

## **RESPONSE OF SINGLE BAY TWO STOREYED BRICK MASONRY INFILLED RC PORTAL FRAMES UNDER IN-PLANE CYCLIC LOADS**

Arunkumar A.S.<sup>1,a</sup> and Raghunath S.<sup>2</sup>

<sup>1</sup>Department of Architecture, <sup>2</sup>Department of Civil Engineering  
BMS College of Engineering (Autonomous College affiliated to VTU Belgaum), Bull Temple Road, Bangalore

<sup>a</sup>Corresponding Author E-mail: amrutharu1966@yahoo.co.in

### **ABSTRACT**

Multistoreyed Reinforced Concrete (RC) framed structures with masonry in-fills (MI) have become a widely accepted construction practice. Since, the lateral stiffness of MI has no influence on the design of RC frames under gravity loads, its stiffness can be ignored. However, the lateral load resisting capacity of such frames is largely influenced due to the lateral stiffness and strength contributed by MI and they also influence the post yield response of the structural system significantly. This paper presents the behavior of MI- RC frames under in-plane reversed cyclic lateral loads up to failure. The influence of brick MI on the post yield behavior has been investigated experimentally. Three half-scale single-bay two-storeyed RC frame models were investigated. One of them was a bare RC frame, the second one was RC frame with MI, while the third one was in-filled in the upper storey, to study the open ground storey effect. The post cracking behavior of the system has been experimentally studied and has been presented in detail.

**KEYWORDS:** In-filled RC frame; reversed cyclic loading; Lateral stiffness; stiffness degradation.

### **INTRODUCTION**

Multistoreyed structures are built using masonry, reinforced concrete, structural steel and pre-stressed concrete or its combination as main structural entities. Multistoreyed RC structures which are constructed extensively comprises of beams, columns and slabs along with MI. MI constructed within the RC frame serve as interior or exterior filler walls. These MI-RC framed structures behave well as long as they have to sustain gravity loads. During wind or earthquake induced loads these frames are subjected to lateral loads in addition to the existing gravity loads. The elements of RC frames are generally designed to support the entire gravity and lateral loads ignoring the contribution from MI. MI is usually considered as a non-structural entity and hence only its mass is being considered on the beams in the analysis of MI-RC frames. The interaction of MI with RC frames under lateral loads result in complicated failure mechanisms such as shear failure of columns, cracking of masonry mortar joints, crushing of masonry units etc. These failure mechanisms pose significant challenge in modeling and performance of MI- RC framed structures.

The interaction of MI with RC frame has been the crux of many research investigations done so far. The behavior and the stiffness contribution of MI were first studied by Smith (1967), Smith and Crater (1970) who had developed a rational method to arrive at the stiffness of the diagonal strut which could replace the MI in the MI-RC frame analysis. This concept was further strengthened by studies of Madan et al (1997) by developing a hysteretic model taking into account the effects of stiffness degradation, strength deterioration and pinching. However, they could not address frame in-fill interaction and other local effects. Experimental studies conducted by Ghassan et al. (2002) on single-storeyed MI-RC frames indicated crushing plate mode of failure in in-fills and shear mode of failure in columns. However, it was noticed that there was an enhancement in the shear capacity of in-fill which had been due to confinement of in-fill between the RC frames. The single diagonal strut model so developed by earlier researchers could not represent the actual stressed area within the MI and hence to facilitate the modeling of progressive failure of the infill at the corners a three strut model was proposed by Wael et al. (2003). It is to be noted that there was reduction in the ultimate strength and the modulus of elasticity of masonry along the inclined direction. As an alternative to the diagonal strut concept, modeling MI as equivalent spring models was suggested by Hossein and Toshimi (2004). In their case study on the Bam Telephone center subjected to 2003 Bam Earthquake Strong Motion non-linear analysis of the structure performed using equivalent spring models has predicted the response of the structure with reasonable accuracy.

Experiments were conducted by Khalid et al (1997) on masonry in-filled steel frames subjected to cyclic lateral loads to simulate the behavior of the structure during earthquake. The MI of the models tested failed by developing horizontal cracks along the mortar joints and surprisingly it was noted that the MI did not indicate diagonal strut action. On the contrary, they proposed compression strut to predict the failure which had a larger width at its centre than at its ends. The response of MI-RC frame under seismic activity was studied by Marjani and Ersoy (2002). They tested one-third scaled six-storeyed single-bay MI-RC frame under reversed cyclic load. Their investigation indicated higher lateral resistance when compared with bare RC frame. Further, it is important to note from their study that the plaster of the in-fill also contributed to the lateral load resistance of the MI-RC frame. However, it was felt that for proper prediction of strength and stiffness of MI-RC frame by analytical methods, it becomes necessary to use appropriate value of elastic modulus of materials. In this direction, Karayannis et al. (2005) conducted experimental and analytical investigation on three one-third scale single-story single-bay MI-RC frame specimens subjected to lateral cyclic loads. The tests indicated diagonal strut action of MI and the obvious increase in the initial elastic stiffness of the frame and its lateral load capacity. The post yielding effect of MI on RC frames was studied by Zarnic and Tomazevic (1985) by conducting tests on four half-scale single-storeyed MI-RC frames. Here it was found that after the failure of MI, the entire lateral load was borne by RC members resulting in shear failure of RC columns.

The experimental investigations by Girish and Achyutha (1996) on quarter-scale three-storeyed models with varying storey shears indicated that the response of the frames tested with regard to strength, energy dissipation was largely influenced by the slip and separation of the interface of MI and RC frame. However, the expected increase in base shear resisted by MI-RC frame model was three times that of bare RC frame model. As the elastic analysis was not adequate, an in-elastic model to simulate the realistic behavior of MI-RC frame was developed by Singh et al. (1998) which could predict the response, failure load and mode of failure of the frames. Further, to enhance the performance of MI-RC frames experimental investigation was conducted by Umarani and Govindan (2000) on quarter-scale two-bay five-storeyed RC frame in-filled with confined brick masonry under static lateral cyclic loads. The confinement so provided to masonry increased the lateral load capacity, energy dissipation and stiffness of the frame with reduction in ductility. Similar experimental studies carried out by Murty and Jain (2000) on single-bay single-storey RC frames of 1:2.7 reduced scale models. The models studied were (i) bare RC frame, (ii) RC frame with un-reinforced masonry in-fill and (iii) RC frame with reinforced masonry in-fill. The studies indicated the expected increase in stiffness of MI-RC frame. The increase in average ductility of un-reinforced masonry in-filled RC frame was 4.0 times that of the bare RC frame models while the ductility of reinforced masonry in-filled RC frame was 5.1 times that of bare RC frame. Further, the studies also revealed that there is no significant increase in strength and stiffness of reinforced masonry in-filled RC frame when compared with MI-RC frame with unreinforced masonry in-fill. Parametric studies on the seismic performance of MI-RC frames by Singh and Das (2006) indicated the well-known strut action of MI. Here, it was noticed that high axial and shear forces was generated on columns of MI-RC frames resulting in earlier failure than those of bare RC frame columns. The performance of these frames depend on the natural period of vibration which is used to arrive at the probable design storey shears to be used in the seismic analysis of MI-RC frames. It was found from studies by Kaushik et al. (2006) that there was no consensus on the values of fundamental period of vibration, response reduction factor and allowable drifts provided in seismic design codes of various countries. Also, the mode of failure of MI-RC frames depends on the type and characteristics of masonry used as MI. Experimental investigations done by Suyamburaja et al. (2007) and Suyamburaja and Subramanian (2008) on one-fourth scaled three-bay five-storeyed brick masonry in-filled RC frame indicated shear failure of columns along with excessive buckling. The probable variation in the behavior and mode of failure of MI-RC frames may be due to the complex behavior of the interface between the MI and RC frame. An attempt was made to address this behavior by providing springs between MI and RC frame in the parametric studies conducted by Modal and Jain (2008). They developed a finite element model to arrive at the reduction factor to be applied to diagonal strut width for MI-RC frames with central openings. Further, the force based method of analysis and design suggested in IS code of practice seems to be deficient and hence, Hashmi and Madan (2008) in their investigations indicated the use of a rational non-linear displacement method as the appropriate method of analysis for the design of MI-RC frames.

