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BEHAVIOR OF UNREINFORCED AND REINFORCED BRICK INFILL WALL DURING EARTHQUAKES IN NORTH AND NORTH EAST INDIA USING APPLIED ELEMENT METHOD

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ABSTRACT

In the current year 2020-21, most part of India including North and NE India is recorded with around 100 earthquakes ranging from Magnitude 4 to 6. NE India is the sixth most seismically active zone in the world and falls in Zone 5 in Seismic zonation map of India. People's life and the property are at stake when an earthquake with high magnitude strikes such a region. Infrastructure plays an essential role in the safety of people residing in these areas. In present time new constructions in this region are mostly, RC Structures with Brick infills. Thus, it is necessary to evaluate Brick infill Seismic Performance in RC Structure and eventually devise countermeasures to reduce the causalities during earthquakes. In this study Numerical Analysis is performed, to understand the behavior of Brick infill in RC Frame before and after strengthing during the earthquake. As a strengthening measure, the response of the wall using Horizontal and Vertical Reinforcement is studied. It is found that a reinforced brick infill wall is capable of giving good strength during an earthquake. This numerical study is performed using Applied Element Method, which helps to simulate Brick infill behavior from initial stage of loading till collapse stage and it does not require presumption of crack location, and it is from discrete elements family. Unreinforced masonry wall shows a maximum response to the seismic loading with maximum visible cracks on the wall. The response of the wall gets reduced with provision of reinforcement in the horizontal and vertical direction.

KEYWORDS: Brick Infill, RC Structure, Applied Element Method, Seismic Performance, Horizontal and Vertical Reinforcement, Strengthening Measures, Cracks, Response

INTRODUCTION

Past earthquakes witness severe damage to life and property. Scientist and Researchers in Asian countries are extensively working on this area [1]. As per census 2011, India is having around 649, 481 villages. Past earthquakes in India like Koyna (1967), Kilari (1963), Jabalpur (1997), Bhuj (2001) and the recent earthquake in neighboring country Nepal on 25th April 2015 witness massive loss of the property and life due to damage and collapse of brick masonry infill buildings.

From past earthquakes, Brick masonry failure can be categorized in four main types:

i) Vertical cracks at wall junctions or failure at connection,
ii) Shear cracks or 45^0 bi-directional cracks in wall panels,
iii) Cracks due to in Plane Movement,
iv) Out of Plane (flexure) Failure of Wall leading to collapse.

1. Past Research on Understanding the Behavior of RC Brick Infill Buildings

Analytical and experimental studies were carried out in the past. Past research work focusing on analytical methods is given below.

1.1 Analytical Studies: Modeling Methods

Masonry buildings were analyzed using the macro, micro and simplified micro modelling [2]. In detailed micro modelling, to avoid continuum, Interface between brick and mortar was represented as a crack or slip plane which has initial dummy stiffness [2]. Detailed micro modelling though is capable of showing structural behavior in detail; its cost of computation is more [3]. In simplified micro modelling, no interface was considered as in the previous study, but their properties are considered as combined and fracture planes where represented at joints [2, 4]. In Macro modelling approach, homogenization techniques considered and elements for masonry are considered in a uniform pattern. Smeared cracking and orthotropic material considered using two-stage homogenization technique [5]. The failure mechanism of masonry is very well characterized in terms of cracking in joints, cracking in direct tension, cracking in diagonal tensile units and also splitting into units [2].

1.2 Modelling of Masonry Structures Using Static and Dynamic Loading

Macro model: Macro model approach uses a simplified model of frame and panel to get overall behavior based on real understanding. In past, research work in analytical studies were done since 1961 till 1971 [6-9], In past studies, Panel was also represented by many structs [10].

Effect of the single strut and multi strut versus four-node panel elements was studied by Crisafulli [11, 12]. It was understood that If the single strut is considered for the analysis, it was not capable of resisting internal forces in frames which are tensile and compressive. However, when a 4-node panel was considered then element allows strength and stiffness to masonry panel. For dynamic analysis and static nonlinear analysis, strength and stiffness degradation concerning opening and closing of masonry gaps was studied [13]. Equivalent braced frames with infill walls were studied [14]. Study on reduction of the width of the diagonal strut was conducted. A reduction factor for the diagonal strut to account for the central window opening in the infill reinforced the concrete frame [15].

Micro model: Modeling of units as Elastic continuum elements were done by Page [16], initially units were modelled as by considering them in elastic continuum and interface considered as a bond between them. Based on the experimental study, the elastic interface was developed. In the yield, the surface contains two compressions and one tension branch. Compression with a tension cut-off was represented by strain-softening model [17].

A tension cut-off and compression were considered under Von Mises strain-softening model. Friction and cohesion for tension cutoff and softening were considered for joints for brittle behavior. Experimental results on shear walls were checked with the collapse load obtained from the model. Analytical work has been done to incorporate all types of failure related to brick masonry [2, 18].

The damage was concentrated in the comparatively weak joints. The joint interface yield surface considered including all the failure mechanisms except tensile cracking. Interface cap model was developed. The interface model comprises of a compression cap which captures inelastic behavior for masonry in compression. Around the same period, Gambarotta et al. have developed a continuum model in 1996 for brick masonry [19]. In-plane stress condition, the constitutive equations were developed. Brick masonry is considered as a stratified medium with two layers, i.e. the mortar head joints and brick unit's representative layer and bed mortar joint layer.

In a detailed study of brick joint action, Rots [20], performed a standard compressive test. It has been observed that horizontal compressive stress arises in the mortar and horizontal tensile stress arise in bricks. The later stresses govern ultimate failure under compression; also, it has been studied that the mortar joint, the peak stress in brick amounts to 6.5 times the average stress and is likely to initiate cracking of brick and delamination along with the brick / joint interface Rots [20]. Brick masonry panels subjected to lateral loading was showing brittle behavior under new homogenization technique in 1996 [21]. Jahangir considered composite material for brick masonry, which showed accurate stress distribution for the prism considered in study [22].

Applied element method for brick was first used for numerical study by Bishnu [23]. At all stages of the loading, capacity of the wall increases if the mortar is of high strength. A significant effect in wall behavior was observed due to lintel, especially on the crack pattern. Cracks can be avoided using the lintel band [23]. Paola studied Earthquake loading and effect of retrofitting, [3]. The behavior of the wall under monotonic lateral loading was studied by considering earthquake loading. Also, sustainability of masonry building was studied after retrofitting. In their work as an advantage of AEM, they could capture crack initiation, a gradual increase in cracks till the full collapse of the building. Around the same period, Guragain et al. [24], did the numerical simulation by AEM for brick masonry walls under lateral loads and especially cyclic loads. The material model considered was the damage model proposed by Gambarotta et al. [19] for cyclic loading case was used for further study in 2006 [24].

Cyclic response of calcium silicate masonry wall was studied recently [25], AEM models used in this work captured crack patterns and energy dissipation in experimental study.

Experimental studies:

In the past, many experimental studies were done to understand the behavior and collapse pattern and crack occurrences in the building and brick, and mortar joints during the earthquake were studied. Damage assessment of modern masonry buildings after the L'Aquila earthquake was studied recently and effectiveness of box like action which helped in avoiding global and local collapse however in plane shear capacity was reduced due to severe damage to piers and spandrels [26].

Masonry wall can be considered in two ways, depending on their functional use. First is the bare wall and second is Infill wall. Different types of loading conditions considered in past experimental studies, i.e. In-plane and out of plane loading, cyclic and dynamic loading, pseudo-dynamic loading, quasi-static loading [27].

In this numerical study, RC framed infill brick masonry wall is considered.

