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In-plane Shear Behaviour of Unreinforced Brick Masonry – A Literature Review

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ABSTRACT: During the post-earthquake damage surveys carried out in different parts of the world, it is noted that masonry buildings are one of the most vulnerable types of construction when subjected to earthquake forces. Realising this, recently number of investigations are being carried out. The lateral load resistance of masonry buildings is mainly due to in-plane shear resistance of the masonry elements/piers. Therefore detailed investigation on the in-plane shear behaviour of masonry piers thus becomes necessary. This paper contains a review of models proposed for predicting in-plane shear resistance and deformability of unreinforced brick masonry piers, and a brief over view of applicability of these models for checking safety of unreinforced masonry buildings against earthquakes. Probabilistic capacity assessment of one of the walls of a monumental building is carried out using the strut model and the results are discussed.

1 INTRODUCTION

Brick masonry construction is the most preferred one for low rise buildings, especially in the developing and under developed countries because of its ease in construction, economy etc,. It has a very long history that dates back to 2000 B.C. or so. Unreinforced brick masonry has been used for the construction of a number of historical/monumental buildings. Nevertheless, brick masonry is the least understood in the aspect of strength and other performance related parameters (Abrams 2001), because of its complex behaviour and its non homogeneity even in deci-scale. Many earthquake damage reports point out the devastating damage to masonry buildings including the recent Iran earthquake May-07, 2006. In particular the unreinforced brick masonry buildings are seen to be highly vulnerable even to moderate earthquakes. The 1997 Umbria earthquake which occurred in Central Italy with a magnitude of 4.2 on Richter scale had resulted in considerable damage to the unreinforced masonry buildings (Priya et al. 2005). In India, during the earthquakes at Bhuj (2001), Uttarkashi (1991), Koyna (1967) etc., the unreinforced brick masonry structures underwent devastating damage (Priya et al. 2005). At the same time the performance of a few buildings in past earthquakes, have demonstrated that masonry buildings do possess good characteristics if well engineered and constructed (Abrams 2001). It is also seen that when a masonry building is subjected to earthquake forces, the contribution of in-plane shear resistance of the brick masonry towards the lateral load resistance will be high when comparing to that of the out-of-plane shear resistance.

In India, there are many monumental structures made of brick masonry viz; Qutub Minar at New Delhi, Rajendra Chola Madhil at Gangaikonda Cholapuram, The Great Stupa at Sanchi, Santhome Church at Chennai etc,. Monumental structures represent the cultural heritage and in the recent years growing attention has been devoted to the protection of these structures against extreme events. As pointed out by Augusti et al. (2001), predicting the response of monumental buildings is a challenging task. Attempts have been made to evaluate the lateral resistance adopting suitable theoretical models (Orduña and Lourenço 2001). Investigations have been carried out by various researchers to determine the seismic vulnerability of monumental buildings (Augusti et al. 2001, Petrini and Casolo 1998). This paper highlights the features of some of the popular models. After presenting a review of different models proposed for determining the in-plane shear resistance of masonry piers, an example problem of probabilistic capacity assessment of a typical wall of a building (which can also be a wall of a monumental building) with unreinforced brick masonry is carried out using Monte Carlo simulation technique. More studies are required to examine the applicability of the shear capacity prediction models, reviewed in this paper, for monumental structures (using the in-situ properties obtained from field investigations). Studies are being continued in this direction at SERC.

2 MODES OF FAILURE

When a brick masonry pier is subjected to in-plane horizontal force, it may fail in any of possible failure modes viz; flexure or rocking mode of failure, diagonal shear mode of failure, sliding shear mode of failure (Magenes and Calvi 1997). The 'failure mode' is governed by geometric aspect ratio (ratio of height of pier 'H' to width of pier 'D'), boundary conditions at top and bottom of pier, vertical compression stress and relative strengths of brick and mortar used. One of the important failure modes to be considered for assessing the safety of historical brick masonry structures is due to fatigue loads. Depending upon the magnitude of the sustained vertical compressive loads and the age of the structure (including the presence of defects in the bricks), the capacity of the structure to resist the amplitude of the fatigue load varies (Ronca et al. 2005). However, number of experimental studies is required to develop S-N curves and also the interaction curves similar to those proposed by Ronca et al. (2005).

