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STUDY OF EFFECT OF VERTICAL STIFFNESS IRREGULARITY ON THE BEHAVIOUR OF FRAMED STRUCTURE WITH MASONRY INFILL IN THE NON-LINEAR RANGE

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ABSTRACT

In Indian construction scenario, majority of medium rise structures is of framed reinforced concrete structural system with masonry infills. Further, depending on the functional requirements, the floor heights are not uniform thereby introducing vertical stiffness irregularities. The computer modelling using software normally takes care of the stiffness irregularity but designers seldom model the masonry infills. In usual practice, while the mass of the infills is considered, their stiffness contribution is ignored. Moreover, with the increasing requirement to make the structure fully earthquake resistant, behavior of structural system which has been designed using Response Spectrum method has to be studied in the non-linear range using Pushover analysis. This paper reports the findings of exhaustive analysis carried out on a ground plus eleven story single bay frame with and without infills, and with and without vertical stiffness irregularity in the linear and non-linear range. Further the study has been expanded to include the effect of different seismic zones as classified in Indian codes.

Keywords: multi-storey building, non-linear seismic analysis, masonry infill, equivalent diagonal strut, vertical stiffness irregularity, pushover method & SAP2000.

1. INTRODUCTION

1.1 Infills

It has long been known that the Masonry infill walls influence the strength and stiffness of the infill frame structures. In earthquake zones, neglecting the frame panel does not necessarily lead to a conservative design, because, under the lateral loads, the infill walls significantly increase stiffness by acting as a strut (compressive member). While designing a structure with infills, it is modelled in the form of equivalent diagonal struts.

1.2 Finding Equivalent Strut

A panel of masonry infill is modelled as an equivalent compressive strut. The ends of diagonal strut are assumed as pin joints connecting to RC frame. Thereafter for unreinforced masonry (URM) infill wall the equivalent depth of the infill is found out using IS 1893(Part 1):2016 [13] and the width of the infill is taken equal to thickness of masonry wall.

$E_m = 550\ f_m$

(f_m = Compressive strength of masonry prism-IS-1905[2])

$\Theta = \tan^{-1}\frac{h_i}{L_i}$

$\alpha_h = h.\left(\sqrt[4]{\frac{E_m.t.\sin 2\theta}{4E_f.I_c.h}}\right)$

Width of equivalent diagonal Strut: $W_{ds} = 0.175\ \alpha_h^{\ -0.4} L_{ds}$

Details of equivalent diagonal strut are shown in Figure 1.

L_{ds}= Diagonal length of strut(mm)

E_m = Modulus of elasticity of URM infill(N/mm)

E_f= Modulus of elasticity of MRF(N/mm^2)

I_c = MOI of adjoining column. (mm^4)

t = Thickness of infill (mm)

θ = Angle of diagonal with horizontal

h_i= Height of infill

L_i= Length of infill

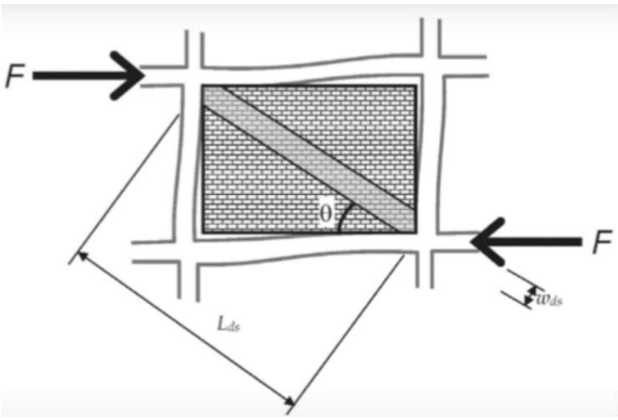

Figure-1. Details of equivalent diagonal struts.

1.3 Vertical Stiffness Irregularity

Vertical stiffness irregularity occurs when lateral stiffness of one storey is different, from the lateral stiffness of storeys above or below. Normally, such a vertical stiffness irregularity occurs when storey heights vary over the height of the structure. The different storey heights depend upon the functionality of the different storeys.

Difference in storey heights between the storeys introduces soft storey which is a stiffness irregularity. In this paper effect of the presence of soft storey at different heights, on the overall behaviour of the structure when subjected to seismic forces, is studied.

