Comparative Evaluation of Performance of Multistoreyed RC Special Moment Resisting Framed Structure

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***Abstract -** Earthquakes are known to produce one of the most destructive forces on earth. It has been seen that during past earthquakes many of the building were collapsed. Reinforced concrete special moment resisting framed structures are one of the widely used lateral load resisting system known for its enhanced ductility capacity and used for the same in high seismic risk zones. In this study the performance of RC SMRF building is studied. It is well known that users of any software for structural analysis and design do not know whether the program is having any bugs or its correctness while using. Since any program developed may contain some error or bugs it is necessary for the users to check the analysis and design results manually. Hence in this project a four storey RC special moment resisting framed building located at zone–IV was compared by modeling, analysis and designing done analytically as well as by STAAD.Pro software .The results were compared for base shear and steel reinforcement obtained from both the cases. An earthquake load is calculated by static method or equivalent lateral force method using IS 1893 (PART-I):2002. The beams and columns have been modeled as frame elements, brick infill walls is considered with no openings and the base of foundation is assumed to be fixed. For design purpose IS 13920:1993 was also used to ensure the structure to be SMRF and compared with the design of IS 456:2000.*

***Key Words**: Analysis and comparison, Design of structure, IS 13920(1993), IS 456(2000), SMRF, STAAD.Pro, Ductility, Base shear.*

1. INTRODUCTION

A detailed design of a four-storey office building having a regular layout, which can be divided into a number of similar vertical frames has been considered to illustrate the analysis and design of a frame. Only one frame in transverse direction has been considered for analysis and design. A standard computer program STAAD. Pro on personnel computer has been carried out for the analysis. The design is carried out according to IS 13920:1993 following the IS 456:2000 and SP 16:1980. The detailing of reinforcement at level 2 (as shown in fig 3.2) in the frame considered has been presented.

1.1 Special moment resisting frame

Reinforced concrete special moment resisting frames are one of the widely used lateral load resisting systems. Special moment resisting frames (SMRF) are known for its enhanced ductility capacity and used for the same in high seismic risk zones. A special moment resisting frame should be expected to sustain multiple cycles of inelastic response if it experiences design-level ground motion. The proportioning and detailing requirements for special moment frames are intended to ensure that inelastic response is ductile. Two main goals are: (1) To avoid shear failure, (2) To provide details that enable ductile flexural response in yielding regions.

2. OBJECTIVES OF STUDY

- Comparative study of structures detailed according to the IS 456:2000 and IS 13920:1993.
- Study the performance of RC SMRF structure designed as per IS 13920:1993.
- To study the behaviour of column beam joint under the application of seismic load and axial load as per IS 13920:1993.
- Detailing of selected members as per IS 13920:1993.
- Performance comparison of the designed buildings on the basis of nodal displacements and reactions.
- To check whether the input data while modeling the structure is correct or not following comparisons are made:
- Analysis and design of (G + 3) storey RC SMRF multi-storeyed building analytically and by STAAD.Pro the design methods used in analysis are Limit State Design conforming to Indian Standard Code of Practice.
- Comparison of results of base shear calculated by analytical analysis and by STAAD.Pro analysis.
- Comparison of average response acceleration coefficient (Sa/g) calculated by analytical analysis and by STAAD.Pro analysis.
- The design of beam and columns of sub frame 4-4 (as shown in fig 3.2) is carried out according to IS 13920:1993 following the IS 456:2000 and after designing comparison of results of area of steel reinforcement in columns and beams calculated by manual analysis and by STAAD.Pro software.

3. Modeling and analysis

In the present study the modeling and analysis of a multistoried RC special moment resisting frame building is carried out an earthquake load is calculated by equivalent lateral force method using IS 1893 (PART-I):2002 and the design of beam and columns of sub frame 4-4 at level 2 (as shown in Figure 3.2) is carried out according to IS 13920:1993 following the IS 456:2000. The storey shear and base shear under seismic load have been calculated manually and by STAAD. Pro software and the variation in result from both the calculation is compared. The plan is regular in nature in the sense that it has all columns equally placed. Thus, entire building space frame can be divided into a number of vertical frames. An interior frame 4-4 (as shown in Figure 3.1) is considered for analysis and design.

