EVALUATION OF SEISMIC PERFORMANCE OF CONTAINMENT REINFORCED MASONRY BUILDINGS THROUGH NON-LINEAR FINITE ELEMENT ANALYSIS

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Abstract

The unreinforced masonry (URM) buildings have shown very poor performance during earthquakes all across the globe, owing to their large mass, high initial stiffness, low tensile strength, brittle material behavior, flexible roof diaphragms, and irregularities in plan. Additionally, openings in the walls reduce their strength and stiffness and make them amenable to out-of-plane rocking behavior leading to life threatening collapse. However, masonry is still one of the most popular construction materials, owing to simplicity of construction and low cost. A novel form of near surface mounted (NSM) reinforcement termed as containment reinforcement has been demonstrated to be effective in mitigating seismic risk of masonry buildings, through detailed experimentation performed on half scale, one storey box type masonry building model at Indian Institute of Science, Bangalore. Moreover, provision of containment reinforcement even in very small quantity has been shown to be effective in mitigating seismic risk under out-of-plane rocking behavior of the masonry building. Corner containment reinforcement has been shown to ensure integral behavior of walls of masonry building. Additionally, containment reinforcement has been shown to improve in-plane shear strength and ductility of masonry. Considering limitations on payload capacity, experimental evaluation of seismic performance of multi-storey masonry buildings on shake table becomes difficult task and often a compromise is made on scale of the test structure. The present study explores the role of containment reinforcement and reinforced concrete (RC) bands at lintel and sill level in improving seismic performance of the symmetric and asymmetric two storey masonry buildings, through a non-linear finite element analysis. Masonry buildings have been modeled using the commercial finite element software, Abaqus (Simulia, 2011). The finite element model of containment reinforced/unreinforced masonry has been validated using the experimentally obtained response and failure patterns of containment reinforced and unreinforced masonry assemblages/ building models to static and/or dynamic loads. The present work concludes the following, (a) provision of containment reinforcement reduces the spread and magnitude of maximum principal plastic strain (that correlates to cracking damage), (b) the containment reinforcement provided even in a very small quantity is helpful to maintain integrity of the two-storey masonry buildings under severe seismicity, and (c) provision of containment reinforcement in asymmetric and symmetric two storey masonry buildings successfully restrains inter storey drift ratio well below that corresponding to immediate occupancy performance level (in moderate to severe seismic event) defined by Federal Emergency Management Agency guidelines (FEMA 273, 1996).

Keywords: containment reinforced masonry, masonry modelling, seismic performance, inter storey drift

1. Introduction

The seismic vulnerability of the URM buildings all across the globe can be attributed to their large mass, high initial stiffness, low tensile strength, brittle material behavior, flexible roof diaphragms, and irregularities in plan. Moreover, openings in the masonry walls reduce their strength and stiffness and make them amenable to out-of-plane rocking behavior leading to life threatening collapse. In spite of poor seismic performance, masonry is still one of the most popular construction materials owing to simplicity of construction and low cost. In the developing counties like India, two to three storey URM buildings are most common forms of dwellings. Different types of plan irregularities and vertical irregularities as described in IS 1893, part-I [1], owing to asymmetric distribution of stiffness or mass, significantly affect seismic performance of the masonry buildings.

To improve seismic performance of masonry buildings, the vertical reinforcing steel bars are provided at the mid-thickness of the masonry walls (termed as core reinforcement). Such arrangement of reinforcement in masonry renders half of the masonry wall thickness ineffective in resisting bending moments due to seismic actions [2, 3]. Furthermore, due to reversal of stresses under seismic action, tensile cracks initiate on both faces of the masonry walls. The provision of reinforcement at core of the masonry wall results in development of few flexural cracks that grow rapidly and reach core reinforcement, rendering insufficient ductility [4, 5]. Jagadish et al. suggested an improved way of reinforcing masonry walls in vertical direction, termed as 'containment reinforcement' [5, 6]. The containment reinforcement was suggested to be provided in one of the following two ways [2],

a) The vertical reinforcement can be provided on the surface of masonry wall and held in the position by horizontal ties at every/alternate bed joints. Horizontal ties will ensure integral behavior of masonry and containment reinforcement. However, exposed containment reinforcement needs protection against corrosion. (Fig.1)
b) Grooved masonry unit can be laid in such a way that a continuous vertical groove is created to accommodate the vertical reinforcement. The vertical groove can later be grouted (Fig.2).

