SEISMIC ASSESEMENT OF MASONRY INFILL RC FRAMED BUILDING WITH SOFT GROUND FLOOR

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Abstract: Construction of multistoried buildings with open ground floor is a common trend of urbanization of cities of many parts of many countries. Social and functional need to provide parking space at ground level outweighs the seismic vulnerability of such buildings. Generally these buildings are designed as RC framed structures without regards to structural action of masonry infill walls present in the upper floors. In the present paper an investigation has been made to study the behavior of RC frames with various arrangement of infill when subjected to dynamic earthquake loading. Result of bare and infill frame are compared and some conclusions are made in view of *IS* -1893(2002) code.

Key words: *Masonry infill, RC frames, Soft storey*

1. Introduction

Reinforced concrete frames with masonry in-fills are a popular form of construction in high-rise buildings. Social and functional needs for vehicle parking, shops, reception, etc. are compelling to provide an open first storey in high rise building. Parking floor has become an unavoidable feature for the most of urban multistoried buildings. Though multistoried buildings with a parking floor (soft storey) are vulnerable to collapse due to earthquake loads, their construction is still widespread. These buildings are generally designed as framed structures without regard to structural action of the masonry infill walls. They are considered as non structural elements. Due to this, in a seismic action RC frames purely acts as moment resisting frames leading to variation in expected structural response. The effect of infill panels on the response of R/C frames subjected to seismic action is widely recognized and has been a subject of numerous experimental and analytical investigations. In the current practice of structural design in India, masonry infill panels are treated as nonstructural elements and their strength and stiffness contributions are neglected. In reality, the presence of infill wall changes the behavior of frame action into truss action thus changing the lateral load transfer mechanism.

In the present study, seismic performance of various configurations of infill panels in RC frames are compared with bare frame model using nonlinear analysis. The main objectives of this study were to investigate the behavior of multistory, multi-bay soft storey infilled frames and to evaluate their performance levels when subjected to earthquake loading.

2. Description of Structural Model

Significant experimental and analytical research is reported in the literature, which attempts to understand the behavior of infilled frames. Different types of analytical models based on the physical understanding of the overall behavior of an infill panel were developed over the years to mimic the behavior of infilled frames. The single strut model is the most widely used as it is simple and evidently the most suitable for large structures (Das and Murthy, 2004). Thus RC frames with unreinforced masonry walls can be modeled as equivalent braced frames with infill walls replaced by equivalent diagonal strut which can be used in rigorous nonlinear pushover analysis. Using the theory of beams on elastic foundations (Smith and Carter, 1969) suggested a non dimensional parameter to determine the width and relative stiffness of diagonal strut. Mainstone suggested another model representing the brick infill panel by equivalent diagonal strut. The strut area, A_e, was given by following expression:

$$A_e = w_e t \quad (1)$$

where,

$$w_e = 0.175\ (\lambda\ h)^{-0.4}\ w \quad (2)$$

$$\lambda = \sqrt[4]{\frac{E_i\, t \sin(2\theta)}{4 E_f\, I_c\, h'}} \quad (3)$$

where,

E_i = the modulus of elasticity of the infill material
E_f= the modulus of elasticity of the frame material
I_c = the moment of inertia of column
t = the thickness of infill
h = the centre line height of frame
h'= the height of infill
w'= the diagonal length of infill panel
θ= the slope of infill diagonal to the horizontal.

In this study, five different models of an eight storey building, symmetrical in the plan are considered. Usually in a building 40% to 60% of masonry in-fills (MI) are effective as the remaining portion of the Masonry Infills (MI) are meant for functional purpose such as doors and windows openings (Pauley and Priestley, 1992). In this study the buildings are modeled using Masonry Infills (MI) but arranging them in different manner as shown in the Figure 1. The building has four bays in North-South and East-West directions with the plan dimension 20 m× 16 m and a storey height of 3.0m each in all the floors. Further inputs include unit weight of the concrete is 25 kN/m^3, unit weight of masonry is 20 kN/m^3, Elastic modulus of steel is 2×10^8 kN/m^2, Elastic Modulus of concrete is 22.36×10^6 kN/m^2, Strength of concrete is 20 N/mm^2 (M20), Yield strength of steel is 415 N/mm^2 (Fe-415) and Live-load is 3.5 kN/m^2. The modulus of brick masonry and strut width is obtained using FEMA (306, 1998) recommendations i.e. $E_m = 550\times f_m$=2035 N/mm^2. Window openings are assumed tiny relative to the overall wall area thus not included in the as they have no appreciable bearing on the general behavior of the structure (Jain, *et al.,* 1997).