Table -3.1: Structural properties of

PROPERTIES	DIMENSIONS/FEATURES
Type of structure	Multi-storey rigid jointed frame
Zone	IV
Layout	Shown in figure 3.1
Number of stories	Four (G+3)
Ground storey height	4.0 m
Floor to floor height	3.35 m
External walls	250 mm thick including plaster
Internal walls	150 mm thick including plaster
Live load	3.5 KN/m^2
Materials	M 30 and Fe 415
Seismic analysis	Equivalent static method (IS 1893 (Part I):2002)
Design philosophy	Limit state method conforming to IS 456:2000
Ductility design	IS 13920: 1993
Size of exterior column	300 X 530 mm
Size of interior column	300 X 300 mm
Size of beam in longitudinal and transverse direction	300 X 450 mm
Total depth of slab	120 mm
Soil type	Medium

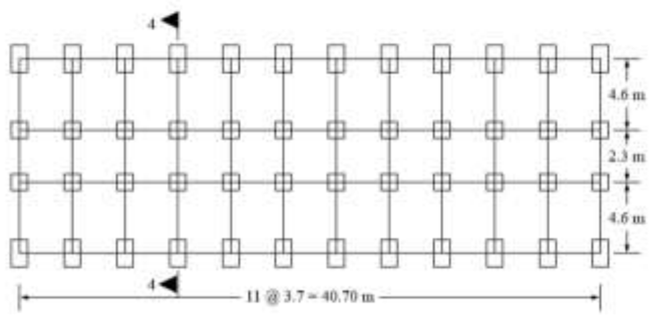

Fig -3.1: Typical plan of the building

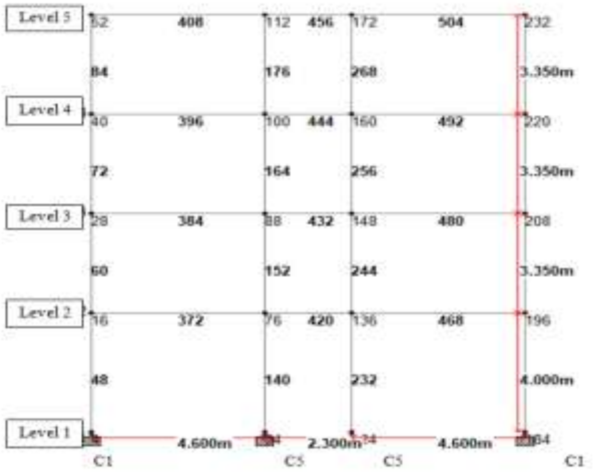

Fig -3.2: Details of sub frame 4-4

4. CALCULATION

4.1 Load calculations

Concentrated mass at roof = 5074.98 KN

Concentrated mass at 2nd and 3rd floor= 6622.80 KN

Concentrated mass at 1st floor =6622.8 KN

Total weight = 5074.98 + 2×6578.95 + 6622.8 = 24855.69 KN

Total base shear = $\alpha_h W$ = 0.06 × 24855.69 = 1491.34 KN

4.2 Beam design: as per IS 13920:1993

(i).Reinforcement at top (A_t) of section 16 (2@16φ +4@22φ) =1922 mm^2

(ii).Reinforcement at bottom (A_b) of section 16 = (2@16φ +2@22 φ) = 1162 mm^2)

(iii).Reinforcement at top (A_t) of section 76 =(2 @ 16φ+2@22φ =1162 mm^2)

(iv).Reinforcement at bottom (A_b) of section 76 = (2 @ 16φ+1@12) = 515 mm^2

(v). Shear reinforcement = 8 mm bar at a spacing of 100 mm and beyond distance 2d from Support 200mm c/c.

4.3 Column design: as per IS 13920:1993

(i).Exterior column main reinforcement = (16@16φ =3217mm^2)

(ii).Transverse reinforcement = Provide 8 mm φ two-legged stirrups about 150 mm c/c

(iii).Special confining reinforcement = 12 mm dia. bar, 80 mm c/c (iv).Interior column main reinforcement =14 @ 16 mm φ

= 2814.86 mm^2

(v). Transverse reinforcement = Provide 8 mm φ two-legged stirrups about 150 mm c/c