2.1 Flexure or Rocking Failure

This mode of failure may be associated with slender piers and when the vertical compression stress is small. Cracking will be seen on the tension side and the load will be transferred to the compression zone. The final failure will be characterised by the failure of masonry in compression (see Fig. 1a). This mode of failure is also reported in literature as toe compression failure. As this seems to be common mode of failure, attempts have been made by several researchers to describe the same analytically some of them are given below.

Tomazevic (1999)

$$H_{f,w} = \frac{D\,t\,p}{2\alpha_v}\left(1 - \frac{\gamma_m\,p}{f_k}\right) \tag{1}$$

where $H_{f,w}$ = design lateral resistance of masonry in rocking mode, D = width of the pier, t = thickness of pier, p = design value of compressive stress, f_k = Characteristic compressive strength of masonry, γ_m = Partial safety factor for masonry, α_v = Shear ratio (ratio of effective height to width of pier $= \psi\, H/D$ where, ψ = 1.0 for fixed-free pier and 0.5 for fixed-fixed pier).

Abrams (2001)

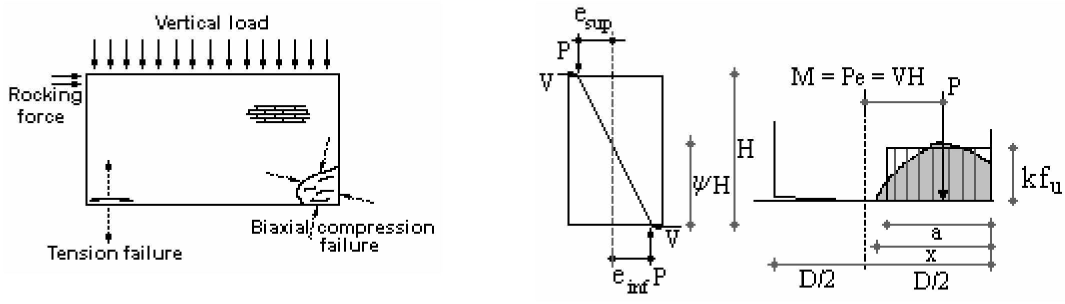
$$V_r = \alpha P_{lb}\left(\frac{D}{h_{eff}}\right)\left(1 - \frac{f_a}{0.7\,f'_m}\right) \tag{2}$$

where V_r = lateral rocking resistance, P_{lb} = lower bound of vertical compressive force, α = effective height factor (for fixed-fixed pier – 1.0 and for fixed-free pier – 0.5), f_a = lower bound of vertical compressive stress, f'_m = lower bound of masonry compressive strength and h_{eff} = height of the resultant lateral force.

Magenes and Calvi (1997)

$$V_r = \frac{D\,t\,p}{2\alpha_v}\left(1 - \frac{p}{k\,f_u}\right) \tag{3}$$

where k = coefficient which takes into account the compressive stress distribution in the toe (= 0.85) and f_u = ultimate compressive strength of masonry.



(a) (b)

Figure 1 : Flexural/Rocking mode of Failure

It may be noted that these formulae have been derived basically from the principle of equilibrium of forces. The behaviour of masonry under uniaxial compression is similar to that of concrete. The compressive stress distribution in the toe is idealized as rectangle instead of parabolic (see Fig. 1b), but, it should be noted that the shear capacity is less sensitive to k and compressive strength, and is more sensitive to α_v (Magenes and Calvi 1997). This observation suggests that the use of rectangular stress distribution for compressive stress may not significantly affect the prediction of shear resistance. It is to be noted that all these formulae have not considered the reduction in effective width of the compression zone as the number of cycles of rocking increases.

2.2 Diagonal Shear Failure

This mode of brittle failure can occur for the piers which are neither too slender nor too squat and when the order of vertical compression stress is high. Diagonal (X- type) cracking will be seen in the piers. The cracks may either run along the mortar joints alone or through the bricks also.

2.2.1 Case I: Failure along mortar joints alone

These formulae are based on the Mohr-Coulomb friction criteria. This type of failure may be triggered in the case of masonry with brick units relatively stronger than mortar joints (see Fig. 2a).

Eurocode (from Tomazevic 1999)

$$H_{sd,w} = D\,t\,\frac{(f_{vo} + \mu_c\,\sigma_d)}{\gamma_m} \tag{4}$$

where $H_{sd,w}$ = design shear resistance of masonry, f_{vo} = shear strength due to zero compression, μ_c = constant defining the contribution of compression stress, σ_d = design compression stress perpendicular to shear.

