Performance Analysis Of Infilled RC Frame In Earthquake Region Using STAAD.Pro

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Abstract:

Now a days construction of the RC frame is common because of the simplicity in construction. The masonry walls are mainly used for partition and insulation purposes rather than for structural purposes. However, during the earthquake, this filling contributes to the response of the structure and the behavior of the filling frame is different from that expected for the structure of the bare frame. The fill acts as a compression strut between column and beam. For this purpose, linear dynamic analysis were carried out on the structure of the RC masonry frame to study the influence of the resistance variation of the structure with n without a infill wall, filling effect on dynamic parameters such as the natural period, displacement and state of the hinge. In this rear-end collision effect high building is studied. All analysis are performed by the STAAD PRO v8i software. building modeling and analysis are performed on STAAD PRO v8i. For the analysis the building with G + 10 RCC frame is modeled. In this analysis the width of the strut is calculated manually according to the expression given in the FEMA-356. The infill panels are modeled as equivalent single diagonal struts. Several equations for calculation are considered for these diagonals. In this study the comparison of time verses acceleration , time verses velocity , time verses displacement with respect to the floor is made . The study shows that the influence of filling on the structure is significant. It increases the rigidity of the structure and makes the structure able to withstand a seismic region with respect to the bare frame.

Keywords: Infill walls, Seismic force, base shear , STAAD Pro. , Time History, response spectrum.

I. INTRODUTION

It has always been a human aspiration to create ever higher structures. The moment of reinforced concrete that resists the frames full of masonry walls of unreinforced bricks is very common in India and other developing countries. Masonry is a building material commonly used in the world for reasons that include accessibility, functionality and costs. When it is considered that the masonry in the fillings interacts with the surrounding frames, the lateral load capacity of the structure increases considerably.

The study of buildings damaged by earthquakes further reinforces this understanding. The positive aspects of the presence of fillers are greater resistance and greater rigidity of the filling frames. In skyscrapers, the vertical loads that normally occur, alive or dead, do not represent a big problem, but lateral loads due to wind or earthquake tremors are a cause for great concern and require particular attention in the design of buildings. These lateral forces can produce critical stress in a structure, create unwanted vibrations and, moreover, cause a lateral displacement of the structure that can reach a stage of discomfort for the occupants.

In many countries located in seismic regions, reinforced concrete frames are completely or partially filled with brick masonry panels with or without openings. Although the filling panels significantly increase the rigidity and strength of the frame, their contribution is often not taken into consideration due to the lack of knowledge of the behavior of the frame and the composite filling. The filling wall can be modeled in different ways, such as the diagonal approach of the equivalent upright and the method of the finite element.

II Overview

Seismic Analysis is a subset of structural analysis and is the calculation of the response of a building structure to earthquakes. It is a part of the process of structural design, earthquake engineering or structural assessment in regions where earthquakes are prevalent. Seismic structural analysis methods can be divided into two main categories, static analysis and dynamic analysis. These two main categories can be divided into two main types of analysis, the linear and non-linear analysis. The studied building in this paper is a typical steen-story model of commercial building. The building is comprised of a reinforced concrete structural frame. The overall plan dimension is 22.5m × 22.5 m with 49.1 m in height.

Earthquake resistant design structures are those, which are able to dissipate seismic forces generated due to strong ground motions. In crrent seismic codes,there are several design philosophies which are formulated through experimental studies,computer simulations and observation from past earthquake. To communicate the seismic threat to stake holders (that is owner,contractor,builder,designer and government agencies) structural engineering commmunity has moved towards predictive methods of design, namely performance-based engineering. The concept lies in making structural elements ductile so as to dissipate the cyclic forces and dissipate energy.

A large number of reinforced concrete and steel buildings are constructed with masonry infills Masonry infills are often used to fill the void between the vertical and horizontal resisting elements of the building frames with the assumption that these infills will not take part in resisting any kind of load either axial or lateral; hence its significance in the analysis of frame is generally neglected. Moreover, non-availability of realistic and simple analytical models of infill becomes another hurdle for its consideration in analysis. In fact, an infill wall enhances considerably the strength and rigidity of the structure. It has been recognised that frames with infills have more strength and rigidity in comparison to the bared frames and their ignorance has become the cause of failure of many of the multi-storeyed buildings. The recent example in this category is the Bhuj earthquake on 26 January, 2001. The main reason of failure is the stiffening effect of infilled frame that changes the basic behaviour of buildings during earthquake and creates new failure mechanism. This chapter will discuss the structural action of infill panel and failure modes and modelling of infill walls with and without openings.