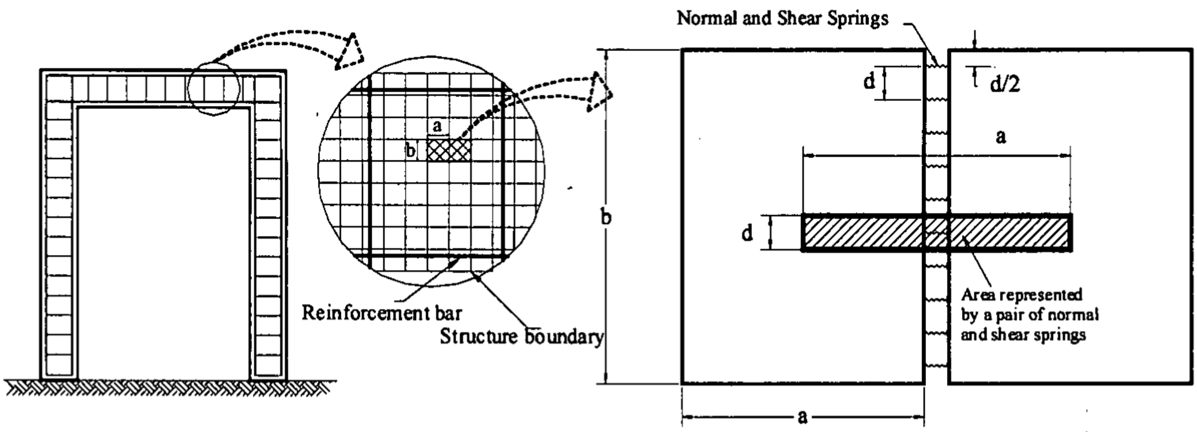
The response of wall under earthquake loading was studied using four cases. A wall with no reinforcement can be called as unreinforced masonry wall (URM) and reinforced masonry wall (RM). Provision of reinforcement in the form of vertical reinforcement, horizontal reinforcement and vertical and horizontal reinforcement together has been studied. Door and windows with frame also been considered. A parametric study by considering an RC framed infill brick masonry wall with one door and one window opening has been done. Details of geometry, boundary conditions and loading are as shown below.

Geometry: RC framed brick infill wall with a door opening of size 2.1 m x 1.05 m and window opening of size 1.05 m x 0.84 m.

2. Methodology

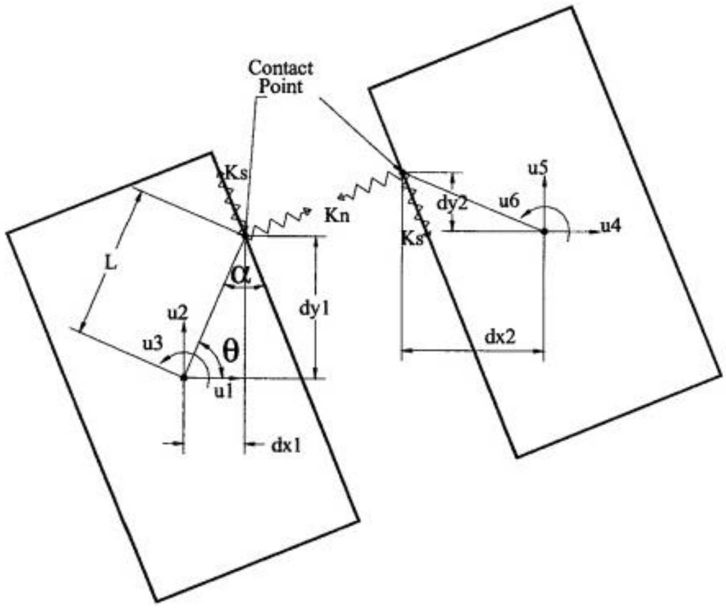
The numerical method used in the study - Applied Element Method (AEM):

The AEM was initially developed at the University of Tokyo in 1998 by Tagel-Din Hatem and Meguro (2000) to study the behaviour of 2D structures during an earthquake. Applied element method is a FORTRAN program. It is a simple program and gives results with high accuracy. Computational timings are less when compared to other methods. This method is capable of incorporating nonlinear behaviour perfectly. Later this method was also developed for large displacement for analyzing structures [28]. Bishnu Pandey initially developed work on brick masonry using AEM in 2004 [23]. However in India, for the first time, further study on brick infill using AEM was conducted in by author [32].



(a) Element generation for AEM (b) Spring distribution and area of influence of each pair of springs

Fig. 1 (a, b) Modelling of structure in AEM - Elements showing degrees of freedom and spring contacts, Hatem and Meguro [5]



(c) Element shape, contact location and degrees of freedom

Fig. 1c Elements showing degrees of freedom and spring contacts, Hatem and Meguro [5]

AEM is a stiffness-based method in which a Global stiffness matrix is formulated, and the equilibrium equations including each of stiffness, mass and damping matrices are nonlinearly solved for the structural deformations (displacements and rotations). Modelling of structure in AEM with element generation and spring distribution and area of influence of each pair of springs is shown in Figure 1a, Figure 1b. The Global Stiffness matrix is formulated considering the stiffness of all pairs of normal and shear springs located at the element contact points, i.e., distributed around the edges, as shown in Figure 1c. Those springs are responsible for the transfer of normal and shear stresses among adjacent elements. After calculation of the global matrix equation using three degrees of freedom, stress and strains are calculated at the edges of elements. For more details, kindly refer work by [5, 28, 29]. At every contact point pair of springs are used, i.e. normal and shear springs. K_s represents the stiffness of shear spring, and K_n represents the stiffness of normal springs. Normal and shear spring position at one point of contact is as shown in Figure 1c.

2.1 Discretization of Brick Masonry in Applied Element Method

In AEM, brick and mortar joint is considered in two phases. Normal and shear springs connecting small brick elements are represented as Unit Springs and Brick–mortar joint springs as Joint Springs Refer Figure 2 for more details.

In Figure 2, a brick is called as unit and it is divided in four elements, each element connected to all other elements with the help of springs. Joint spring is showing brick mortar joint. Each spring assumes the property of corresponding material it represents. The stiffness of unit springs and joint springs, which simulate forces and displacements in bricks and brick-mortar joint is calculated from Equations 1 and 2 and are denoted by K_{nunit}, K_{sunit}, K_{njoint} and K_{sjoint} respectively. Stiffness matrix considered contribution of all springs around the element to form global matrix. So different phases of material were not considered but only contribution towards stiffness were considered.

K_{nunit} represents stiffness for brick unit in normal direction, K_{sunit} represents stiffness for brick mortar in normal direction whereas K_{njoint} and K_{sjoint} represents stiffness of springs at joint for brick and brick mortar joint in normal and shear direction respectively.

$$K_{nunit} = \frac{E_u t d}{a};\ K_{njoint} = \frac{E_u E_m t d}{E_u x t_h + E_m(a - t_h)} \tag{1}$$

$$K_{sunit} = \frac{G_u t d}{a};\ K_{sjoint} = \frac{G_u G_m t d}{G_u x t_h + G_m(a - t_h)} \tag{2}$$

where E_u is Young's modulus for brick unit and E_m is young's modulus for mortar, respectively, whereas G_u and G_m are shear modulus for brick and mortar respectively. $'t'$ represents the thickness of the wall and $'t_h'$ represents the thickness of mortar [23].

