ON UTILIZATION OF SEISMIC RESISTANCE OF MASONRY INFILLS IN DESIGN OF LOW-RISE MIXED R. C. BUILDINGS- A CASE STUDY

Amiya K. Samanta
Department of Civil Engineering, National Institute of Technology Durgapur, India
E-Mail: aksnitd@gmail.com

ABSTRACT

In early stage of civilization, people constructed mostly masonry buildings based on those low-engineered and empirical approaches. In design of new buildings, the structural behaviour of masonry elements is of interest mostly in case of infilled frames or mixed RC frames, where masonry element are also being utilized as participating in structural behaviour of the building. The infilled frames show larger ductility than isolated masonry walls. Very few codes have made provisions on RC frames with brick masonry infill. Euro code (8) and Nepal Building Code (201) make some provisions for ordinary building up to three storeys in the low seismic zones, whereas the Indian seismic code IS: 1893 recommends linear elastic analysis for the bare RC frames excluding the effect of brick infill. In the present case study, the author makes an attempt to evaluate contributions of such infill on RC frames using the literature of Aliaari (2005) and effectively utilizes in design of mixed masonry RC frame structures.

Keywords: masonry, seismic resistance, rc buildings, bare rc frame, mixed rc frame, seismic storey force.

INTRODUCTION

Masonry construction was the leading structural system for low and medium rise buildings until the beginning of the twentieth century. The effective use of masonry construction is seen in load bearing structures where it performs a variety of functions such as supporting loads, dividing space, providing thermal insulation and weather protection. Later on it was replaced by new construction materials viz. reinforced concrete and structural steel. However it is still in use particularly for low cost housing as it is easy to procure, economical and good for insulation and finishing. Since many existing conventional masonry buildings suffered damage due to earthquake, it has become imperative to assess the same as earthquake resistant structures. Also earthquake prone countries like Japan do not allow going for unreinforced masonry structures beyond 9m or three stories. The major difficulties of such structures are due to its poor performance in earthquake. This is due to 1) material itself is brittle and as a result severe strength degradation occurs under cyclic load due to earthquake, 2) heavy weight, 3) wide variation in strength depending on quality of construction. Evidences show that even masonry structures designed and constructed taking care of these factors suffers damage due to earthquake but to a less extent. It's a well established fact that load-deformation relation for a masonry structure in flexure is approximately elastoplastic type and the ductility in shear is very low. As a result, the behaviour of whole system is brittle. That's why provisions are made in the code of practice for masonry structures as a box system and prescribes to design such systems against earthquake by considering a response factor twice that for a moment resisting frame.

Normally, compressive strength of masonry elements such as solid clay burnt bricks is 7 to 10 MPa and the strength of cement mortar is 1.5 to 5 MPa. Hence the compressive strength of prisms made up of clay-burnt bricks and cement mortar lies in between strength of the individual components and it increases with increasing mortar strength and with decreasing mortar thickness. The Poisson's ratio of mortar at the joint increases as compressive stress approaches maximum strength and as a result tensile stress develops in the masonry element causing cracks.

When subjected to lateral forces, masonry walls with or without opening, shear failure often takes place in the form of diagonal cracks due to relative horizontal movements. This type of shear failure is more likely to occur as the height to length ratio becomes smaller. The general principle of earthquake resistant masonry structure says that the aspect ratio of height to width should be minimum, as also ratio of opening area-to-solid wall should be minimum preferably should not exceed one-third. If the same is satisfied it may be assumed that distribution of stresses will be reasonably uniform. As long as this objective of distributing stress uniformly is implemented, a tendency of incremental failure /collapse may develop.

When a masonry wall surrounded by reinforced concrete frame is subjected to shear, the wall panel and the frame separate at a load equal to 50 to 70 percent of maximum capacity and the wall then acts as a compression strut. Final failure occurs when maximum capacity of the compression strut having width approximately one-fourth of the length of the diagonal is exceeded or sliding resistance is exceeded. Once the sliding takes place, the external shear is being resisted by only the frame-columns as friction between the sliding surfaces reduces. Thus the infilled frames show larger ductility than isolated masonry walls. The strength and the energy dissipation capacity of infilled frames are much higher than those of bare frame due to higher stiffness.