Experimental and analytical studies by researchers namely, Ricardo (2005), Ozgur and Sinan (2006), Mehmet and Tugce (2010), Ioannis et al. (2011) has clearly indicated the increase in initial stiffness in the case of MI-RC frames but, it is important to note that there has been no consensus on the range of increase of stiffness, lateral load resistance and mode of failure of RC frames due to the presence of MI. This might be due to the type, characteristics, interface behavior and workmanship associated with the construction of MI as being noticed by Mehmet and Tugce (2010) in their studies on MI-RC frames. Numerical models of MI-RC frames developed have not predicted the failures precisely. This has been reported by the investigations carried out by Ioannis et al. (2011) wherein the two-third scale model developed a combination of diagonal/sliding cracks in the MI together with a major shear crack near the mid height of the columns while analytical studies indicated shear failure at the column base. Studies on MI-RC frames indicate a significant strength and stiffness degradation after the failure of MI which clearly shows the participation of MI in the lateral load resistance of MI-RC frames.

There have been scanty experimental studies on the behavior of masonry in-filled RC frames, particularly in the Indian context wherein the strength and stiffness of brick masonry in-fill are relatively low.

### **OBJECTIVES OF THE PRESENT INVESTIGATION**

The strength, elastic properties and construction practices of masonry adopted in other countries are significantly different from those of Indian masonry. Indian brick masonry in particular has very low strength and low modulus of elasticity. Therefore, the contribution of brick masonry as MI in resisting seismic loads needs to be investigated. The building frames are being analyzed for lateral loads by assuming the in-fills as non-participating walls wherein the mass of MI is considered on the beams. However, the response of the frames in the presence of MI under lateral loads will be different from those when analyzed as bare RC frames. Further, multistoreyed framed structures have open lower storey used for parking purpose. The absence of MI in the lower storey would change the response of MI-RC frames. In view of the above there is a need to study the behavior of frames as being analyzed and constructed in practice. The objectives of the present study are;

- To study the response of bare RC frame with the mass of MI being simulated by stacking equivalent number of bricks on the beams subjected to reversed cyclic lateral loads.
- To study the response of MI-RC frame under reversed cyclic lateral loads
- To study the open ground storey effect on the response of MI-RC frame under reversed cyclic lateral loads

### **PRESENT INVESTIGATION**

In the present investigation three half scaled single bay two storeyed RC frames were considered. The columns and beams were 100mm x 100mm in cross section reinforced with four bars of 8mm HYSD bars with 6mm mild steel stirrups at 50mm centre to centre as shown in Fig. 1. The reinforcement has been fabricated conforming to the IS: 13920 (1993). The details of the models tested are as under,

- i) Bare RC frame model wherein the mass of the wall on the beams was simulated by stacking an equivalent number of bricks on the beams which is as shown in Fig. 8.
- ii) MI-RC frame model wherein the MI was constructed after casting the RC frame.
- iii) MI-RC frame with open ground storey constructed similar to MI-RC frame but without MI in the ground storey.

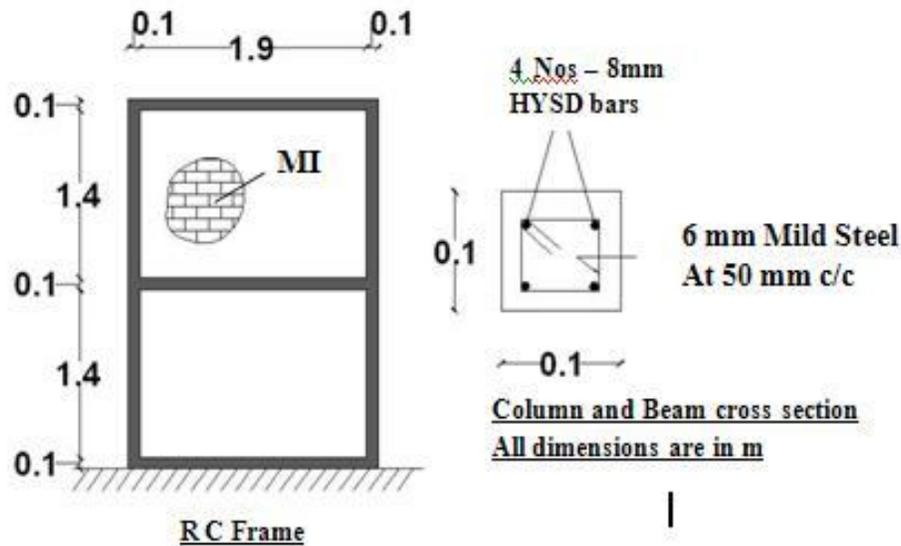


Fig. 1 Details of Test Specimens

#### CHARACTERISTICS OF MATERIAL USED IN THE EXPERIMENTAL STUDY

The RC members of the test specimen were small in dimensions with closely spaced reinforcements and to facilitate proper placement and compaction of concrete, self-compacting concrete (SCC) was used for avoiding the need for vibration of concrete. Ordinary Portland cement (C 53 grade) conforming to IS: 12269 requirements was used in the investigation. Fly ash obtained from Raichur Thermal Power Station (RTPS), Karnataka was used as filler. Natural river sand (conforming to zone II of IS: 383-1970) was used as fine aggregate and crushed granite stone was used as coarse aggregate. The coarse aggregates used were 12.5mm and down size. Commercially available modified polycarboxylic ether based super plasticizer (Glenium B233) was used as a chemical admixture. From the studies conducted by Girish (2010), method based on absolute volume concept starting from a volume of paste was used for mix proportioning of SCC. The optimum dosage of super plasticizer was obtained by conducting Marsh cone test with powder and water. The volume of paste of 0.41 with water content of 180 l/m<sup>3</sup>, cement content of 300.0 kgs/m<sup>3</sup> and water to cement ratio of 0.6 was used to achieve concrete cube strength of 44.2 MPa (fcu). This mix was used in preparing bare RC frame model. For the above parameters and with a cement content of 350.0 kg/m<sup>3</sup> concrete mix used to prepare MI-RC frame had cube strength of 53.0 MPa (fcu).