2.1 STRUCTURAL AND CONSTRUCTIONAL ASPECT OF INFILLS

The presence of masonry infill is the cause of (i) Unequal distribution of lateral forces in the different frames of a building overstressing of some frames ; (ii)vertical irregularities in strength and stiffness-soft storey or weak storey as a result higher interstorey drifts and higher ductility demands of RC elements of the soft storey in comparison to remaining stories ; (iii) horizontal irregularities-significant amount of unexpected torsional forces since the centre of rigidity is moved towards the stiffer infilled frames of increased stiffness and as a result occurrence of very large rotation and large displacements in the extreme bare frames;

(iv) inducing the effect of short column or captive column in infilled frame-a captive column is full storey slender column whose clear height is reduced by its part-height contact with a relatively stiff masonry infill wall, which constraints its lateral deformation over the height of contact (CEB, 1996) resulting in premature brittle failure of columns and

(v) failure of masonry infills-out-of-plane and in-plane failure results which become the cause of casualties. A significant amount of research work has been carried out on the consideration of stiffening effect of infill panels and its constructional details. A clear decision has to be taken by the structural engineers, whether the infill walls will be made to participate in resisting the load or not. Depending upon its load resisting mechanism of infills the construction details will be followed as :

a) Only axial load–– infill walls tight to the under side of the floor system - arching action is the dominant mechanism,

   (ii) Axial and lateral load friction or mechanical anchorage along the top to transfer lateral load to the wall-connection must be able to transfer the reaction

b) Only lateral load –– wall built tight to the columns and a movement joint at the top of wall, and no axial and lateral movement joints along all the sides of walls and must be sufficiently thick to isolate the effects of inter-storey drift, floor deflection and ual movement-this type of wall is called partition wall (Drydale, Hamid and Baker, 1994).

2.2 FAILURE MECHANISM OF INFILLED FRAME

The failure mechanism of an infilled frame is quite complex and depends upon a number of factors such as relative strength and stiffness properties of infill and frame, frame wall interface gaps, openings, shear connectors, and such other characteristics. Figure shows the five most common modes of failure of masonry infilled frame under increasing intensity of lateral force (Buonopane et al., 1999). In principle, failure mechanism of an infilled frame depends to a extent on the relative strength of the frame and the infill (El-Dakhakhni et al.,2003,Mehrabiet al. (1996) .

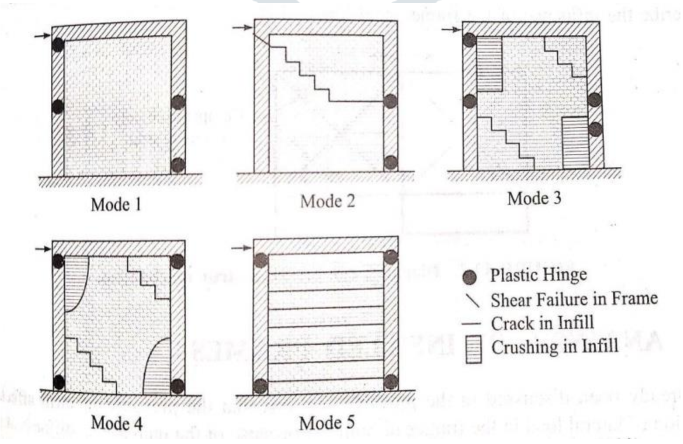


Fig :1 failure mechanism of infilled frame .

2.3 ANALYSIS OF INFILLED FRAMES

It has already been discussed in the previous sections that the presence of infill affects the distribution of lateral load in the frames of building because of the increase of stiffness of some of the frames. The distribution of lateral forces in the frames of building basically depends upon of rigidity of the building and the resultant of the applied lateral loads. If both nearly distribution of lateral load remains straightforward ie. in the ratio of their relative fresh. If it is not the case, large torsional forces are introduced in the building. These type of structures can be better analysed on the basis of 3D analysis of building after considering the increased stiffness of the infilled frames .

The study of interaction of infill with frames has been attempted by using sophisticated analysis like finite element analysis or theory of elasticity. But due to uncertainty in defining the interface conditions between the infilled with the frames, an approximate analysis method may be better acceptable. One of the most common approximation of infilled walls is on the basis of equivalent diagonal strut i.e. the system is modeled as a braced frame and infill walls as web element. The main problem in this approach is to find the effective width for the equivalent diagonal strut. Various investigators have suggested different values of width of equivalent diagonal strut .