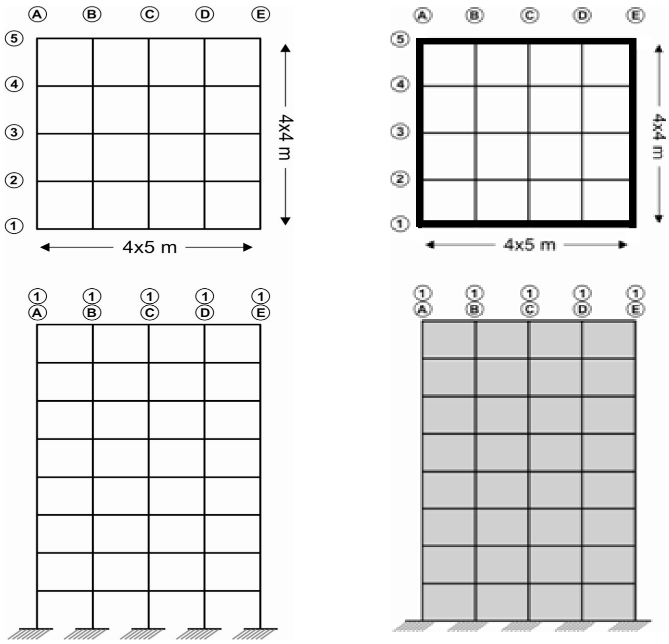
Following five different models are investigated in the study.

1. Model I : Bare frame
2. Model II : Masonry infill are arranged in outer periphery
3. Model III: Masonry infill are arranged in outer periphery with soft storey
4. Model IV: Masonry infill are arranged as inner core with soft storey
5. Model V : Masonry infill are arranged as (+) cross in plan with soft storey

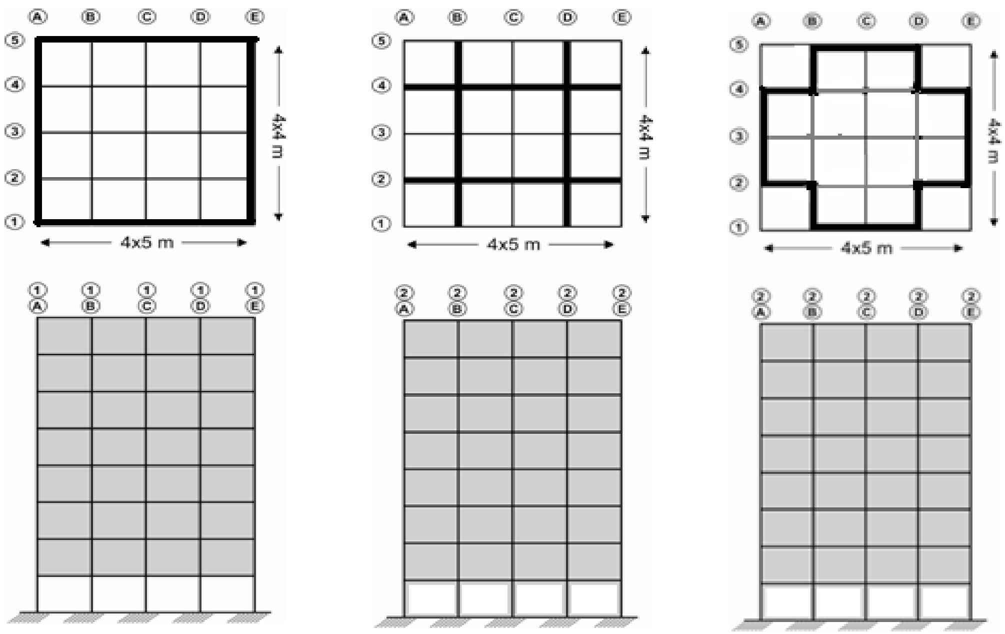
3. Nonlinear Analysis

Nonlinear analysis is the method used for determining the earthquake response of the structural systems. This method varies in methodology as nonlinear static pushover analysis and nonlinear dynamic time history analysis. In this study, nonlinear static pushover analysis is used to determine earthquake response of the structure using ETABS 9.5 (Computers and Structures) software.