Grimm (1975)

$$V_d = D\,t\left(1.4 f_b + \phi f_p\right) \tag{5}$$

where V_d = ultimate shear resistance of masonry (in lb), f_b = bond strength of brick-mortar (in lb/in^2), f_p = axial compressive stress (in lb/in^2) and ϕ = coefficient of internal friction.

Magenes and Calvi (1997)

The ultimate shear resistance V_d, of masonry is the least of the values obtained using,

$$V_d = D\,t\left(\frac{c' + \mu' p}{1 + \alpha_v}\right) \tag{6}$$

$$V_d = D'\,t\left(\frac{1.5\,c' + \mu' p}{1 + \dfrac{3\,c'\,\alpha_v}{p}}\right) \tag{7}$$

where $c' = Kc$ and $\mu' = K\mu$

$$K = \frac{1}{1 + \left(\dfrac{2\mu\,\Delta_y}{\Delta_x}\right)} \tag{8}$$

$$D' = 3\left(\frac{1}{2} - \frac{V}{P}\alpha_v\right)D \tag{9}$$

where c = cohesion of bed joints, μ = coefficient of friction of bed joints, Δ_y = height of brick unit and Δ_x = length of brick unit.

In Eurocode formula, f_{vo}, μ_c are global strength parameters of masonry whereas, in Magenes and Calvi (1997) formula, c, μ are local material properties. The global strength parameters and local material properties can be obtained by conducting tests. Among these, the tests required for the estimation of local material properties are comparatively easier when compared to the tests for estimation of global strength parameters. Nevertheless, the fruitfulness lies in the accuracy of model relating the local material properties to obtain the shear capacity of the masonry pier. The influence of head joints is considered by Magenes and Calvi (1997) as per the correction suggested by Mann Muller (Eq. 8) from Magenes and Calvi (1997), which is more realistic. Further, the effect of flexural cracks, if any, is considered by Magenes and Calvi (1997). For the estimation of in-plane shear capacity, Grimm (1975) considers the bond strength of brick-mortar whereas Eurocode considers the shear strength due to zero compression. Nevertheless, they are physically analogous quantities. Furthermore, the change in the constants of Grimm (1975) formula is on account of its applicability only in FPS unit system. The Mohr-Coulomb's linear friction model, which is the basis of these formulae for diagonal shear mode of failure, is not valid for very low compression stress range according to Andreas (1996).

2.2.2 Case II: Failure through bricks

This type of failure may be triggered in masonry where the brick strength and mortar strength are comparable or where the mortar is relatively stronger than brick (See Fig. 2b).

Tomazevic (1999)

$$H_{sd,w} = D\,t\,\frac{f_{tk}}{\gamma_m b}\sqrt{1 + \frac{p\gamma_m}{f_{tk}}} \tag{10}$$

where f_{tk} = characteristic tensile strength of brick masonry and b = a constant depending on geometric aspect ratio (b =1 for H/D ≤ 1; b = H/D for 1< H/D<1.5 and b = 1.5 for H/D ≥ 1.5).

Abrams (2001)

$$V_{dt} = D\,t\,f'_{dt}\,\frac{D}{h_{eff}}\sqrt{1 + \frac{f_a}{f'_{dt}}} \tag{11}$$

where f'_{dt} = lower bound of the masonry diagonal tension strength.

Magenes and Calvi (1997)

$$V_d = D\, t \frac{f_{bt}}{2.3(1+\alpha_v)} \sqrt{1+\frac{p}{f_{bt}}} \tag{12}$$

where f_{bt} = tensile strength of bricks.

The formula given by Tomazevic (1999) considers the masonry as homogeneous, isotropic material and the failure is according to maximum shear stress theory. According to the formulae given by Tomazevic (1999) the shear capacity is a function of geometric aspect ratio only. But the formula proposed by Magenes and Calvi (1997) expresses the shear capacity in terms of shear ratio (α_v). Proposal by Magenes and Calvi (1997) seems to be more realistic, because shear ratio takes into account the effect of boundary conditions, whereas geometric aspect ratio is not capable of including the effect of boundary conditions.