2.3.1 Equivalent diagonal strut

The width of the equivalent diagonal strut (w) can be found out by using a number of expressions given by different researchers

The geometric and material properties of the equivalent diagonal strut are required for total braced frame analysis to determine the increased stiffness of the infilled frame. The geometric properties are of effective width and thickness of the strut. The thickness and material properties of strut are similar to the infill wall. Many investigators have proposed various approximations for the width of equivalent diagonal strut. Originally proposed by **Polyakov (1956)** and subsequently developed by many investigators, the width of strut depends on the length of contact between the wall and the columns, $\mathbf{a_h}$ and between the wall and beans, $\mathbf{a_L}$. shown in Figur . The proposed range of contact length is between one-fourth and one-tenth of the length of panel. Stafford Smith (1966) developed the formulations for $\mathbf{a_h}$

and $\mathbf{a_L}$ on the basis of beam on an elastic foundation. The following equations are proposed

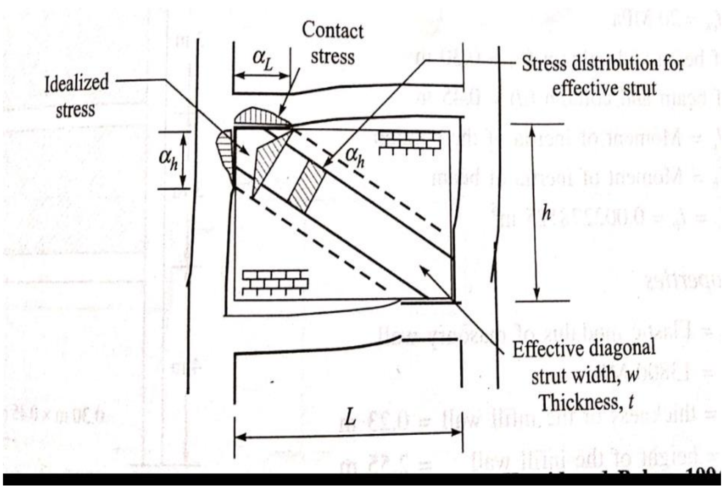


Fig :2 Equivalent diagonal strut

to determine , **a**$_h$, and **a**$_L$, which depend on the relative stiffness of the frame and infill and on the geometry of the panel.

$$\alpha_h = \frac{\pi}{2}\sqrt[4]{\frac{4E_f I_c h}{E_m t \sin 2\theta}}$$

$$\alpha_L = \pi\sqrt[4]{\frac{4E_f I_b L}{E_m t \sin 2\theta}}$$

1) **Henry (1998) has** proprosed the following equation to determine the equivalent or effective strut width w , where the strut is assumed to be subjected to uniform compressive stress

$$w = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_L^2}$$

2) **Holems (1963)** recommended a width of the diagonal strut equal to one third of the diagonal length of the panel , whereas New Zealand Code (NZS 4230) specifies a width equal to one quarter of its length .

3) **FEMA- 356**

The masonry infill walls are replaced with diagonal compression member (or) strut with appropriate mechanical properties. The thickness of the strut is equal to the thickness of the wall. The strut is assigned with hinges at both ends in order to take care of moment at strut frame intersection. As per FEMA-356 the equivalent width of diagonal strut is given by expressions:

$$a = 0.175 \times r_{inf} \times (\lambda_1 \times h_{col})^{-0.4}$$

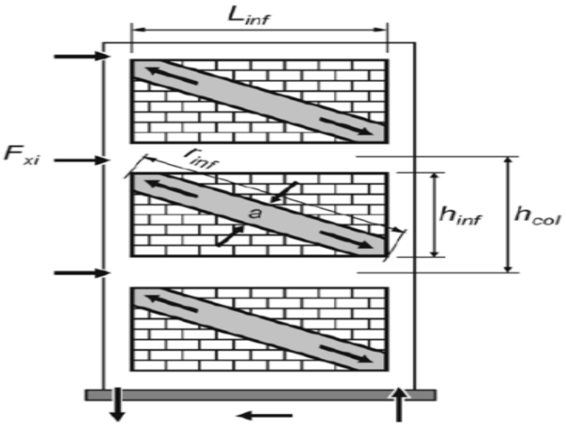


Fig:3. Diagonal strut

4) **Holmes (1961)** states that the width of equivalent strut to be one third of the diagonal length of infill, which resulted in the infill strength being independent of frame stiffness

$w = \frac{1}{3} d_{infill}$

Where , d_{infill} is the diagonal length of infill

5) **Stafford Smith and Carter (1969)** proposed a theoretical relation for the width

of the diagonal strut based on the relative stiffness of infill and frame.

$W=0.58 (1 / H)^{-0.445}.(\lambda_h.H_{infill})$

$\lambda_h = \sqrt[4]{\frac{Einf .t.\sin 2\theta .}{4.Ec .Ic .Hinf}}$

6) **Mainstone (1971)** gave equivalent diagonal strut concept by performing tests on model frames with brick infills. His approach estimates the infill contribution both to the stiffness of the frame and to its ultimate strength.