A nonstructural wall not separate from other structural elements increases stiffness and hence brings about a higher earthquake response. Such a wall causes stress concentrations and torsional deformation of the frame. On the otherhand, it increases story shear strength and energy dissipation capacity. From early 20th century, mixed solutions i.e. RC or steel frames with partly of fully masonry infill has become very popular. A significant mixed construction of reinforced or unreinforced masonry has also been carried out in European countries in particular. But due to inherent nonhomogeneous features, very little research work has been carried out, especially experimental. In general, various codes support a designer only by suggesting few methodological principles.

Very few codes have made provisions on RC frames with brick masonry infill. Euro code 8 considers brick masonry infilled RC frames as dual systems. In such cases, seismic effects on RC frames are modified by a factor equal to design spectrum ordinate of average natural period of infilled to the same for bare RC frames. Nepal building code 201 provides one thumb rule for ordinary building upto three storeys in the low seismic zones. The Indian seismic code IS: 1893 recommends linear elastic analysis for the bare RC frames excluding the effect of brick infills, which is discussed and utilized for mixed frames by modifying as per Aliaari (2005). In the present study, the author makes an attempt to evaluate contributions of such infill on RC frames and effectively utilizes in design of mixed masonry RC frame structures.

DESIGN METHODOLOGY

A multi-bay multi-story frame is subjected to design seismic forces are shown in Figure-2. The residential building being located at Durgapur, IS1893: 2002 (Part-I) is followed to calculate design horizontal seismic co-efficient (A_h) for Zone-III. For Case-I (moment resisting RC frame building without brick infill panels), approximate fundamental natural time period

$$T_a = 0.075h^{0.75} = 0.075(3x3 + 1.5 + 0.6)^{0.075} = 0.456$$

Sec

Where h = total height of the frame from the base/foundation level

From response spectrum curve for medium soil with 5% damping, average response acceleration co-efficient $S_a / g = 2.5$. With Zone factor $Z=0.16$, Importance factor $I=1.0$ and Response reduction factor $R=3.0$, design horizontal seismic co-efficient

$$A_h = \frac{Z.I}{2.R}.\frac{S_a}{g} = 0.0667 .$$

Design seismic base shear is calculated as $V_i = A_h.W$, where W is the seismic weight of the building at the corresponding floor level combined with appropriately reduced imposed load. This base shear is distributed in the form of lateral force or story shear along the height of the building as per

$$F_i = V_i \cdot \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Where n is the number of stories in the building under consideration. At the same time, the story shear force is the sum of all loads at above the story under consideration. Thus $V_i = \sum_{j=i}^{n} F_j$, where V_I is the calculated story shear force and F_i is the lateral force at the i-th story level. On the other hand, the story shear resistance may be taken as the sum of resistances of masonry infill wall and the same of reinforced concrete column. So once the story shear resistances due to masonry wall panels are estimated, one may evaluate the rest to be shared by the reinforced concrete columns. As far as the story shear resistance of the masonry wall panels is concerned, a damage level may be defined either in terms of initial cracks or major cracks. The initial and major cracking loads of a masonry wall panel may be approximated as about 40% and 70% of its ultimate load respectively, as the past literature [Tomazevic 1999] indicates. The ultimate load of an unreinforced masonry wall is the minimum of its shear and flexural capacities. It is often preferred to eliminate flexural failure of an unreinforced masonry wall panel. According to Tomazevic and Lutman (1988), the nominal shear resistance of an unreinforced masonry wall can be estimated by the following equation;

$$V_c = A\left(\frac{f_t}{b}\right)\sqrt{\left(1 + \frac{\sigma_0}{f_t}\right)} \qquad \text{Eq. (1)}$$

Where V_C is the nominal shear resistance, A is the horizontal cross sectional area, f_t is the tensile strength of the masonry, and σ_0 is the average compression stress due to vertical load and b is the shear stress distribution factor depending on aspect ratio (height/width) of the wall. For aspect ratio less than unity, b = 1.0, if aspect ratio greater than 1.5, b = 1.5 and for aspect ratio ranging in between 1.0 and 1.5, b is equal to aspect ratio. The average compressive stress may be easily calculated from the vertical load above the story level considered. If the initial cracking load (40% of the ultimate resistance) of the masonry wall panel is considered as the limiting criterion and a safety factor (FS) is applied, the above equation i.e. the capacity of the masonry wall panel may be evaluated as ;