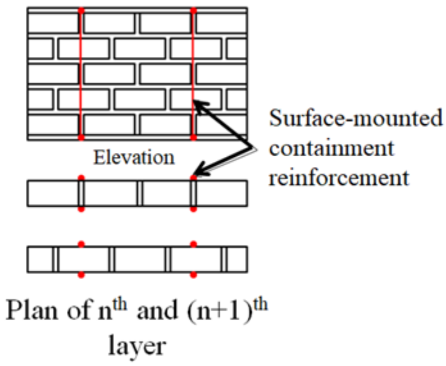

Fig.1 Surface mounted reinforcement

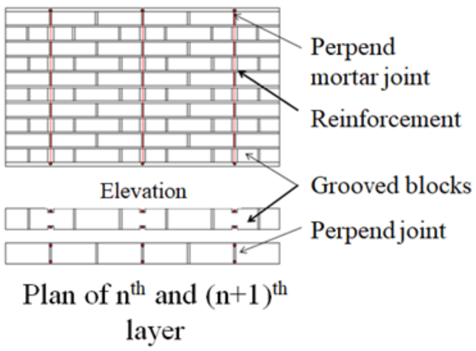

Fig.2 Near surface mounted reinforcement

Enhanced performance of masonry reinforced with near surface mounted reinforcement as shown in Fig.2 is evident from the work of Rao and Joshi [7, 8]. Additionally, brief summary of experimental investigations to evaluate seismic performance of one storey masonry half scale model with near surface mounted reinforcement termed as containment reinforcement is presented by Rao [9] and Joshi [10]. Due to limitations of payload capacity, testing of multi-storey masonry buildings on shake table becomes difficult and a compromise is made on scale of the test structure. The present work thus explores seismic performance of two-storey symmetric and asymmetric containment reinforced masonry buildings through non-linear finite element analysis (FEA).

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2. Scope and objectives

In the present work, role of containment reinforcement and RC bands at sill and lintel level in improving seismic performance of two-storey symmetric and asymmetric (in plan) masonry buildings is evaluated through non-linear FEA. Various material modeling parameters for reinforced masonry are calibrated using experimental tests performed on unreinforced/reinforced masonry assemblages at Indian Institute of Science, Bangalore, India [10]. Reinforced masonry elements for above mentioned tests were constructed by employing different bonding arrangements of stabilized earth blocks (SEB) and cement-soil-sand (1:1:6) mortar (viz. stretcher bond, English bond). The finite element model of containment reinforced/unreinforced masonry is validated using the experimentally obtained response and failure patterns of containment reinforced and unreinforced masonry assemblages/ buildings [10]. Following the model validation, the non-linear FE model is extended to evaluate seismic performance of two-storey symmetric and asymmetric containment reinforced masonry buildings. The objectives of the present work can be stated as follows

(a) Exploring role of near surface mounted containment reinforcement in moderating storey drift ratios of two-storey symmetric and asymmetric masonry buildings subjected to seismic events.

(b) Exploring role of near surface mounted containment reinforcement in enhancing seismic performance of two-storey symmetric and asymmetric masonry buildings.

3. FE model description

Masonry is a heterogeneous, composite material. The mechanical behavior of the masonry is governed by the behavior of the masonry unit, masonry mortar and interface. In the present work masonry is modelled using macro modelling as described by Lourenco [11]. The containment reinforcement is modeled as smeared layer in masonry. Following subsections describe the FE modelling of containment reinforced masonry buildings.

3.1 Discretization

Masonry building is modeled using the commercial finite element software, Abacus [12]. Discretization of the masonry was carried out using 4-noded general purpose shell element, S4R (with reduced integration and hourglass control). The formulation of this element is based on Mindlin-Reissner theory. Neither does this element suffer from transverse shear locking nor does it have any unconstrained hourglass mode. Reinforcement is modeled using rebar layer option of Abacus [12]. The rebar layer represents the smeared reinforcement layer with a constant thickness, t. The value of thickness is calculated as area of reinforcing bar (A) divided by the spacing (s).

3.2 Material modelling and calibration

Abaqus [12] material library provides following material models for quasi-brittle materials: (a) concrete smeared cracking, (b) cracking model for concrete and (c) concrete damaged plasticity (CDP). Concrete smeared cracking model is suitable for monotonic loading at slow strain rates. Cracking model for concrete assumes linear elastic behavior under compression. CDP outperforms the other two material models as it has capacity to model nonlinear-plastic compressive behavior and it can be used to model non-linear response under cyclic loading protocols. CDP combines isotropic damaged elasticity and multi-hardening plasticity with non-associated flow rule to represent irreversible damage that occurs during the process of fracturing [13]. Owing to its capacity in representing compressive behavior of masonry, very close to reality, CDP is used to model inelastic behavior of the masonry.