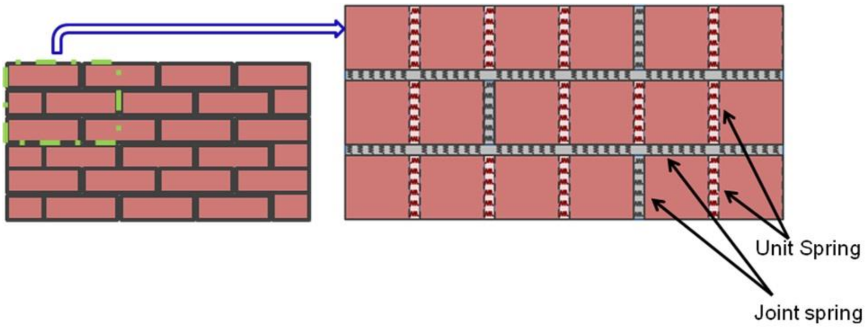


Fig. 2 Masonry details considered in AEM: Brick represented as Unit Spring and Brick-Mortar joint represented as Joint Spring

2.2 Material Model for Masonry

The composite Material model is considered for masonry. This material model takes into account the constitutive relation, which effectively considers elastic and plastic behavior for hardening and softening. Brick units springs were assumed to follow linear elastic behavior and principal stress failure criteria. In the tensile state, splitting of bricks which are reaching the elastic limit for such cases, normal stress and shear stress are assumed that they are not transferred to the cracked surface.

Failure envelope for the brick spring's failure is as given by Equation 3:

$$\frac{f_b}{f_b'} + \frac{f_t}{f_t'} = 1 \tag{3}$$

Brick masonry represents a composite structure. The material model should be such that which takes into account hardening and softening of brick and mortar and composition. So, elastic and plastic behavior is considered to represent constitutive relation between them. Where f_b represents principal compression stress and f_t is principal tensile stresses whereas f_b' is uniaxial compression and f_t' uniaxial tensile strength.

Figure 3(a), Figure 3(b) represents cohesion and bond degradation, respectively [23]. Figure 3(c) represents failure criteria for joint spring [18, 30]. Figure 3(c) represents failure which comes when jointly brick unit and mortar in the maximum compression zone, its represented by compression cap. Figure 3(d) is showing hardening and softening brick masonry.

Equation 4 shows the tension cut-off, f_1, Equation 5 shows the sliding along joints, f_2, exhibit behaviour towards softening, whereas Equation 6 represents f_3, the compression cap which shows hardening and softening respectively. The failure surfaces used in this study are derived before [18], Refer Equations (4), (5) and (6), [Figure 3(c)] [18, 30].

$$f_1(\sigma, K_1) = \sigma - f_t\, exp\left(-\frac{f_t}{G_f^I} K_1\right) \tag{4}$$

$$f_2(\sigma, K_2) = |\tau| + \sigma\, tan(\phi_1) - c\, exp\left(-\frac{c}{G_f^{II}} K_2\right) \tag{5}$$

$$f_3(\sigma, K_3) = |\tau| + \sigma\, tan(\phi_2)\{(\sigma_3(K_3) - \sigma)\} \tag{6}$$

Parameters K_1 is hardening and softening parameters for tension, K_2 hardening and softening parameters for shear and K_3 hardening and softening parameters for the compression. Also, G_f^I is fracture energy in tension, and G_f^{II} is fracture energy in $t$shear.

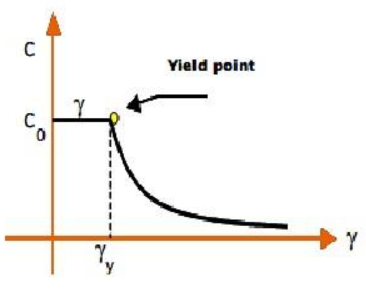


Fig. 3(a) Degradation in cohesion [23]

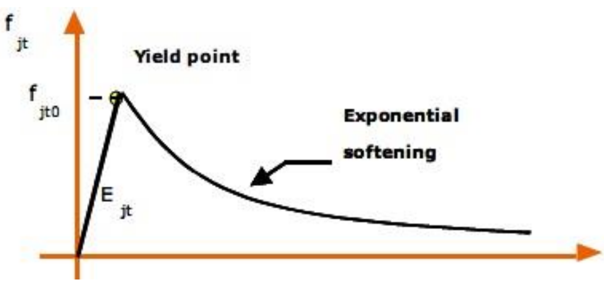


Fig. 3(b) Degradation in Bond [23]

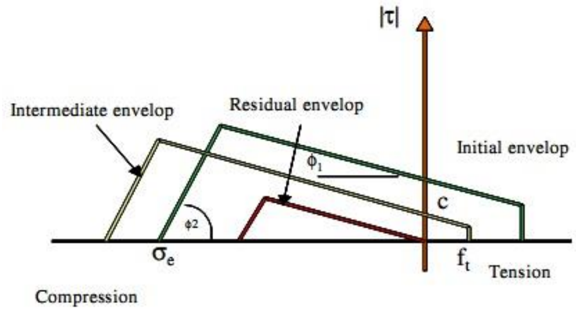


Fig. 3(c) Failure criteria for joint spring [18, 30]

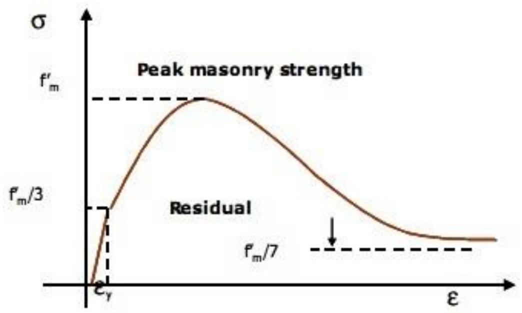


Fig. 3(d) Hardening and softening for joint spring [18]

2.3 Modelling of the Interface Between Wall and Frame

The interface between wall and frame is considered similar to the interface between mortar and brick units. Constitutive law for tension, shear and compression are considered as discussed before. Detaching of the panel from the frame is an indication of a failure of interface spring in tension. Friction due to large displacement range contributes to shear sliding between frame and panel.

2.4 Numerical Scheme

Spring characteristics of concrete, masonry and interface are taken into separate consideration before assembling their stiffness contribution to the corresponding element centroids. Displacement vector obtained from global solution has been assigned to corresponding spring to get the spring force in proportion to their stiffness. Stress is derived by considering the participation of masonry and concrete as an interface.

2.5 Material Models: For Concrete and Steel

Maekawa's compression model is considered to represent concrete [Figure 4(a)]. The material model for shear is as shown in Figure 4(b). The material model for steel is as shown in Figure 4(c). For more details, refer [11].

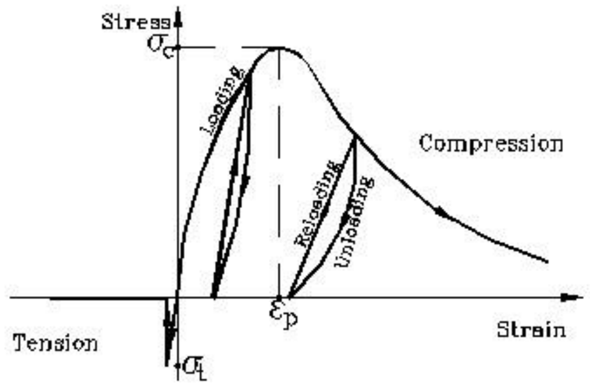


Fig. 4(a) Tension and compression in concrete [4]

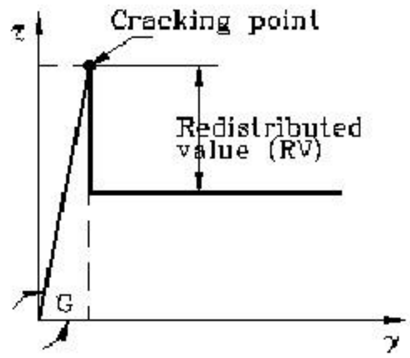


Fig. 4(b) Shear for concrete [4]

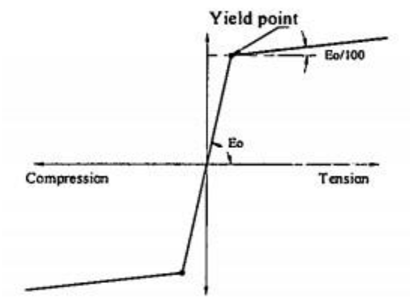


Fig. 4(c) Material models for steel [11]

2.6 Failure Criteria

One of the failures can be diagonal crack. Modelling diagonal failure and also detection of a crack is challenging.