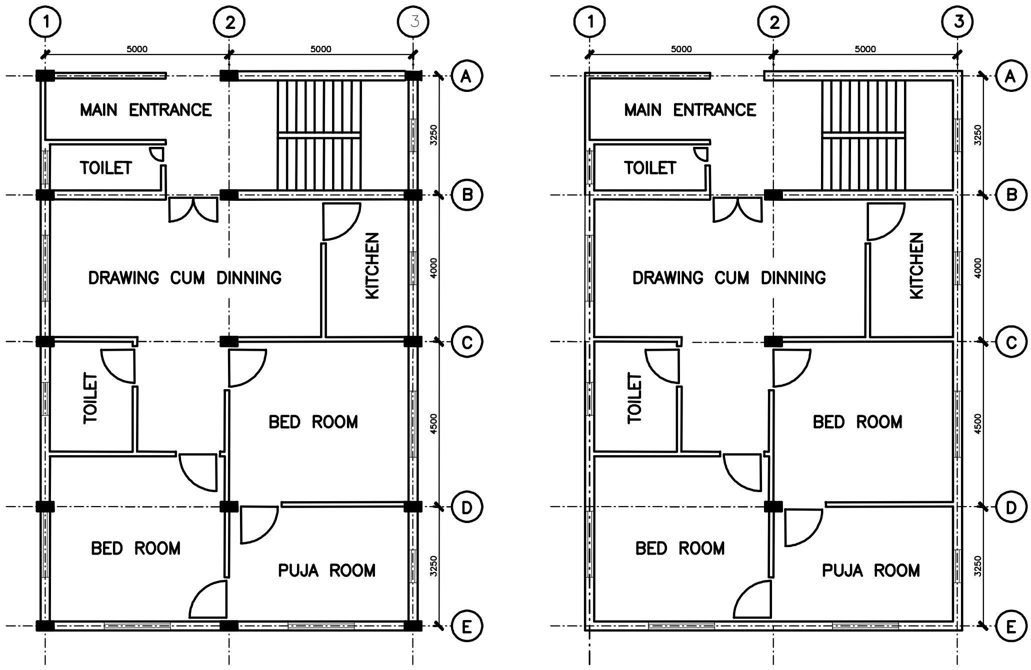
$$V_c^{'} = A\left(\frac{f_t}{b}\right)\sqrt{\left(1 + \frac{\sigma_0}{f_t}\right)}.\left(\frac{0.40}{FS}\right) \qquad \text{Eq. (2)}$$

Three-story design example

A typical three-story residential building with four bay in the longitudinal direction and two bays in transverse direction as shown in Figure-1 is considered to show the effectiveness in applying the methodology

VOL. 4, NO. 4, JUNE 2009 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

explained as above. The plan area of the building is 15.50 m by 10.0 m and the typical story height is 3.0 m. Ground floor(with tie beams and ordinary floor resting on earth) is 0.6 m above existing ground level (EGL) and the depth of foundation is 1.5m below EGL and located at Durgapur. The arrangement of brick masonry walls are also shown in the plan drawing. The building has been analyzed for earthquake loads in two different ways, viz. Case-I and II. The concrete grade used is M20 and steel reinforcement grade is Fe415. The modulus of elasticity for concrete is $5000\sqrt{f_{ck}}$ MPa as per IS456, f_{ck} being the 28-days characteristic strength, poisons ratio is 0.15. The unit weight of concrete and brick are 2.5 ton/m^3 and 1.92 ton/m^3, respectively. The RC floor thickness has been considered as equal to 125mm for all floors including roof slab. The imposed load on floor and the same on roof are considered as 0.3 ton/m^2 and 0.15 ton/m^2 respectively. The load combinations are also used as per IS1893 along with appropriate reduction in the values of imposed load.



Case-I (RC bare frame) **Case-II** (Mixed RC frame)

Figure-1. Plan of 3-storey residential building.

