Abstract - Keeping in line with the increased need for developing guidelines for the analysis and design of infilled frames in multistoried buildings under the effect of dynamic loads, we discuss in this manuscript an analysis technique for the seismic design of infilled frames, which has practical utilities. Infilled frames should be designed to withstand lateral forces that can result from seismic ground motion and the accuracy of the predicted forces depends on the calculated dynamic characteristics of the structures, namely natural frequencies, free vibration modes, and damping. Here we have modelled the infill in the reinforced concrete frame as diagonal strut or Finite Element model, the mass matrix and stiffness matrix is assembled and by the Eigen value analysis the time period could be found out. This presents a simple practical analysis technique for the seismic design of infilled frames, which can be used by practicing engineers.

Key words: Infilled frames, RC framed structure, Finite Element model, Eigen value analysis, Natural time period

1. INTRODUCTION

The availability of improved analytical and experimental techniques in recent years have led to an increased recognition among structural engineers that elements of a building, which are generally considered “non-structural”, may affect its characteristics especially under the effect of dynamic load. For eg., lateral loads can produce critical stress in a multistoried structure leading to undesirable vibrations during earth quakes[1-3]. Such lateral forces are a matter of great concern and need special considerations in the design of buildings. Reinforced concrete skeleton frames are filled with brick or concrete block masonry walls in multistoried buildings to meet the architectural and functional requirements. In such situations, combination of frame and filler walls forms an ‘infilled frame’. These infill walls, though constructed as secondary structural elements, behave as a constituent part of the structural system and determine overall behavior of the structure, especially when it is subjected to lateral loads[4]. However, designers tend to treat these infill walls as “non-structural” elements and treat the frames as conventional bare frames in practice, which is far from representing the true behavior.

The neglect of infills in seismic design can be attributed to the common misconception that masonry infill in frames can only increase the overall lateral load capacity, and therefore, must always be beneficial to seismic performance[1]. However, a number of structural damages, as recorded in some of the recent earthquakes were traced to modifications of the structural frame due to presence of infilled walls. Hence, there is increasing recognition among structural engineers that the infilled walls affect the characteristics of the building under the effect of dynamic loads, specifically, wind and earthquake loads and should be taken into consideration in the analysis and design. Infilled frames, like any other structures should be designed to withstand lateral forces that can result from seismic ground motion[3]. The accuracy of the predicted forces depends on the calculated dynamic characteristics of the structures, namely natural frequencies, free vibration modes, and damping[5]. The dynamic properties are significantly influenced by the presence of infill. Empirical relationships, as proposed in design codes do not properly account for the effect of infill. Furthermore, errors greater than ± 50% were observed when the natural frequencies, computed using empirical expressions, were compared with measured natural frequencies[1]. Hence, the dynamic characteristics of an infilled frame should be determined by accounting for the effect of infill.

The problem of building frames having infill panels with different materials and configurations has henceforth generated interest among researchers in this field. Frames could be steel, reinforced concrete or concrete encased steel frames and the materials of infill panels could be solid or hollow bricks, reinforced or non-reinforced concrete blocks, lightweight concrete, composite materials or reinforced concrete[6]. A review of the literature on the behavior of infilled frames indicated that the addition of infills may cause significant changes in the dynamic characteristics of building and influence their behavior during earthquakes. An infill that is properly designed and connected to frame offers conceptual and practical advantages, particularly if the basic structural system is a moment resisting frame. Also, seismic resistant design should be based on the principle of avoiding unnecessary
masses and using necessary masses structurally to resist seismic effects. Thus, when walls and partitions are needed, attempts should be made to use masonry of walls and partitions as structural elements. Unfortunately, the lack of simple practical method for the analysis of infilled frame has led, in some cases, to the design of buildings without accounting for the effect of infill. Literature review on infilled frames revealed the need for developing guidelines for analysis and design of infilled frame. In this paper we present an analysis technique for the seismic design of infilled frames, which can be used by practicing engineers.

2. THEORETICAL FORMULATION

We have formulated a theoretical analysis where a diagonal strut-frame combination represents the infilled frame[4], beam and truss elements idealise the structure for the elastic analysis[7]. The analysis accounts for infills in computing seismic load, determining forces in members and determining the strength of different components of the composite system. Preliminary design gives an estimate of member sizes of composite system. Modal analysis establishes natural periods and mode shapes, and static analysis yields member forces and displacements. We have compared these forces with the strength of infill and frame corresponding to possible modes of failure.