1.4 Response Spectrum Analysis (RSA)

This is the first stage analysis which is also known as linear dynamic analysis. RSA is a fundamental approach to obtain response of the structure during earthquake using mode shapes**.** With the help of mass and stiffness distribution, structural response is determined.

1.5 Pushover Analysis (PA)

Earthquake is random in nature and are unforeseeable. Further design philosophy of earthquake resistant structures envisages the structure to undergo inelastic deformation but without suffering overall collapse. Pushover analysis (PA) is a procedure in which the structure is loaded in the lateral direction in an incremental manner with certain pre-defined patterns and is a second stage analysis. This procedure helps in evaluating and estimating strength of the structure beyond its elastic stage and helps in predicting potential weak areas in a structure by keeping track of each and every member by the formation of hinges they hold. As the load increases, the structure starts undergoing yielding at some locations with the formation of plastic hinges and consequently after such yielding stiffness of the structure is modified and hence its load carrying capacity and deformation pattern. This modified structural property is considered for further analysis with incremental lateral loading till next plastic hinge is formed and the procedure is continued till the structure attains prescribed level of displacement. A typical Pushover curve between the displacement and the lateral force at various stages of analysis indicated therein, is presented in Figure-2.

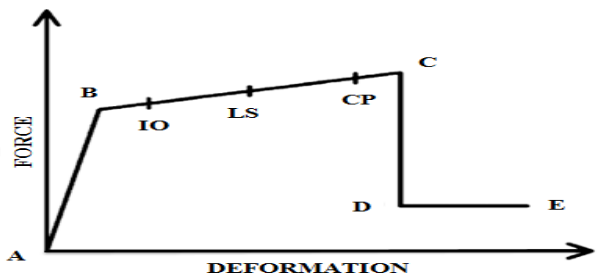

Figure-2. Pushover curve.

The salient points in the curve are:

a) Immediate Occupancy (IO) - Less damage but serviceable

b) Life Safety (LS) - Little more damage to non-structural members but serviceable

c) Collapse Prevention (CP) – Large damage to structural members & not serviceable.

1.6 Methods of Pushover Analysis

There are several static push-over methods, all with the same overall steps but different details. This paper considers two methods namely ATC 40 Capacity Spectrum Method (ATC 1996) and FEMA 356 Coefficient Method (FEMA 2000).

2. LITERATURE REVIEW

Rahul Lesli (2013) entitled ‘The Pushover Analysis, explained in its Simplicity’ presented comparison between pushover analysis (PA) and conventional structural analysis (SA) on the basis of the structural capacity, accuracy, loads, factored loads & lateral loads. Seismic behavior of the structure gets influenced due to the presence of masonry infill suggesting modelling using diagonal struts in PA. Elastic model preferred for SA whereas PA used non-linear model.

V. Mani Deep *et al* (2017) presented the paper ‘Pushover analysis of RC building: Comparative study on seismic zone of India’. Formation of hinges was within CP Limited damage experienced by the structure followed by retrofitting of column of lower storey. Severity of seismic activity on structure (G+9: 31m) was noticed when base shear, time period and displacements progressively increased from zone II to zone V respectively. The performance point changed from elastic to IO, to LS level from seismic zone II to zone V.

Numerical modelling of masonry wall (by Anna University) chapter 4- Detailed study is carried out in this chapter regarding equivalent diagonal struts. The concept of analysing the masonry infill as equivalent diagonal strut can be effectively used in the Finite Element analysis to find the natural frequencies of the structures. Smith & Carter in 1969 proposed numerical method to estimate width of the equivalent diagonal strut to replace the masonry wall. One of the most common and popular approximation is, replacing the masonry infill by equivalent diagonal strut whose thickness is equal to the thickness of the masonry infill. The problem faced in this approach was to find the effective width of the equivalent diagonal strut. Here, contact length, ‘α_h’ is related with the relative stiffness of the infill to the frame by the approximate equation whereas ‘λ_h’, is an empirical parameter and the beam contact length’ α_L’ is always approximately half of its span, and width was estimated. Comparison of width of equivalent diagonal strut by various methods were carried out.

Sarosh Hashmat Lodi *et al* (2012), entitled ‘Nonlinear Static Analysis of an Infill Framed Reinforced Concrete Building’. Adopted equivalent strut approach for modelling infill panels carried by PA for evaluating RC framed structure with G+8 storeys with masonry infill panels on the exterior and interior walls, the building has shops located at the ground floor and the mezzanine floor has offices, above floors have residential apartments with reinforced concrete lift core. The foundations have mainly isolated footings. The structure consists of beam slab system. Which is not centrally located. Detailed evaluation concluded that the structure has deficiencies which further needs to be retrofitted for enhancing the strength.