Case-I RC bare frame

The building is entirely supported on reinforced concrete beams, column and RC foundation i.e. conventional RC frame structure. It has got external /peripheral wall 250 thick, which is not load bearing, but provided for security purpose only and all internal /partition wall 125 thick except staircase block. As the plan shows, the longitudinal direction is considered with four bays and lateral direction with two bays. The moment resisting frames are transferring all the loads including brickwork to the foundation and are designed as bare frame. No shear resistance of masonry wall panels has been taken into consideration. The bare frames are analyzed for dead loads, imposed loads as well earthquake load as per IS1893-2002 (Part-I). Then the building is designed as per the latest code of practice and an attempt has also been made for the estimation of the whole unit.

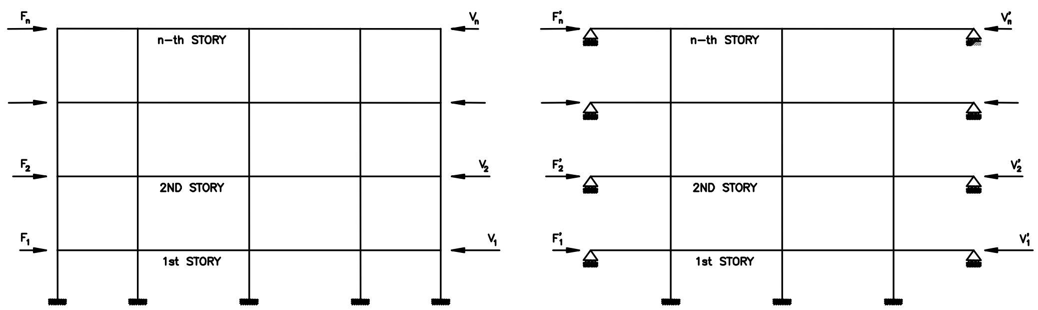
Case-II Mixed RC frame

The same building is being considered in a different way. It has got three reinforced concrete column with isolated foundation located along the central longitudinal direction. These columns are connected with RC beams in both directions at all floor levels. But it has the 250 thick load bearing masonry wall with strip foundation all along the periphery of the building as no RC column is being considered along the external periphery of the building. The RC beams are being considered supported on central columns and the load bearing mason wall panels. Like case-I, all internal /partition walls are 125 thick too except staircase block. In this case, a typical arrangement of the frame is shown in Figure-1(b) and with a reduced lateral earthquake load by the shear resistance calculated by the Eq-2. All the RC frames in both directions have got no column at either

VOL. 4, NO. 4, JUNE 2009 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

www.arpnjournals.com

ends, instead it is simply-supported on masonry wall panels, which is load bearing one.



(a) **Case-I** (b) **Case-II**

Figure-2. Multi-bay 3-storey frame subjected to lateral load.

The permissible tensile strength of the masonry (*f_t*) for cement-mortar grade M1 has been considered as 0.07 MPa as per IS: 1905-1987 and the shear stress distribution factor (b) of the wall has been taken as unity as aspect ratio (height/width) of the wall as less than unity. A factor of safety (FS) equal to 1.25 is considered here. Based on these assumptions, capacity of the masonry wall panel (referred as Aliaari storey force) and seismic story force as per IS1893 have been evaluated for the frames (case-II) along line-D and line-2 as shown in Tables-1 and 2 respectively. These tables also list the various storey forces (referred as design storey force) that need to be considered for the analysis of the corresponding RC frame in case the capacity of the masonry infill is taken into consideration.

Table-1. Design storey force along line-D.