Fig.3 shows the definition sketch for uni-axial compressive behavior and uni-axial tensile behavior of quasi brittle material modeled with CDP. Details of loading and unloading paths in this figure are presented in Table 1. In Fig.3, σ_{t0} and σ_{c0} are uni-axial tensile strength and uni-axial compressive strength of quasi brittle material respectively, w_c and w_t are compression and tension recovery factors respectively.

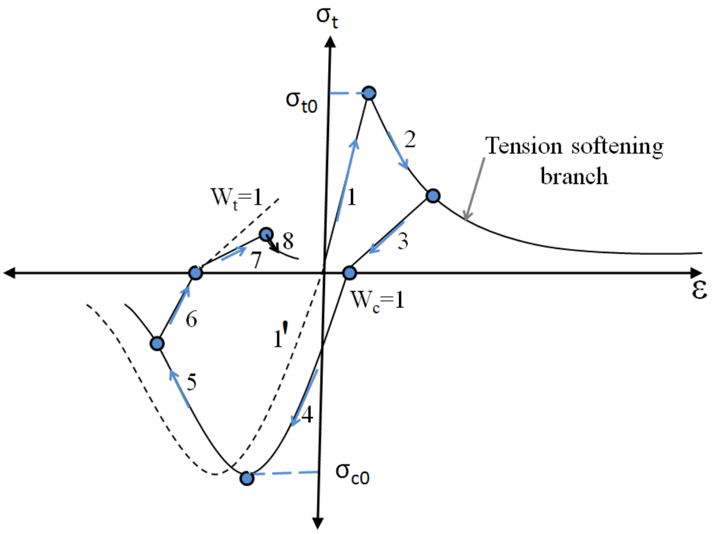

Fig.3 Description of loading and unloading paths of CDP model

Table 1 – Details of loading and unloading of CDP with reference to Fig.3

Line	Slope	Remark
1'	E0	Compression
1	E0	Tension loading
2	-	Tension loading
3	(1-Dt)E0	Tension unloading
4	E0	Compression loading
5	-	Compression loading
6	(1-Dc)E0	Compression unloading
7	(1-Dc)(1-Dt)E0	Tension loading
8	-	Tension loading

Note: Dt and Dc are scalar damage parameters under uni-axial tension and uni-axial compression, respectively.

The stress-strain behavior under uni-axial compression of stretcher bond and English bond masonry prisms has been explored at Indian Institute of Science, Bangalore, India [10]. These stress-strain curves are employed as constitutive models of masonry assemblages in compression. The tensile strength of about 10% of compressive strength of masonry is adopted for analysis of masonry elements/buildings. In order to avoid mesh-sensitive results due to the small amount of reinforcement in the structure, the tensile post peak behavior is adopted as fracture energy cracking criterion by specifying stress-displacement curve instead of a stress-strain curve. Table 2 presents the summary of details of the CDP model of masonry. The compression recovery wc = 0.7 and tension recovery wt = 0 is adopted. Other required material properties for CDP, were assumed to be defaults of Abaqus [12] (dilation angle = 30°, flow potential eccentricity = 0.1, ratio of initial equi-biaxial compressive yield stress to initial uni-axial compressive yield stress = 1.16, ratio of second stress invariant = 0.667 and viscosity parameter = 1×10^{-5}).

Containment reinforcement is modeled as elastoplastic material of Abaqus [12] material library. Bauschinger effect is neglected and behavior of steel in uni-axial compression is assumed to be the same as its behavior in tension.

Table 2 – CDP model parameters [10]

Material	Modulus of elasticity (MPa)	Compressive strength (MPa)	Tensile strength (MPa)	Post peak tensile behavior (stress-displacement equation)
Stretcher bond masonry loaded perpendicular to the bed joints	4650	6.95	0.1	$\sigma = (0.1)\ .\ e^{-(2000\ X\ y)}$; $0.00085<y<0$ (σ : stress in MPa, y: displacement in m)
Stretcher bond masonry loaded parallel to the bed joints	11920	7.49	0.7	$\sigma = (0.7)\ .e^{-(2000\ X\ y)}$; $0.00085<y<0$
English bond masonry	5490	8.16	0.1	$\sigma = (0.1).e^{-(2000\ X\ y)}$; $0.00085<y<0$