$W = 0.16\, d_{infill}(\lambda_h\, H_{inf,})^{-0.3}$

7) **Mainstone & Weeks and Mainstone (1974**), also based on experimental and analytical data, proposed an empirical equation for the calculation of the equivalent strut width

$W = 0.175\, d_{infill}(\lambda_h\, H_{inf,})^{-0.4}$

8) **Bazan and Meli (1980)** , on the basis of parametric finite-element studies for one bay, one-story, infilled frames, produced an empirical expression to calculate the equivalent width *w* for infilled frame:

$W = (0.35 +0.22\beta)\, h$

$\beta = \frac{Ec .Ac .}{Ginf \quad Ainf}$

9) **Liauw and Kwan (1984)** proposed the following equations based on experimental

and analytical data

$$w = \frac{0.95\, Hinf\, .\cos\theta}{\sqrt{\lambda h\, .Hinf}}$$

10) **Paulay and Preistley (1992)** pointed out that a high value of *w* will result in a stiffer structure, and therefore potentially higher seismic response. They suggested a conservative value useful for design proposal, given by:

$w = 0.25 d_{inf}$

11) **Durrani and Luo (1994)** analyzed the lateral load response of reinforced concrete infilled frames based on Mainstone's equations. They proposed an equation for effective width of the diagonal strut, w, as

$$w = \gamma\sqrt{L^2 + H^2}\,\sin 2\theta$$

where:

$$\gamma = 0.32\sqrt{\sin 2\theta}\left[\frac{H^4 E_{\inf} t}{m E_c I_c H_{\inf}}\right]^{-0.1};\; m = 6\left[1 + \frac{6E_c I_b H}{\pi E_c I_c L}\right]$$

III. formulation of work

The example RRC frame represents a medium rise G+10 framed building. Following figure shows the typical layout of the RCC frame. This RCC frame represents a commercial building in the seismic zone-IV, as per IS 1893, on a medium soil type. The height of a ground floor storey is 4.1m and other floor heights are 5m, and the beam spans 7.5m. The spacing between the frames is 7.5m. Firstly the width of the strut is calculated by using the FEMA356. Model of the RCC frame is created in STAAD.prov8i . Two model are created, first without considering diagonal strut and second is with considering the diagonal strut . Time history analysis is performed for both the model by considering the time verses acceleration data for earthquake region IV .

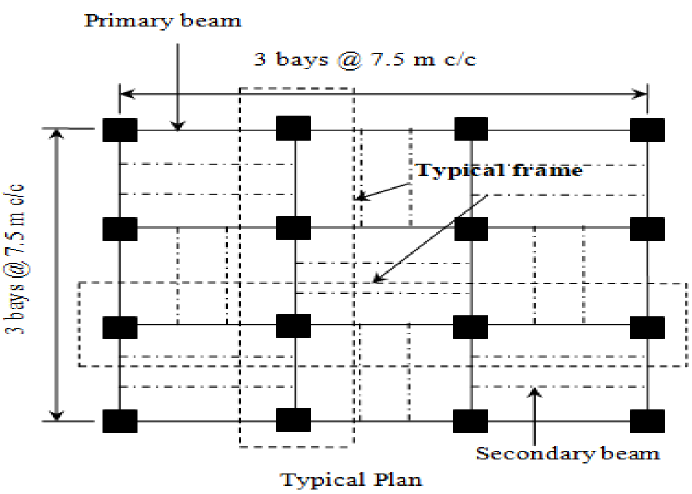


Fig:4 Typical layout of example RC frame

The characteristics of these RCC frame are presented in the following table.

Table : 1 Material property

Material property	Concrete M25 grade	Steel Fe415 grade
Weight per unit volume (KN/m^3)	25	76.97
Mass per unit volume (KN/m^3)	2.548	7.849
Modulus of elasticity (KN/m^2)	25 x 10^6	2x10^8
Characteristics strength (KN/m^2)	25000(for 28 days)	415000(yield)
Minimum tensile strength(KN/m^2)	-	485000
yield strength(KN/m^2)	-	456500
tensile strength (KN/m^2)	-	533500

3.1 Calculation Width Of Strut

The masonry infill walls are replaced with diagonal compression member (or) strut with appropriate mechanical properties. The thickness of the strut is equal to the thickness of the wall. The strut is assigned with hinges at both ends in order to take care of moment at strut frame intersection. As per FEMA-356 the equivalent width of diagonal strut is given by expressions:

$a = 0.175 \times rinf \times (\lambda_1 \times h_{col})^{-0.4} \quad \times \quad \Upsilon_{infill}$