Standard table moulded bricks available in Bangalore were used in the MI-RC frame models which conformed to IS: 3495 (part-1):1992 having a unit strength of 6.31 MPa. Brick masonry in-fill was constructed within the RC frame in cement sand mortar (1:6) having a water cement ratio of 1.2. Stack bonded brick masonry prisms were cast, cured for 28 days and were tested under compression. The cubes and cylinders of SCC and mortar cylinders were cast cured for 28 days and were tested under compression to evaluate the compressive strength and modulus of elasticity of concrete and cement mortar used for the construction of the models. The test results are shown in Table 1. Reinforcing steel used for the present investigation conforming to IS: 1786-1985 having yield strength of 415.0 MPa was used. Mild steel reinforcements having yield strength of 250.0 MPa was used for stirrups of beams and laterals of columns.

**Table 1: Strength and elastic properties of materials used in the models**

Sl. No	Material	Compressive strength (MPa)	Modulus of Elasticity (MPa) (initial tangent modulus)
1	Brick Masonry	$f'_m = 3.45$ (corrected prism strength)	1560 (prism tests)
2	Concrete	$f_{cu} = 44.2$ (bare RC frame) $f_{cu} = 53.0$ (MI-RC frame) (cube crushing strength)	26310.0 31477.0 (compressive tests on concrete cylinders)
3	Cement mortar	4.56 (cube crushing strength)	6370.0 (compressive tests on mortar cylinders)

### ESTIMATION OF FAILURE LOAD

Based on the cross-sectional details of the RC members of the test specimen and the grade of concrete used, moment of resistance of the section was calculated by ultimate load method. Here, the strain was assumed to vary linearly across the depth of the section. The bending compressive stress distribution in concrete was considered to be parabolic with a maximum compressive stress of  $0.67 f_{cu}$  for the respective grades of concrete used for the models. Yield stress of reinforcing steel was taken as 415.0 MPa. Using the above parameters, the moment of resistance of the section, at failure, worked out to 3028.0 N-m and 3244.0 N-m for the respective concrete grades which has been presented in Table 2.

**Table 2: Estimated failure load of the model**

SL. No	Type of the model	Ultimate Moment of resistance of the frame section (N – m)	Max bending moment in the frame as per linear static analysis (N-m)	Base shear computed (N)
1	Bare RC Frame	3028.0	3075.0	6248.0
2	MI-RC Frame	3244.0	3267.0	88040.0
3	MI-RC Frame with open ground storey	3244.0	3109.0	7810.0

The aim of the experimental investigation was to study the behavior of the models under in-plane cyclic lateral loads. To arrive at the amount of lateral load to be applied at each floor level the following procedure was adopted.

Eigen value analysis of the models was performed and the fundamental periods of vibration were 0.15s, 0.04s, 0.134s for bare frame, MI-RC frame and MI-RC frame with open ground storey respectively. In the present study only dead loads due to concrete and brick MI have been considered.

The calculated seismic weight on the RC frame was 13.025 kN. According to IS 1893 (part-1)-2002, the weights of column and walls should be equally distributed to the floors above and below. However, the construction practices adopted in India is such that there exists a structural gap at the interface of MI and the soffit of the beam which is due to improper filling of the joint.

It can be noted that the masonry is a non-structural 'in-fill' with inadequate compatibility at the top edge. As a consequence of such construction practices, the entire mass can be assumed to be lumped on to the beam below. Also, the extremely poor shear bond strength and tensile strength between the soffit of beam and top of MI, the wall readily separates even for a small lateral drift. This means that the mass is transferred to the beam below. In view of this, to simulate the conditions prevailing at the site, it was decided to lump the entire weight of masonry wall on to the beam below it.

The corresponding spectral acceleration coefficient ( $S_a/g$ ) using the design response spectra and the fundamental period of vibration for the bare frame model and MI-RC frame with open ground storey model as per IS: 1893-2002 (Part-1) worked out to be 2.5 while that for MI-RC frame model was 1.6. The

design base shear for the bare frame model and MI-RC frame with open ground storey model was calculated and was found to be 0.55kN and the design base shear for the MI-RC frame model was 0.352kN.

The design lateral load at each storey levels of the models was estimated based on the equivalent lateral load method conforming to IS: 1893 (Part-1) -2002.

Based on the calculated design base shear, the design lateral forces of first and second storey were found to be in the ratio of 1.86:1. The details are presented in Table-3.

**Table 3: Proportion of lateral forces applied at each storey of the models.**

Level	Seismic weight (kN)	Design lateral force ( Q )	Ratio (Q <sub>1</sub> /Q <sub>2</sub> )
Base	5.9	0	1.86
Storey-1	6.275	0.65Vb	
Storey-2	0.85	0.35Vb	

Static load analysis of the models subjected to combined vertical loads (dead loads) and lateral loads were carried out. Here, the lateral loads were applied in the ratio 1.86:1. In the present study, it has been assumed that the mass of MI is entirely lumped on to the beam underneath the MI. As a consequence of this assumption, the ratio of the lateral forces along the vertical direction will be different from the ratio calculated as per the provisions of IS 1893 (2002), although the base shear remains un-altered. Further, by trial and error method, the magnitude of lateral loads required to produce a maximum bending moment in the frame equal to the ultimate moment of resistance of the respective sections was determined. The location and the magnitude of the bending moments at the beam-column junctions of the models as being determined from the static load analysis are shown in Figs. 2 to 4. The summation of the lateral loads which is equal to the base shear applied to the models was deemed as the theoretical failure lateral load of the respective models and same has been presented in Table 2. It is to be noted that the axial load on the columns being small, the axial load–moment interaction curves was not used to estimate the failure lateral load of the models.

The base shear thus calculated was applied on the respective models in cycles of increasing magnitude as shown in Figs. 5 to 7.