4. FE model validation

The proposed FE model is validated by comparison of numerical and experimental responses obtained in the following type of tests

(a) Containment reinforced masonry beams tested under monotonic four point bending: experimental load-deformation curves are presented in [8, 10]

(b) Containment reinforced diagonal shear specimens tested under diagonal shear test: experimental shear behavior is presented in [7, 10]

(c) One storey half scaled containment reinforced masonry building model tested under base motions on shock table: experimental results presented in [10]

Due to space constraints, details of test specimens, experimentation and results of (a), (b) and (c) are not presented in this paper. However, these details are elaborately described and presented in [7, 8, 10]

Comparison of numerical and experimental load-deformation plots obtained under test (a) are presented in Fig.4 and Fig.5. Similarly, comparison numerical and experimental behavior of containment reinforced masonry under shear is presented in Fig.6. Fig.7 presents FE discretization of one storey half scale containment reinforced masonry building model used in test (c). The comparison of numerically predicted damage zone and experimentally observed crack patterns obtained after shock table tests [10] is presented in Fig.8.

The comparison of experimental and numerical results under various loading scenarios indicate that the proposed FE model of reinforced masonry, not only predicts force deformation relations in close conformity with experiments, but also simulates experimentally observed failure patterns of building models.

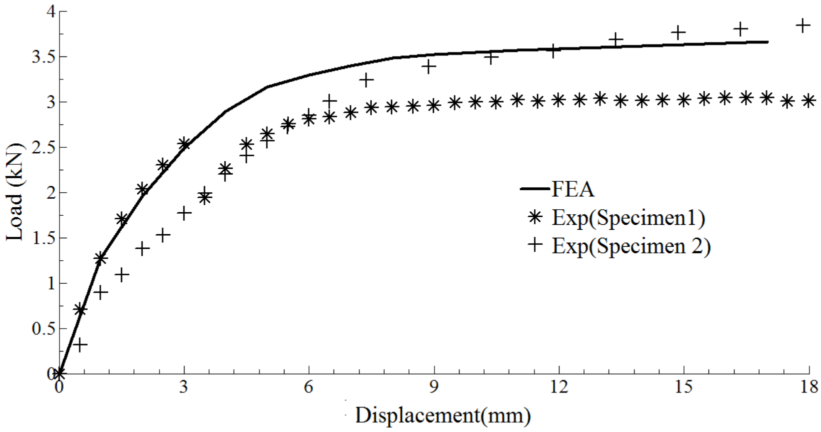

Fig.4 Comparison of numerical and experimental behavior of containment reinforced Stretcher bond beams under monotonic four point bending [10]

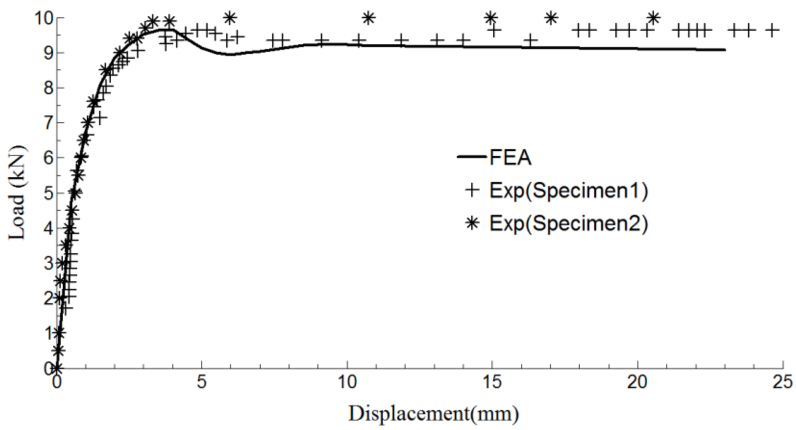

Fig.5 Comparison of numerical and experimental behavior of containment reinforced English bond beams under monotonic four point bending [10]