Dynamic analysis has been used to predict seismic loads, and truss elements idealise infilled frame and a frame analysis gives computed dynamic properties. A modal analysis procedure uses these dynamic properties to calculate the base shear. Maximum base shear is distributed as lateral forces in different storeys, based on the relative masses of each storey, and eventually the corresponding lateral force is distributed to each column of a storey. Total load acting on infilled frame system is established with due consideration to load factors and load combination factors given in the code of practice. Unsymmetrical distribution of infills leads to additional loads from torsion. Using all predicted loads, a static analysis for frame can give an estimate of the member forces. These member forces are used to design columns, beams and infills.

In the present work the infills are idealised as equivalent diagonal strut having two degrees of freedom at each node[4] i.e. it will act as a truss element. The degrees of freedom for frame element are three for each node. The mass degrees of freedom are only translational; the rotational degrees of freedom are ignored in the analysis. First the stiffness matrix for the frame elements and the brick infills are derived, and then Eigen vector analysis (free vibrational analysis) is performed to get the natural frequencies and period of vibration. The modal analysis is also carried out by using SAP 2000, using that natural period and by the IS code expressions, the dynamic analysis is performed[3].

2.1. Stiffness matrix for plane frame elements

In preparation for the analysis of plane frames, the member stiffness matrix for a typical plane frame member is developed. The matrix is formulated with respect to member axes and then transformed to structure axes by the method of rotation of axes. The frame elements are standard prismatic bending elements having three degrees of freedom at each node. The material is idealized as elastic and isotropic. Depending on their angle (\( \theta \)) with X-axis, they are divided as Beam-Frame elements and Column-Frame elements for \( \theta = 0^\circ \) and \( \theta = 90^\circ \), respectively.

The element stiffness matrix for general Frame elements is calculated as presented in Figure 1.
Figure 1. Element stiffnesses for general frame element.

The frame elements shown in Figure 1 (a) shows the deformation directions required for deriving stiffness matrix. Figures 1 (b), (c) and (d) represents forces produced due to unit deformation at first joint along direction 1, 2 and 3 directions, respectively. Similarly one can easily find stiffnesses for second joint. The resulting 6 x 6 member stiffness matrix for member axes is given in Eqn. (1).

The member stiffness matrix $S_M$ is then transformed to the stiffness matrix for structure axes $S_{MS}$. In order to transform the stiffness matrix from member axes to structure axes, rotation transformation matrix $R_f$ for a plane frame member is required.

$$
S_M = \begin{bmatrix}
\frac{AE}{L} & 0 & 0 & -\frac{AE}{L} & 0 & 0 \\
0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\
0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\
-\frac{AE}{L} & 0 & 0 & \frac{AE}{L} & 0 & 0 \\
0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\
0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L}
\end{bmatrix}
$$  \hspace{1cm} (1)
As a first step the 3 x 3 rotational matrix $R$ will be expressed in terms of the direction cosines of the member. This may be accomplished by expressing the direction cosine $\lambda$ of

$$ R = \begin{bmatrix} \lambda_{11} & \lambda_{12} & \lambda_{13} \\ \lambda_{21} & \lambda_{22} & \lambda_{23} \\ \lambda_{31} & \lambda_{32} & \lambda_{33} \end{bmatrix} = \begin{bmatrix} \cos \theta & \sin \theta & 0 \\ -\sin \theta & \cos \theta & 0 \\ 0 & 0 & 1 \end{bmatrix} $$

The rotation transformation matrix $R_T$ for a plane frame member can be shown to take the same form as Eqn. (1).

$$ R_T = \begin{bmatrix} R & 0 \\ 0 & R \end{bmatrix} $$

Having the rotation transformation matrix on hand, one may then calculate the member stiffness matrix for structure axes using equation.

$$ S_{MS} = R_T^T S_M R_T $$

Thus final stiffness matrix for frame element is,

$$ [S_{MS}]_{6x6} = \begin{bmatrix} K_c C^2 + K_s S^2 & (K_1 - K_2) CS & K_c S & -(K_1 C^2 + K_2 S^2) & -(K_1 - K_2) CS & K_c S \\ (K_1 - K_2) CS & K_c S^2 + K_s C^2 & -K_c C & -(K_1 - K_2) CS & -(K_s S^2 + K_c C^2) & -K_c C \\ K_c S & -K_c C & K_c & -K_s S & K_c & K_c \\ -(K_1 C^2 + K_2 S^2) & -(K_1 - K_2) CS & K_c S & K_c S^2 + K_s C^2 & (K_1 - K_2) CS & K_c S \\ -(K_1 - K_2) CS & -(K_s S^2 + K_c C^2) & K_c & K_c C & K_s S & K_c C \\ K_c S & -K_c C & K_c & -K_s S & K_c & K_c \end{bmatrix} $$