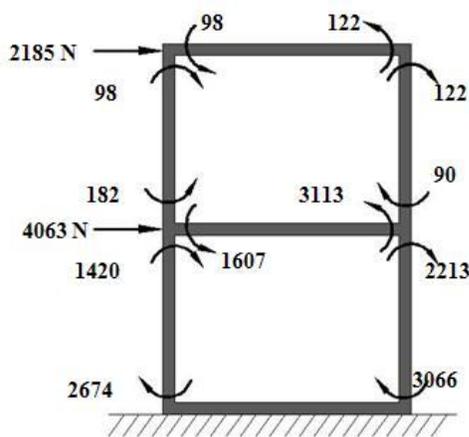


Fig. 2 Bending moments (N-m) in Bare RC frame model

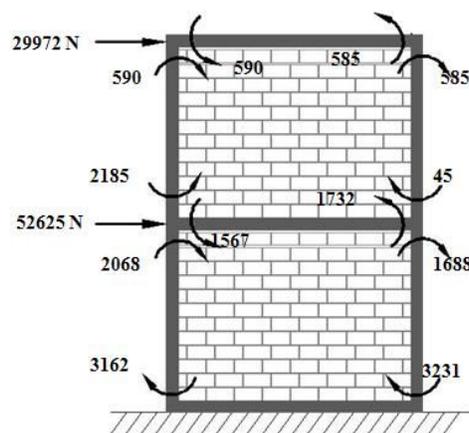


Fig. 3 Bending moments (N-m) in MI-RC frame model

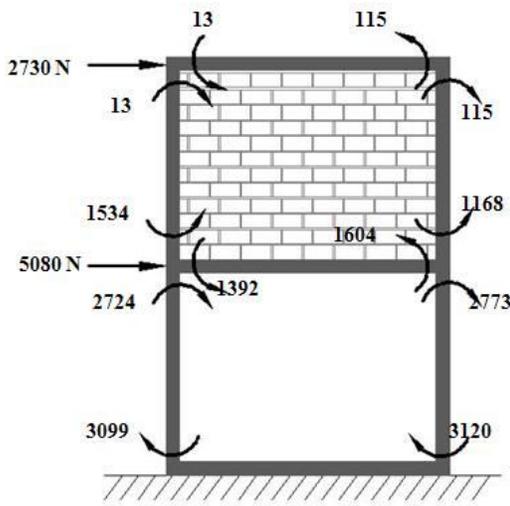


Fig. 4 Bending moments (N-m) in MI-RC frame model with open ground storey

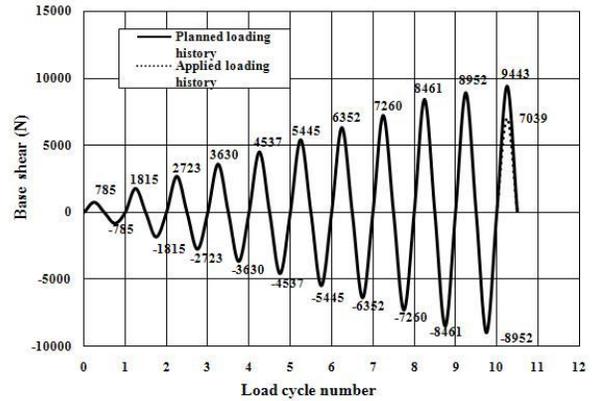


Fig. 5 Loading history for Bare RC frame model

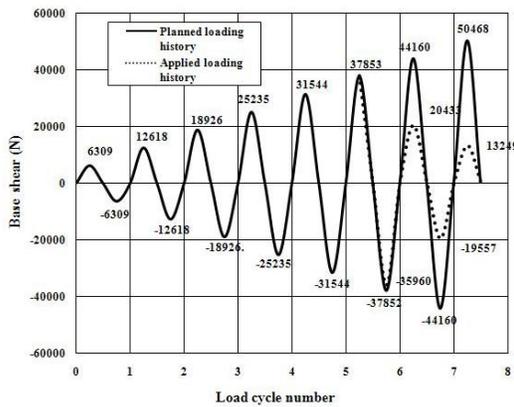


Fig. 6 Loading history for MI-RC frame model

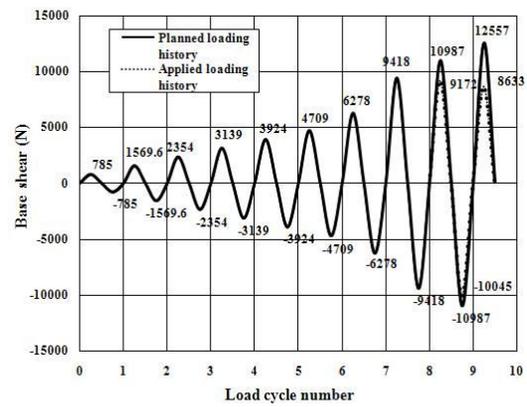


Fig. 7 Loading history for MI-RC frame model with open ground storey

### EXPERIMENTAL PROCEEDINGS

The RC frames were provided with RC flanges of 100.0 mm x 100.0 mm in section and 450.0 mm long at the bottom on either side of the columns to facilitate fixed support conditions at the base. The test setup of the models is as shown in Figs. 8 to 10. These RC flanges of MI-RC frame models were firmly secured to the base of the loading frame using adequate mild steel channels and plates which were welded to the steel base of the loading frame. The bottom flanges of the bare RC frame models were clamped at the base using lipped channels which were anchored using 10 mm diameter anchor bolts. Suitable restraint arrangements with the help of I-sections and rollers hung from the top beam of the loading frame were provided to prevent the probable out-of-plane deformation of the models so as to ensure that the models are subjected to in-plane deformations only. Jacks were mounted at each of the storey levels and digital dial gauges were mounted on the models at each of the storey levels as well as at the mid-span of the top beam of the model. All the three models were tested in a 2000.0 kN loading frame for reversed cyclic lateral loads.

The models were subjected to in-plane lateral loads using manually operated jacks which had been mounted at each of the storey levels. The lateral loads applied at the bottom and top storeys were in the ratio of 1.86:1. The lateral loads applied by the jacks were monitored using proving rings. The max base shear of each cycle shown as per the loading history of the respective models (Figs. 5 to 7) was applied through static lateral loads at the storey levels which were increased stepwise up to the maximum value and later, the lateral load was unloaded in the similar fashion. The respective storey drifts and the deflection of beams at corresponding lateral load steps were recorded using the digital dial gauges. The direction of the applied lateral loads were reversed by applying the lateral loads from the other end of the model and the dial gauge readings corresponding to the applied lateral load was recorded. Hence, in each first half of cycle of loading, the lateral load was applied gradually to the maximum value and then unloaded while, the same process was repeated in the second half of the cycle of loading by reversing the direction of the applied lateral loads.

The maximum magnitude of the base shear of each loading cycle applied to the models in the form of storey lateral forces was increased gradually up to the failure of the frame. The models were white washed to facilitate the visual observation of the cracks. The crack pattern and its progress were recorded and photographed from both faces.

