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Non-Linear Dynamic Analysis of RC-Framed Structure

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Abstract: ***In this earthquake prone world, design and construction of earthquake resistant structure has greater value. In generally constructed buildings, masonry in-fills are non-structural elements and their stiffness are not considered which lead to dangerous design. If masonry infill walls are constructed as secondary elements then it act as constituent part subjected to seismic load. IN this paper, the earthquake response of RC-Framed structure is studied by manual calculation and with the help of SAP 2000. This paper will provides complete guideline for time history analysis of RC-Framed structure.***
Keywords: Joint Displacement, Storey Drift, Non-linear Analysis, Time History, Base Shear.

I. INTRODUCTION

Due to increase in population there is high demand of tall buildings. Earthquakes have potential to damage the tall buildings. Therefore it is need of designing the building by considering the earthquake loads. Nowadays it has become a need of analyzing the beam for various earthquake intensities. Earthquake intensities vary with respect to their location.

(1)SAP 2000 (Software Analysis Program) is leading software which is used to design and analysis of any structure. Many design company's use this software for their project design purpose. SAP 2000 offers the various time history of various intensities which could be applied in analyzing of a building. So, the paper mainly deals with time history analysis of RS-Framed Structure using SAP-2000 software and comparing it with manual calculations. RC stands for Reinforced Concrete Structure.

A. Case Study Details

In this analysis low rise, mid-rise and high rise buildings are considered. A building frame of G+3, G+8, G+12 are considered. It consists of 4 bays along x direction and 2 bays along y direction. The spacing along x and y direction is 5 meters. Heights of each model are 10.2m, 22.2m, 34.2m.

1) *Design Data:* Floor to floor height = 3m, Column Size = 600mm*450mm, Beam Size= 450mm*450mm, Depth of Slab = 150mm, Frame Type = OMRF, Location = Pune, Seismic Zone =III, Type of soil = Hard soil Type I, Thickness of wall = 0.23m, Dead Load = 1KN/m2, Live Load =2.5KN/m2, Wall Load = 1KN/m2, Floor Finish Load = 1KN/m2, Roof Load = 1KN/m2, Live Roof Load = 1KN/m2
2) *Description of Building Frame:* No. Bays along X axis : 4, No. Of bays along Y axis : 2, Spacing along X axis : 5m, Spacing along Y axis : 5m, Story height: 3m, No. Of floors: G+3, G+8, G+12, Size of column : 600mm x 450mm, Size of beam : 450mm x 450mm, Slab : 150mm thick

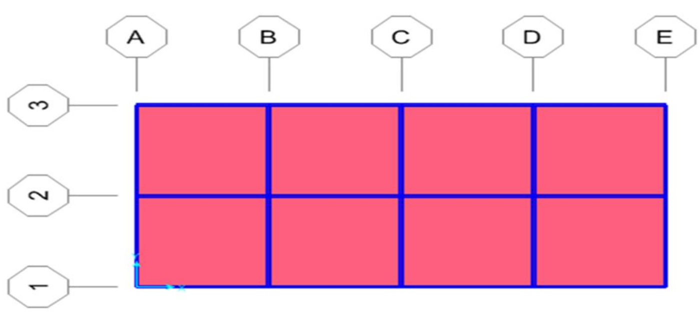

Fig.1 Plan OF Building

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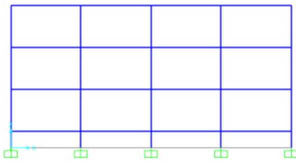
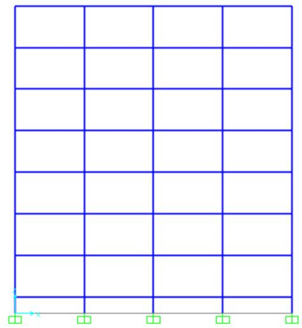
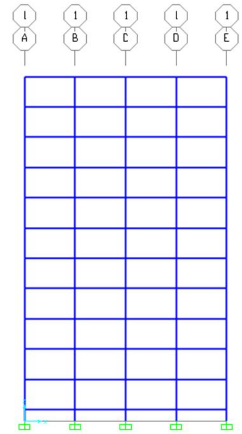
ELEVATION OF G+3