Neethu K. N. *et al* (2013), presented ‘Pushover Analysis of RC Building’. The main objective was to check the kind of performance a building can give when designed as per Indian Standards. The existing frame for pushover analysis was safe for seismic activity as the performance point base shear was greater than design base

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shear. Demand curve and capacity curve intersected near the elastic range, making structure a good resistive and safe against collapse. The behaviour of detailed RC building demonstrated appropriate.

3. RESEARCH METHODOLOGY

A G+11 story two-dimensional moment resisting framed structure with single bay of width 8 m and total height 36 m is considered in this analytical work. The structure is first analyzed without any masonry infills using RSA (SAP2000) and designed in accordance with Indian code IS 456:2000 [3].

Seismic parameters like zone factor (Z), soil properties and importance factor (I), response reduction factor (R) taken from the Indian code IS 1893 (Part1): 2002[1] are factored in the design. The key specifications of the structure are shown in Table-1.

Table-1. Data for design and analysis.

Grade of concrete	M30
Grade of steel	Fe 415
Spacing of frame	3.5 m
Floor finish	25 mm thick at $\gamma_w{}^{*}$= 24 kN/m^3
Thickness of water proofing treatment in roof	50 mm at γ_w= 24 kN/m^3
Thickness of slab of floor	150 mm at γ_w= 25kN/m^3
Thickness of Roof slab	160 mm at γ_w= 25kN/m^3
Storeys: 230mm thick brick wall	γ_w= 20kN/m^3
Roof: 1000 mm high parapet wall	γ_w= 20kN/m^3
Imposed load	Floors: 3 kN/m^2
	Roof: 1.5 kN/m^2
Column Dimension (in mm)	600 × 450
Beam Dimension (in mm)	650 × 300
E=**5000**$\sqrt{fck}$	2.7× 10^{7}kN/m^2
Struts (in mm)	540 × 230

*γ_w – Unit weight of the material

After proportioning and checking the dimensions of the member against relevant codes, the model is used for all subsequent analytical investigation.

For pushover analysis hinges are assigned for columns (P-M2-M3), beams (M3) and equivalent strut (P). The typical behaviour of the hinges in the linear and non-linear range as incorporated in SAP2000 is shown in Figure-3.

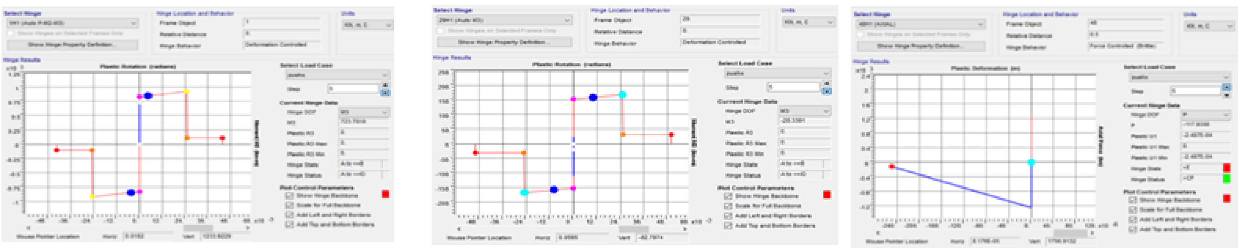

Figure 3(a) Behaviour of P-M2-M3-Column **Figure 3(b) Behaviour of M3 - Beam** **Figure 3(c) Behaviour of Axial -strut**

Figure-3. Typical M-∅ relations of Hinges.

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The analytical investigation which includes both RSA and PA consist of the following steps:

a) Analysis of bare frame models (without infills) without any stiffness irregularity as shown in Figure-4(a).
b) Analysis of frames with infills modelled as equivalent frame without any stiffness irregularities as indicated in Figure-4(b)
c) Analysis of bare frame models with vertical stiffness irregularity introduced at different storey heights starting from the ground storey to the topmost one as shown in Figure-5(a).
d) Analysis of infill frame models with vertical stiffness irregularity introduced at different storey height starting from the ground storey to the topmost one as indicated in Figure-5(b). The lateral stiffness of the soft storey is kept at 30% of the stiffness of other remaining stories.