(vi). Special confining reinforcement = 10 mm dia. bar, 80 mm c/c

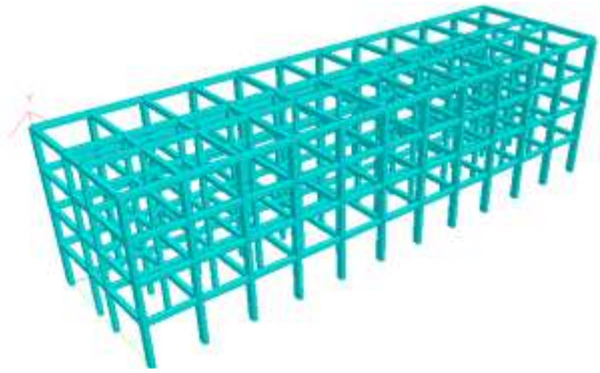

Fig -4.1: 3D rendering view of the structure in STAAD.Pro software

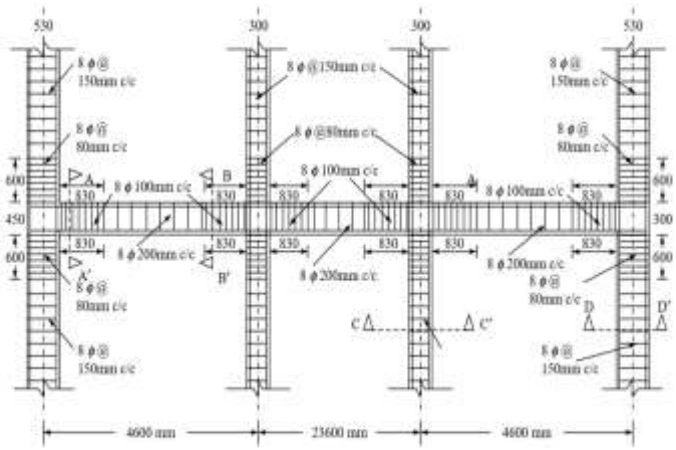

Fig -4.2: Detailing of reinforcement at a level 2 of sub frame 4-4 as per IS 13920:1993

5. RESULTS

5.1 Results of seismic analysis of sub frame 4-4

Table -5.1: Designed lateral loads at each floor

Level	Lateral loads by manual analysis (KN)	Lateral loads by STAAD.Pro analysis (KN)	Percentage (%) variation in loads wrt manual calculation
Roof (Level 5)	674.085	715.62	6.16
Third floor (Level 4)	507.05	566.5	11.72
Second floor (Level 3)	238.62	267.34	12.03
First floor (Level 2)	71.58	83.71	16.9
Total	1491.34	1633.23	9.5

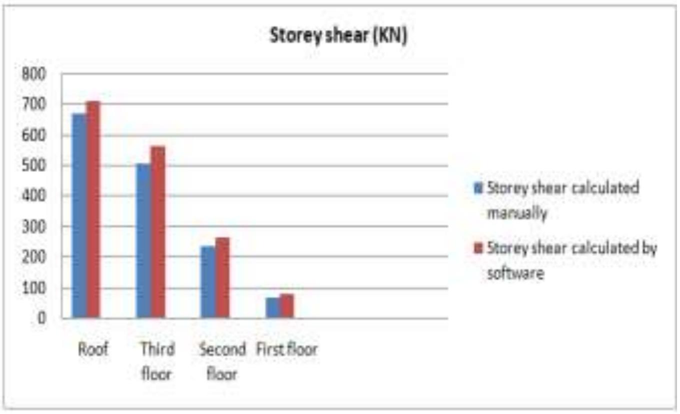

Chart - 5.1: Comparison of storey shear calculated manually and by software

Base shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. The base shear calculated by STAAD.Pro gives 9.5% more value with respect to the calculated base shear. Storey shear increases with increase in the height of the structure. The SMRF building experience less base shear in comparison with other ordinary moment resisting framed building due to one of its high response reduction factor (5).

5.2 Beam design (As per IS 13920:1993 following IS 456:2000) result and discussion

Table -5.2: Beam 372 design results

	Manual Design		STAAD.Pro Design	
Longitudinal reinforcement at	A_S (mm^2)	Reinforcement details	A_S (mm^2)	Reinforcement details
Top of sec.16	1795.61	2@16 φ +4@22 φ	1793.05	9@16 φ
Bottom of sec.16	874.91	2@16 φ +2@22 φ	896.53	8@12 φ
Top of sec. 76	961.53	2@16 φ +2@22 φ	951.18	5@16 φ
Bottom of sec.76	451	2@16 φ +1@12 φ	475.59	5@12 φ
Shear reinforcement	2 legged 8 mm φ at 100 mm c/c spacing beyond 2d dis. from support 200 mm c/c spacing		2 legged 8 mm φ at 100 mm c/c spacing beyond 2d dis. from support 160 mm c/c spacing	

