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CONTAINMENT REINFORCEMENT FOR EARTHQUAKE RESISTANT MASONRY BUILDINGS

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SUMMARY

The two recent earthquakes of Latur (1993) and Kachchh (2001) in India have caused large-scale damage to buildings and in particular to un-reinforced masonry buildings. In this paper an attempt is made to understand the behaviour of masonry buildings during earthquakes based on post earthquake survey. These studies have revealed the inadequacies in the current provisions for earthquake resistant design. The paper also reports the results of the dynamic analysis of typical Indian brick masonry buildings subjected to three different earthquake ground motions. The stress distributions obtained from this analysis have shown that out-of-plane flexural stresses and in-plane shear stresses exceed their strength limits. Based on stress analysis and observed damage patterns of masonry buildings, it may be concluded that out-of-plane flexural failure of walls is primarily responsible for collapse of masonry buildings during earthquakes. In order to prevent this kind of failure and to improve the ductility of masonry walls, a new and innovative way of reinforcing masonry in the vertical direction called as "containment reinforcement" has been developed. Laboratory studies on masonry building models with such reinforcement in addition to horizontal bands have shown significant improvement in flexural ductility and energy absorption capacity of masonry.

INTRODUCTION

Most of the fatalities during an earthquake are caused by collapse of buildings. The behaviour of masonry structures during earthquake ground motions has received inadequate attention in the available literature on earthquake engineering. The building damage reports of various post earthquake studies, however, invariably indicate that greater damage to buildings and loss of life takes place in one and two storeyed masonry buildings rather than in framed structures. This is especially true of masonry buildings of developing countries like India. This paper attempts to look at some aspects of the design of masonry buildings to resist earthquakes.

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Bruneau [1] provides a review of earlier information on the behaviour of masonry buildings when subjected to earthquakes. Among other features of the responses of masonry to dynamic inputs, he highlights the in-plane and out-of-plane failure of masonry walls during earthquakes. The Bureau of Indian Standards [2] recommendations for the design of masonry buildings incorporate features like horizontal bands of reinforced concrete at lintel and roof levels and vertical steel through the core of the masonry at corners, junctions of walls and at jambs of door and window openings. These practices were based on static model tests carried out by Jai Krishna and Brijesh Chandra [3] and Jai Krishna et al [4]. The static tests essentially involved applying lateral loads concentrated at the roof level. The tests also made it clear that this approach was based on the concept of strengthening the walls parallel to the ground motion, which develop in-plane stresses due to lateral dynamic loads.

PERFORMANCE OF MASONRY STRUCTURES DURING EARTHQUAKES

It is always useful to study the behaviour of masonry buildings after an earthquake as it gives an insight into the performance of various kinds masonry materials used and earthquake resistant features adopted in the buildings. Such studies will also bring out the deficiencies in the existing provisions of earthquake resistant design and help in choosing appropriate materials for reconstruction. The paragraphs to follow give a brief description of failure patterns of masonry buildings observed in several places after the Latur and Kachchh earthquakes of 1993 and 2001 respectively.

The out-of plane failure of brick masonry wall in a place called Sastur after the Latur earthquake is shown in Plate 1. This clearly demonstrates the vulnerability of masonry buildings with light roof during an earthquake. Plate 2 shows another shop building built out of brick masonry in Sastur having walls on three sides and door opening on the remaining one side with all the walls practically intact after the earthquake. This building had mud roof supported on timber posts and beams. Some of the timber posts were adjacent to the walls as could be seen in the photograph (The timber posts have been removed for reuse elsewhere). The presence of timber posts at close intervals adjacent to the walls must have acted as an earthquake resistant feature in the vertical direction and hence prevented the flexural collapse of the walls.

