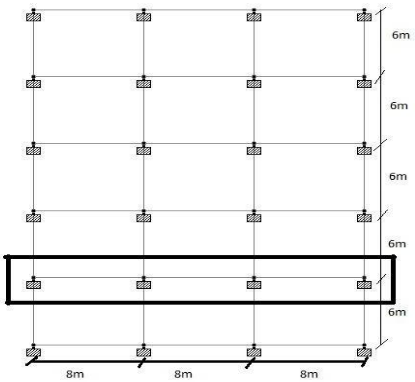

Fig. 2: Plan of the Building

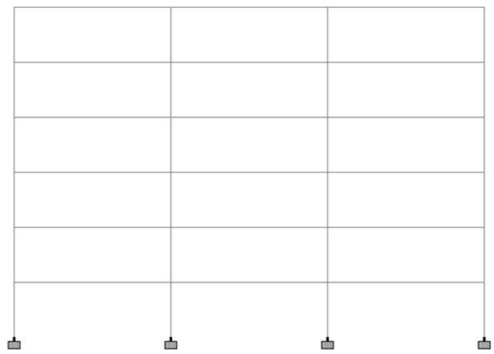

Fig. 3: Elevation of the Building Frame

3.1 LOADING PARAMETERS

The following loading types and pattern is considered for the steel frame designed.
Dead Load Considered = 4.5 kN/m^2
Imposed Load Considered = 3.5 kN/ m^2
Load on the Beam along the Horizontal Direction:

Self Weight of the Structure	=	37.5 m^2 X 4.5KN/m^2	=	168.75 KN
Uniformly Distributed Load	=	168.75 / 7.5	=	22.5 KN/m
Imposed Load over the Structure	=	37.5 m^2 X 3.5KN/m^2	=	131.25 KN
Uniformly Distributed Load	=	131.25 / 7.5	=	17.5 KN/m

Load combinations as per IS 1893-2002 :

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

3.2 METHODOLOGY

The first step includes the modeling and design of steel frame. The main process includes the member section selection for the considered frame. As we know that the dynamic analysis of the structures effects is the basic function of stiffness of member. However, the process includes large number of iterations.

The considered example includes the structure having frame type as moment resistant type (MRF) which acts as resistance against seismic response along x-direction and y-direction. These frames i.e., MRF are called as the flexible structures. Hence, their design analysis is generally characterized by the requirement to fulfill the criteria of deformation under serviceable seismic loading, or to overcome the drawback of effects of P-Δ under seismic design loading. Due to which, the rigid joints in the form of connections are usually preferred. The Preliminary design steps are:

- To design the cross-sections of beam, implementation of checks of deflection and the criteria of resistance under the action of gravitational loading.
- To follow the process of iteration, by the use of following steps to fulfill all the criteria of designs.

This process of iteration involves the use of two methods, i.e., the lateral force method and spectral response method.

1. To design the beam cross-section based on IS: 800 - 2007.
2. To define the column sections and apply the criteria of “weak beam strong column” to check the stability of connections.

3. To check the buckling/ compression at ground floor under the action of gravity loads.
4. To calculate the seismic weight based on IS: 1893 - 2002.
5. To analyze frame statically under lateral loads using STAAD. Pro.
6. To analyze frame statically under gravity loads using STAAD. Pro
7. To apply the check for stability of structure by the use of P-Δ effects (parameter ϴ) under the condition of the seismic loading.
8. To apply the check for deflection under the implementation of seismic loading.
9. To follow the analysis of response spectrum, the same procedure is followed. But, in step 5, the static analysis is primarily replaced by response spectrum analysis of the frame so that effects of seismic actions may be implemented.

4. ANALYSIS AND DISCUSSION

4.1 COMPARISON OF ABSOLUTE STOREY DRIFT IN BOTH METHODS

The absolute storey drift is determined for each storey using both the methods. Table 5.1 shows all the values of storey drift at each storey and figure 5.6 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 5.1: 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