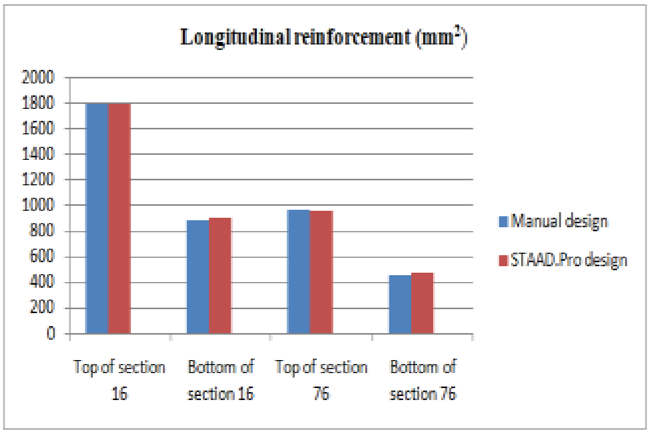

Chart -5.2: Comparison of longitudinal reinforcement of beam 372 calculated analytically and by software

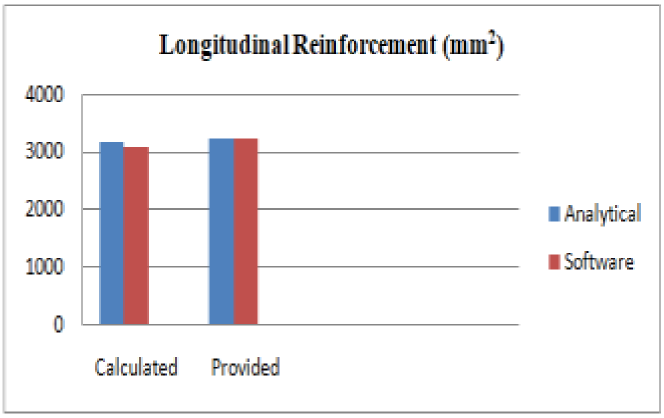

Chart -5.3: Comparison of longitudinal reinforcement of column 48 calculated analytically and by software

Sufficient reinforcement should be available at any section along the length of the member to take care of reversal of loads or unexpected bending moment distribution. As per IS 13920 design the main reinforcement provided in beam is same as designed by IS 456. Hoop spacing is same throughout the member length in case of IS 456 but IS 13920 provide hoop spacing criteria closely spaced hoops at the two ends of the beam are recommended to obtain large energy dissipation capacity and better confinement.

5.3 Exterior column design (As per IS 13920:1993 following IS 456:2000) result and discussion

Table -5.3: Exterior column 48 design results

Longitudinal reinforcement	Manual design results	STAAD.Pro design results
Area of steel provided (mm²)	3216.99	3216.99
Area of steel required (mm²)	3180	3093.01
Reinforcement details	16@16 φ	16@16 φ
Transverse Reinforcement	2 legged 8 mm φ stirrups about 150 mm c/c	2 legged 8 mm φ stirrups about 150 mm c/c
Confining Reinforcement	12 mm dia. ,80 mm c/c,l_0=600 mm towards the mid span of column	12 mm dia. ,75 mm c/c,l_0=670 mm towards the mid span of column

The area of longitudinal reinforcement provided by IS 13920 and IS 456 is found to be same. As transverse reinforcement serves to provide shear reinforcement to the member, IS 456 allows the hoop spacing to be equal to the least lateral dimension of the column while IS 13920 restricts it to half the least lateral dimension so that closer spacing of hoops is desirable to ensure better seismic performance. IS 13920 provide special confining reinforcement over the length of l_o because this region may be subjected to large inelastic deformations during strong ground shaking hence to ensure adequate ductility and to provide restraint against buckling to the compression reinforcement special confining reinforcement is provided.

5.4 Interior column design (As per IS 13920:1993 following IS 456:2000) result and discussion

Table -5.4: Interior column 140 design results

Longitudinal reinforcement	Manual design results	STAAD.Pro design results
Area of steel provided (mm²)	2814.86	3216.99
Area of steel required (mm²)	2700	2853
Reinforcement details	14@16mm φ	16@16mm φ
Transverse Reinforcement	2 legged 8 mm φ stirrups about 100 mm c/c	2 legged 8 mm φ stirrups about 100 mm c/c
Confining Reinforcement	10 mm dia ,80 mm c/c,l_0=600 mm towards the mid span of the column	16 mm dia ,80 mm c/c,l_0=670 mm towards the mid span of the column