ELEVATION OF G+8



ELEVATION OF G+12



II. CALCULATION OF DESIGN BASE SHEAR

A. *IS Code Method*

VB = Ah*∑W

Where,

VB = Design Seismic Base Shear

Ah= Design Horizontal Seismic Coefficient

∑W= Seismic weight of structure

B. *Horizontal Seismic Coefficient*

Ah= Z/2 * I/R * Sa /g

From IS 1893,

Z= Zone Factor

I = Importance Factor

R = Response Reduction Factor

S_a/g = average response acceleration Factor

Average response acceleration coefficient is calculated by determining time period.

$T_a = 0.09H/\sqrt{d}$

C. *Calculations of Seismic Weight (∑W)*

∑W = Total load of Plinth level + Total load of First Floor + Total load of Second Floor+…+Total load of Twelfth floor

Where, Load= Area * Density

Table-1 Manually Calculated Values of Base Shear

		Horizontal seismic coefficient. A_h	Seismic weight of structure ∑W	Base Shear V_B
Low Rise G+3	X	0.067	11165.75	748.10
	Y	0.067	11165.75	748.10
Mid Rise G+8	X	0.45	23249.15	1394.69
	Y	0.059	23249.15	1371.69
High Rise	X	0.018	35332.55	635.98

G+12	Y	0.027	35332.55	953.97

D. Software Analysis

Table-2 Software Calculated Base Shear

MODELS	BASE SHEAR BY SOFTWARE(KN)	
	EX	EY
High Rise G+12	429.476	414.380
Mid Rise G+8	624.082	542.392
Low rise G+3	665.043	643.206

III. COMPARISON OF JOINT DISPLACEMENT

A. G+3 Storey

1) TH-X:

Table-3 Joint Displacement in X-Direction

JOINT	JOINT DISPLACEMENT(M)		
	ALTANDENA	HOLLISTER	NEWHALL
1	0	0	0
2	0.65668	0.5908399	0.096606
3	0.742962	0.6768483	0.177661
4	0.858279	0.6981787	0.230574
5	0.986126	0.7341545	0.355206

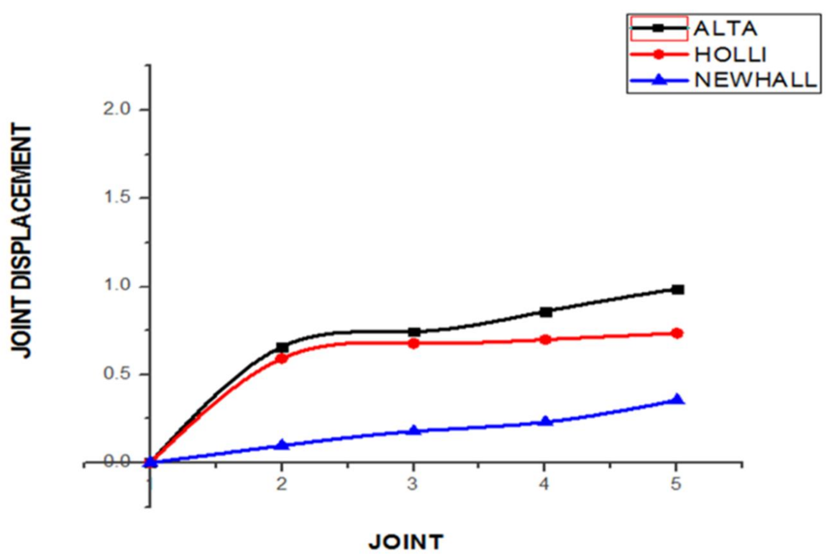


Fig.2 Joint Displacement in X-Direction for G+3

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2) Th-Y:

Table-4 Joint Displacement in Y Direction

JOINT	JOINT DISPLACEMENT(M)		
	ALTANDENA	HOLLISTER	NEWHALL
1	0	0	0
2	0.516742424	2.32436	1.281
3	0.872348485	5.8406	3.2981
4	1.480681818	5.98749	3.5412
5	2.590348485	6.66734	5.39

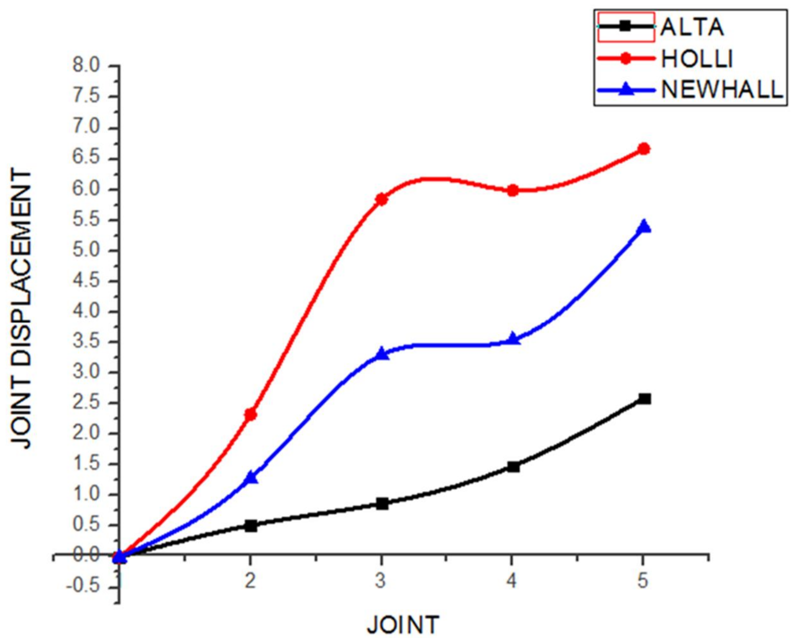

Fig.3 Joint Displacement in Y-Direction for G+3

B. G+8 Storey

1) TH-X:

Table-5 Joint Displacement in X Direction

JOINT	JOINT DISPLACEMENT(M)		
	ALTANDENA	HOLLISTER	NEWHALL
1	0	0	0
2	0.01995	0.07726	0.04785
3	0.16071	0.19624	0.14644
4	0.31031	0.23974	0.29737
5	0.40449	0.34864	0.42819
6	0.46255	0.55514	0.50255
7	0.54574	0.69084	0.59119
8	0.65186	0.78997	0.71435
9	0.72016	0.9237	0.85245

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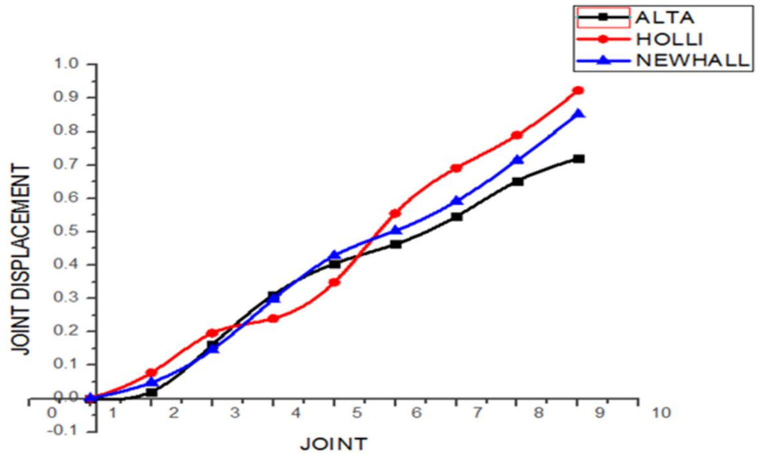