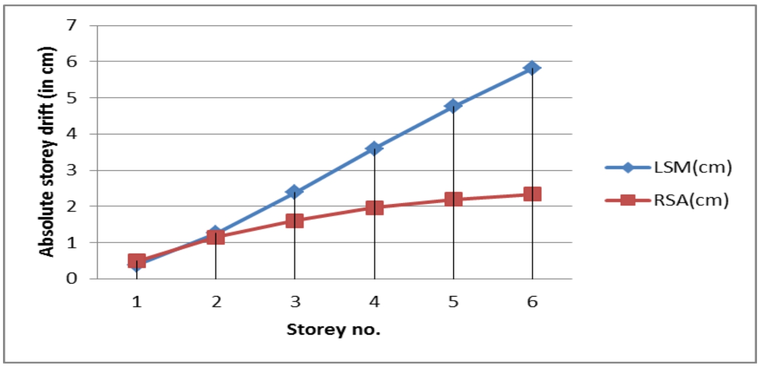

Fig. 5.6: Comparison of Absolute Storey Drift

4.2 COMPARISON OF STOREY SHEAR IN BOTH METHODS

The storey shear is determined for each storey using both the methods. Table 5.2 shows all the values of storey shear at each storey and figure 5.7 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 5.2: 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

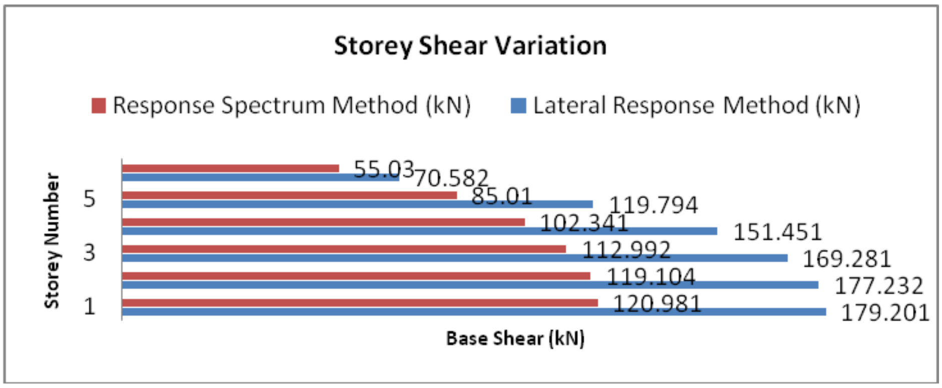

Fig. 5.7: Storey Shear Variation as per Response Spectrum and Lateral force Method

4.3 COMPARISON BETWEEN PRE-DESIGN DRIFT AND POST-DESIGN DRIFT

Table 5.3 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. 5.8 shows the variation between the values of drift pre-design and post-design members.

Table 5.3: 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

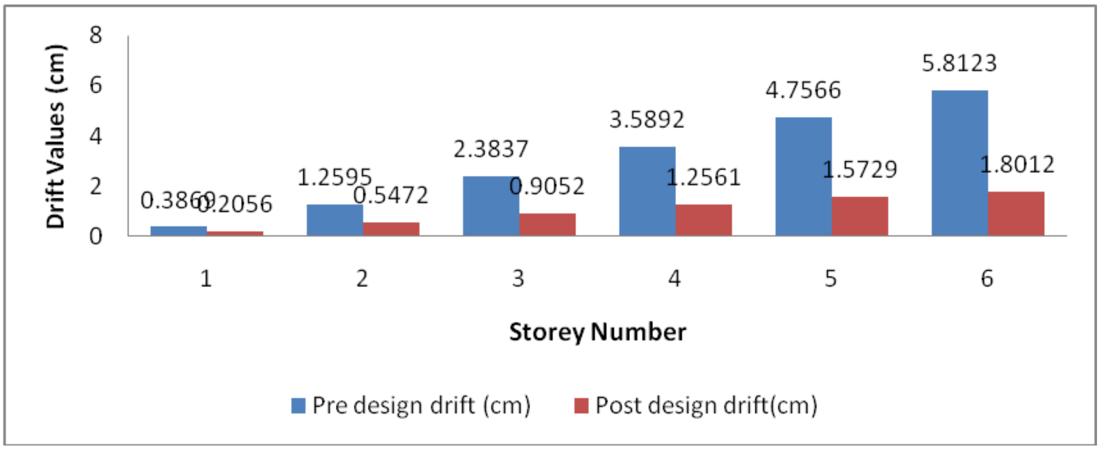

Fig. 5.8: Graphical Variation of Pre-Design Drift and Post-Design Drift Values with Storey Number

6. CONCLUSIONS

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.

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