Stresses are calculated around each element and crack detected based on the solution. Refer Figure 5. At point A, shear and normal stresses are calculated from the respective springs at that location. Finally, the principal stresses at each location are determined.

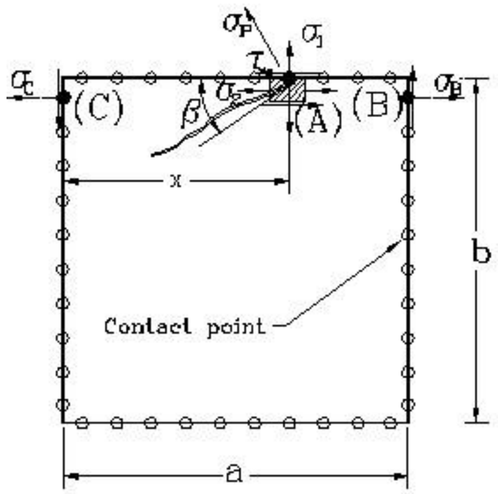


Fig. 5 Failure Criteria [5]

The secondary stress (σ_2) can be calculated by Equation 7 from normal stresses at location *B* and *C*, refer Figure 5.

$$\sigma_2 = \frac{x}{a}\,\sigma_B + \frac{(a-x)}{a}\sigma_C \tag{7}$$

Equation 8 shows principal stress calculation

$$\sigma_p = \left(\frac{\sigma_1+\sigma_2}{2}\right) + \sqrt{\left(\frac{\sigma_1-\sigma_2}{2}\right)^2 + (\tau)^2} \tag{8}$$

Tension resistance of the material is compared with principal stresses as calculated in Equation 8. Redistribution of forces is done if the principal stresses exceed tension resistance at the critical stage. These redistributed forces are applied in reverse direction in next increment. Then, these forces are transferred at the center of the element in terms of force and moment. Proper crack propagation can be followed by redistribution of forces along with the crack location. After crack occurrence shear stiffness is assumed as zero though there might be some resistance due to interlocking between crack faces and friction.

Inclination angle (β) towards local crack and element edge is calculated using Equation 9.

$$tan(2\beta) = \left(\frac{2\tau}{\sigma_1+\sigma_2}\right) \tag{9}$$

If the value of shear stress is zero, it implies that crack is coinciding with the element edge. This is a simple technique and no special consideration required to represent the crack. If shear is not predominant, then angle tends to zero indicating edge of the element and crack is in parallel, which gives highly accurate results.

3. Verification With Experimental Study

3.1 Validation with experimental results

Numerical results of the brick masonry wall using Applied Element method [23] was validated with experimental wall test [31]. Good comparable results were found. Please refer to Figure 6a and 6b for more details. The wall of dimension as 990 mm x 1000 mm was considered for the study. Brick considered of size 210 mm x 52 mm x 100 mm and mortar thickness 10 mm. Figure 7 shows a comparison between experimental and numerical study.

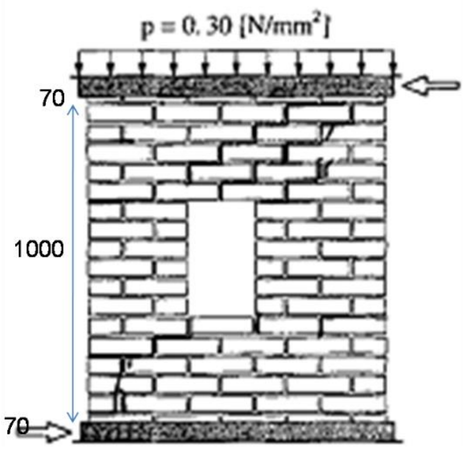


Fig. 6(a) Experimental study showing a crack pattern [31]

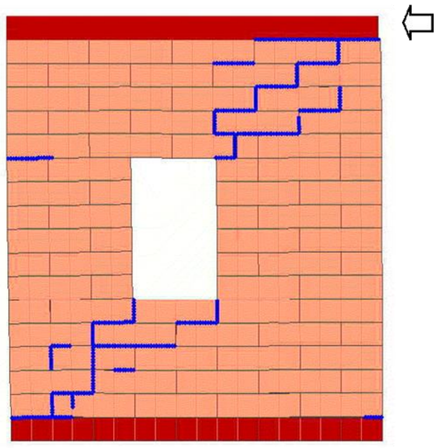


Fig. 6(b) Numerical study showing a crack pattern [23]

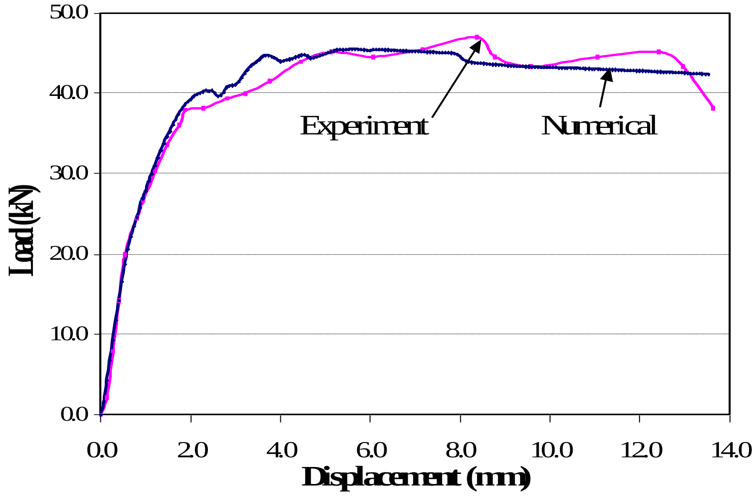


Fig. 7 Comparison with the experimental study [23, 31]

Current Study: This study is an extension of a previous study [32] which was conducted using lateral loads.

4. The Geometry of Wall, Beam, Column and Steel Details: A Typical RC Framed Brick Masonry Wall was Considered for the Study

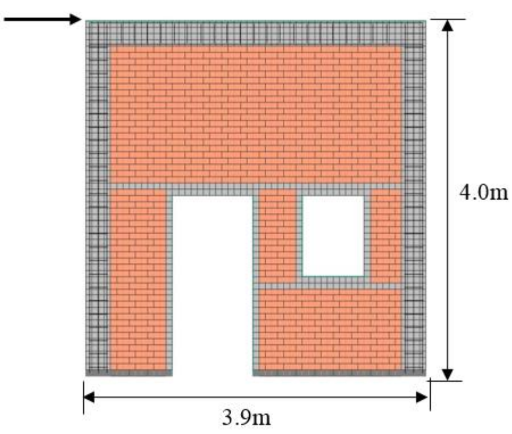


Fig. 8(a) Numerical model of brick infill wall

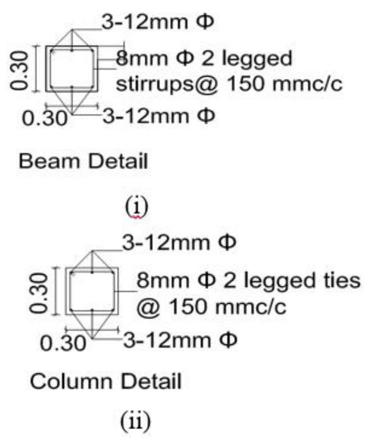


Fig. 8(b) Reinforcement details for (i) Beam & (ii) Column

Figure 8(a), (b) refers to the geometry of the wall and reinforcement details considered for the beam and column.