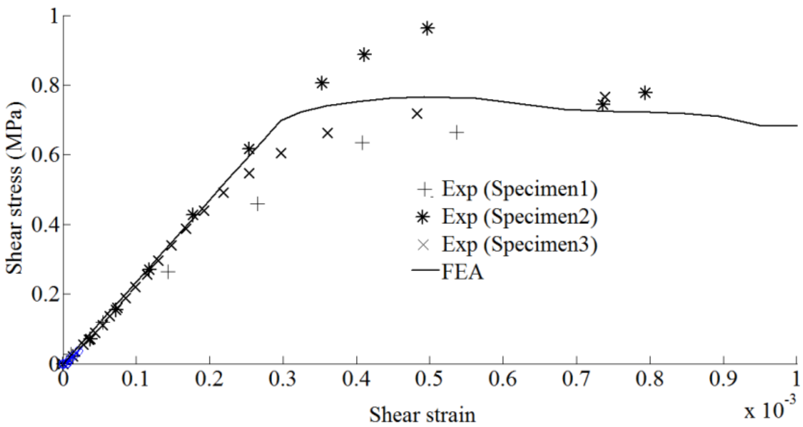

Fig.6 Comparison of numerical and experimental behavior of containment reinforced masonry in shear [10]

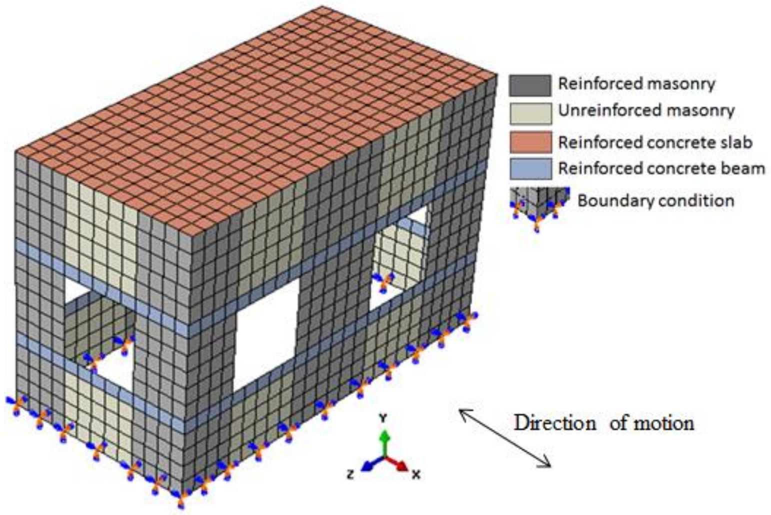

Fig.7 FE model of one storey half scaled containment reinforced masonry building model [10]

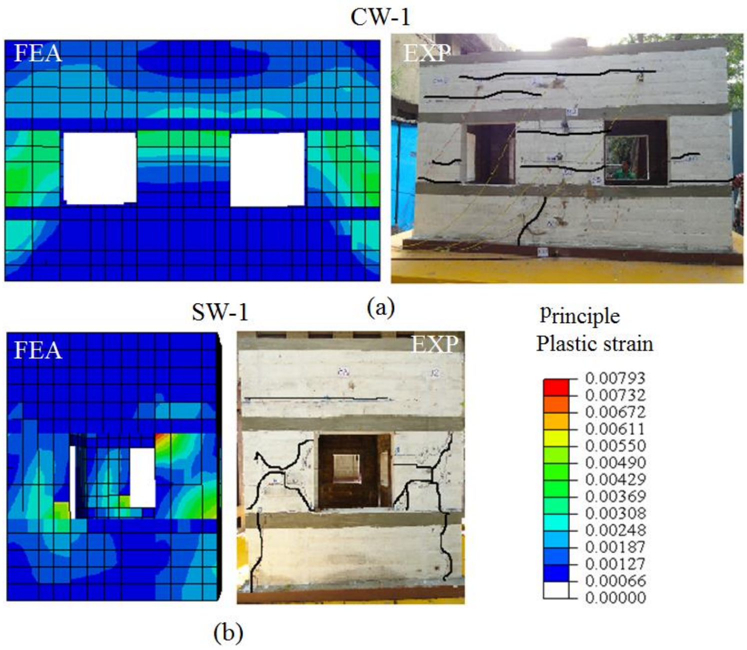

Fig.8 Comparison of numerical and experimental damage pattern of containment reinforced masonry building subjected to shock table motions [10]

5. Assessment of seismic performance of two-storey masonry buildings through FEA

The proposed non-linear FE model is used to access seismic performance of masonry buildings listed below,

(a) Symmetric half scale unreinforced masonry building with sill and lintel bands (URLSB-S)

(b) Symmetric half scale containment reinforced masonry building with sill and lintel bands (RLSB-S)

(c) Asymmetric half scale unreinforced masonry building with sill and lintel bands (URLSB-AS)

(d) Asymmetric half scale containment reinforced masonry building with sill and lintel bands (RLSB-AS)

Geometric details of these models are presented in Fig.9. Each face of the masonry walls of these buildings is reinforced with 0.03 % (of gross wall area) steel in the form of near surface mounted containment reinforcement. Sill band and lintel bands are reinforced concrete layers running around the masonry building provided at bottom and top levels of openings respectively.