Plate 1: Out-of plane collapse of wall of a school building (Sastur)

Plate 2: Timber post supported wall of a shop building intact after earthquake (Sastur)

The old buildings in the town of Morbi were essentially made of sand stone units with lime mortar. All such buildings performed badly in the Kachchh earthquake, where as neighboring brick-in-cement mortar buildings survived even though they developed extensive cracks. The collapse of buildings was essentially due to out-of-plane failure of walls. Further, buildings with light roofs were more prone to such type of failure. Plate 3 shows one such typical failure.

Plate 3: Out-of-plane failure of sandstone in lime mortar masonry wall (Morbi)

In the new town of Bhuj the masonry buildings built by Central Public Works Department had earthquake resistant features like lintel band and corner reinforcement. Two such buildings were studied. In one building the wall below the lintel band suffered out-of-plane failure and the lintel band also had come down (Plate 4). Plate 5 shows another building, the corners of which has been badly damaged. This building had corner reinforcement. The provision of corner reinforcement is not possible without having a continuous vertical joint, which could trigger separation of walls meeting at the corner at the first jolt during an earthquake.

Plate 4: Out-of plane failure of wall leading to collapse of lintel band (Bhuj)

Plate 5: Corner failure in presence of corner reinforcement (Bhuj)

The observations made during the field investigations discussed above indicate frequent occurrence of out-of-plane failure of masonry building walls. It is obvious that such a behaviour is indeed caused either by the out-of-plane inertial loads on walls when they are moving out-of-their planes or by resonant response when the building as a whole is vibrating in one of its modes. Such vibrations will invariable involve out-of-plane flexure, as a study of normal modes of masonry buildings frequently indicate (Raghunath [5] and Satish [6]). Pankaj and Rai [7] have also reported damages to masonry buildings due to out-of-plane loading in the Jabalpur earthquake of 1997. It is hence clear that the current provisions of Bureau of Indian Standards [2] take inadequate cognizance of the out-of-plane dynamic response of walls. It is also necessary to recognize that collapse of masonry wall, whether it be due to in-plane failure or out-of-plane failure, will invariable involve a movement of the wall element out-of-the-plane of the wall. There is hence a need to increase the ductility of the masonry walls in out-of-plane flexure. This issue can also be examined through a dynamic analysis of masonry building subjected to earthquake ground motion.

STRESSES IN MASONRY WALLS DURING EARTHQUAKE GROUND MOTIONS

The walls of a masonry building offer resistance against lateral dynamic loads by developing flexural and shear stresses, during earthquake ground motions. In an attempt to identify the regions in a masonry building where the flexural stresses and shear stresses are a maximum during an earthquake, a linear dynamic analysis has been carried out on a typical single storeyed masonry building of plan dimensions 6.0m x 3.0m, when subjected to earthquake ground motions. It is assumed that the earthquake ground motion is perpendicular to the cross walls of 6.0m length. Two openings were provided in the cross-walls and none in the shear-walls. The following three structural features were considered for the analysis. The schematic diagrams of the buildings are shown in Figure 1 and 2.

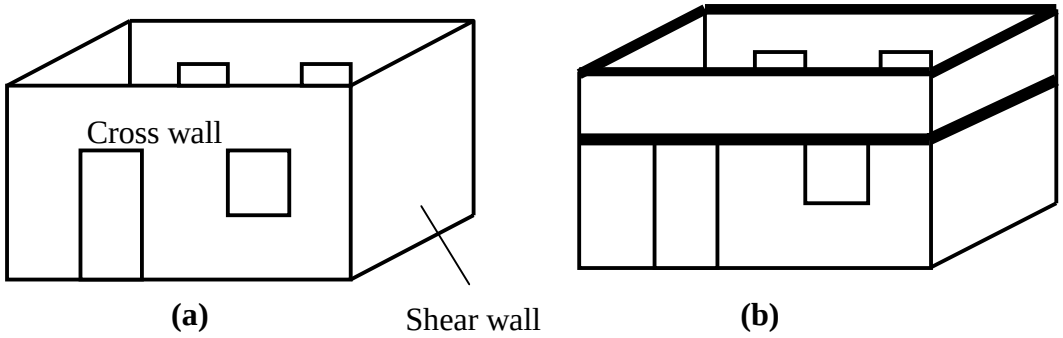


Figure 1: Buildings without roof (a) without bands (b) with RC lintel and roof bands