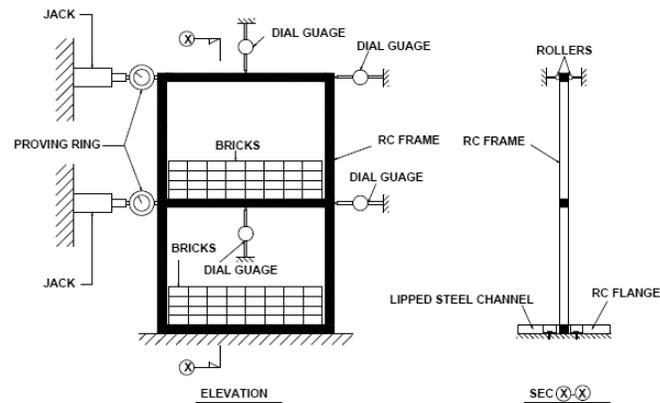


Fig. 8 Test setup for Bare frame model

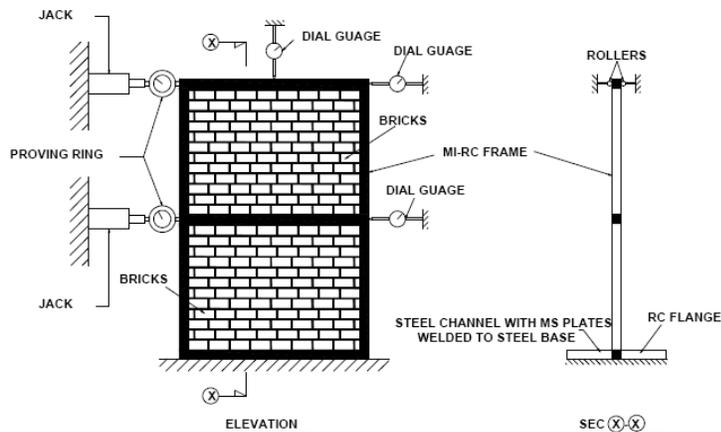


Fig. 9 Test setup for MI-RC frame model

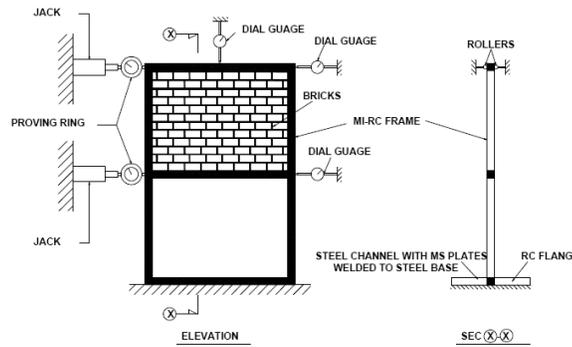


Fig. 10 Test setup for MI-RC frame model with open ground storey

## EXPERIMENTAL RESULTS AND DISCUSSIONS

### a) Top Storey drift:

The loading sequence and the maximum lateral loads applied in each cycle of loading of the respective models are shown in Figs. 5 to 7. The bare frame model was subjected to 11 numbers of cycles of loading and a maximum base shear of 8952.0 N was reached in the 10th cycle of loading. There was a drastic reduction of base shear after the 10th cycle with large drift values. Bare RC frame model exhibited a maximum drift of 310 mm at the end of 11th cycle.

The MI-RC frame model was subjected to a maximum of 8 load cycles with a maximum base shear of 37853.0 N at the 6th load cycle. During the last quarter of the 6th cycle the MI failed by horizontal shearing of mortar bed joint (Fig. 20). This resulted in a very large drift of the frame during the 6th to 8th cycles of loading. The base shears also reduced to 50% of the maximum value in the subsequent cycles of loading.

The MI-RC frame model with open ground storey was subjected to 10 numbers of cycles of loading. The frame was loaded to a maximum base shear of 10045.0 N in the 9th cycle after which there was a reduction of base shear and after midway of the 10th cycle the frame was unable to support further loading.

The top storey drifts of the frames for the base shears at ultimate loads on the respective models are shown in Fig. 11. The chart (Fig. 11) shows that Bare RC frame model exhibits highest amount of ductility while the ductility of MI-RC frame model was the least.

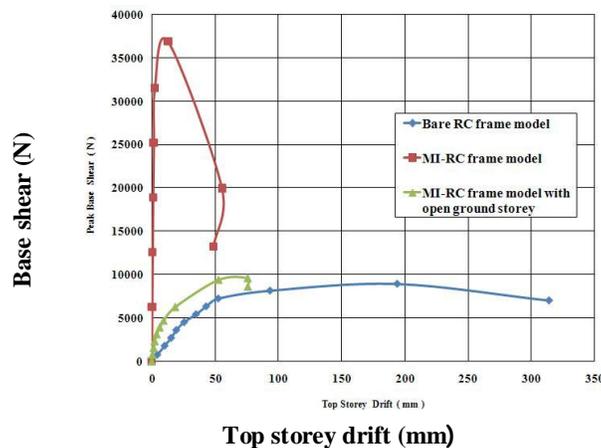


Fig. 11 Base shear v/s top storey drift at each cycle of loading

### b) Stiffness degradation:

Stiffness of the frame is defined as the amount of base shear required to produce a unit drift at the top storey level. The initial stiffnesses of Bare RC frame model, MI-RC frame model and MI-RC frame model with open ground storey at the first cycle were 178.0 N/mm, 26750.0 N/mm and 1414.0 N/mm respectively. The stiffness of the Bare RC frame model degraded beyond the 8th loading cycle. There was a stiffness degradation upto 85% at the end of 10th cycle and the recorded residual stiffness at the end of the experimentation was 27.0 N/mm.

The presence of MI showed a very high increase in stiffness in MI-RC frame model upto 150 times the initial stiffness of the Bare frame model. The stiffness of the MI-RC frame model degraded suddenly during the 6th loading cycle. The stiffness of MI-RC frame model degraded upto 99% at the end of 8th cycle and the residual stiffness recorded at the end of the experimentation was 271.0 N/mm.

The absence of MI in the bottom storey has rendered the MI-RC frame soft at the bottom storey such that the initial stiffness of the MI-RC frame model with open ground storey was 1414.0 N/mm which is 5.3% of the initial stiffness of MI-RC frame model and around 8 times the initial stiffness of Bare RC frame model. The stiffness of this frame degraded steadily upto the 5th cycle and thereafter at a faster rate. It was noticed that the frame stiffness had degraded to 92% of the initial stiffness and the residual stiffness of the frame at the end of 10th cycle of loading was 114.0 N/mm.

The stiffness degradation of the three models tested corresponding to the cycles of loading has been shown in Fig. 12. It can be noticed that the residual stiffness of the MI-RC frame model and MI-RC frame model with open ground storey model was 150% and 64% of the initial stiffness of Bare RC frame model. Further the residual stiffness of MI-RC frame model was highest while the residual stiffness of Bare RC frame model was the least.

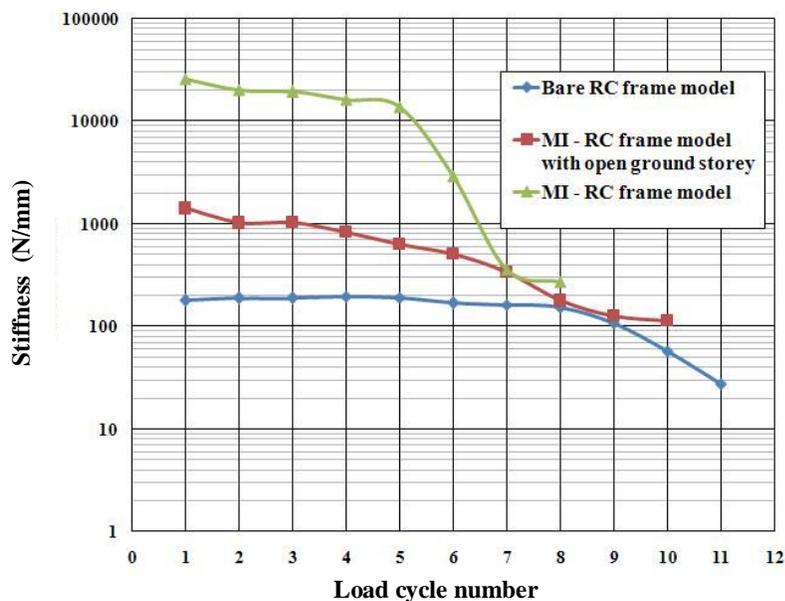


Fig. 12 Stiffness degradation chart

### c) Ductility:

It is well known that structures which have a high value of deformable capability and still retains its vertical load carrying capacity perform very well during seismic activity. These types of structures are preferred because the structure will have a capacity to absorb considerable energy during earthquake. This performance of the structure will prevent it from total collapse. To perform well during earthquakes, the structural resistance to seismic and other lateral loads demands a higher energy absorption and ductility in addition to retaining of gravity load carrying capacity. Ductility factor with respect to the top storey drift

of the test model is defined as the ratio of the maximum drift at any load level to the yield drift. Cumulative ductility was calculated which is defined as the sum of the ductilities at maximum base shear levels attained in each cycle up to the load cycle considered.