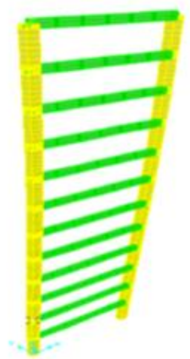

Figure-4(a). Without masonry infill.

Figure-4(b). With masonry infill.

Figure-4. Regular RC frame structure (2D) Model).

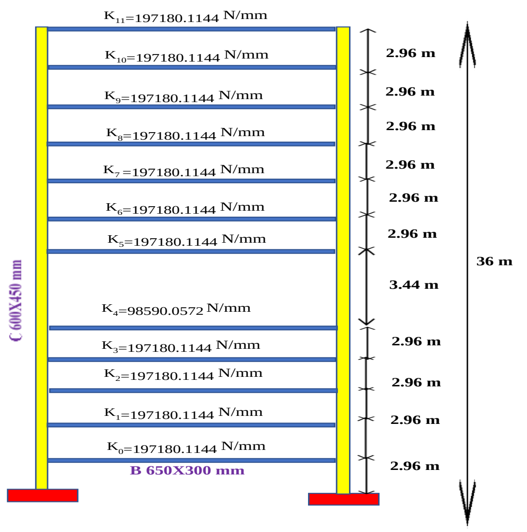

Figure-5(a). Without masonry infills

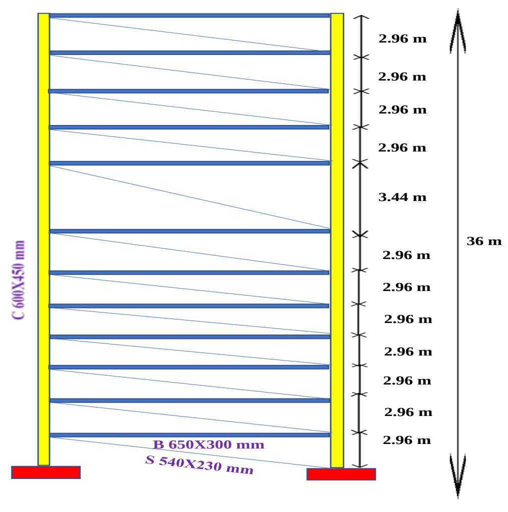

Figure-5(b). With masonry infills.

Figure-5. Vertical stiffness irregularity in the form of storey height at G+5 and G+7 floors for the frame structure with and without masonry infills.

The analyses have been carried out for different seismic zones starting from Zone III to V as per IS: 1893-2002 [1]. In total, 96 models have been analysed.

Typical mode I deformation pattern of the strucutre under RSA is shown in Figure-6.

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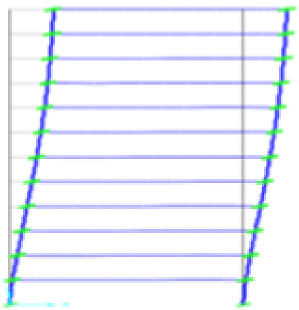

Figure-6(a). Without masonry infill.

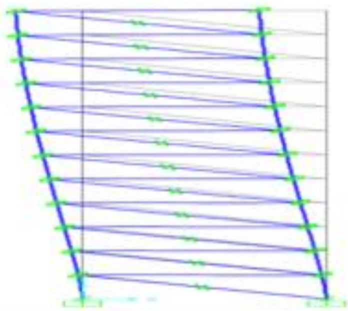

Figure-6(b). With masonry infill.

Figure-6. Response spectrum deformed shape (Mode I).

Formation of plastic hinges under PA is depicted in Figure-7.

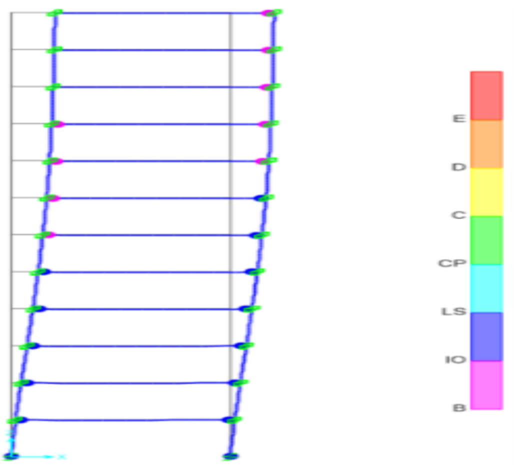

Figure-7(a). Without masonry infills.