A parametric study was done by considering an RC framed infill brick masonry wall with one door and one window opening.

Geometry of door and window opening: RC framed brick infill wall with a door opening of size 2.1 m x 1.05 m and window opening of size 1.05 m x 0.84 m (Figure 8(a)).

4.1 Material Properties and Boundary Conditions and Loadings as Below

Material Property for Concrete and Steel:

Material properties are assigned to springs which represent a brick unit, mortar, concrete and steel, Material properties as considered from micro test results [23].

Young's modulus of brick, E_u = 1670 N/mm^2; Young's modulus of mortar, E_m = 0.79 N/mm^2; tensile strength of brick, f_{ut} = 2 N/mm^2; Tensile strength of joint, f_{jt} = 0.25 N/mm^2; cohesion = 0.35 N/mm^2 and Friction angle considered as ϕ_1 = 36.50. Compressive strength of masonry f_m' assumed as 20 N/mm^2 and cap mode angle as, ϕ_2, is selected as 400 as suggested by Sutcliffe et al., 2001, as 20^0 - 70^0.

Material property for concrete and steel:

Concrete considered as M25 grade and steel considered as grade Fe 250

Young's modulus for concrete and steel is considered as shown.

For concrete, E_c = 25000 N/mm^2, and steel, E_{st} = 2 X 10^5 N/mm^2,

Reinforcement detail: Steel is embedded in the wall. i.e. Normal and shear springs (as explained in Figure 1) in the particular region will have properties of steel as considered in the study.

Case 1: No reinforcement (Figure 9)

Case 2: Horizontal reinforcement in provided i.e. Lintel and sill (See Figure 10)

2 bars of 12 mm diameter and Stirrups 6 mm diameter spaced at 150 mm as per IAEE guidelines (33)

Case 3: Vertical reinforcement provided – As per IAEE guidelines (33) (See Figure 11)

16 mm dia bars are used, spacing between the bars is 210 mm

Case 4: Combination of Case 2 and Case 3 (See Figure 12)

Boundary Conditions: All elements are fixed at the base.

Loading: RC framed infill has been subjected to following earthquake loading

4.2 Earthquake components considered in the Study

North India and NE India always faces the earthquake challenge, so past earthquake acceleration data of has been considered as being in an earthquake-prone region. Predominant period of earthquake which causes damage is mentioned here. Station owner: IIT Roorkee

1. The NE India earthquake: PGA of 1.110 m/s^2 with the time period of 0.02 sec and total duration 16.9 sec NE-India Earthquake, Sept. 10, 1986 13:20 IST, 86.0010.L.

   Station: UMMULONG, 25 31 N 92 10 E COMP: N87E.
2. Uttarkashi Earthquake: PGA of 2.48 m/s^2 with time period of 0.02 sec and total time of 36.1 sec Station: Bhatwari, India, Latitude and Longitude: 30.8, 78.22, Oct 19, 1991.
3. Chamoli Earthquake: PGA of 1.95 m/s^2 with time period of 0.02 sec and total time of 25.38 sec Station: Gopeshwar, India, Latitude and Longitude: 30.4, 79.33, March 28, 1999

Four cases have been considered for the study with door and window frame:

1. RC framed wall with brick infill with no reinforcement (Figure 9)
2. RC framed wall with brick infill with horizontal reinforcement (Figure 10)
3. RC framed wall with brick infill with vertical reinforcement (Figure 11)
4. RC framed wall with brick infill with vertical, horizontal reinforcement (Figure 12)

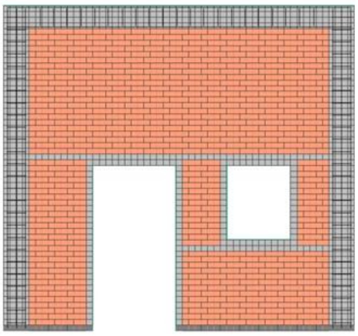


Fig. 9 Case 1 - No reinforcement

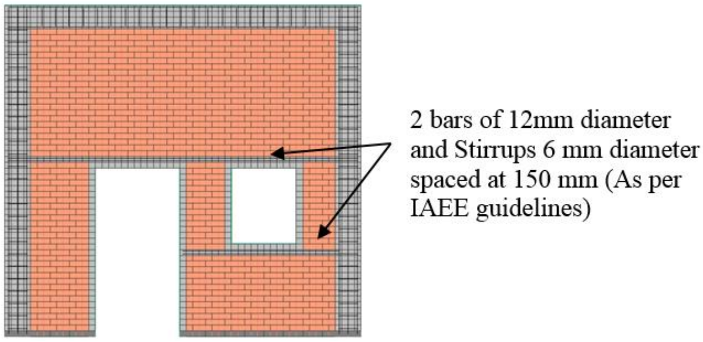


Fig. 10 Case 2 - Horizontal reinforcement

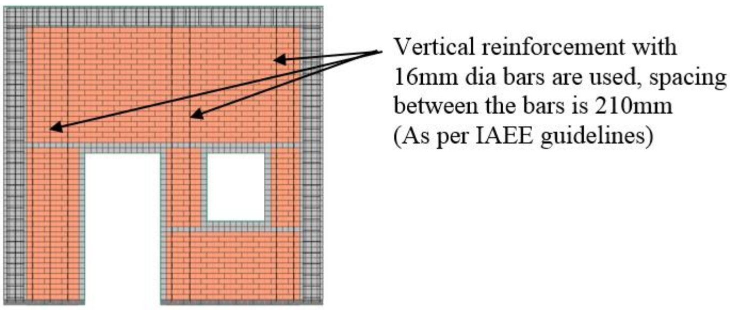


Fig. 11 Case 3 - Vertical reinforcement

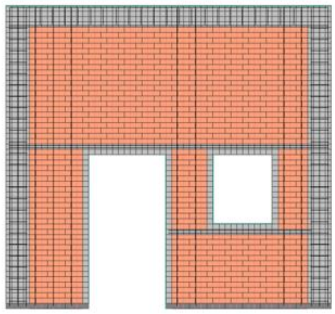
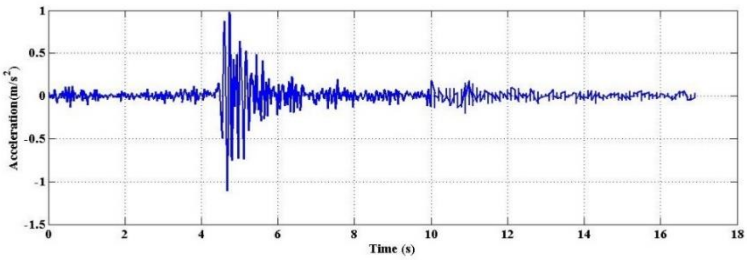
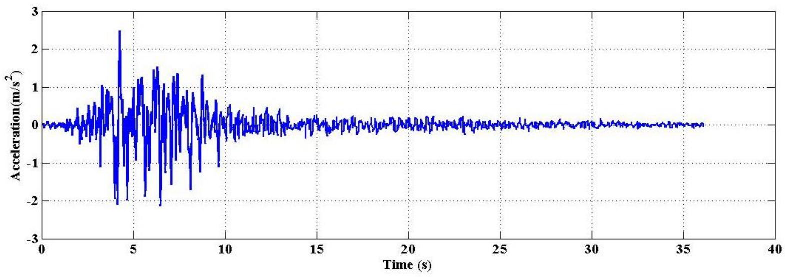