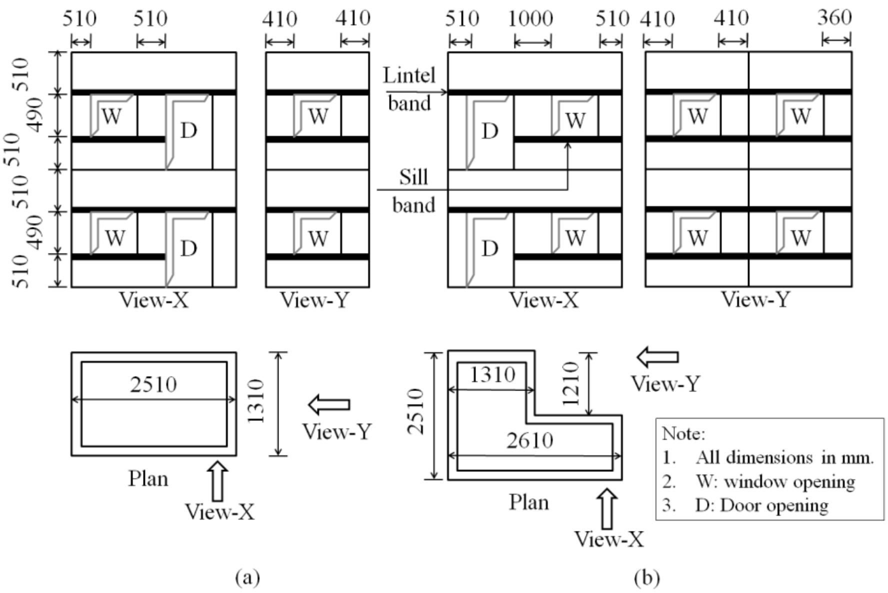

Fig.9 Geometric details of a) symmetric and b) asymmetric masonry building models

The response of these masonry buildings is simulated under scaled spectral compatible time histories. The Chamoli earthquake (India), March 1999 (magnitude 6.8 on Richter scale) is chosen as base acceleration record. This earthquake record showed peak ground acceleration, peak velocity, strong motion duration, Arias intensity, Housner's intensity and acceleration spectral intensity of 3.66 m/s^2, 0.42 m/s, 8.98 s, 0.8 m/s, 1.33 m, 2.60 m/s respectively [10]. The spectrum compatible time histories are obtained from base acceleration record, using program SeismoMatch [14], for following spectra; (a) Zone-II: maximum considered earthquake as per IS 1893 (Part-1): 2002 [1], (b) Zone-III: maximum considered earthquake as per IS 1893 (Part-1): 2002 [1], (c) Zone-IV: maximum considered earthquake as per IS 1893 (Part-1): 2002 [1], (d) Zone-V: maximum considered earthquake as per IS 1893 (Part-1): 2002 [1]. The spectrum compatible time histories are scaled by employing the scale factors for acceleration and time to satisfy similitude requirements as described in section 7.6 of [10]. These acceleration time histories are presented in Fig.10.

6. Results and discussion

Table 3 presents the inter-storey drifts exhibited by symmetric and asymmetric masonry buildings under scaled spectral compatible time histories of increasing severity. These drifts can be readily compared with the values presented in FEMA 273 [15] for immediate occupancy, life safety and collapse prevention. Fig. 11 and Fig. 12 present contours of principal plastic strain in the walls of symmetric and asymmetric masonry buildings for zone-II and Zone-V seismicity. Following observations can be made based on Table 3, Fig. 11 and Fig. 12,

(a) Provision of containment reinforcement drastically reduced the inter storey drift.

(b) Percent inter storey drift ratios of symmetric masonry buildings were well below the drift ratio for immediate occupancy FEMA 273 [15], under Zone II, Zone III and Zone IV compatible time histories.

(c) Asymmetric unreinforced masonry buildings did exhibit first story drifts crossing life safety and collapse prevention in Zone-IV and Zone-V compatible time histories.

(d) Containment reinforced asymmetric buildings were observed to be safeguarded against collapse even at Zone-V compatible time histories.

(e) Introduction of plan irregularity makes the masonry building vulnerable to damage even under low seismic intensity. Unlike symmetric masonry buildings, asymmetric masonry buildings show large spread of damage even for Zone-II compatible time histories.