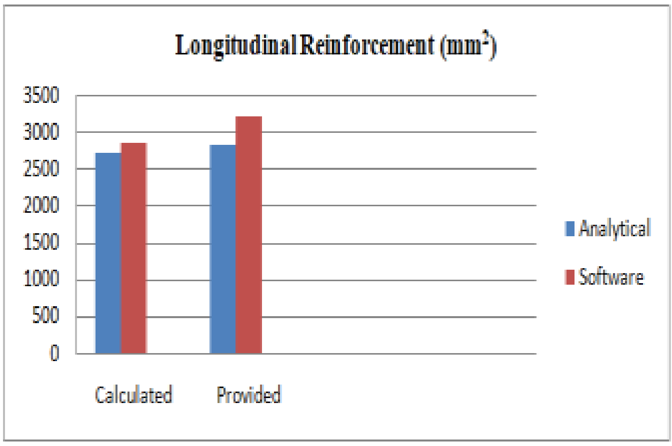

Chart -5.4: Comparison of longitudinal reinforcement. Of column 140 calculated analytically and by software

The area of longitudinal steel reinforcement required for interior column 140 designed by STAAD. Pro. showed a little more (0.575%) steel requirment when compared to manual design and more steel requirnments for confining reinforcement. IS 456 allows the hoop spacing to be equal to the least lateral dimension of the column while IS 13920 restricts it to half the least lateral dimension so that closer spacing of hoops is desirable to ensure better seismic performance. IS 13920 provide special confining reinforcement over the length of l_o.

5.5 Maximum nodal displacement of sub frame 4-4

Table -5.5: Maximum nodal displacement of sub frame 4-4

Node No.	Load Combination	X (mm)	Y (mm)	Z (mm)	Resultant (mm)
16	1.5(DL+EQX)	16.013	-0.871	-0.023	16.036
28	1.5(DL+EQX)	27.832	-1.397	0.003	27.867
40	1.5(DL+EQX)	37.283	-1.721	-0.014	37.323
52	1.5(DL+EQX)	42.666	-1.841	0.049	42.706
76	1.5(DL+EQX)	17.061	-1.489	-0.004	17.126
88	1.5(DL+EQX)	29.149	-2.39	0	29.247
100	1.5(DL+EQX)	39.193	-2.95	-0.003	39.304
112	1.5(DL+EQX)	44.945	-3.172	0.009	45.057
136	1.5(DL+EQX)	17.061	-1.489	0.004	17.126
148	1.5(DL+EQX)	29.149	-2.39	0	29.247
160	1.5(DL+EQX)	39.193	-2.95	0.003	39.304
172	1.5(DL+EQX)	44.945	-3.172	-0.009	45.057
196	1.5(DL+EQX)	16.013	-0.871	0.023	16.036
208	1.5(DL+EQX)	27.832	-1.397	-0.003	27.867
220	1.5(DL+EQX)	37.283	-1.721	0.014	37.323
232	1.5(DL+EQX)	42.666	-1.841	-0.049	42.706

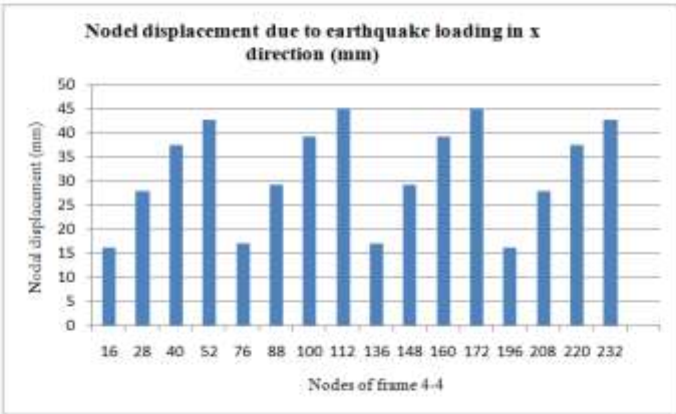

Chart -5.5: Nodal displacement of sub frame 4-4 due to earthquake load in x direction

Table 5.5 shows the value of nodal displacement for all the nodes of sub frame 4-4, the bar diagram of nodal displacement is shown in chart 5.5. From bar diagram it is analyzed that displacement of the node increases with the increase in the height of structure .The nodal displacement is more for interior node than the exterior node of same storey.