Level	Load Type	Total load	DL+ 0.25LL	Length of wall	Wall Thk	P/A	σ_o	V_c	V_c'	Aliaari Story Force	IS1893 Story Force	Design Story Force
		(ton)	(ton)	(m)	(mm)	(MPa)	(MPa)	(ton)	(ton)	(ton)	(ton)	(ton)
Roof Lev	DL	17.7	17.70	4.0	125.0	0.354	0.354	8.61	2.76	2.76	4.70	1.94
	LL	4.6										
2nd Fl Lev	DL	23.0	25.28	4.0	125.0	0.506	0.860	12.75	4.08	1.32	4.20	2.88
	LL	9.1										
1st Fl Lev	DL	23.0	25.28	4.0	125.0	0.506	1.365	15.85	5.07	0.99	1.70	0.71
	LL	9.1										
Gr. Fl Lev	DL	11.8	11.80	4.0	125.0	0.236	1.601	17.10	5.47	0.40	0.10	0.00
	LL	0.0										
		Total =	80.1							5.47	10.70	5.53

Reduction in design story shear = 51.7 %

VOL. 4, NO. 4, JUNE 2009 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

www.arpnjournals.com

Table-2. Design storey force along line-2.

Level	Load Type	Total load	DL+ 0.25LL	Length of wall	Wall Thk	P/A	σ_o	V_c	V_c'	Aliaari Story Force	IS1893 Story Force	Design Story Force
		(ton)	(ton)	(m)	(mm)	(MPa)	(MPa)	(ton)	(ton)	(ton)	(ton)	(ton)
Roof Lev	DL	23.6	23.61	7.5	125	0.252	0.252	14.07	4.50	4.50	8.30	3.80
	LL	7.0										
2nd Fl Lev	DL	35.2	38.26	7.5	125	0.408	0.660	21.19	6.78	2.28	8.60	6.32
	LL	12.3										
1st Fl Lev	DL	35.2	38.26	7.5	125	0.408	1.068	26.46	8.47	1.69	3.40	1.71
	LL	12.3										
Gr. Fl Lev	DL	15.8	15.81	7.5	125	0.169	1.237	28.35	9.07	0.61	0.20	0.00
	LL	0.00										
		Total =	115.94							9.07	20.50	11.83

Reduction in design story shear = 57.72 %

DISCUSSIONS

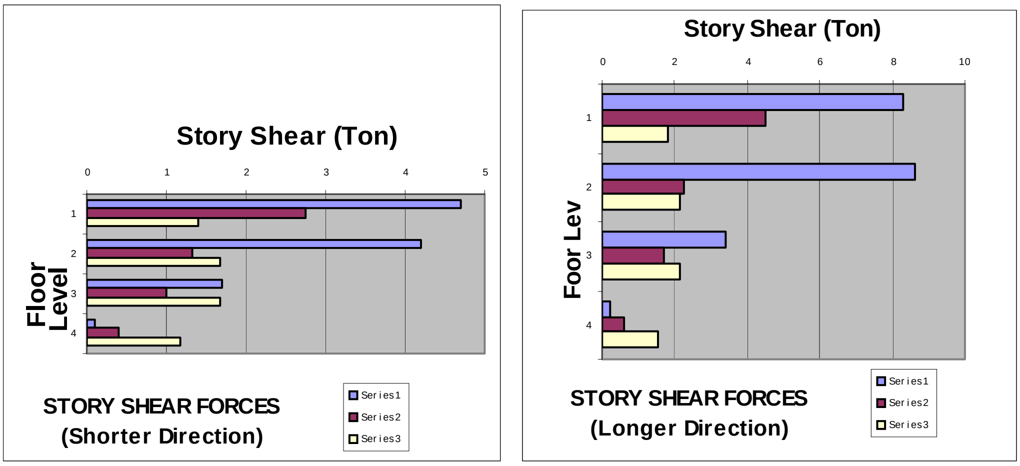
A comparison of seismic story force, Aliaari story force and lateral force due to wind load has been shown in Table-3 and Figure-3. In general residential buildings upto three stories do not required to be checked for lateral forces due to wind loads as capacity /resistance of the masonry infill is almost equal to the lateral force due to wind and Aliaari story force (approximate masonry capacity) is almost 50% of the story force due to earthquake.

The bare frames of Case-I were analyzed for various combinations (as prescribed by code of practice) of dead load, live load and lateral forces due to IS1893 seismic story force using STAADPro. Similarly the frames for Case-II were also analyzed for the same loading condition of dead load, live load and derived (reduced) design /Aliaari story force. In both cases the buildings were designed manually. Case-I includes 15 nos. RC column along with isolated foundation, whereas Case-II includes only 3 nos. RC columns with isolated foundation along with strip footing all along the load bearing external masonry wall foundation as shown in Figure-4. In Case-I, design of column requires all columns restricted to 400x250 except the column marked C2 as 400x400, whereas in Case-II all the 3-columns were 400x400. Also an assessment of the cost implication (for major items only) for both the cases was done as listed in Table-4. It clearly depicts that such mixed RC frames (Case-II) becomes an economic solution for lowrise buildings and the project cost reduces approximately by 10% on lower end.