Where,

$$ K_1 = \frac{AE}{L}, \quad K_2 = \frac{12 EI}{L^3}, \quad K_3 = \frac{6 EI}{L^2}, \quad K_4 = \frac{4 EI}{L}, \quad K_5 = \frac{2 EI}{L}, \quad C = \cos \theta, \quad S = \sin \theta $$

From the matrix $[S_{MS}]$, stiffness matrix for Beam-frame element and Column-frame element can be derived by substituting corresponding values of $\theta$.[6,7]

2.2. Stiffness matrix for infill panel element

The contribution of infills to the composite system can be accounted for in this analysis by using the equivalent diagonal strut method.[4] When the frame-diagonal strut idealization is used, the columns and beams of the frame are modelled as frame elements, with three degrees of freedom at each node, namely rotation and two displacements, while for the diagonal strut, the truss element will be used.[7]. For the truss element the value of moment of inertia becomes zero in equation 5, hence the stiffness matrix for truss elements becomes

$$ S_{MS} = \frac{AE}{L} \begin{bmatrix} C^2 & CS & -C^2 & -CS \\ CS & S^2 & -CS & -S^2 \\ -C^2 & -CS & C^2 & CS \\ -CS & -S^2 & -CS & S^2 \end{bmatrix} $$


2.3. Element mass matrix for frame and infill element

Modal analysis is used to determine the vibration modes of a structure in order to understand their structural behaviour[9,10]. The determination of mass influence coefficients for axial effects of a beam element may be carried out by any of two methods indicated for the flexural effects, namely (i) The lumped mass model and (ii) The consistent mass model. In the lumped mass method, the mass allocation to the nodes of the frame element is found from static a consideration, which for a uniform beam gives half of the total mass of the frame segment allocated at each node. Then for a prismatic beam segment, the relationship between nodal axial forces and nodal acceleration is given by

$$ \begin{bmatrix} P_1 \\ P_2 \end{bmatrix} = \frac{mL}{2} \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \begin{bmatrix} u_1'' \\ u_2'' \end{bmatrix} $$

...
where \( m \) is the mass per unit of length. The combination of the flexural lumped mass coefficient and axial mass coefficient gives, in reference to the nodal co-ordinates in Fig. 2, the diagonal matrix given in equation (8).

\[
\begin{bmatrix}
P_1 \\
P_2 \\
P_3 \\
P_4 \\
P_5 \\
P_6
\end{bmatrix} = mL \begin{bmatrix}
1 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & 0 & 1
\end{bmatrix} \begin{bmatrix}
u_1 \\
u_2 \\
u_3 \\
u_4 \\
u_5 \\
u_6
\end{bmatrix}
\]

(8)

In the formation of mass matrix the rotational moment of inertia of the members are neglected and only translational inertia is considered. Hence the \( P_3 \) and \( P_6 \) becomes zero.

2.4. Natural period

Consider a single degree of freedom (SDOF) system. When some initial disturbance (displacement or velocity) is given to this SDOF system, it undergoes a free vibration and time required to complete one oscillation of free vibration is defined as the natural period of SDOF system. In a multi-degree freedom system, masses at different locations can undergo free vibration oscillations in different normal mode shapes of the deformation. In each of these normal modes of vibration, the structure takes a definite amount of time to complete one cycle of motion; this time to complete one cycle of motion is called natural period of motion of that normal mode of vibration. This phenomenon is particularly important for determining seismic forces. In fact, without dragging it back and forth, it is not possible to make an object vibrate at anything other than its natural period. Natural period primarily is a function of building height. Other factors such as the building construction materials, which affect the stiffness of the structure and building geometric proportions, also affect the period, but height is the most important consideration.

The approximate fundamental natural period of vibration in seconds, \( T_a \), for moment resisting frame buildings without brick infill panel can be estimated by using the following expression:

For RC frame buildings:
\[
T_a = 0.075 h^{0.75} \quad (9)
\]

For Steel frame buildings:
\[
T_a = 0.085 h^{0.75} \quad (10)
\]

And the approximate fundamental natural period of vibration in seconds for other types of buildings including moment resisting building with infill can be estimated as:

\[
T_a = \frac{0.09 h}{d^{1/2}} \quad (11)
\]

where, \( h \) = Height of building in meters. This excludes the basement storeys where basement walls are connected with ground floor deck or fitted with building columns, however, it includes the basement when they are not connected, \( d \) = base dimension of the building at the plinth level in meters along considered direction of the lateral force [3].