5.6 Maximum vertical reactions at support of sub frame 4-4

Table -5.6: Maximum vertical reaction at support of sub frame 4-4

S. No.	Node	1.5(DL+ELX) (KN)	Remark
1.	4	-63.244	Exterior column
2.	64	-40.961	Interior column
3.	124	-40.961	Interior column
4.	184	-63.244	Exterior column

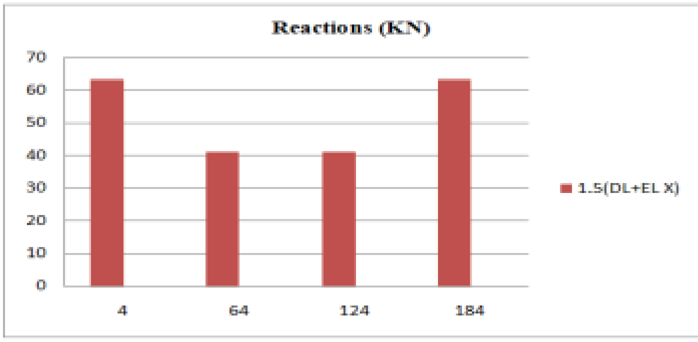

Chart -5.6: Vertical reaction at support of sub frame 4-4 due to maximum load combination 1.5(DL+EL X) (above the axis show negative Y direction)

The reaction at supports implies that the rigidness of support and to ensure that the capability of a column to transfer the load without settlement of support. From the above table it has been observed that maximum nodal reaction due to load combination 1.5(DL+EL X) in y direction is more for exterior columns than for interior columns.

6. CONCLUSIONS

1. Comparison is made between the structures detailed according to the IS 456:2000 and IS 13920:1993 and found that requirement of steel reinforcement is more in case of IS 13920 than designed by IS 456.

2. For designing beam shear reinforcement IS 13920 provide hoop spacing criteria that, more closely spaced hoops at the two ends of the beam are recommended to obtain large energy dissipation capacity and better confinement which is not provided by IS 456.

3. For providing shear reinforcement to the column, IS 456 allows the hoop spacing to be equal to the least lateral dimension of the column while IS 13920 restricts it to half the least lateral dimension so that closer spacing of hoops is obtained to ensure better seismic performance

4. IS 13920 provide special confining reinforcement over the length of l_o because this region may be subjected to large inelastic deformations during strong ground shaking hence to ensure adequate ductility and to provide restraint against buckling to the compression reinforcement special confining reinforcement is provided which is not provided by IS 456.

5. IS 13920 covers the demands of design and ductile detailing of the reinforced concrete structure under seismic condition.

6. Performance comparison of the designed buildings on the basis of nodal displacements and reactions, displacement of the node increases with the increase in the height of structure and the nodal displacement is more for interior node than the exterior node of same storey. Vertical reactions at support for exterior column are more than the interior column.

7. After analyzing the G+3 storey RC SMRF building it is concluded that the performance of that structure is more better under seismic condition thus RC SMRF building structure is found to be suitable in high seismic zone.

8. Manual analysis results were compared with the STAAD results and identified that the values of base shear in software is 9.5% more with respect to the value of base shear calculated by analytical analysis, this might be due to some simulation differences, which is very slight.

9. Area of main steel reinforcement in beam is approximately equal in design as per IS 13920 and by software, the only difference in the spacing of shear reinforcement.

10. Area of main steel reinforcement in columns is approximately equal in design as per IS 13920 and by software, the only difference in the spacing of confining reinforcement.

11. The value of average response acceleration coefficient (Sa/g) calculated by software is same as calculated manually.

12. In structural members the rearrangement of reinforcement for practical considerations in case of software design is required.

13. It is not possible to show each and every member of details that is reinforcement, so it is required to create a grouping of members and provide reinforcement details.

14. The use of software in structural analysis and design has been proven to be effective from the results output. It was observed that the time for performing the design work is reduced. The software programs can be easily misused without observing proper precautions in the analysis and design procedures which can lead to structural failures, costly disputes and poor performing structures. Hence it is important to check the results manually.

7. FUTURE SCOPE OF THE STUDY

1. Comparison of analytical results with that of the experimental results should be studied.

2. Another field of wide research could be the analysis and design of moment resisting frames considering the infill walls and shear walls as a part of the structure.

3. The study of seismic behavior of structural system could be extended using one another software

4. The study of seismic behavior of structural system could be extended considering two or more than two seismic zones

5. Construction of SMRF structure gives all time protection for the building not only while the times of earthquakes but also against vibrations created by blasts

6. The SMRF building can be designed and provided for the buildings having more than 3 floors.

7. Nonlinear analysis by push over.

8. Dynamic nonlinear analysis by time history method.

9. Capacity based design of structure.

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