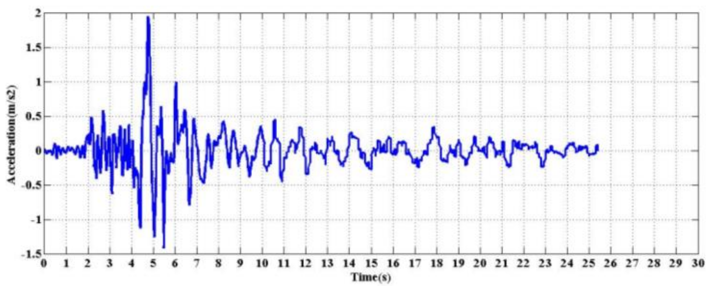
Fig. 12 Case 4 -Vertical and Horizontal reinforcement

5. Results and Discussions

Table 1 shows Acceleration time history of NE INDIA, Uttarkashi and Chamoli earthquake.

Table 1: Acceleration Time history of Earthquake and station details

Earthquake details	Acceleration Time History	Station
The NE India earthquake PGA of 1.110 m/s^2 with total duration 16.9 sec, NE-India Earthquake, Sept 10, 1986 13:20 IST, 86.0010.L		Ummulong, 25 31 N 92 10 E COMP: N87E
Uttarkashi Earthquake: PGA of 2.48 m/s^2 with total duration of 36.1 sec, Oct 19, 1991		Bhatwari, India, Latitude and Longitude: 30.8, 78.22,

Chamoli Earthquake: PGA of 1.95 m/s^2 with total duration of 25.38 sec, Mar 28, 1999		Gopeshwar, India, Latitude and Longitude: 30.4, 79.33,

5.1 Results with NE India Earthquake

Table 2: Behaviour of wall during NE earthquake with and without reinforcement

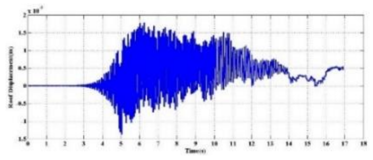
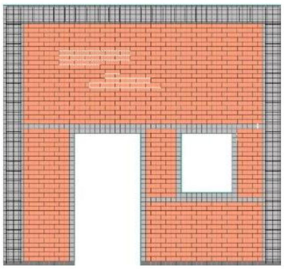
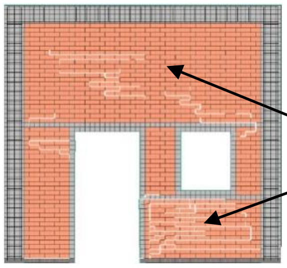
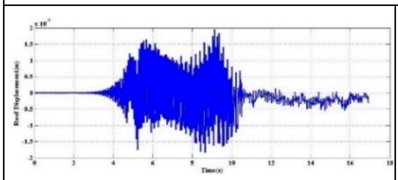
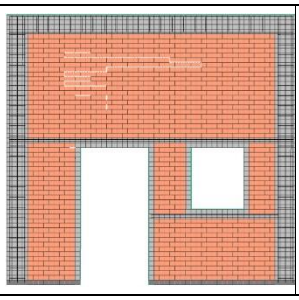
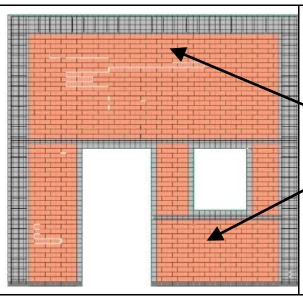
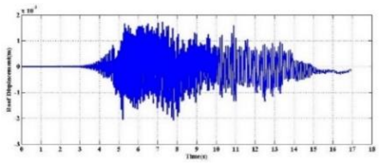
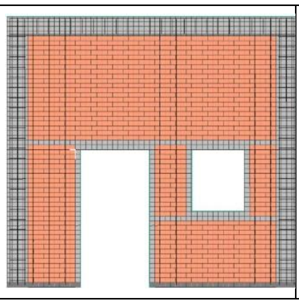
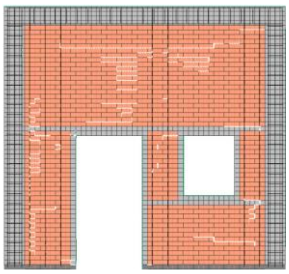
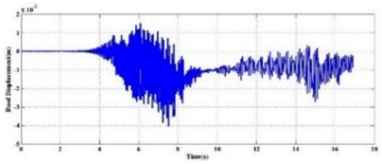
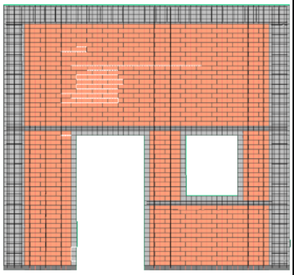
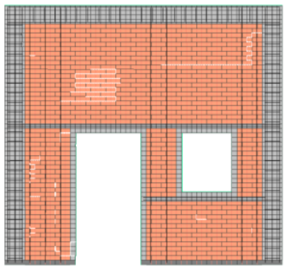
Case 1: No Reinforcement			
Roof displacement versus time response	**Cracks at PGA**	**Cracks at the end of duration**	
			**Cracks**
Case 2: Horizontal Reinforcement			
			**Cracks reduced**
Case 3 : Vertical Reinforcement			
			**Minor cracks at the end.**
Case 4: Horizontal and Vertical Reinforcement			
			**Overall Strengthening**

Table 2 shows behavior of RC framed infill wall panel for NE India earthquake: Column 1 shows roof displacement with time response of the wall panel. Response is gradually reduced with horizontal and vertical reinforcement.

Case 1: A wall with no reinforcement is a typical construction practice in rural areas mostly with the help of unskilled labors. Such houses, when checked with NE earthquake, following observations are found. Under the seismic action, there is no response initially, however crack initiation started at around 5 sec at PGA. Case 1 shows cracks in the upper part and at the right pier of the wall. This is due to aspect ratio which is contributing to predominant shear. Strut action could not complete due to openings in the wall.

Case 2: Wall with Horizontal Reinforcement: In case 2, horizontal reinforcement has been provided in the form of the lintel on door and window. Provision of lintel shows the considerable strengthening effect. Cracks in piers as seen in Case 1 are taken care when lintel is provided. Horizontal reinforcement develops 5 strong components or pockets or regions in the wall which resists earthquake forces effectively (See Figure 13).

When compared with Case 1, the provision of lintel gives very good strength to the wall and cracks reduced considerably. Cracks at the right pier are entirely avoided due to provision of horizontal reinforcement.