The Bare RC frame model on loading exhibited an appreciable increase in the top storey drift during the 8th cycle of loading. This is evident in the load – displacement chart shown in Fig. 13 wherein the curve deviates more during the 8th cycle of loading. This drift at the 8th cycle was considered as the top storey yield drift of the Bare RC frame model. Ductility factor at the end of each load cycle and the corresponding cumulative ductility factor were calculated.

From the load – displacement chart shown in Fig. 14 for the MI-RC frame model, the drift of 1.56 mm magnitude at the 5th cycle was considered as the yield drift. Subsequently the cumulative ductility factor for the MI-RC frame model has been evaluated.

For the MI-RC frame model with open ground storey the first yield drift was considered at the 5th cycle of loading which is evident from the load – displacement chart shown in Fig. 15.

The variation of the cumulative ductility factors of the three models is represented in Fig. 16. The tested Bare RC frame model showed higher ductility while the MI-RC frame model was less ductile than other two frames.

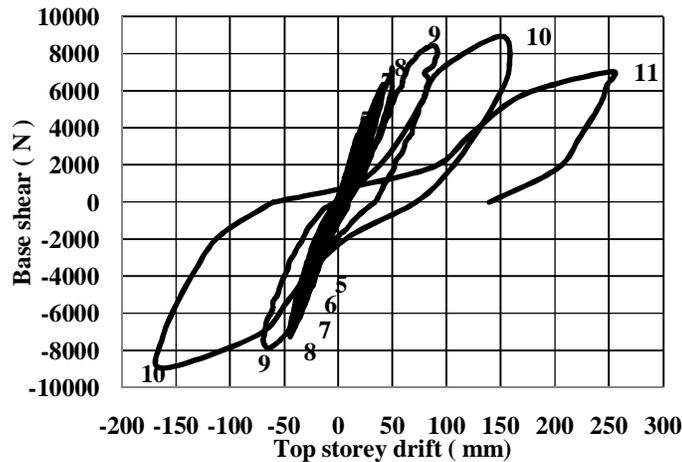


Fig. 13 Load–displacement chart for Bare RC frame model

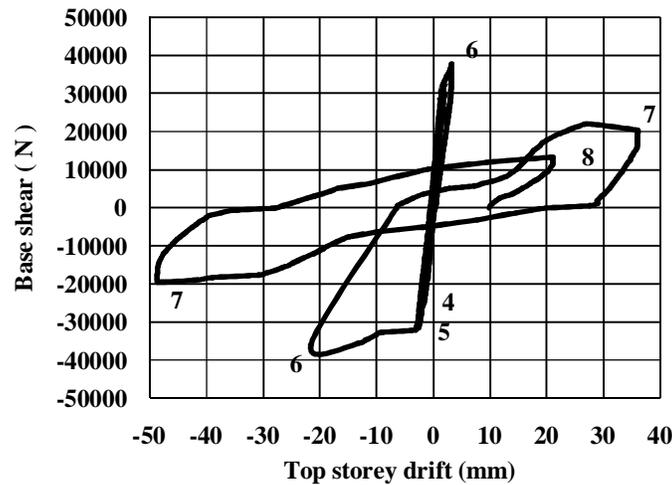


Fig. 14 Load–displacement chart for MI-RC frame model

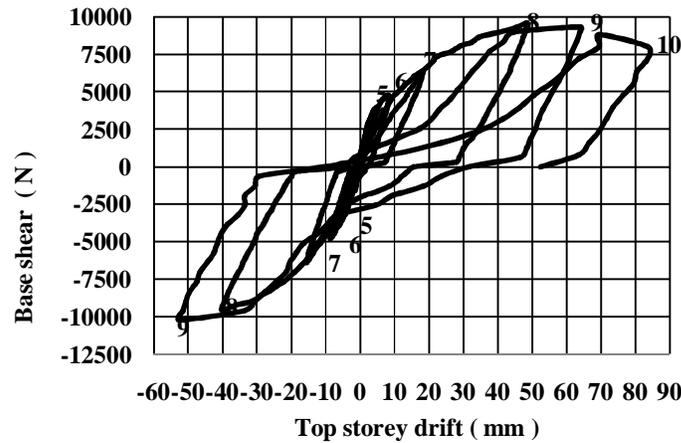


Fig. 15 Load–displacement chart for MI-RC frame model with open ground storey

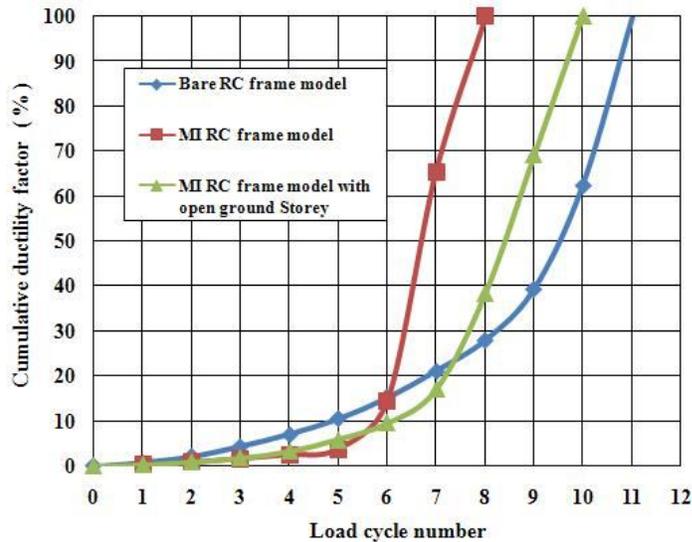


Fig. 16 Cumulative ductility chart

#### d) Energy dissipation:

The energy dissipation capacity of the structure per cycle is the area under the hysteretic loop of each cycle and the cumulative energy dissipated has been calculated. The energy dissipated by the Bare RC frame model in the first cycle was 1.3 kN-mm while the cumulative energy dissipated at the failure stage (11th cycle) was 2911.0 kN-mm. The energy absorbed by the MI-RC frame model and MI-RC frame model with open ground storey in the first cycle was 0.68 kN-mm and 0.12 kN-mm respectively, while the cumulative energy dissipated at the failure stage by MI-RC frame model (8th cycle) was 2006.0 kN-mm and by MI-RC frame model with open ground storey (10th cycle) was 1355.0 kN-mm.

The comparison of the cumulative energy dissipated by the three models is shown in Fig. 17. The total energy dissipated by MI-RC frame model was about 68% and that of MI-RC frame model with open ground storey was 47% of the energy dissipated by Bare RC frame and hence MI-RC frame model and MI-RC frame model with open ground storey exhibited much lower ductility and absorbed less energy than Bare RC frame. It is to be noted that the energy dissipated at the 8th cycle of loading of the MI-RC frame model was 5.8 times the energy dissipated by Bare RC frame at the 8th cycle of loading.