Where, $\lambda_1 = \{(E_i \times t \times \sin 2\theta)/(4 \times E_f \times I_c \times h_{inf})\}^{1/4}$, $\theta = \tan\text{-}1(h_{inf}/ l)$

h_{col} - Column height between centre lines of beam (m) = 5m

E_i - Modulus of elasticity of infill material (kN/m2) = 550x 7 = 3850

E_f - Modulus of elasticity of frame material (kN/m2) = 25 x 10^6

T - Thickness of wall (m) = 0.30 m

h_{inf} - Height of the infill (m) = 4.4 m

L- Length of the infill (m) = 7.0 m

I_c - Moment of inertia of column (m^4) = 5.208 x 10^{-3}

θ - Slope of infill diagonal to the horizontal

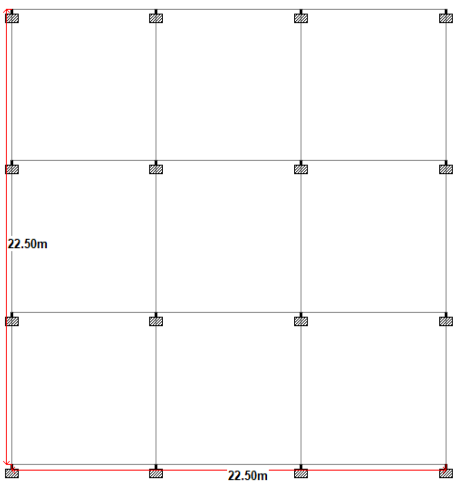
$$\theta = \tan^{-1}\frac{hinf}{Linf} = \tan^{-1}\frac{4.4}{7} = 32.15$$

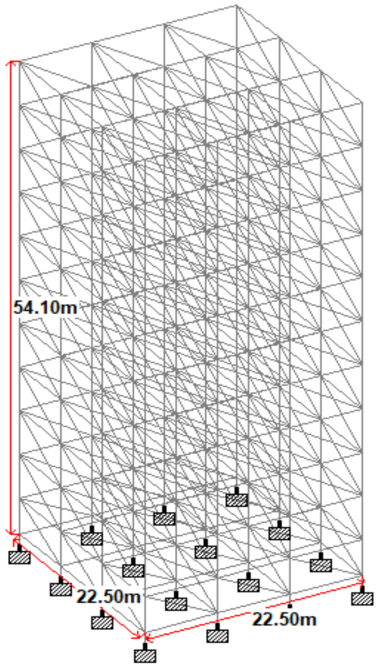
r_{inf} - Diagonal length of infill panel= 8.26 m

$$\lambda_1 = \sqrt[4]{\frac{(Ei \times t \times \sin 2\theta)}{(4 \times Ef \times Ic \times hinf)}} = \sqrt[4]{\frac{3850 \times 0.3 \times \sin 2 \times 32.15}{4 \times 25 \times 10^{6} \times 5.208 \times 10^{-3}}} = \mathbf{0.145}$$

$$a = 0.175 \times (0.145 \times 4.4)^{-0.4} \times 8.26 = 1.730 \text{ m}$$

3.2 modeling in staad.pro





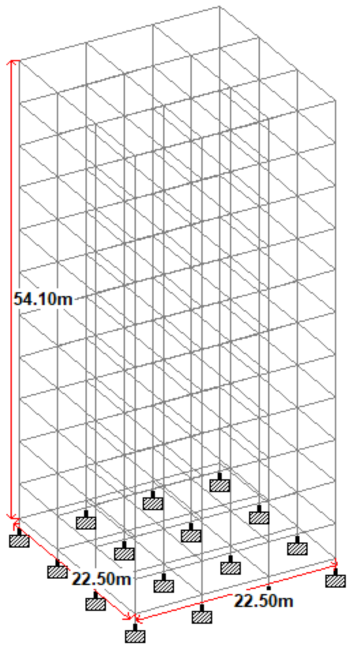


fig :5 typical plan **fig :6 with strut frame** **fig :7 bare frame**

VI . Result

4.1 Mode shape for bare frame for the following frequency and period in second .

Table : 2 calculated frequency for load case 3

MODE	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	0.272	3.67266
2	0.272	3.67266
3	0.318	3.14331
4	0.789	1.26748
5	0.817	1.22451
6	0.817	1.22451