2.5. Modal analysis

Modal analysis is used to determine the vibration modes of a structure. These modes are useful to understand the behavior of structure. Mode shapes depend on distribution of mass and stiffness in building. In tall buildings, higher modes can be quite significant and irregular buildings mode shapes may be somewhat irregular. Hence, for tall and irregular buildings, dynamic analysis is generally preferred. There are two types of modal analysis to choose from when defining a modal analysis (i) Eigenvector analysis which determines the undamped free-vibration mode shapes and frequencies of the system. These natural modes provide an excellent insight into the behavior of the structure. They can also be used as the basis for...
response-spectrum or time-history analyses. (ii) Ritz-vector analysis which find modes that are excited by a particular loading. Ritz vectors can provide a better basis than do eigen vectors when used for response-spectrum or time-history analyses that are based on modal superposition. Modal analysis is always linear. It may be based on the stiffness of the full-unstressed structure, or upon the stiffness at the end of a nonlinear analysis case. We have performed eigen vector analysis in the work discussed here. Eigenvector analysis involves the solution of the generalized eigenvalue problem

\[ [K] - \omega^2 [M] \{\phi\} = 0 \quad (12) \]

where \([K]\) is the stiffness matrix, \([M]\) is the diagonal mass matrix, \(\omega^2\) represents eigenvalues, and \(\{\phi\}\) is the matrix of corresponding eigenvectors (mode shapes). Each eigenvector eigenvalue pair is called a natural vibration mode of the structure[10]. The modes are identified by numbers from 1 to \(n\) in the order in which the modes are found by the program.

The eigenvalue is the square of the circular frequency for that mode (unless a frequency shift is used, see below). The cyclic frequency \(f\) and period \(T\) of the Mode are related too by:

\[ T = \frac{2\pi}{\omega} \quad \text{and} \quad f = \frac{1}{T} \quad (13) \]

2.6. Equivalent static analysis for evaluation of lateral loads as per IS-1893 (Part-I): 2002

For the purpose of determining seismic force, the country is classified into four seismic zones, which are presented in Figure 1, of IS 1893(part-I): 2002. The total design lateral force (design seismic base shear) along any principal direction shall be calculated by following expression

\[ V_a = A_h W \quad (14) \]

where, \(V_a\) = design seismic base shear, \(A_h\) = design horizontal seismic coefficient for various structures and \(W\) = seismic weight of building.

The design horizontal seismic coefficient for a structure shall be determined by the following expression

\[ A_h = \frac{Z \cdot I_m \cdot S_a}{2 \cdot R \cdot g} \quad (15) \]

Where \(Z\) = zone factor, \(I_m\) = importance factor, \(R\) = response reduction factor, \(S_a/g\) = average response acceleration coefficient. These parameters are determined as follows.

(i) Zone factor (Z): It is a factor used to obtain the design acceleration spectrum depending upon perceived seismic hazard in the zone in which structure is located. The basic zone factors included in I.S. code are reasonable estimate of peak ground acceleration. Zone factor given in Table 2 of I.S. 1893: 2002 shows the values of zone factor depending upon the seismic intensity.

(ii) Importance factor (I_m): Seismic design philosophy assumes that a structure may undergo some damage during severe shaking. However critical and important facilities must respond better during an earthquake than an ordinary structure. Importance factor is meant to account for this by increasing the design force level for critical and important structures. Importance factor depends upon functional use of structures; characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value or economic importance. The importance factor is given in Table. 6 of I.S. 1893 (part I): 2002 depending on the importance of structure.

(iii) Response reduction factor (R): The structure is allowed to be damaged in case of severe shaking. Hence the structure is designed for seismic force much less than what is expected under ground shaking if structure were to remain linearly elastic. The response reduction factor depends upon the perceived seismic damage performance of the structure characterized by ductile and brittle deformations. However, the ratio \((I/R)\) shall not be greater than 1.0. The values of \(R\) for buildings are given in Table 7 of I.S. 1893 (part I): 2002 depending on the type of structure.

(iv) Average response acceleration coefficient (Sa/g): Average response acceleration for Rock and soil sites as given in Figure 2 of I.S. 1893 (part I): 2002 based on appropriate natural periods and damping of structure. These curves represent free field ground motion. Here, Figure 2 shows proposed 5% spectra for different soil sites and Table 3 of the I.S. gives multiplying factors for obtaining spectral values for various other damping.