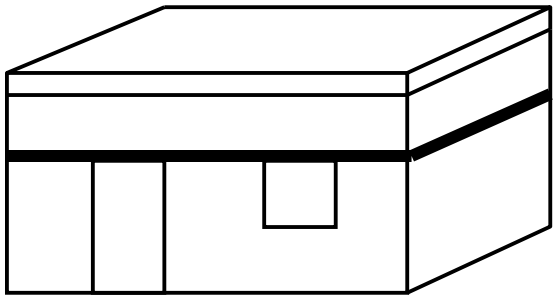


Figure 2: Building with RC roof and lintel band

1. Masonry buildings without any earthquake resistant features and with light roofing material that is loosely placed at the top edge of the wall. The top edge of walls of such buildings could be assumed to be un-restrained (B–1).
2. Masonry buildings with RC lintel and roof level bands and un-restrained at the top edge. The lintel and roof level bands were in accordance with BIS [2] codal provisions (B–2)
3. Masonry buildings with heavy RC roof and RC lintel band (B–3)

The behavior of masonry buildings under lateral dynamic loads is rather complicated basically due to complexities of the walls such as orthotropy, presence of openings, continuity at junctions of cross-walls and shear-wall etc. However, all these can be conveniently modeled using finite element technique. In the present analysis the finite element model was developed using a commercially available software [8]. The properties used for the FE analysis are presented in Table 1. Free vibration analysis was carried out to obtain the natural frequencies, mode shapes and modal participation factors before carrying out the ground motion analysis. The first four natural frequencies (in Hz) of the buildings analyzed are presented in Table 2. The fundamental mode shape of building without roof and with roof is shown in Figures 3 and 4 respectively.

Table 1: Details of finite element analysis

Parameter	Property
Size of cross-wall (height x length) Size of shear-wall (height x length)	3.0m x 6.0m; one cross-wall with a door and a window opening, other cross-wall with two window openings 3.0m x 3.0m; no openings in shear-walls
Masonry	0.23m (1 – brick thick); table moulded burnt bricks of Bangalore; mortar: CM 1:6
Reinforced concrete	RC lintel and roof bands: 0.15m thick; 0.23m wide; RC slab: 0.15m thick
Boundary conditions	Base clamped
Masonry properties [5] Modulus of elasticity normal-to-bed-joints (E_y) Modulus of elasticity parallel-to-bed-joints (E_x) Modulus of rigidity (G_{xy} assumed) Poisson's ratio (ν, assumed) Flexural strength normal-to-bed-joints Flexural strength parallel-to-bed-joints Shear strength [9] Density	 600.0 MPa 1800.0 MPa 800.0 MPa 0.2 0.137 MPa 0.36 MPa 0.06 MPa Masonry: 2000.0 kg/m^3
Dynamic analysis	Linear transient dynamic analysis (base acceleration input); no. of modes chosen: 10
Element adopted	Masonry: 4 noded orthotropic shell element, each node having 6 d-o-f RC lintel and roof band: 2 noded 3d beam element, each node having 6 d-o-f RC roof: 4 noded orthotropic shell element, each node having 6 d-o-f

Table 2: Natural frequencies (Hz) of buildings

Mode no.	Buildings without roof		Building with roof
	B-1	B-2	B-3
1	6.43	8.17	14.87
2	6.88	9.05	17.11
3	14.01	18.61	18.95
4	15.92	20.12	20.03