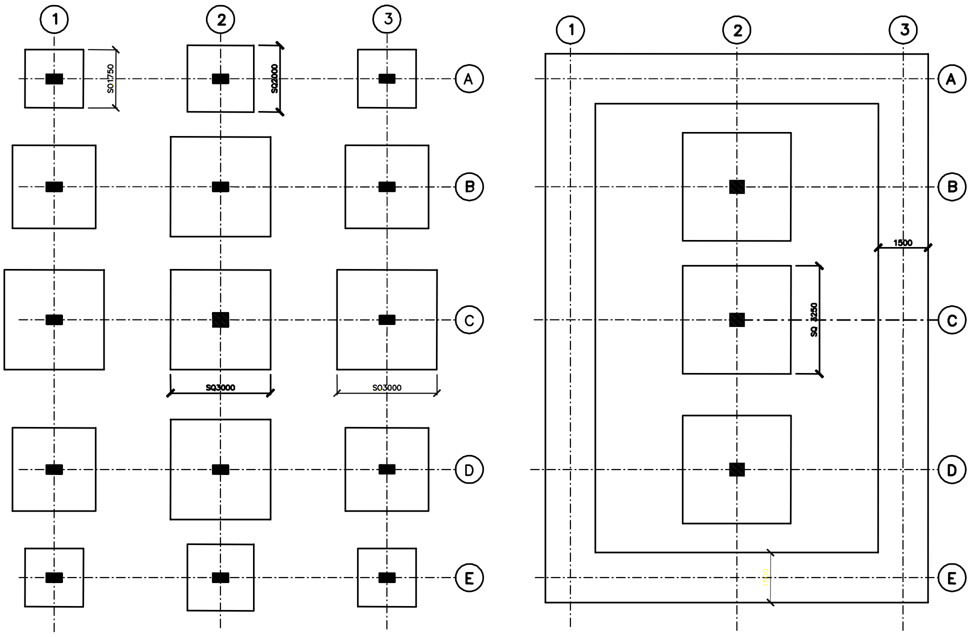
Table-3. Story shear capacity vs. story shear resistance.

	Level	1893 Story Shear	Aliaari Shear Capacity	Wind Story Shear	Aliaari Shear Capacity to wind shear ratio	Aliaari Shear Capacity to 1893 Story Shear Ratio
		(ton)	(ton)	(ton)		
Line-D	Roof Lev.	4.7	2.76	1.395	1.98	0.59
	2nd Fl Lev.	4.2	1.32	1.674	0.79	0.31
	1st Fl Lev.	1.7	0.99	1.674	0.59	0.58
	Gr. Fl Lev	0.1	0.4	1.172	0.34	4.00
Line-2	Roof Lev.	8.3	4.5	1.836	2.45	0.54
	2nd Fl Lev.	8.6	2.28	2.16	1.06	0.27
	1st Fl Lev.	3.4	1.69	2.16	0.78	0.50
	Gr. Fl Lev	0.2	0.61	1.512	0.40	3.05

ARPN Journal of Engineering and Applied Sciences



Series 1: IS1893, series 2: masonry seismic capacity, series 3: wind loads
Figure-3. Multi-bay 3-story frame subjected to lateral load.



(a) **Case-I** (b) **Case-II**

Figure-4. Foundation plan.