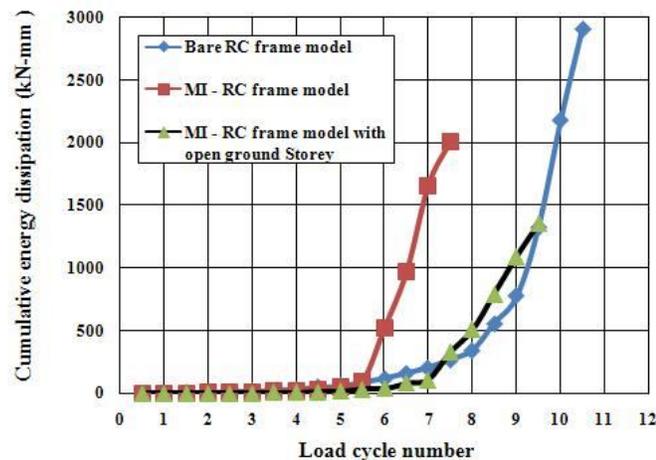


Fig. 17 Cumulative energy dissipation chart

### MODE OF FAILURE

The first visible crack at the beam column junction of the bottom storey developed midway during the third cycle of loading of the Bare RC frame model. This corresponded to a base shear of about 2722.0 N. Diagonal crack occurred exactly at the geometric centre of the beam-column junction. Later the reversal of load led to another crack at the same location. Thus a typical X – type crack was noticed. During the immediate next cycle, similar cracks were noticed at either end of the bottom storey beam-column junction. During the ninth cycle of loading, plastic hinges had begun to form at all junctions viz. at the base, at the top and at the bottom storey levels. During the last stages of loading, spalling of concrete was noticed (Fig. 18). The final deformed shape of the Bare RC frame model is shown in Fig. 19. At this stage a few negative moment cracks were noticed at the top of the beams. It was possible to achieve a maximum top storey drift of 314.0 mm and no horizontal shear cracks were noticed in columns.

The observed maximum drift value of MI-RC frame model during the first 6 cycles of loading was around 2.06 mm at a base shear of 37850.0N which is due to a relatively high stiffness of the frame due to the presence of MI. During the second half of the 6th cycle there was a brittle failure of the MI and on further loading cracks developed in the MI and in the RC frame. It was noticed that there was a large separation of MI from the RC frame and due to the drastic deduction in the stiffness of the frame, numerous cracks were triggered immediately at the beam-column junctions and in lower storey columns which is shown in Fig. 20. On further loading, the frame went into the plastic state with a sudden increase in the drift by an amount of 18.42 mm which is illustrated in Fig. 14 and at the end of the 6th cycle there was a residual permanent drift of 6.54 mm. During the next cycles of loading the load resisting capacity of the frame drastically reduced as shown in fig. 14 and the cracks in the frame increased which resulted in shear failure of lower storey column associated with crushing of concrete at the beam-column junction which is illustrated in Fig. 20. The final deformed shape of the MI-RC frame model is shown in Fig. 20.

There was a slight improvement in the performance of MI-RC frame model with open ground storey over the Bare RC frame model. Due to the low stiffness of the bottom storey the upper storey experienced a rigid body displacement in the initial 5 cycles of loading. Cracks in the bottom storey column at 150 mm below the beam and at beam-column junction was initiated mid way of the 6th cycle of loading. Beyond the 7th cycle, the model showed increased drift values indicating the yielding of the frame and at the 8th cycle there was separation of vertical interface of MI and the frame shown in Fig. 21. This further increased the crack widths and at the 10th cycle model was on the verge of failure at beam-column junction and columns showed failure cracks at beam-column junction and at the base as shown in Fig. 21. It is to be noted that no visible cracks was noticed in the masonry in-fill. The final deformed shape of the MI-RC frame model with open ground storey is shown in Fig. 21.



Fig. 18 Formation of plastic hinge at bottom storey beam-column junction - Bare RC frame model



Fig. 19 Frame at failure stage. - Bare RC frame model



Fig. 20 Sliding shear of masonry leading to shear failure of column – MI-RC frame model



Fig. 21 Frame at failure stage indicating rigid body displacement of upper storey- MI-RC frame model with open ground storey

## CONCLUSIONS

The present investigation focused on the in-plane lateral load response of MI-RC frames. Three half scaled models namely (i) Bare RC frame model, (ii) MI-RC frame model and (iii) MI-RC frame model with open ground storey were subjected to in-plane reversed cyclic lateral loads through a series of experiments. Based on the studies the following sets of conclusions are high lightened;

- a) The peak base shear resisted by the Bare RC frame model was 8952.0 N as against the theoretically calculated value of 6248.0 N. Further, the peak base shears resisted by MI-RC frame model and MI-RC frame model with open ground storey model were 37853.0 N and 9418.0 N respectively. The corresponding theoretically calculated values were 88040.0 N and 7810.0 N for the respective models. It is thus clear that the presence of in-fill enhances the lateral load capacity.
- b) It was possible to achieve a maximum top storey drift of 314.0 mm for the Bare RC frame model. The maximum top storey drifts for MI-RC frame model and MI-RC frame model with open ground storey were 55.56 mm and 76.0 mm respectively. The influence of in-fills in increasing the stiffness is clearly noticed.
- c) The initial stiffnesses of Bare RC frame model, MI-RC frame model and MI-RC frame model with open ground storey at the first cycle were 178.0 N/mm, 26750.0 N/mm and 1414.0 N/mm respectively. Stiffness degraded at a faster rate in the MI-RC frame model when compared with that of the Bare RC frame model. The average stiffness degradation of Bare RC frame model was 13.7 N/mm per cycle. The average stiffness degradation of MI-RC frame model and MI-RC frame model with open ground storey were 33107 N/mm per cycle and 1307 N/mm per cycle respectively. Further the residual stiffness of MI-RC frame model was highest while the residual stiffness of Bare RC frame model was the least.
- d) Bare RC frame model exhibited higher post yield ductility while that of MI-RC frame model was the least. Further, the energy dissipated in MI-RC frame model in the 8th cycle was 5.8 times that of the Bare RC frame model which was due to the presence of MI. However the energy dissipation capacity of MI-RC frame drastically reduced after the failure of the MI. Bare RC frame model exhibited better ductility when compared with MI-RC frame model.
- e) Bare RC frame model exhibited typical X- type cracks at the lower storey beam-column junction which indicates the formation of plastic hinges at the joints. A brittle fracture of the masonry in the MI-RC frame model has resulted in sudden increase in the lateral load transferred to the columns leading to shear failure and crushing of concrete in the lower storey column. In case of MI-RC frame model with open ground storey the MI did not develop any crack and it was noticed that the upper storey exhibited a rigid body translation and finally shear-flexure cracks have developed in the lower storey columns.
- f) In the present study, it has been assumed that the mass of MI is entirely lumped on to the beam underneath the MI. As a consequence of this assumption, the ratio of the lateral forces along the vertical direction will be different from the ratio calculated as per the provisions of IS 1893 (2002), although the base shear remains un-altered. This can be considered as the limitation of the present study.