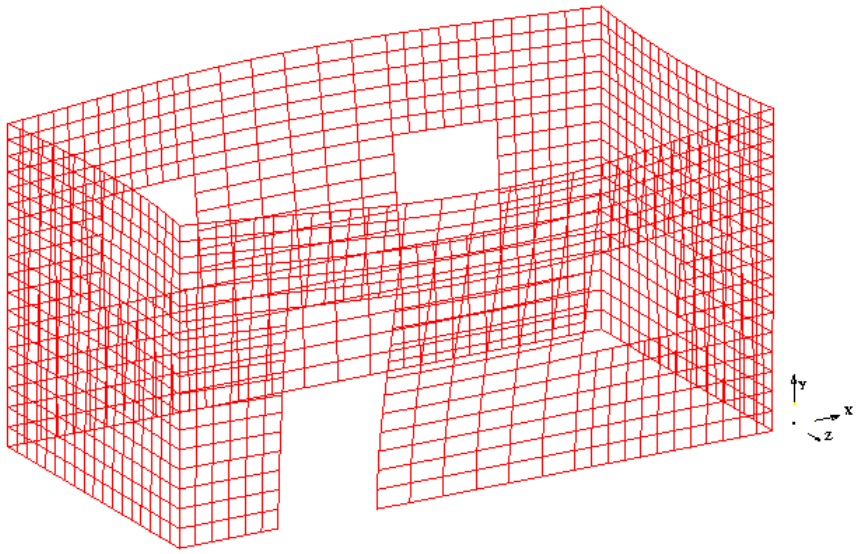


Figure 3: Fundamental mode shape of building without roof

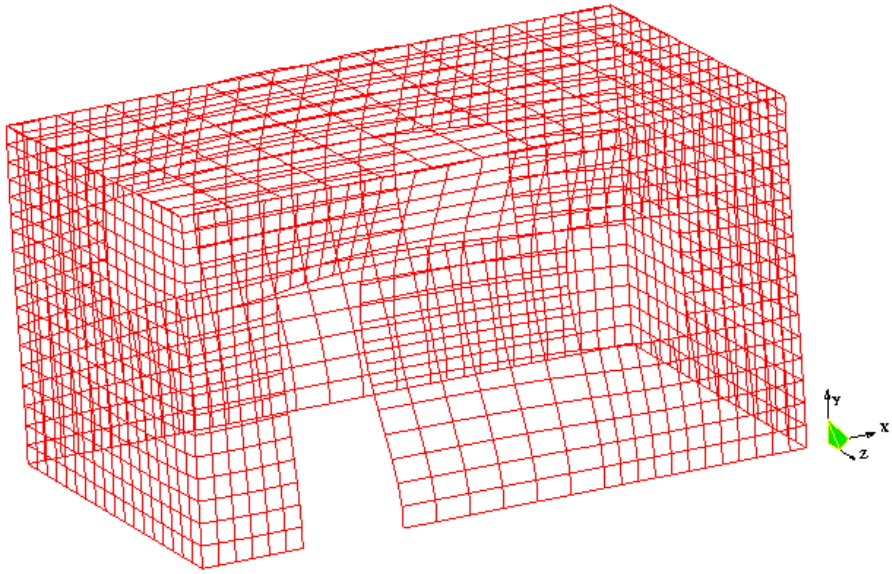


Figure 4: Fundamental mode shape of building with roof

The peak stresses for the ground motion input were obtained by carrying out a time history analysis. The ground motion acceleration was input normal to the direction of the cross-walls. Three earthquake records have been used as input for the models. The details of the three earthquakes are presented in Table 3.