The main objective of this paper is to demonstrate that utility of mixed RC frames where resistance of masonry infills are effectively utilized, can promote a better and economic solution particularly for lowrise buildings through cited theoretical case study from the author's own design experience. In this regard, it is needless to mention that interactive use of brick masonry and RC frames is still an important issue nowadays by the process of deep learning and thereby making the analysis and design process more interesting and innovative to the learners and practicing engineers. In particular for underdeveloped countries, which is subjected to low seismic forces and which requires low-cost housing at the rural areas, this kind of mixed mode RC and masonry may provide a better, safe and economic solution. Also there is a negative attitude, in general, towards the use of structural masonry for new buildings in seismic areas because of the inadequate performances of unreinforced low-engineered masonry structures. With this case study, the author makes an attempt to encourage the use of structural masonry in association with RC frames in the mixed mode, where only RC frames are not being used to reduce cost of the project in one side and on the other hand effectively utilizes the contribution of masonry infills to resist the lateral forces such as seismic forces etc together with a safe and economic solution, in particular to the low-cost housing area low seismic zones.

Table-4. Cost comparison.

Sl. No.	Major Items	Unit	Quantity		Rate	Project Cost	
			Case-I (RC Frame)	Case-II (Masonry)		Case-I (RC Frame)	Case-II (Masonry)
1	Earthwork in Excavation	Cum	128.50	216.38	40	5140	8655
2	Earthwork in Filling	Cum	102.70	117.75	24	2465	2826
3	Brick Flat Soling	Sqm	219.25	257.38	145	31791	37319
3	PCC (M15)	Cum	59.47	76.69	1600	95155	122701
4	DPC (25 thk)	Sqm	--	51.00	110	--	5610
5	Brick Masonry Work						
	a) Sub-structure	Cum	--	50.36	1560	--	78562
	b) Super-structure	Cum	97.30	148.72	1560	151788	232003
7	RCC Work	Cum	167.98	110.34	2830	475378	312262
8	Steel Reinforcement	MT	18.90	12.99	32000	604800	415680
9	Formwork	Sqm	1092.80	889.68	127	138785	112989
10	Plastering						
	a) 6 thick	Sqm	480.30	480.30	19	9126	9126
	b) 12 thick	Sqm	1059.60	1059.60	39	41324	41324
	c) 20 thick	Sqm	869.40	869.40	58	50425	50425
11	Two Coats White wash	Sqm	2396.56	2396.56	5	11983	11983
						1618160	1441466

CONCLUSIONS

Although computer outputs (deformation pattern etc.) are not presented in detail, it may be concluded from the study that appropriate modeling of a structure considering resistance of masonry infills may substantially reduce the project cost in comparison with conventional bare RC framed structures particularly for lowrise buildings. In general, the exterior wall thickness of the building structure is always kept on higher side for safety, protection of fire and weather, insulation irrespective of the fact that it is a load bearing masonry structure or RC frame structure. This presentation makes an attempt to utilize this aspect of the building construction to effectively reduce the cost of the project by utilizing the resistance of masonry infills by an approximate method, where one can go for a choice in between purely RC frame construction and masonry construction. It proposes the exterior (higher thickness) wall as load bearing one and suggests going for a few interior RC columns connected to the building by floor beams and slab. It is apparent that such a mixed mode of construction may be encouraged also for regular and box shaped structures upto three stories for low cost housing which may combat lateral forces due to ground motion at low seismic zones. Of course the work has to be extended for experimental verification also before it may be put into actual practice and inclusion in the form of various codal provisions.

REFERENCES

Dowrick D. J. 1977. Earthquake resistant design: A manual for engineers and architects. John Wiley and Sons Ltd., London.

Wakabayashi M. 1986. Design of Earthquake resistant buildings. McGraw-Hill Book Company. Tokyo.

Aliaari M. and Memari Ali M. 2005. Analysis of masonry infilled steel frames with seismic isolator subframes. Engineering Structures Elsevier. 27: 487-500.

IS: 875-1987 (Part 3). Code of Practice for design wind loads. Bureau of Indian standards. New Delhi.

IS: 1893 (Part-1). 2002. Criteria for earthquake resistant design of structures, General provisions and building Bureau of Indian standards, New Delhi.

IS: 1905-1987. Code of Practice for structural use of unreinforced masonry. Bureau of Indian standards, New Delhi.

SP 20 (S and T). 1991. Handbook on masonry design and construction. Bureau of Indian Standards, New Delhi.

IS: 456-2000. Code of Practice for Plain and Reinforced Concrete. Bureau of Indian standards, New Delhi.