(a) failure along mortar joints (b) failure through the bricks

Figure 2 : Diagonal shear mode of failure

2.3 Sliding Shear Failure

Squatter piers may encounter this type of failure when the vertical compression stress is low. The failure will be in the form of sliding along the bed joints (see Fig. 3). Eurocode (from Tomazevic 1999), Magenes and Calvi (1997) express this state as follows;

$$H_{sl,w} = \mu_c\, P \tag{13}$$

where $H_{sl,w}$ = design sliding resistance of masonry, μ_c = coefficient of friction between brick and mortar joint and P = vertical compressive force.

Abrams (2001)

$$V_{bjs} = \nu_{me}\, D t \tag{14}$$

where ν_{me} = expected bed joint sliding shear strength

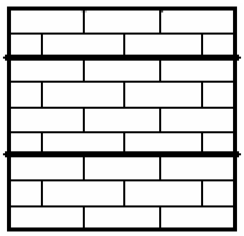


Figure 3 : Sliding shear mode of failure

In close observation it could be seen that the formula given in Eq. 13 is similar to the diagonal shear failure formulae. Only difference is that shear capacity under zero compression stress is neglected. But in reality, there will be lateral resistance due to adhesion of the mortar joint with bricks and due to the self weight of the wall.

3 PROVISIONS GIVEN IN IS: 1905-1987 FOR ESTIMATION OF SHEAR CAPACITY

The Bureau of Indian Standards prescribes simple formulae considering the different modes of failure and the levels of vertical compression stress (Eq. 15). Allowable maximum shear stress shall be the least of the three values evaluated by,

$$\begin{aligned} F_{v,all} &= 0.5\,\mathrm{MPa} \\ F_{v,all} &= 0.1 + 0.2\,\sigma_d \\ F_{v,all} &= 0.125\sqrt{f_m} \end{aligned} \tag{15}$$

where $F_{v,all}$= allowable maximum shear stress (in MPa), σ_d= average axial compression (in MPa) and f_m= compressive strength of masonry (in MPa).

Allowable maximum shear stress for unreinforced masonry is based on experimental studies on various failure modes. At low values of vertical compression stress, sliding type of failure mode may be triggered, and so a Mohr-Coulomb type failure theory is more appropriate and shear capacity is increased with the increase in the vertical load. At larger values of vertical compression stress, tensile cracking of masonry is more likely which are expressed in terms of square root of compressive strength of masonry.

4 DEFORMABILITY

In addition to the interfacial stresses, deformability exhibited as drift is also a major factor contributing towards the stability of the masonry structures. For instance the Eurocode has given the formula for displacement as given in Eq. 16.

$$\delta = \frac{V h^3}{12 E I_w} + \frac{\kappa V h}{G A_w} \tag{16}$$

where δ = in-plane displacement of at the top of the pier, E = modulus of elasticity of masonry, G = shear modulus of masonry, h = height of the pier, I_w = moment of inertia of the cross section of pier in plan about its minor axis, A_w = area of the cross section of pier in plan and κ= shear coefficient (1.2 for rectangular section).

Magenes and Calvi (1997) suggests the limiting values for the displacement (drift) as 1.0% of height of pier for flexure and sliding mode of failure and 0.5% of height of pier for the diagonal shear mode of failure. This is because, in the flexural and sliding modes of failure, large displacements are possible without the loss of structural integrity of masonry, whereas, in diagonal shear mode, it is not so. The Eq. 16 is meant for walls with fixed-fixed condition only, whereas the same equation is rearranged as,

$$K_e = \frac{G A_w}{\kappa h \left[1 + \alpha' \frac{G}{E} \left(\frac{h}{D} \right)^2 \right]} \tag{17}$$

where K_e = in-plane elastic stiffness of masonry pier.

Equation (17) is applicable to cantilever walls also as the effect of boundary conditions is taken care by the constant α' (equal to 0.83 for fixed-fixed condition and 3.33 for fixed-free condition)

Considering the different possible modes of failure of piers and the boundary conditions, attempts have been made to evaluate the seismic safety of brick masonry structures and the approaches are discussed in the following sections.