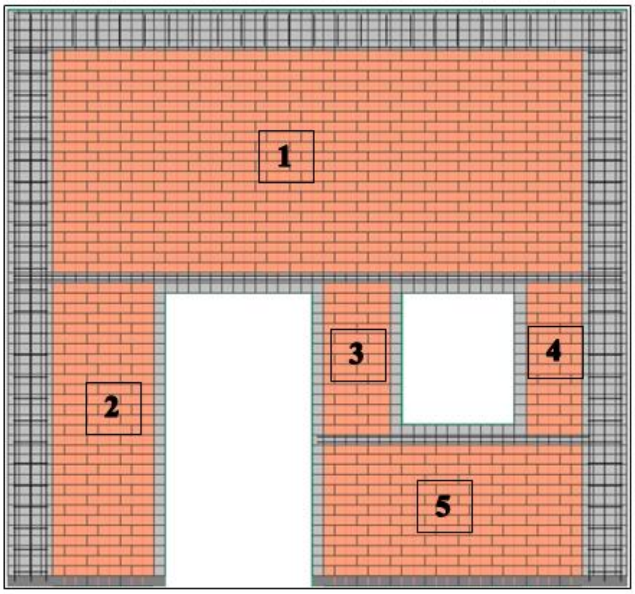


Fig. 13 Horizontal reinforcement develops 5 strong components in wall

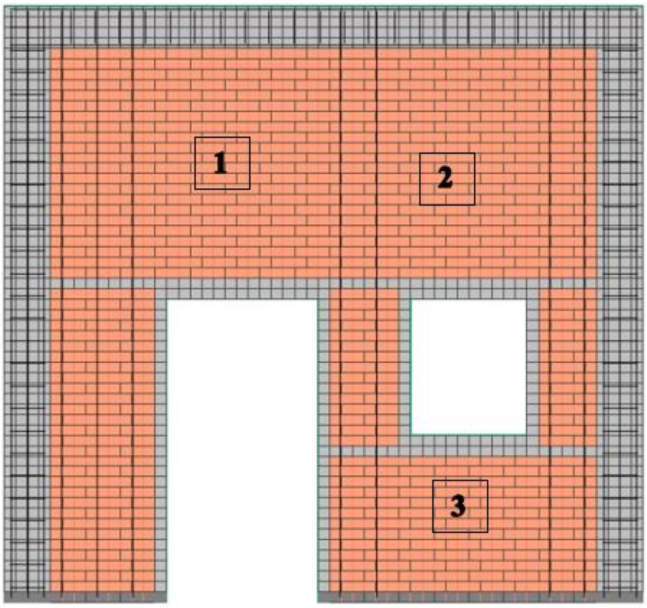


Fig. 14 Vertical reinforcement develops 3 strong components in wall

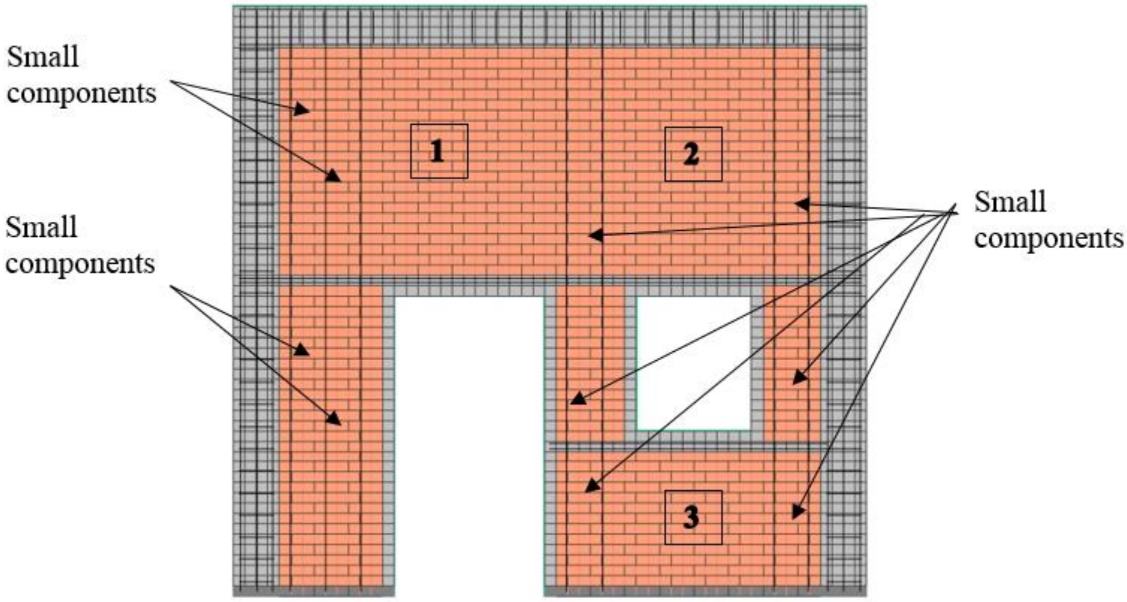


Fig. 15 Horizontal and Vertical reinforcement develops 3 major strong components and 10 small strong components in wall

Case 3: Wall with Vertical Reinforcement: Case 3 shows the provision of vertical reinforcement to the wall. Effect of vertical reinforcement on the wall can be seen in a response plot of time versus roof displacement ratio in Table 2. Figure 14 shows development of strong components in wall due to vertical reinforcement, which resist the earthquake forces and reduces cracks in wall. No major cracks were seen in right pier of the wall as in Case 1. Minor cracks were observed when vertical reinforcement is provided. It shows the strengthening effect due to vertical reinforcement. Flexure cracks are seen above the door, as no vertical reinforcement provided above door and window. (Refere Table 2)

Case 4: Wall with Horizontal and Vertical Reinforcement: A good strengthening effect can be seen due to provision of horizontal and vertical reinforcement. Table 2 shows the time versus roof displacement response of the wall for case 4, overall response is reduced due to provision of reinforcement. Figure 15 shows details of strong major and minor components developed due to provision of horizontal and vertical reinforcement, which enhances the strength and stiffness of the wall.

5.2 Results with Uttarkashi Earthquake

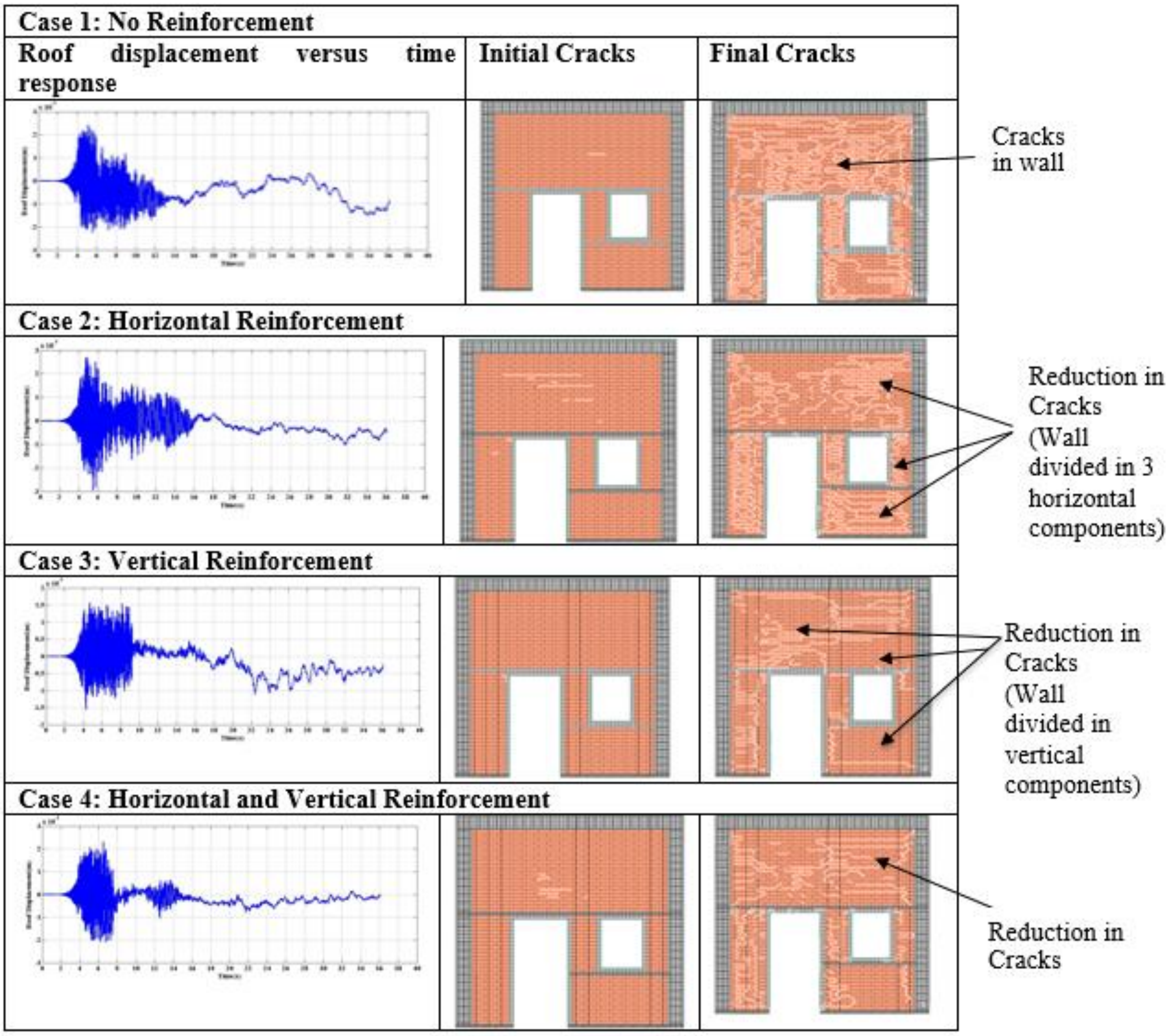
Table 3 shows response of wall during Uttarkashi earthquake.