Provided that for any structure with approximate natural period of vibration, \(T \leq 0.2\) s, the value of \(A_h\) will not be taken less than \(2/2\) whatever the value of \(Im/R\).

Seismic weight of each floor is its full dead load plus appropriate amount of imposed load. While computing seismic weight of each floor, the weight of columns and walls shall be equally distributed to the floors above and below the storey. Any weight supported in between storeys shall be distributed to the floors above and below in inverse proportion to the distance from the floors. The percentage of imposed load to be considered in seismic weight calculations is given in Table 8 of I.S. 1893. As uniformly distributed imposed load up to 3.0 kN/m² percentage of imposed load is 25% and above 3.0 kN/m² it is 50%.

In the limit state design of reinforced concrete structures, following load combinations shall be accounted as per I.S. 1893 (Part I) 2002, where the terms DL, IL and EL stand for the response quantities due to dead load, imposed load and earthquake load, respectively.

1) 1.5 (DL + IL)
2) 1.2 (DL + IL + EL)
3) 1.5(DL + EL)
4) 0.9 DL + 1.5 EL

2.7 Dynamic analysis

Dynamic analysis shall be performed to obtain the design seismic force, and its distribution into different levels along height of a building and to the various lateral load-resisting elements. To perform dynamic analysis most important thing is to carry out free vibration analysis of a frame, to obtain dynamic properties i.e. natural periods and mode shapes of frame. Expressions for design load calculation and load distribution with height are based on...
the following assumptions: (i) Fundamental mode dominates the response (ii) Mass and stiffness are evenly distributed with building height, thus giving a regular modeshape. Mode shapes depend on the distribution of mass and stiffness in the building. In tall buildings, higher modes can be quite significant and in irregular buildings mode shapes maybe somewhat irregular. Hence, for tall and irregular buildings, dynamic analysis is generally preferred. Industrial buildings may also require dynamic analysis because they may have large spans, large heights, and considerable irregularities.

The dynamic analysis procedure for the calculation of design seismic force is valid when a building can be modelled as a lumped mass model with one degree of freedom per floor. This method of analysis does not imply that (a) Structure deforms only in the shear mode with no rotations or vertical translations at the floor levels, and (b) Beams in a structure are flexurally rigid and hence undergo no rotations[9].

(i) Free vibration analysis: Undamped free vibration analysis of the entire building shall be performed as per established methods of mechanics using the appropriate masses and elastic stiffness of the structural system, to obtain natural periods (T) and mode shapes (φ) of those of its modes of vibration that need to be considered[9].

(ii) Modes to be considered: Number of modes to be used in analysis for a considered direction of earthquake shaking should be such that sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass and missing mass correction beyond 33 percent. If modes with natural frequencies beyond 33 Hz are to be considered, the modal combination shall be carried out only for modes up to 33 Hz and the effect of higher modes with natural frequencies beyond 33 Hz shall be included by considering the missing mass correction procedure.

In a multi-degree-of-freedom system, when the ground shakes in a particular direction, only a part of the total mass of the whole structure vibrates in each mode of vibration. Thus, the net mass accounted for in the modes of vibration considered may be less than the total mass of the structure. Difference between total mass of the structure and net masses accounted for in the modes considered is called missing mass. Often, this missing mass corresponds to modes of vibration whose natural periods are very small (or whose natural frequencies are very large). Thus, in missing mass correction procedure, it is assumed that the missing mass corresponds to modes of vibration that have natural periods close to zero.

In dynamic analysis, it is sufficient to consider only first three modes, since higher modes donot significantly alter the response of a frame for buildings less than ten storeys high [6].

3. CONCLUSIONS

There has been an increased realization among the structural engineers that the presence of infills influences the behavior of frame system in multistoried buildings and can be either beneficial or detrimental, depending on the design situation. An increased need for developing guidelines for the analysis and design of infilled frames under the effect of dynamic loads is felt and in this manuscript we discuss an analysis technique for the seismic design of infilled frames, which has practical utilities. The infill in the reinforced concrete frame can be modelled as diagonal strut or Finite Element model. In the Finite Element model the frame is modelled as frame elements having three degrees of freedom. The stiffness matrix and the mass matrix can be formulated by knowing the size and the properties of the model. The infill is modelled as frame element having two degrees of freedom. Finally the mass matrix and stiffness matrix is assembled and by the Eigen value analysis the time period can be found out.

4. REFERENCES