5 SEISMIC SAFETY EVALUATION OF UNREINFORCED BRICK MASONRY BUILDINGS

Augusti et al. (2001) used the macro-element approach for seismic vulnerability analysis of monumental buildings. In their approach, the whole structure is considered as assemblages of few components called macro-elements and the probability of failure of the structure against collapse during an earthquake is calculated using a system reliability analysis procedure. The macro-elements can further be sub-divided into assemblage of masonry piers. Discretisation of the macro-elements into piers should be carried out with care so that the overall behaviour of wall is represented realistically. Then, the stiffness matrices for the piers are evaluated and assembled to get the stiffness matrices for the macro-elements and in turn assemble the global stiffness matrix of the structure. While obtaining the stiffness matrix of a macro-element, the load sharing path has to be considered. In a similar way, the load sharing path amongst the macro-elements has to be considered while assembling the global stiffness matrix of the structure. Dynamic analysis of the structure has to be carried out to estimate the spatial distribution of earthquake forces. This equivalent static force is distributed to every macro-element in proportion of its stiffness, and then to piers. Piers are checked against their shear capacities and the limiting drift. Further, while considering the elements adjacent to the wall junctions, they will have the additional stiffness as a contribution from the out-of-plane stiffness of adjoining/intersection wall, which has to be considered. A detailed investigation on this issue is mandatory for a realistic assessment of the response.

6 PROBABILISTIC CAPACITY ASSESSMENT OF A WALL OF A MONUMENTAL BUILDING USING STRUT MODEL

The probabilistic capacity assessment of one of the walls of a monumental building, made of un-reinforced brick masonry, subjected to in-plane loading is presented in this section. The strut model is used for determining the in-plane shear capacity of un-reinforced masonry walls (Lang 2002, Lang and Bachmann 2003). The unreinforced masonry wall is considered to be consisting of several piers, one per storey, separated by relatively stiff joint regions. The wall, which is a part of the monumental building, is of two-storey height with a total height (H_{tot}) of 5.84 m and a width of 1.68 m. The length, thickness and height of the masonry pier are 1.68m, 0.39m and 1.5m, respectively. In order to take into account the possible variations in material properties of masonry, the compressive strength of brick, compressive strength of mortar, modulus of elasticity of masonry and angle of internal friction of masonry are treated as random variables. The statistical properties of random variables considered are presented in Table 1. It is assumed that the random variables presented in Table 1 are independent of each other. The compressive strength of masonry perpendicular to the mortar bed is computed using the equation proposed by Haller (from Sahlin 1971) and is given by

$$f_{mx} = [(1 + 0.15 f_b)^{0.5} - 1][8 + 0.057 f_m] \tag{18}$$

where f_m is the compressive strength of mortar (kg/cm^2), f_b is the compressive strength of brick (kg/cm^2) and f_{mx} is the compressive strength of masonry perpendicular to mortar bed (kg/cm^2). The strength of masonry parallel to mortar bed is computed using the relation $f_{my} = 0.3 f_{mx}$ (Lang 2002) and the shear modulus of masonry is computed using the relation $G_m = 0.4 E_m$ (FEMA-273). Since elastic modulus of masonry, E_m is a random variable; shear modulus of masonry, G_m also will be random with mean value equal to 0.4 times the mean value of E_m and cov equal to the cov of E_m. Since the compressive strength of brick and mortar, modulus of elasticity of masonry and angle of internal friction of masonry are considered as random variables, the capacity curve for the masonry wall will be also be a random variable. Hence, a probabilistic analysis needs to be carried out for determining the capacity. In the present study, the probabilistic capacity assessment of the wall is carried out using Monte Carlo simulation technique. Two thousand samples are considered in the analysis.

Table 1 : Statistical properties of random variables considered

Random variable	Distribution	Mean	Coefficient of variation* (cov)
Compressive strength of brick (N/mm^2)	Lognormal	19.15	0.2
Compressive strength of mortar (N/mm^2)	Lognormal	4.8	0.2
Elastic modulus of masonry (N/mm^2)	Lognormal	2629	0.3
Angle of repose of mortar (Degrees)	Lognormal	36.87	0.15

(Note: * - assumed)

6.1 Results and Discussion

The mean values of the shear capacity and, yield and ultimate displacements at the top of the wall for the un-reinforced masonry wall, subjected to in-plane loading are obtained as 80.15kN, 2.49mm and 9.47mm, respectively, with values of cov as 0.041, 0.245 and 0.231, respectively. The very low value of cov of shear capacity indicate that the shear capacity of the wall is relatively independent of the variations in the compressive strengths of brick and mortar, and the modulus of elasticity and angle of internal friction of masonry. The frequency distributions of the shear capacity and, yield and ultimate displacements at the top of the wall are shown in Figs. 4-6. It is noted that while the frequency distributions of the shear capacity and yield displacement are uni-modal, that of the ultimate displacement is multi-modal.