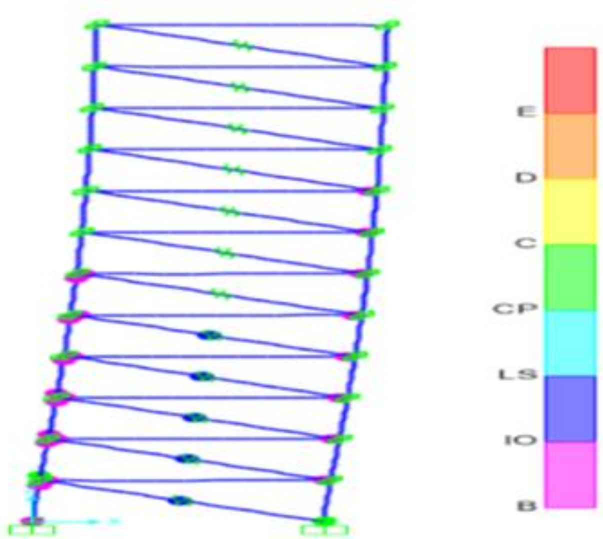

Figure-7(b). With masonry infills.

Figure-7. Formation of plastic hinges.

4. RESULTS OF THE INVESTIGATION

4.1 Frames without Vertical Stiffness Irregularity

The results of RSA and PA using ATC 40-Capacity Spectrum and FEMA 356 for strucutral models without infills in seismic zones III to V are shown in Table-2 and corresponding results for models with infills are shown in Table-3.

Table-2. Performance Point (Regular frame structure without masonry infill).

						ATC 40-CAPACITY SPECTRUM		FEMA 356	
ZONE	I	R	Z	T(MODE 1) (Sec)	Base shear (kN)	Base Shear, V (kN)	Displacement, D (m)	Base Shear, V (kN)	Displacement, D (m)
V	1.5	5	0.36	1.66214	101.499	395.206	0.144	404.22	0.178
IV	1.5	5	0.24	1.66214	67.666	367.756	0.099	404.22	0.178
III	1.5	5	0.16	1.66214	45.111	316.133	0.057	404.22	0.178

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Table-3. Performance point (Regular frame structure with masonry infill).

						ATC 40-CAPACITY SPECTRUM		FEMA 356	
ZONE	I	R	Z	T(MODE 1) (Sec)	Base shear (kN)	Base Shear, V (kN)	Displacement, D (m)	Base Shear, V (kN)	Displacement, D (m)
V	1.5	5	0.36	1.20196	149.013	977.249	0.12	1099.869	0.137
IV	1.5	5	0.24	1.20196	99.342	715.286	0.083	1099.869	0.137
III	1.5	5	0.16	1.20196	66.228	516.776	0.056	1099.869	0.137

ATC-40 Capacity Spectrum (Pushover Curve) between Spectral Acceleration and Spectral Displacement for G+11 framed structure with and without masonry infills is obtained for different seismic zones is shown in Figure-8 and Figure-9 respectively.

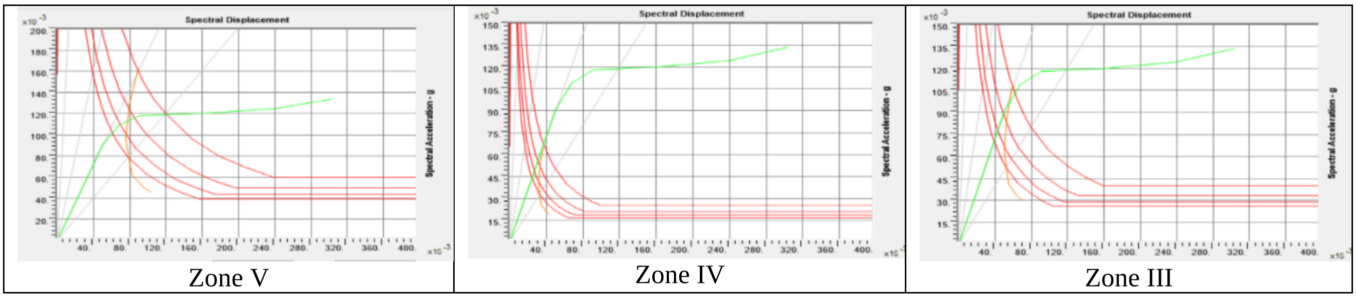

Figure-8. ATC-40 Capacity spectrum (Pushover Curve) regular frame structure without masonry infills.