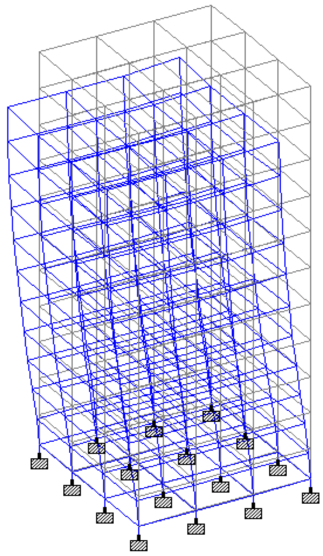


Fig :8 Mode shape 1

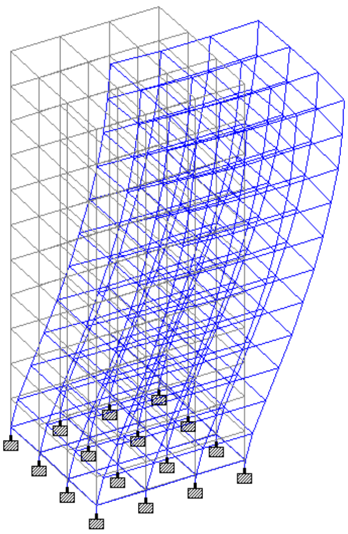


Fig :9 mode shape 2

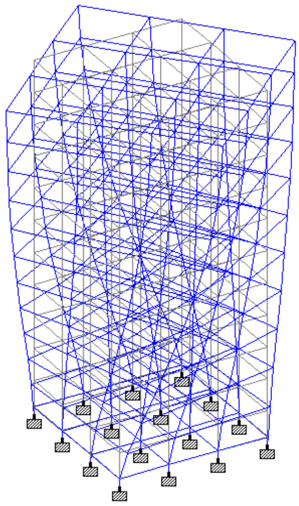


Fig :10 mode shape 3

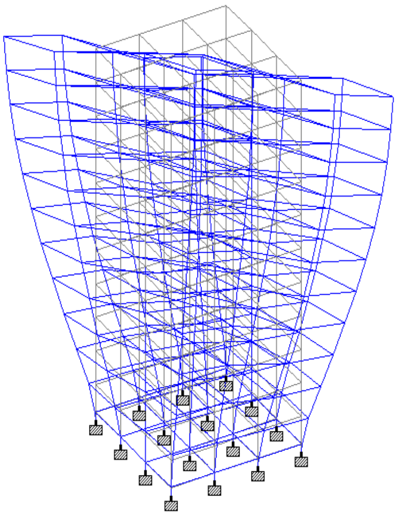


Fig :11 Mode shape 4

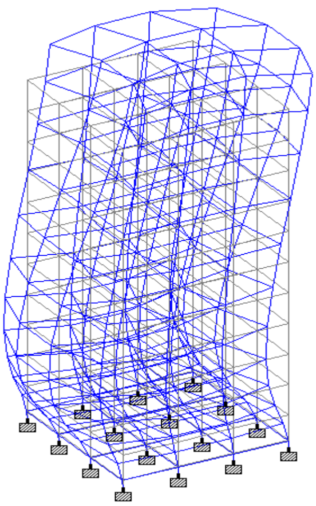


Fig:12 mode shape 5

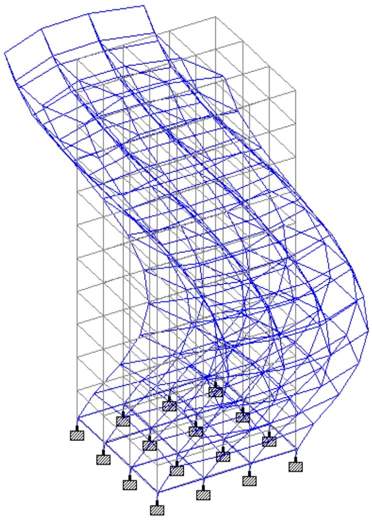
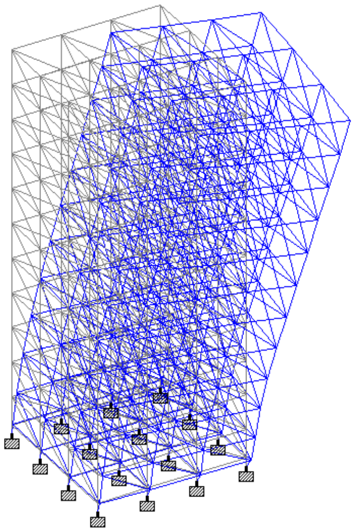