(f) The damage can be seen to be more pronounced at re-entrant and other corners of bottom storey.

(g) The out-of-plane crack initiation of cross walls and shear sliding shear of shear wall initiated at lower seismicity for asymmetric buildings than that for symmetric buildings. Hence, for asymmetric buildings the containment reinforcement needs to be provided, even if the building is situated in area with low seismicity.

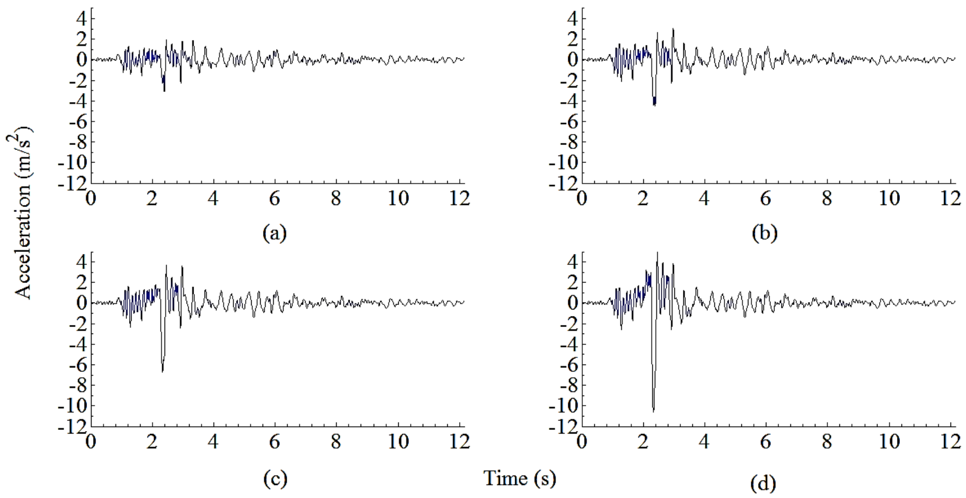

Fig.10 Scaled spectrum compatible time histories for (a) Zone-II (b) Zone-III (c) Zone-IV and (d) Zone-V spectra as per IS 1893 (Part-I) : 2002

Table 3 – Inter-storey drifts under spectral compatible time histories

Type of masonry building	Storey	Percent inter storey drift ratio under scaled spectrum compatible base motion (%)				Limiting percent drift ratios by FEMA 356 (2000)	
		Zone II	Zone III	Zone IV	Zone V	Performance level	Limiting percent drift (%)
URLSB-S	First storey	0.01	0.02	0.04	0.21	Immediate occupancy	Unreinforced: 0.3 Reinforced: 0.2
	Second storey	0.01	0.02	0.03	0.21		
RLSB-S	First storey	0.01	0.02	0.03	0.08		
	Second storey	0.01	0.02	0.03	0.07	Life safety	Unreinforced: 0.6 Reinforced: 0.6
URLSB-AS	First storey	0.07	0.37	0.85	1.17		
	Second storey	0.01	0.04	0.13	0.28		
RLSB-AS	First storey	0.06	0.29	0.68	0.77	Collapse prevention	Unreinforced: 1 Reinforced: 1.5
	Second storey	0.01	0.03	0.15	0.19		