For all the four cases cracks initiated at 4-5 sec at PGA. Cracks are seen due to following reasons:

1. Shear is dominant due to contribution of aspect ratio.
2. Strut action is not complete due to door and window opening in the wall.
3. Cracks are continuously increasing due to dynamic loading.
4. Duration of earthquake: Uttarkashi earthquake duration is 36.1 sec due to which a greater number of cracks can be seen in case 1 which is reduced due to provision of reinforcement.

Provision of horizontal reinforcement divides wall in three components horizontally (See Table 3). Its providing strength and stiffness to wall and hence cracks are reduced in this region. Provision of vertical reinforcement gives strengthening an effect as wall gets divided in components and provides stiffness to wall. Provision of reinforcement in both the direction overall increases strength in the wall.

Table 3: Behaviour during Uttarkashi earthquake

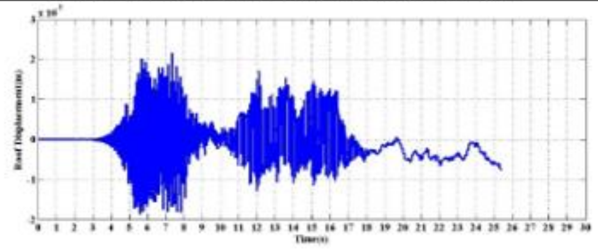
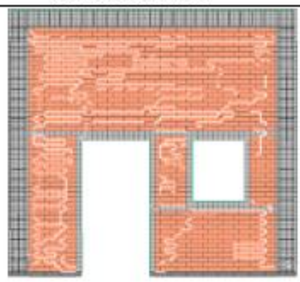
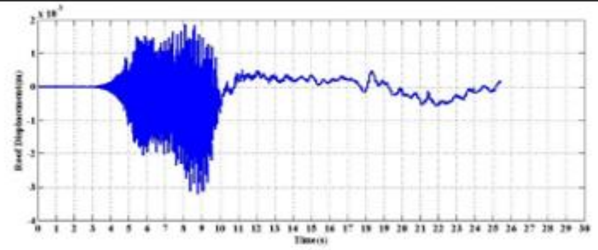
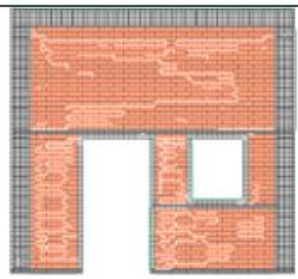
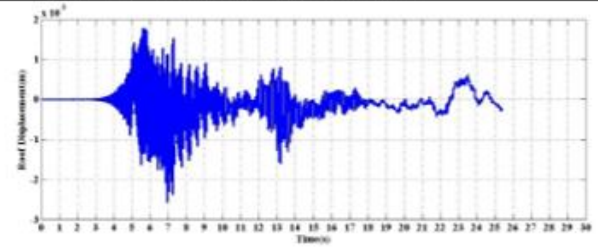
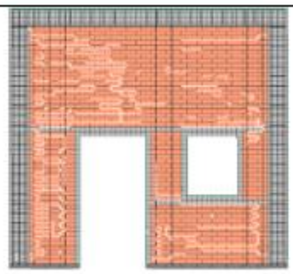
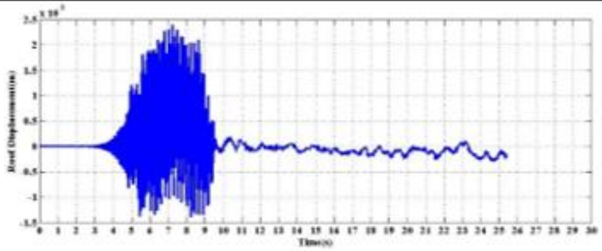
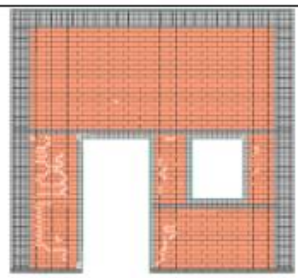


5.3 Results with Chamoli Earthquake

Chamoli has PGA 1.95 m/s^2 and duration of 25.38 sec.

Table 4 shows response of wall during Chamoli earthquake. Final cracks at the end of earthquake is considered in this table. As discussed in previous case, cracks are seen due to dominant shear, incomplete strut action, dynamic loading and duration of earthquake. However, provision of horizontal reinforcement divides wall in three components (See Table 4). Its providing strength and stiffness to the wall and hence cracks are reduced in this region. Similarly, Provision of vertical reinforcement gives strengthening effect. Due to reinforcement wall gets divided in components and provides stiffness to wall. Provision of reinforcement in both the direction overall increases strength in the wall.

Table 4: Behaviour Jahangir, Bakhtiari during Chamoli earthquake

Roof displacement versus time response	Final Cracks	
Case 1: No Reinforcement		
		Cracks at 25.38 secs
Case 2: Horizontal Reinforcement		
		Cracks are reduced
Case 3: Vertical Reinforcement		
		Cracks are reduced
Case 4: Horizontal and Vertical Reinforcement		
		Good strengthening effect

CONCLUSIONS

- RC framed Brick infill wall behavior depends on the type of earthquake, PGA, duration of earthquake and aspect ratio.
- Horizontal and vertical reinforcement play an essential role in strengthening of the wall by increasing strength and stiffness.
- Horizontal reinforcement provided in terms of lintel and sill divides wall in 3 high strength components i.e. wall above the lintel, wall between lintel and sill and wall below the window, which effectively resists the earthquake and reduces crack propagation.

- Vertical reinforcement provided in the wall, strengthens the wall by creating 2 components i.e. one above the door and other above the window and one below the window, all behave as strong units which are capable to resist strong ground motion.
- When horizontal and vertical reinforcement provided, it gives overall strengthening to the wall and helps in reducing response of the wall.

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