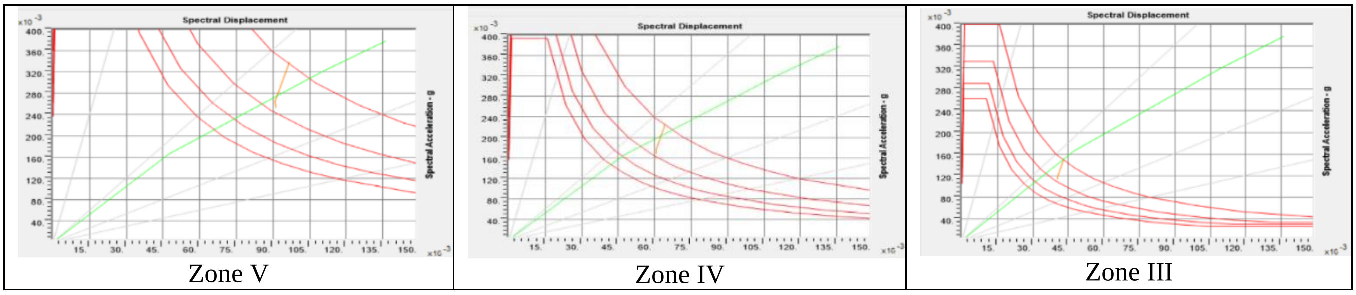

Figure-9. ATC-40 Capacity spectrum (Pushover Curve) regular frame structure with masonry infills.

4.2 Frames with Vertical Stiffness Irregularity

Analysis of vertically irregular frames using RSA and PA is done for different models by shifting the position of soft story starting from ground floor to the top most 11th story both for bare frames and frames with infills. The variation of mode I Time period with the position of soft story as determined using RSA is shown in Figure-10.

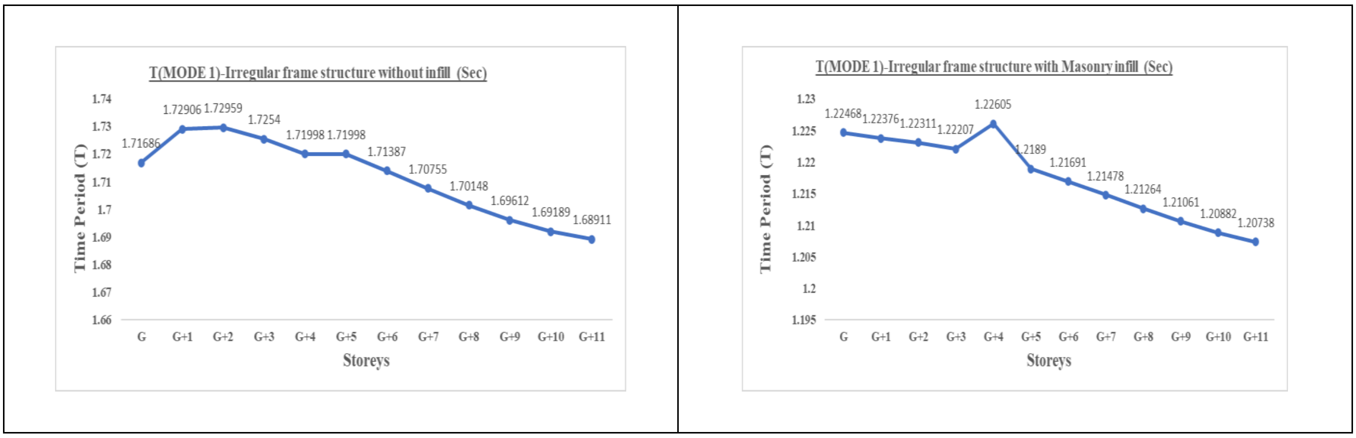

Figure-10. Time period (Mode I) at different storeys with and without infill having 30% vertical stiffness irregularity.