The mean capacity curve for the unreinforced wall against in-plane loading is shown in Fig. 7. Similarly, the capacity curves for the other walls of the monumental building considered can be developed, and the capacity curve for the structure can be determined. This combined with definitions of damage grades (as given by European Macroseismic Scale, Grunthal (1998), for instance) can be used for generating the vulnerability function for the building, and for the fragility analysis Priya et al. (2005).

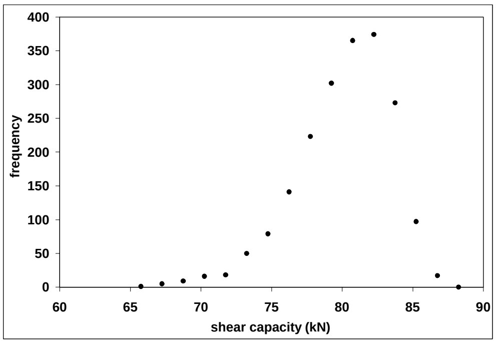


Figure 4 : Frequency distribution of shear capacity for the un-reinforced brick masonry wall

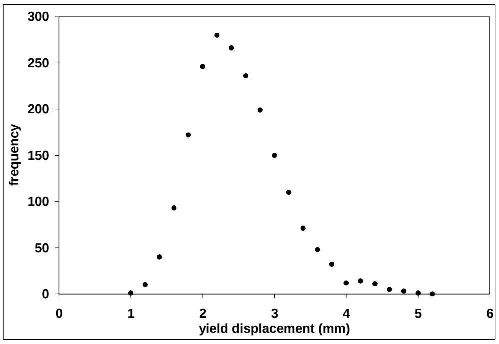


Figure 5 : Frequency distribution of yield displacement at the top of the un-reinforced brick masonry wall

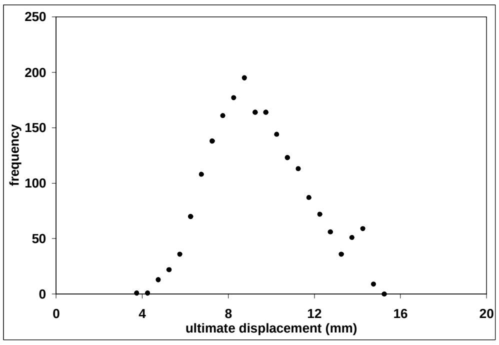


Figure 6 : Frequency distribution of ultimate displacement at the top of the un-reinforced brick masonry wall

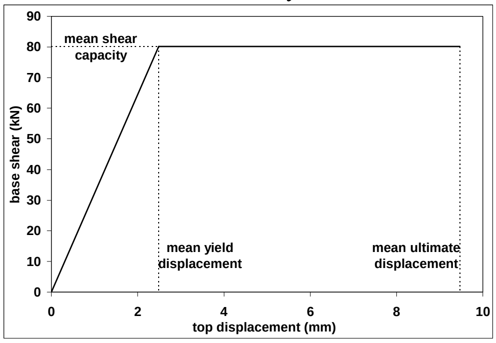


Figure 7 : Mean capacity curve for the un-reinforced brick masonry wall against in-plane loading

7 CONCLUSIONS

Based on the brief review of literature, it is noted that for in-plane lateral resistance, the formulae proposed by Magenes and Calvi (1997) are improvements over those given in Eurocode and are rational. The Indian code provisions are not comprehensive and have to be modified in accordance with various failure modes. Perhaps, adoption of the refinement of models proposed by Magenes and Calvi (1997) to Indian context could be one among the good choices. An approach integrating the limit analysis method with the dynamic analysis is presented in section 5 for seismic safety evaluation.

The probabilistic capacity assessment of one wall of a monumental building made of unreinforced brick masonry against in-plane loading using strut model is presented. From the results obtained, it is noted that the shear capacity of the wall is relatively independent of the variations in random variables considered, while the ductility (which depends on Δ_y and Δ_u) is sensitive to variations in random variables considered (namely, the compressive strengths of brick and mortar, and the modulus of elasticity and angle of internal friction of masonry). It is also noted that while the frequency distributions of the shear capacity and yield displacement are uni-modal, that of the ultimate displacement is multi-modal.

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