Typical pushover analysis was achieved using displacement control strategy, where in the whole structure was pushed to evaluate the seismic performance of the buildings using preselected lateral load pattern until the roof displacement reaches the target value. The lateral load pattern was distributed along the height of the structure in such a way that each floor is subjected to a concentrated force. Two invariant load patterns were utilized to represent the likely distribution of inertia forces imposed on the building during the earthquakes. The invariant load patterns used are the following:



Model I Model II



Model III Model IV Model V

Figure 1: *Plan and Elevation of Eight Storeys Reinforced Concrete Building*

- **Elastic First mode Lateral Load Pattern** :

The first mode load pattern is related to the first displacement mode shape (Φ) of vibration. The lateral force of any storey is proportional to the product of the amplitude of the elastic first mode and mass (m_i) at that storey i.e.

$$F_i = m_i\Phi_i / \sum m_i\Phi_i \quad (4)$$

where,

Φ_i = Amplitude of the elastic first mode of the storey.

- **Codal Lateral Load Pattern**:

This method uses the equivalent lateral forces due to fundamental period of vibrations. The code lateral load shape represents the forces obtained from the predominant mode of the vibration and uses the parabolic distribution of lateral forces along the height of the building. The following expression has been used to calculate the load pattern as per IS 1893 (Part-I): 2002.

$$V_B = A_h W \quad (5)$$

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \quad (6)$$

Where,

V_B = Design Base Shear as per IS 1893(Part-I): 2002
Q_i = Lateral Force at Floor ***i*** **,**
W_i = Seismic weight of floor ***i*** **,**
h_i = Height of floor ***i*** measured from base and
n = Number of storey in the building.

In addition to these lateral loadings, the structures are subjected to dead loads and live loads. The displacement control method of pushover analysis was utilized with the target displacement 4% of total height of the building (ATC 40, 1996). The results were presented in the form of base shear vs. top displacement (Pushover Curves). The results of various models were discussed separately to have proper comparison between various load patterns and with that of the bare frame model. FEMA and ATC provide the frame work for performance based seismic design (FEMA 356, 2000, ATC 40, 1996). Prescribed performance levels in the FEMA-356 are the discrete damage states that the buildings can experience during the earthquake. In this study, inter storey drift capacity corresponding to the desired performance levels and two intermediate structural performance ranges were used. The discrete structural performance levels are Immediate Occupancy **(IO),** Life Safety (**LS**) and Collapse Prevention **(CP**).

3.1 Interstorey Drift

The inter storey drift is one of the commonly used damage parameter. The inter storey drift is defined as

$$SDi = \frac{\delta_i - \delta_{i-1}}{h_i} \quad (7)$$

Where, $\delta_i - \delta_{i-1}$ is the relative displacement between successive storey and h_i is the storey height. Acceptable limits of storey drift for various structural systems, associated with different performance levels were mentioned in section 3.2.

3.2 Results and discussions

As per FEMA-356, drift criteria for RC moment frames are **1%, 2% and 4%** for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance levels, respectively. The drift criteria for unreinforced masonry infilled frames are **0.1%, 0.2% and 0.6%** for IO, LS and CP performance levels, respectively. Capacity curves along with Performance levels of building models for various load patterns are shown in Figure 2 (a-e). Fundamental natural time period as per IS 1893-2002 and as per analysis using ETABS software of various models are tabulated in Table 1. Base shear and top displacement at performance levels are tabulated in the Table 2 and Table 3 respectively for the First mode load pattern and Codal load pattern.