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## REFERENCES

1. Ghassan Al-Chaar, Mohsen Issa and Steve Sweeney (2002). "Behavior of Masonry-Infilled Nonductile Reinforced Concrete Frames", J. Struct Eng, Vol. 128, No. 8, pp. 1055-1063.
2. Girish, S. (2010). "Influence of Powder and Paste on the Fresh and Hardened Properties of Self Compacting Concrete", Ph.D Thesis, BMS College of Engineering, Bangalore.
3. Girish, K.B. and Achyutha, H. (1996). "Nonlinear Finite Element Analysis of Reinforced Concrete Frames with Brick Masonry Infills under Lateral Loads", Journal of Structural Engineering, Vol. 22, No. 4, pp. 211-220.

4. Hashmi, Arshad K. and Madan, Alok (2008). "Damage Forecast for Masonry Infilled Reinforced Concrete Framed Buildings Subjected to Earthquakes in India", *Current Science*, Vol. 94, No. 1, pp. 61-73.
5. Hossein Mostafaei and Toshimi Kabeyasawa (2004). "Effect of Infill Masonry Walls on the Seismic Response of Reinforced Concrete Buildings Subjected to the 2003 Bam Earthquake Strong Motion: A Case Study of Bam Telephone Center", *Bull. Earthq. Res. Inst. Univ. Tokyo*, Vol. 79, pp. 133-156.
6. IS: 383 (1970). "Specifications for Coarse and Fine Aggregates from Natural Sources for Concrete", BIS, New Delhi, India.
7. IS: 1786 (1985). "Specifications for High Strength Deformed Steel Bars and Wires for Concrete Reinforcement", BIS, New Delhi, India.
8. IS: 12269 (1987). "Specifications for 53 Grade Ordinary Portland Cement", BIS, New Delhi, India.
9. IS: 3495 (part-1) (1992). "Methods of Tests of Burnt Clay Building Bricks", BIS, New Delhi, India.
10. IS: 13920 (1993). "Code of Practice for Ductile Detailing of RC Structures Subjected to Seismic Forces", BIS, New Delhi, India.
11. IS: 456 (2000). "Code of Practice for Reinforced Concrete Design", BIS, New Delhi, India.
12. IS: 1893(part-1) (2002). "Criteria for Earthquake Resistant Design of Structures", BIS, New Delhi, India.
13. Ioannis Koutromanos, Andreas Stavridis, Benson Shing, P. and Kasper Willam (2011). "Numerical Modeling of Masonry-Infilled RC Frames Subjected to Seismic Loads", *Comput Struct*. Vol. 89, pp. 1026-1037.
14. Karayannis, G., Kakaletsis, D.J. and Favvata, M.J (2005). "Behavior of Bare and Masonry Infilled RC Frames Under Cyclic Loading: Experiments and Analysis", *WIT Transactions on the Built Environment*, Vol. 81, pp. 429-438.
15. Kaushik, Hemant B., Rai, Durgesh C. and Jain, Sudhir K. (2006). "Code Approaches to Seismic Design of Masonry-Infilled Reinforced Concrete Frames: A State-of-the-Art Review", *Earthquake Spectra*, Vol. 22, No. 4, pp. 961-983.
16. Khalid M. Mosalam, Richard N. White and Peter Gergely (1997). "Static Response of Infilled Frames Using Quasi-Static Experimentation", *J. Struct Eng*, Vol.123, No. 11, pp. 1462-1469.
17. Madan, A., Reinhorn, A.M., Mander, J.B. and Valles, R.E. (1997). "Modeling of Masonry Infill Panels for Structural Analysis", *J. Struct. Eng.*, Vol. 123, No. 10, pp. 1295-1301.
18. Marjani, F. and Ersoy, U. (2002). "Behavior of Brick Infilled Reinforced Concrete Frames Under Reversed Cyclic Loading", *ECAS2002 International Symposium on Structural and Earthquake Engineering*, Turkey, pp. 142-150.
19. Mehmet Baran and Tugce Sevil (2010). "Analytical and Experimental Studies on Infilled RC Frames", *International Journal of the Physical Sciences*, Vol. 5(13), pp. 1981-1998.
20. Mondal, Goutam and Jain, Sudhir. K. (2008). "Lateral Stiffness of Masonry Infilled Reinforced Concrete (RC) Frames with Central Opening", *Earthquake Spectra*, Vol. 24, No. 3, pp. 701-723.
21. Murty, C.V.R., and Jain, Sudhir K. (2000). "Beneficial Influence of Masonry Infill Walls on Seismic Performance of RC Frame Buildings", *Proceedings of Twelfth World Conference on Earthquake Engineering*, pp. 1-6.
22. Ozgur Anil and Sinan Altin (2006). "An Experimental Study on Reinforced Concrete Partially Infilled Frames", *J. Engstruct* 27, pp. 449-460.

23. Ricardo Perera (2005). "Performance Evaluation of Masonry-Infilled RC Frames Under Cyclic Loading Based on Damage Mechanics", *J. Engstruct* 27, pp. 1278-1288.
24. Singh, Harpal, Paul, D.K. and Sastry, V.V. (1998). "Inelastic Dynamic Response of Reinforced Concrete Infilled Frames", *Comput Struct.* 69, pp. 685-693.
25. Singh Yogendra and Das Dipankar (2006). "Effect of Urm Infills on Seismic Performance of RC Frame Buildings", 4<sup>th</sup> International Conference on Earthquake Engineering, Taipei, Taiwan. Paper No. 64.
26. Smith, B.S, (1967). "Methods for Predicting the Lateral Stiffness and Strength of Multistory Infilled Frames", *Build. Sci.*, Vol. 2, pp. 247-257.
27. Smith, B. S. and Crater, C. (1970). "A Method of Analysis for Infilled Frames", *Proc., Instn. of Civ. Engrs, London*, Vol. 44, pp. 31-48.
28. Suyamburaja Arulselvam, Subramanian, K., Perumal Pillai, E. B. and Santhakumar, A.R. (2007). "RC Infilled Frame- RC Plane Frame Interactions for Seismic Resistance", *Journal of Applied Sciences*, 7(7), pp. 942-950.
29. Suyamburaja Arulselvam and Subramanian, K. (2008). "Experimental Investigation on Three Dimensional RC Infilled Frame - RC Plane Frame Interactions with Slab for Seismic Resistance", *American Journal of Applied Sciences*, 5(4), pp. 328-333.
30. Umarani, C. and Govindan, P. (2000). "Experimental Study of RC frame Infilled with Confined Brick Masonry", 6<sup>th</sup> International Seminar on Structural Masonry for Developing Countries, Bangalore, pp. 307-316.
31. Wael W. El-Dakhkhni, Mohamed Elgaaly and Ahmad A. Hamid (2003). "Three-Strut Model for Concrete Masonry-Infilled Steel Frames", *J. Struct Eng*, Vol. 129, No.2, pp. 177-185.
32. Zarnic, R. and Tomazevic, M. (1985). "The Behavior of Masonry Infilled Reinforced Concrete Frames Subjected to Cyclic Lateral Loading", *Proceedings of Sixth World Conference on Earthquake Engineering*, pp. 863-870.