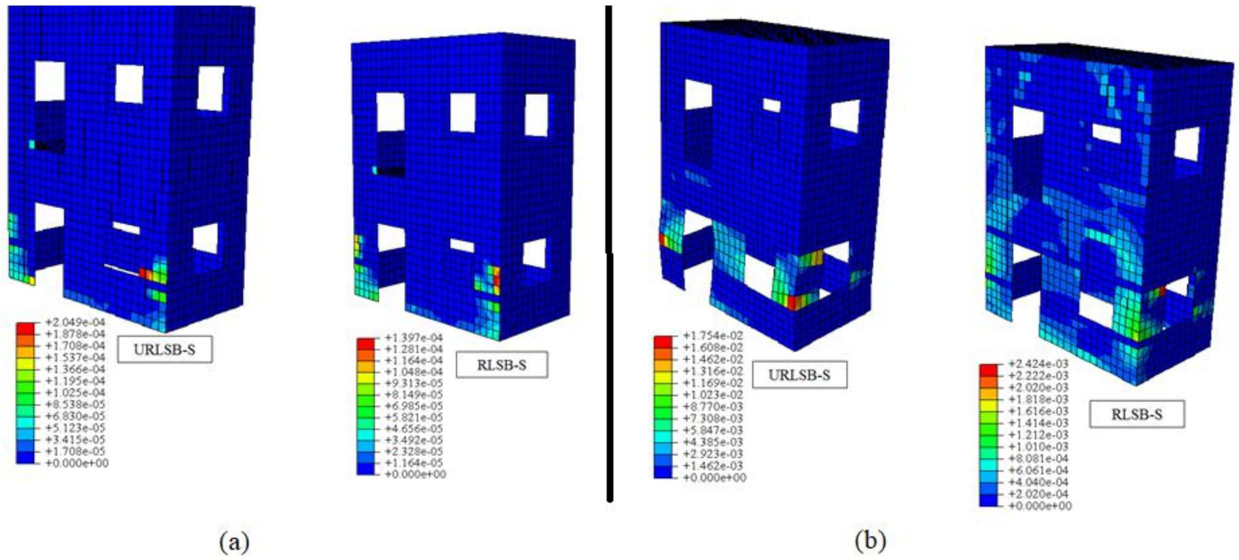

Fig.11 Contours of principal plastic strain distribution in symmetric masonry building subjected to base motion corresponding to (a) Zone-II and (b) Zone-V compatible time history

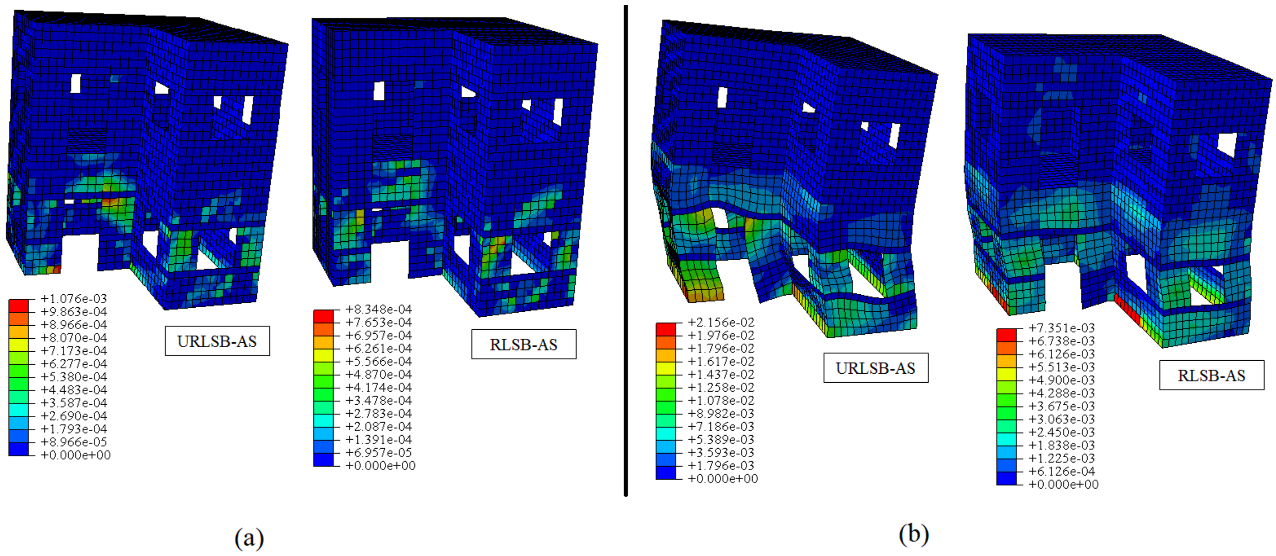

Fig.12 Contours of principal plastic strain distribution in asymmetric masonry building subjected to base motion corresponding to (a) Zone-II and (b) Zone-V compatible time history

7. Conclusions

Following conclusions are evident from the present work,

(a) Modestly reinforcing masonry buildings with near surface mounted containment reinforcement ensures collapse prevention even in severe seismic event.

(b) Provision of containment reinforcement reduces the spread and magnitude of principal plastic strain (that correlates to cracking damage).

(c) Reinforced concrete sill band and lintel band without containment reinforcement successfully mitigates seismic risk of two to three storey symmetric masonry buildings in India.

(d) Reinforced concrete sill band and lintel band without containment reinforcement successfully mitigates seismic risk of two to three storey asymmetric masonry buildings situated in Zone-II and Zone-III as per seismic zoning map of India.

(e) Near surface mounted containment reinforcement in addition to reinforced concrete sill band and lintel band is required to successfully mitigate seismic risk of two to three storey asymmetric masonry buildings situated in Zone-IV and Zone-V as per seismic zoning map of India.

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