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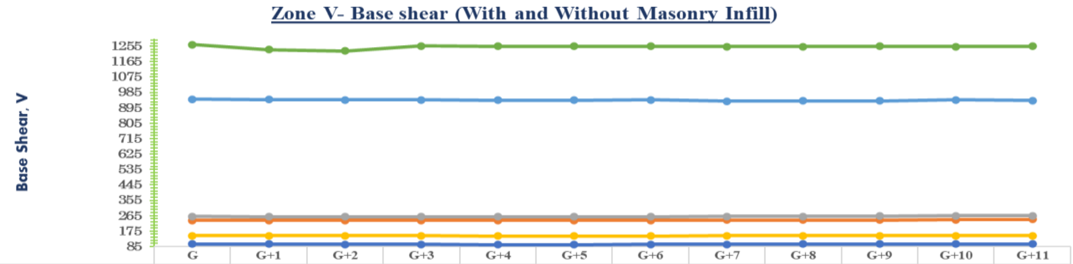
In addition to the time period, the following results have been compared for frames with and without infills having stiffness irregularity in the form of soft story located at different story levels for the zones III to V

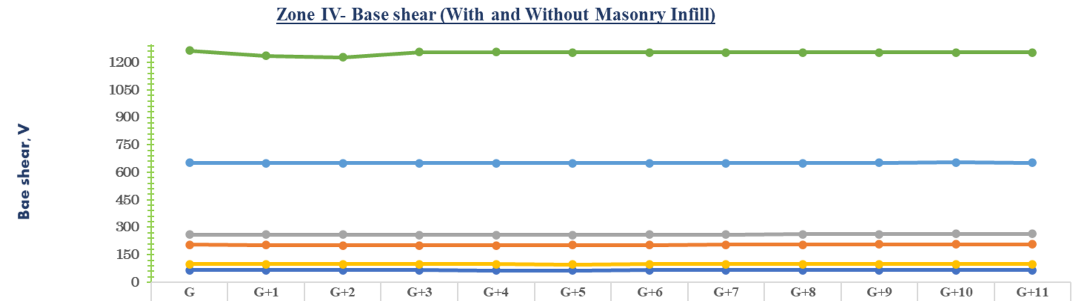
a) RSA base shear
b) PA base shear - ATC-40
c) PA base shear - FEMA 356

The results are shown in Figure-11(a), 11(b) and 11(c) for Zones V, IV and III respectively.



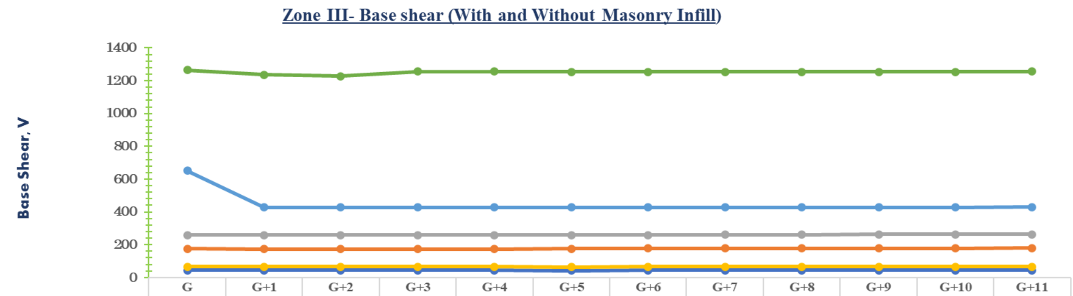
	G	G+1	G+2	G+3	G+4	G+5	G+6	G+7	G+8	G+9	G+10	G+11
Base shear without infill (kN)	99.917	99.188	97.896	97.622	97.608	97.608	97.825	98.22	98.722	99.247	99.705	100.023
ATC 40 -Base shear, (V) without infill (kN)	238.225	237.458	237.181	237.125	237.433	237.433	237.983	238.861	240.118	240.516	242.374	242.721
FEMA -Base shear (V) without infill (kN)	260.088	259.587	258.819	258.727	259.013	259.013	259.415	260.224	261.271	262.483	264.344	264.565
Base shear with Masonry infill (kN)	148.64	148.084	147.588	147.222	147.014	146.971	147.083	147.326	147.66	148.038	148.409	148.725
ATC 40 -Base shear, (V) with Masonry infill (kN)	946.54	943.149	941.976	941.287	939.488	939.565	941.556	934.277	935.3	936.256	941.393	937.862
FEMA 356 -BASE SHEAR, (V) with Masonry infill (kN)	1264.869	1235.809	1227.141	1256.649	1255.788	1254.475	1254.274	1253.847	1253.805	1254.314	1253.883	1255.163

Figure-11(a). Zone- V.