Table 1: *Fundamental Natural Time period (sec.) of Various Structural systems*

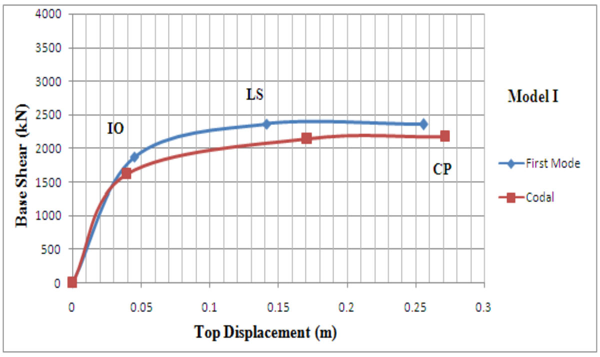
Systems	Model I	Model II	Model III	Model IV	Model V
As per IS 1893:2002	0.8130	0.4830	0.4830	0.4830	0.4830
As per Etabs analysis	1.0941	0.8673	0.8958	0.8954	0.9006

Table 2: *Base shear (kN) and Top displacement (m) at Performance levels for the First Mode Load Pattern*

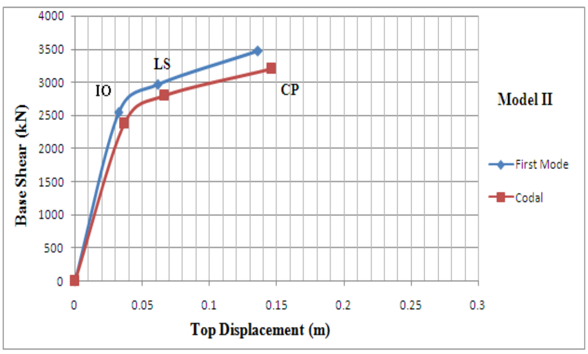
Systems	**IO**		**LS**		**CP**	
	Base Shear	**Top Displacement**	**Base Shear**	**Top Displacement**	**Base Shear**	**Top Displacement**
Model I	1868.34	0.0448	2367.21	0.1414	2352.12	0.2557
Model II	2551.74	0.0325	2970.63	0.0616	3474.98	0.1301
Model III	2494.09	0.0327	3153.58	0.0844	3269.43	0.1324
Model IV	2504.95	0.0331	3164.12	0.0860	3275.20	0.1333
Model V	2487.11	0.0327	3160.29	0.0863	3272.21	0.1342

Table 3: *Base shear (kN) and Top displacement (m) at performance levels for the Codal Load Pattern*

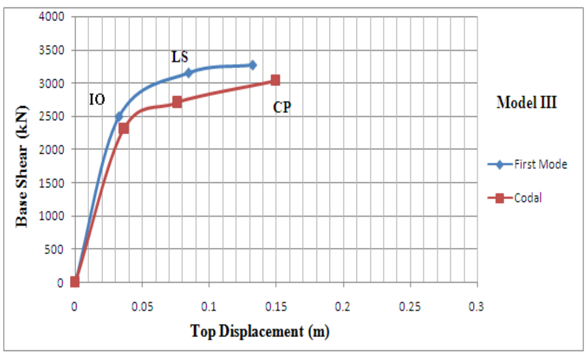
Systems	**IO**		**LS**		**CP**	
	Base Shear	**Top Displacement**	**Base Shear**	**Top Displacement**	**Base Shear**	**Top Displacement**
Model I	1615.48	0.0393	2146.94	0.1708	2174.74	0.2718
Model II	2380.11	0.0366	2796.46	0.0664	3209.57	0.1463
Model III	2307.82	0.0364	2704.41	0.0760	3031.15	0.1499
Model IV	2319.93	0.0371	2721.02	0.0728	3028.85	0.1479
Model V	2329.79	0.0376	2730.02	0.0773	3032.99	0.1511