Table 3: Details of earthquakes used as input

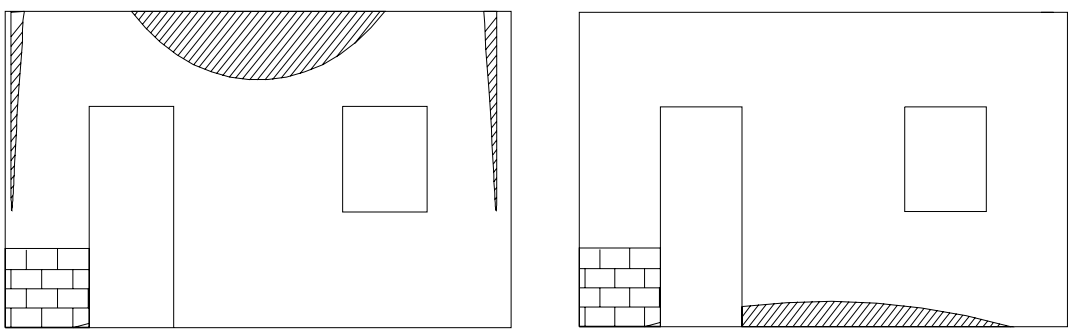
Earthquake	Details
EQ-1	Kangra earthquake, Himachal Pradesh, India; date: 26th April 1986; 3.05 IST; total duration: 20.08s; PGA: 0.248g at 3.04s; median frequency: 5.86Hz
EQ-2	Koyna earthquake, Maharashtra, India; date: 10th December 1967, longitudinal component; total duration: 10.33s; PGA: 0.613g at 3.85s; median frequency: 11.86Hz
EQ-3	Koyna earthquake, Maharashtra, India; date: 10th December 1967, transverse component; total duration: 10.33s; PGA: 0.473g at 3.13s; median frequency: 12.43Hz

Table 4 gives the values of the maximum flexural and shear stresses in masonry. The zones of maximum stresses are shown in Figures 5-7. Table 1 also gives the flexural strength and shear strength of the masonry. It is interesting to compare these strengths with the maximum stresses developed. In the case of buildings without roof and without any seismic bands, the flexural stresses developed in the direction parallel-to-bed-joints (at the centre and ends of top edge of the cross-walls) are exceeding the strength in that direction. This could lead to the development of vertical cracks propagating from the centre of the top edge and at the junctions of the walls. The development of cracks at the junctions of walls could also lead to separation of walls at corners. On the other hand, it can be noticed that the presence of horizontal seismic bands or RC roof brings down these stresses significantly. Again in the case of buildings without roof, the flexural stresses in the direction normal-to-bed-joints (at the base of the cross-walls) are exceeding the strength in that direction. These are responsible for the development of horizontal cracks. It is interesting to note that the presence of bands increases the stresses at the base. The values of these stresses are significantly high between the two openings for the buildings with RC roof as well. Indeed this is the vulnerable portion of the cross-wall. Plate 6 shows the collapse of portion of the wall between openings in a school building in Khavda (Gujarat) during the Kachchh earthquake of 2001. The maximum shear stresses developed in the shear-walls are also exceeding the shear strength of masonry reported by Sarangapani [9]. Expectedly, these values are rather high in the case of building with heavy RC roof. These stresses are responsible for the development of the typical ‘X’ type of cracks in the shear-walls of masonry buildings. It may be remarked that these buildings were analyzed with the ground motion input normal to the cross-walls and hence the flexural stresses are high in the cross-walls and shear stresses are high in the shear-walls. It is possible that these walls could experience a combination of both stresses. It may also be remarked that the separation of walls at the corners hastens the out-of-plane collapse that are noticed very commonly in a majority of low-rise masonry buildings. It is thus clear that there is a need to impart ductility by way of providing reinforcement along both directions in masonry to prevent such catastrophic failure of walls.