	G	G+1	G+2	G+3	G+4	G+5	G+6	G+7	G+8	G+9	G+10	G+11
Base shear without infill (kN)	66.611	66.125	65.623	65.264	65.081	65.0272	65.217	65.48	65.815	66.165	66.47	66.682
ATC 40 -Base shear, (V) without infill (kN)	204.847	203.155	201.275	201.273	201.512	202.18	203.454	204.42	205.553	206.159	206.481	206.692
FEMA -Base shear (V) without infill (kN)	260.088	259.587	259.202	258.819	258.727	259.013	259.415	260.224	261.271	262.483	264.344	264.565
Base shear with Masonry infill (kN)	99.093	98.723	98.392	98.148	98.009	97.981	98.055	98.217	98.44	98.692	98.939	99.15
ATC 40 -Base shear, (V) with Masonry infill (kN)	651.156	650.172	649.536	650.042	649.799	650.132	650.956	649.92	650.614	651.425	653.806	652.399
FEMA 356 -BASE SHEAR, (V) with Masonry infill (kN)	1264.869	1235.809	1227.141	1256.649	1255.788	1254.475	1254.274	1253.847	1253.805	1254.314	1253.883	1255.163

Figure-11(b). Zone- IV.



	G	G+1	G+2	G+3	G+4	G+5	G+6	G+7	G+8	G+9	G+10	G+11
Base shear without infill (kN)	44.408	44.048	43.749	43.509	43.387	43.381	43.478	43.654	43.876	44.11	44.313	44.455
ATC 40 -Base shear, (V) without infill (kN)	176.137	174.107	173.497	174.085	174.571	175.414	177.379	178.264	178.677	178.951	179.134	179.293
FEMA -Base shear (V) without infill (kN)	260.088	259.587	259.202	258.819	258.727	259.013	259.415	260.224	261.271	262.483	264.344	264.565
Base shear with Masonry infill (kN)	66.062	65.815	65.595	65.432	65.339	65.32	65.37	65.478	65.626	65.795	65.959	66.1
ATC 40 -Base shear, (V) with Masonry infill (kN)	651.156	427.509	426.963	427.136	427.032	427.189	427.49	427.873	428.375	428.77	428.799	429.693
FEMA 356 -BASE SHEAR, (V) with Masonry infill (kN)	1264.869	1235.809	1227.141	1256.649	1255.788	1254.475	1254.274	1253.847	1253.805	1254.314	1253.883	1255.163

Figure-11(c). Zone- III.

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CONCLUSIONS

Based on the results presented in Tables 2 and 3 and Figures 9-11 the following conclusions are made. Since the analyses give critical values in Zone V, they are only discussed.

a) The base shear found using RSA for the frame with infills and without vertical stiffness irregularity is 1.47 times more than that of bare frames. Similarly, PA too gives higher base shears for structure with infills when compared to bare frames. PA, using ATC-40, gives base shear which is 2.47 times more than that of bare frames whereas it is 2.72 times using FEMA 356. It is inferred that the base shears given by PA are true reflection of what would happen under the action of seismic loads since RSA is a linear elastic analysis subject to provisions of codes. However, the maximum top storey displacements for infill frames given by both the methods of PA are between 0.77 and 0.83 times of bare frames, indicating that resistance to displacement in infilled frames is comparable with that of bare frames.
b) The influence of location soft story which has a stiffness equal to 30% of regular storey on time period ‘T’ is very small as indicated by the fact their standard deviations are only 0.006 for infilled frames and 0.016 for bare frames. However, within this narrow variation it is revealed that location of soft storey has more impact on bare frames than in infilled beams.
c) Location of soft storey nominally influences base shear as indicated by the results obtainable using all the three methods - RSA, PA (ATC40) and PA (FEMA356). Studying the standard deviation (SD) values of base shear reveals that the variation is more under PA (2.01-2.09) as compared under RSA (0.94) for bare frames. But for the frame with infills the SD values are 0.63, 3.52 and 10.07 respectively for RSA, PA (ATC40) and PA (FEMA356). It can be inferred from these results that presence of infills introduces an element of uncertainty in the behaviour of the structure.
d) Presence of infills greatly influences the behaviour of the structure. Under RSA, the average base shear computed by taking the mean of the base shears considering the base shears for every location of soft storey, is 1.5 times more than that bare frame. Similarly, under PA (ATC40) it is 3.93 times and under PA (FEMA356) it is 4.80 times more than corresponding bare frames.
e) Therefore, it is concluded that carrying out PA and considering the infills suitably modelled are essential to understand the complete behaviour of moment resistant framed system under lateral loads.

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