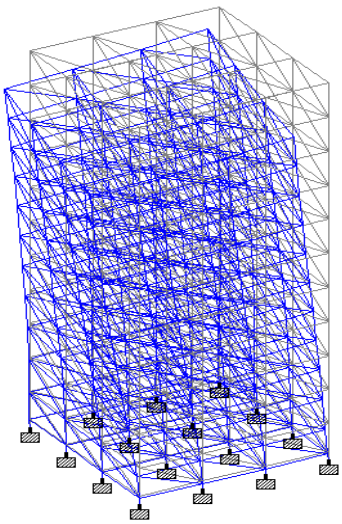
Fig :13 mode shape 6

4.2 Mode shape for the infill frame for the following frequency and period in second .

Table : 3 calculated frequency for load case 3

MODE	FREQUENCY(CYCLES/SEC)	PERIOD(SEC)
1	0.956	1.04575
2	0.967	1.03465
3	1.825	0.54786
4	2.531	0.3951
5	3.149	0.31757
6	3.414	0.29292





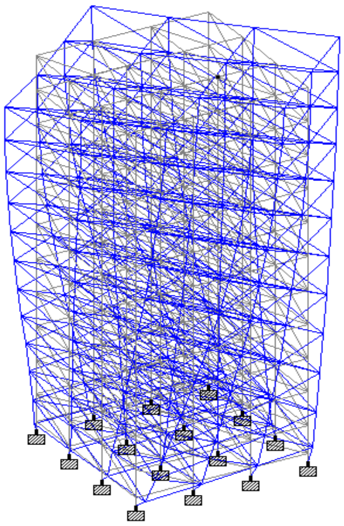


Fig :14 Mode Shape 1 **Fig:15 Mode Shape 2** **Fig : 16 Mode Shape 3**

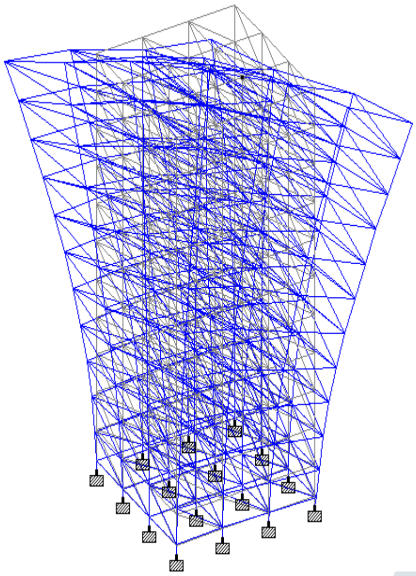


Fig:17 Mode Shape 4

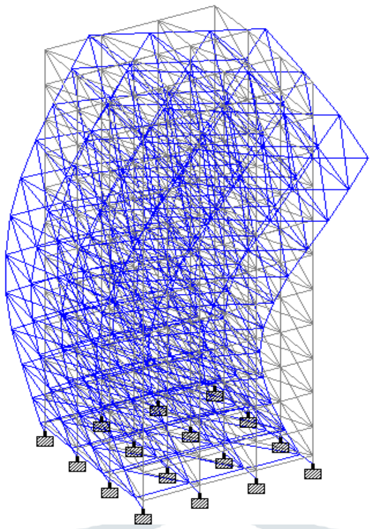


Fig:18 Mode Shape 5

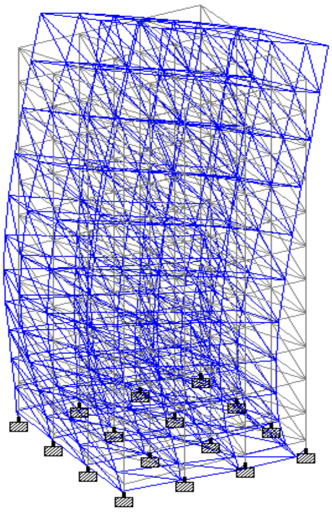


Fig :19 Mode Shape

4.3 Resultant displacement of frame :-

Resultant displacement of the frame is compare with respect to the node for the dead load , static load , dynamic load , combination of load case 4 , combination of load case 5 which are created in staad, PRO for the time history analysis .