Table 4: Results of stress analysis

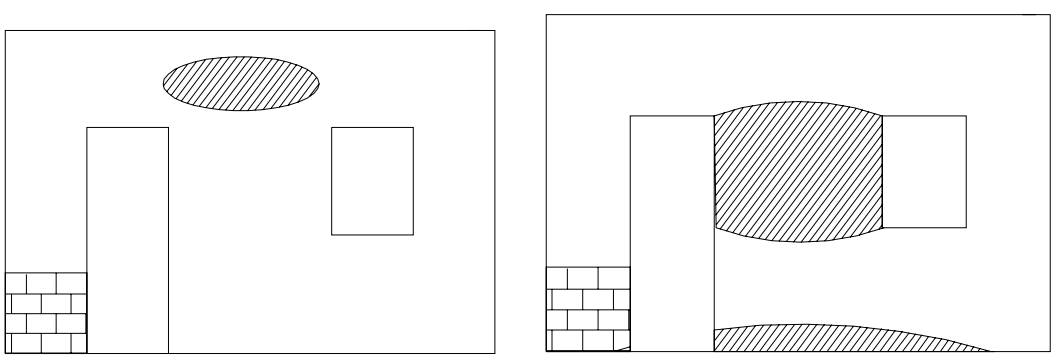
Building type*	Maximum flexural stress (MPa) σ_x at top edge of cross-wall (parallel-to-bed-joints)			Maximum flexural stress (MPa) σ_y at base of cross-wall (normal-to-bed-joints)			Maximum shear stress (MPa) τ_{yz} at the base of shear-wall		
	EQ-1	EQ-2	EQ-3	EQ-1	EQ-2	EQ-3	EQ-1	EQ-2	EQ-3
B-1	0.42	0.368	0.302	0.113	0.12	0.092	0.09	0.09	0.078
B-2	0.14	0.163	0.158	0.156	0.192	0.18	0.095	0.132	0.121
B-3	0.032	0.062	0.055	0.12	0.242	0.186	0.14	0.208	0.172

*** B-1, B-2 : Buildings without roof; B-3: Building with roof**



(a) **(b)**

Figure 5: Regions of maximum flexural stress for buildings without roof (a) σ_x (b) σ_y



(a) **(b)**

Figure 6: Regions of maximum flexural stress for buildings with roof (a) σ_x (b) σ_y

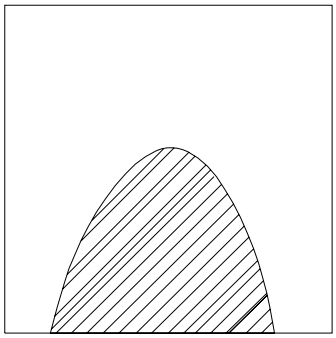


Figure 7: Regions of maximum shear stress in shear-walls (τ)

Plate 6: Collapse of walls between openings (Khavda)

CONCEPT OF CONTAINMENT REINFORCEMENT FOR MASONRY

Based on the field investigations of buildings after an earthquake and dynamic analysis of a typical masonry building subjected to ground motion, it can be concluded that existing provisions of BIS [2] for earthquake resistant design are inadequate to prevent collapse of masonry buildings due to out-of-plane failure of walls. Two important aspects of any earthquake resistant feature are ductility and integral connectivity of the various structural components of the building. It is rather easy to derive both these in the case of reinforced concrete framed structures than in load bearing masonry buildings. Unreinforced brick masonry, generally has poor flexural strength and practically no ductility in flexure. The masonry wall behaves like a plate in two-dimensional bending, when subjected to lateral load during earthquakes. Hence there is a need to reinforce the wall in both horizontal and vertical directions to impart ductility to the wall. It is rather easy to introduce reinforcement and embed it in a thin layer of concrete in the horizontal direction through the bed joints of the masonry, which results in horizontal bands at various