Fig.4 Joint Displacement in X-Direction for G+8

2) Th-Y:

Table-6 Joint Displacement in Y-Direction

JOINT	JOINT DISPLACEMENT(M)		
	ALTANDENA	HOLLISTER	NEWHALL
1	0	0	0
2	0.05725	0.08994	0.04958
3	0.14959	0.17353	0.16683
4	0.28012	0.25982	0.29491
5	0.3512	0.38506	0.34004
6	0.37196	0.45852	0.48312
7	0.4384	0.62506	0.52441
8	0.54218	0.67198	0.626
9	0.6465	0.75655	0.78268

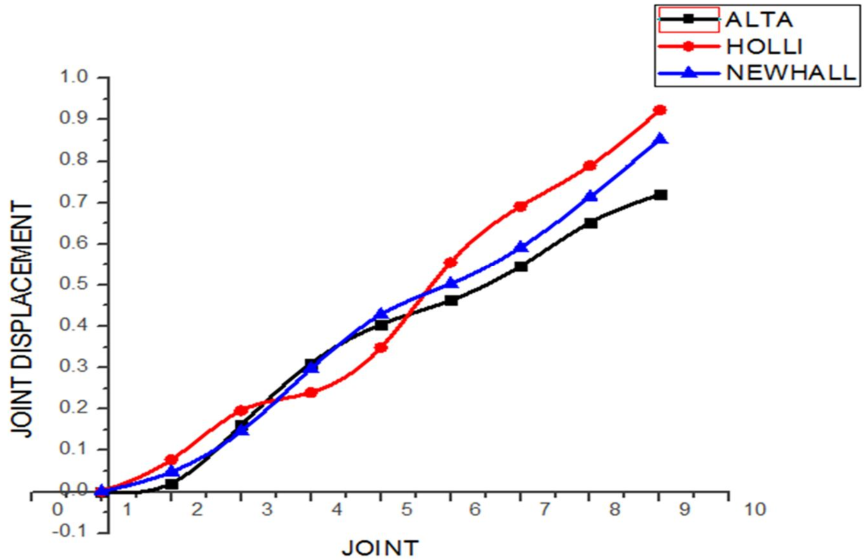

Fig.5 Joint Displacement in Y-Direction for G+8

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C. G+12 Storey

1) Th-X:

Table-7Joint Displacementin X Direction

JOINT	JOINT DISPLACEMENT(M)		
	ALTANDENA	HOLLISTER	NEWHALL
1	0	0	0
2	0.0967	0.0843	0.06316
3	0.12842	0.11659	0.11068
4	0.17147	0.18699	0.15634
5	0.23988	0.24896	0.21353
6	0.29448	0.30588	0.28306
7	0.36193	0.38542	0.35286
8	0.43897	0.46987	0.43395
9	0.54878	0.51235	0.4939
10	0.54942	0.59366	0.52644
11	0.65147	0.68784	0.56347
12	0.71525	0.74699	0.62765
13	0.82549	0.85421	0.68352

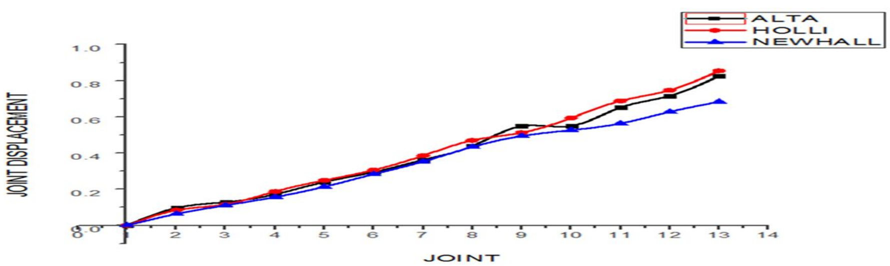

Fig.6 Joint Displacement in X-Direction for G+12

2) Th-Y:

Table-8 Joint Displacement in X Direction

JOINT	JOINT DISPLACEMENT(M)		
	ALTANDENA	HOLLISTER	NEWHALL
1	0	0	0
2	0.01181	0.0646	0.02428
3	0.11981	0.1457	0.13381
4	0.18194	0.21366	0.1936
5	0.26996	0.29687	0.24196
6	0.31163	0.35612	0.30467
7	0.3933	0.42987	0.38536
8	0.45359	0.49639	0.44564
9	0.53944	0.55649	0.51628
10	0.61408	0.63988	0.64756
11	0.69411	0.7136	0.7116
12	0.74324	0.78639	0.81749
13	0.84672	0.85366	0.88282

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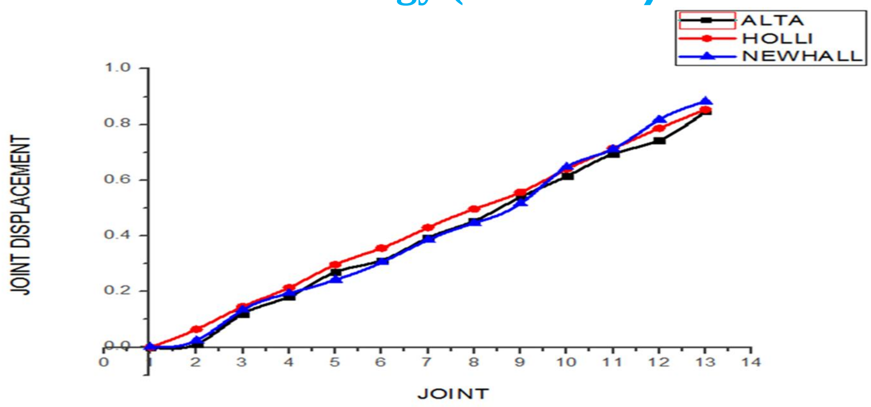

Fig.7 Joint Displacement in Y-Direction for G+12

D. Modal Analysis

Table 9: Modal Analysis

Joint	Ux Sum	Uy Sum	Rz Sum
1	0	0.78772	0.43577
2	0	0.78772	0.67752
3	0.7815	0.78772	0.7854
4	0.7815	0.88613	0.84005
5	0.7815	0.88613	0.8693

E. Time Period

NO.OF STOREY		SOFTWARE	MANUAL
G+3	X_1	0.454	0.2 T_X
	X_2	0.134	
	Y_1	0.533	0.29 T_Y
	Y_2	0.162	
G+8	X_1	0.3280	0.45 T_X
	X_2	0.09584	
	Y_1	0.3849	0.63 T_Y
	Y_2	0.11610	
G+12	X_1	2.07	0.69 T_X
	X_2	0.66	
	Y_1	2.32	0.97 T_Y
	Y_2	0.747	

IV. RESULT AND DISCUSSION

Graphical representation of variation in result as shown in the figure 5 – figure 11. The similar variation in seismic response namely joint displacement with intensities X and Y direction were seen in the graph.

Time period calculated by IS code method and by software is recorded. Modal mass participation factor is being recorded for the structure.

V. CONCLUSION

Based on the analysis and design of multi-storied building the following conclusions are made

A. Codal empirical formula to calculate time period is less compare with time period by modal analysis,imposing higher spectral acceleration which results in conservative design.

B. Modal mass participation factor for the modal in Y direction is higher compare with in X direction, indicating that this is the weak direction for the response during earthquake.

C. Modal mass participation in the torsion mode shows significant contribution and this is due to unsymmetry plan of building

D. Base shear for low rise building is observed to be higher than tall building showing that low rise building are more stiff during earthquake.

E. Due to record to record variability, the deformation responses for the three ground motions are different for the building in X and Y direction.

F. HOLLISTER ground motion record gives maximum responses and might be because this ground motion is having higher PGA, higher frequency content and longer period.

VI. ACKNOWLEDGEMENT

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