(i) Frame without infill

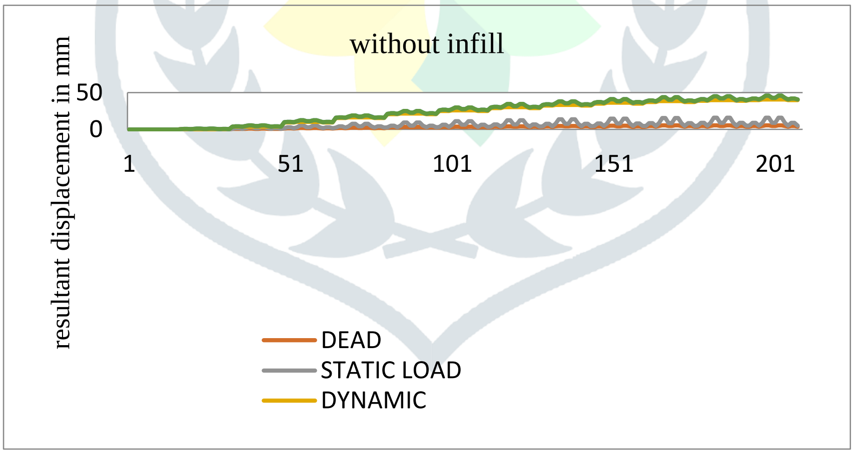


Fig: 20 displacement with respect to node for bare frame

(ii) Frame with infill

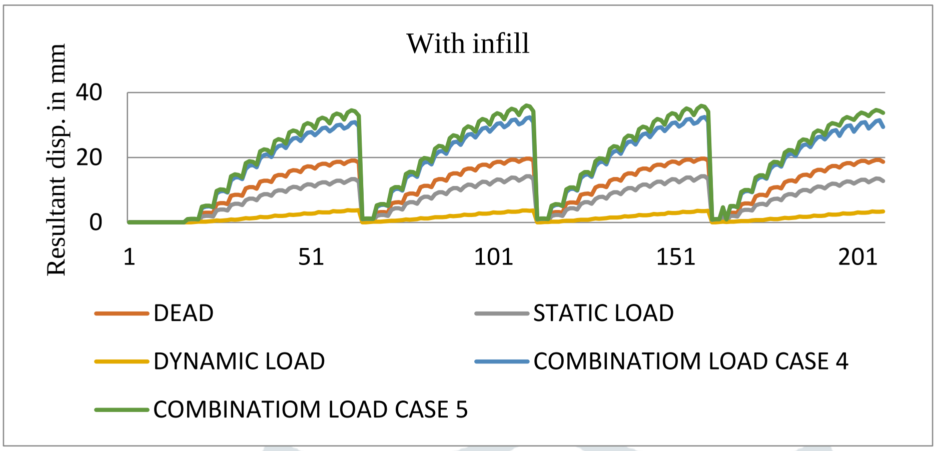


Fig: 21 displacement with respect to node for infill frame

4.4 Base shear :-

It is the total design lateral force at the base of a structure. The total design lateral force or design seismic base shear (VB) along any principal direction shall be determined by the following expression:

$$V_B = A_h W$$

Ah- Design horizontal seismic forces coefficient

W- Seismic weight of the building.

Vertical Distribution of Base Shear to Different Floor Level the design base shear (VB) computed in 7.6.1 shall be distributed along the height of the building as per the following expression:

$$Q_i = \left(\frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \right) V_B$$

Where,

Qi = Design lateral force at floor i,

Wi = Seismic weight of floor i,

hi = Height of floor i measured from base, and

n = Number of storey in the building is the number of levels at which the masses are located

following table show the base shear of the given building which is obtained in the response spectrum analysis done in staad pro . Base shear of the bare frame and the infill frame if compared in the following table

Table no: 4

Storey	level in meter	Peak story shear in KN including shear from

		torsion	
		bare frame	infill frame
12	55.3	72.35	375.93
11	50.3	162.19	810.24
10	45.3	235.84	1126.75
9	40.3	290.48	1348.39
8	35.3	329.15	1514.83
7	30.3	359.71	1671.4
6	25.3	390.56	1849.13
5	20.3	424.99	2049.75
4	15.3	458.4	2249.06
3	10.3	484.4	2414.31
2	5.3	494.88	2521.17
1	1.2	495.57	2567.48

V. Conclusion

1) In this research, the effects of masonry infill on the stability of the building in seismic region is investigated.
2) The response of the structure in the time history analysis and the response spectrum analysis is studied .
3) it is observed that the structure with fully masonry infill is stiffer than the bare structure .
4) The maximum in deflection in bare frame for (g+10) is 47.05mm and in strut frame it is minimum which 36.04mm .. If the effect of infill wall is considered then the deflection has reduced drastically.
5) From this present result it shows that, deflection is very large in case of bare frame as compare to that of infill frame with opening. If the effect of infill wall is considered then the deflection has reduced drastically. And also deflection is more at last storey because earthquake force acting on it more effectively.
6) In the response spectrum analysis the base shear for the bare frame and for infill frame is found out .
7) The base shear for the bare frame is 495.57 KN and for the infill fame is 2567.48 KN .
8) From the response spectrum analysis result it shows that, the base shear for the infill frame is more than the bare frame , hence structure with masonry infill is more stiffer than the bare structure .
9) Finally we can conclude that the structure with infill masonry gives more stability and stiffness to the structure to perform good in the earthquake region than the bare frame structure .

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