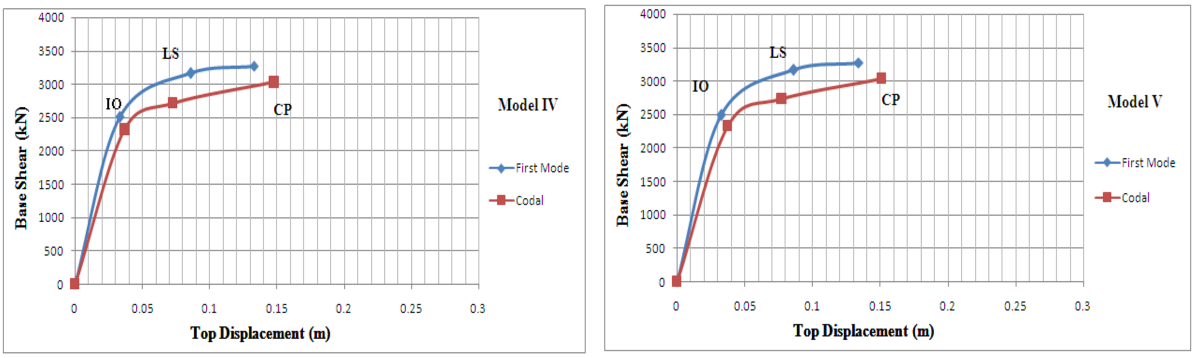
(a) Model I



(b) Model II



(c) Model III



(d) Model IV (e) Model IV

Figure 2: *Pushover Curves Representing Performance Levels*

3.3 Fundamental Natural time period:

The codal (IS: 1893-2002) and analytical (ETABS) natural period of the various models are shown in Table 1. It is observed from Table 1 that the analytical natural period do not tally with the natural periods obtained from the empirical expression of the code. Introduction of infill panels in the RC frame reduces the time period of bare frames and also enhances the stiffness of the structure. Bare frame idealization leads to overestimation of natural periods and under estimation of the design lateral forces. It has been found that in the outer infill configuration (Model II) there was 25% reduction in time period compared to the bare frame (Model I). In all other soft storey models (Model III to V) 20 % reduction in natural period was observed compared to bare frame model (Model I).

3.4 Storey Displacement:

The top storey displacement profiles of models under consideration in Figure 2 show that introduction of infill panels in the RC frame reduces the lateral displacement considerably. From the study it was observed that First mode Lateral load pattern dominates the structures response. From Figure 2 and Table 2 it was observed that for the First Mode lateral load pattern the decrease in the top displacement in the Model II compared to the Bare frame Model (Model I) was nearly 50% and nearly 48% in Model III, IV and V respectively at the collapse prevention performance level. It was also observed that for the Codal load pattern the decrease in the top displacement in Model I compared to the Bare frame Model was nearly 46% for the Model II and nearly 44% in Model III, IV and V respectively at the collapse prevention performance level .On the similar line response of structure was seen at Life safety and immediate occupancy performance level for both lateral load patterns. It has been observed from above result that introduction of infill controls the lateral displacement and storey drift. However, in case of soft storey Models (Model III, IV and V) there was an increase in the top storey displacement by around 5 % compared to outer infill panel frame (Model II) at the Collapse prevention performance level. On the similar line lateral displacements of models were seen at the life safety and the immediate occupancy performance levels.

3.5 Base Shear:

Performance evaluation using the First Mode lateral load pattern resulted in higher base shear than the Codal load pattern. From the results in Table 2 and Table 3 it was observed that for the First mode load pattern the increase in the base shear in Model II was nearly 48% compared to the Bare frame model and was nearly 40% in soft storey models (Model III to V) compared to the Bare frame (Model I) at collapse prevention performance level. Similar to Elastic First mode pattern, the Codal load pattern also governed the structural response. On the similar line response of structure was seen at Life safety and immediate occupancy performance level for both lateral load patterns.

4.0 Conclusions

In this research, the effects of various configurations of masonry infills in the seismic response of gravity load designed RC frame buildings have been discussed. It has been found that the IS code provisions do not provide any guidelines for the analysis and design of RC frames with infill panels. It has been found that calculation of earthquake forces by treating RC frames as ordinary frames without regards to infill results in underestimation of base shear. Therefore it is essential for the structural systems selected to be thoroughly investigated and well understood for catering to soft ground floor, as the presence of masonry infill panels in the frame substantially reduce the overall damage. The performance of fully masonry infill panels was significantly superior to that of bare frame and soft storey frames. The present study also demonstrates use of nonlinear displacement based analysis methods for predicting performance based seismic evaluation.

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