levels. These R.C. bands also integrally connect the various walls of the building together. The provision of vertical reinforcement poses certain difficulties. The conventional approach is to provide reinforcement in the middle of the wall enclosed by concrete. This will be inefficient since half of the wall thickness will be ineffective during the bending of the wall and the ductility of the wall will be limited to compressive strain capacity of the masonry. Further, the interface between the concrete enclosing the vertical steel and the masonry will create a vertical joint which is contrary to the concept of masonry construction. Such an interface between two materials having significantly different elastic properties will facilitate formation of vertical cracks. In order to overcome the above difficulties a new and an innovative way of providing vertical reinforcement on the surface of masonry wall has been developed which is called as **"containment reinforcement"**. In this technique the steel rod is wrapped around the wall in the vertical direction with reinforcement anchored at the top and bottom to the roof and plinth R.C. bands. Further the rods on either faces of the wall are held together at intermittent levels by steel ties/links passing through the bed joints of the masonry. The links are necessary to prevent the buckling of the steel rod present on the compression side of the wall. For the containment reinforcement to be effective, it is essential for it to remain hugged to the wall at all times during an earthquake. Such vertical reinforcement has to be provided at intervals (≈1m) along the length of the wall, adjacent to door and window openings and at corners of the building. Figure 8 shows a schematic view of a masonry building with horizontal bands and containment reinforcement. A similar concept of wrapping the walls with flexible nylon straps both in the vertical and horizontal directions was explored by Ginell et al [10] for retrofitting adobe brick buildings to withstand earthquakes. However, nylon straps are unlikely to be effective since they have low modulus of elasticity (520MPa) compared to steel. McKay [11] suggests a similar way as containment reinforcement for reinforcing masonry retaining walls to assist in resisting lateral pressure

SHOCK TABLE STUDIES ON MASONRY BUILDING MODELS

To understand the complex behaviour of masonry buildings during ground motion, sophisticated facilities like shake table with instrumentation are needed. However, such tests on full-scale prototype are prohibitively expensive. In this context studies on small-scale models become indispensable. Further, in this study a simple version of a shake table called shock table has been adopted to examine the efficacy of masonry building models with containment reinforcement. This test involved imparting base shocks to the table, over which the building model is mounted, using a pendulum impact device. Details of the shock table and various building models investigated with results have been presented elsewhere (Jagadish et al [12]). The building model with only horizontal bands withstood a total energy of 670.71 Nm before it became non-functional, whereas the building model with horizontal bands and containment reinforcement was capable of withstanding a total energy of 1966.7 Nm without collapsing. Plate 7 shows the building model with horizontal bands and containment reinforcement after 60 base shocks amounting to energy of 1966.7 Nm.

CONCLUDING REMARKS

Based on post earthquake field study of masonry building behaviour and finite element analysis of typical masonry building subjected to earthquake ground motions, it is clear that out-of plane flexural failure of walls is primarily responsible for collapse of masonry buildings during an earthquake. It may also be concluded that existing provisions for earthquake resistant design are not enough to prevent collapse of buildings and additional features to improve the ductility of the masonry wall in the vertical direction are needed. This paper reports a new and an innovative way of reinforcing masonry walls on the external faces in the vertical direction called as "containment reinforcement". Its efficacy has been confirmed through laboratory studies conducted on scaled down masonry building models.

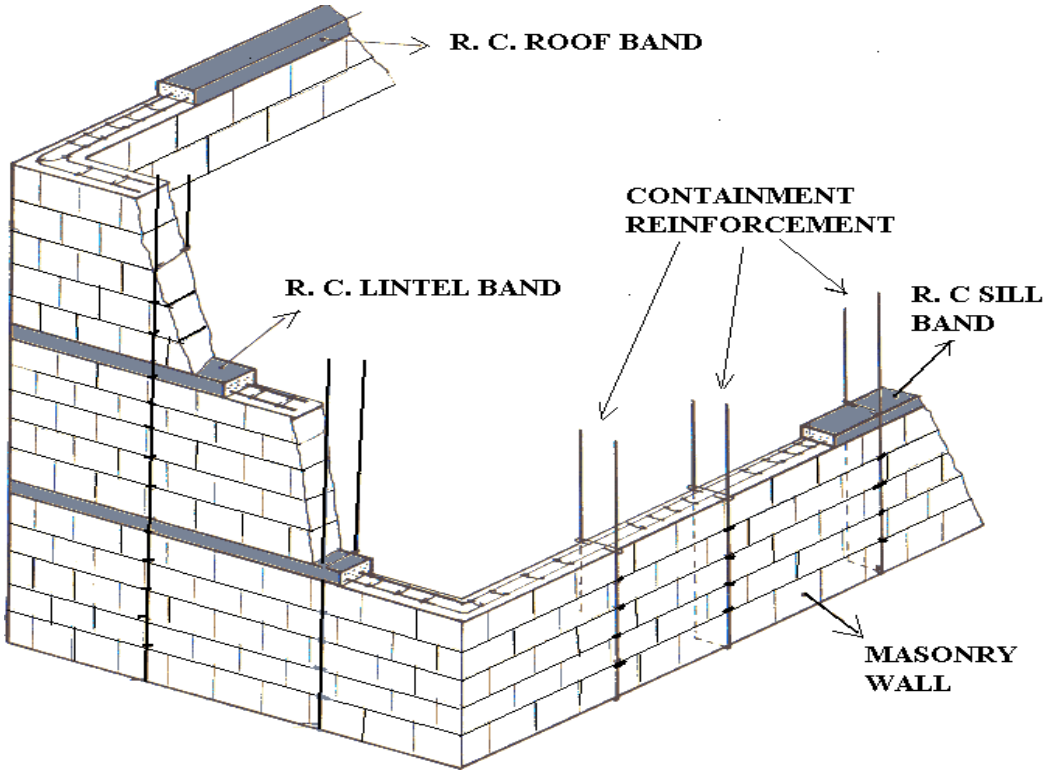


Figure 8: Masonry building with horizontal R.C. bands and containment reinforcement

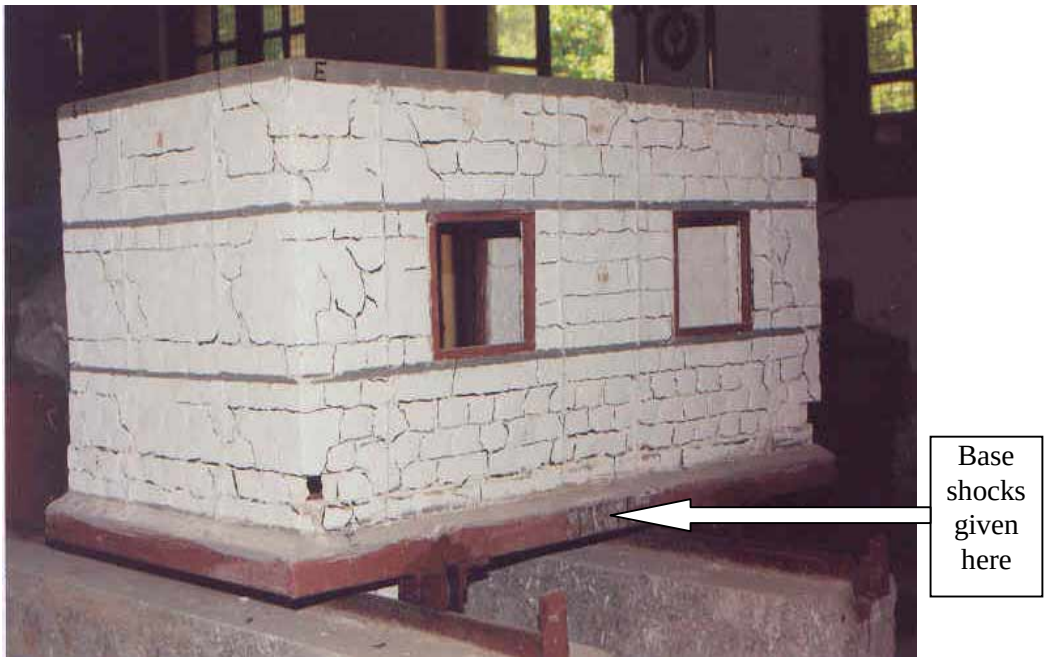


Plate 7: Building model with horizontal bands and containment reinforcement after 60 base shocks

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