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Axial Load-Flexural Strength Interaction of Masonry Walls

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Abstract
In this paper, an analytical investigation of the axial load-flexural strength interaction of reinforced masonry walls is carried. The curvature ductility of masonry walls is also evaluated for masonry walls with different modes of reinforcement configurations under different levels of axial loads. Results from this paper indicate that at low axial loads masonry walls exhibit sufficient ductility to be employed in areas of medium to high seismicity
Keywords-*Reinforced masonry, axial load, flexural strength, interaction, ductility.*

Introduction

Masonry is one of the oldest construction materials in use. On basis of the constituent used for construction, masonry structures are classified as unreinforced masonry structures, reinforced masonry structures and confined masonry structures. Most houses in rural India are masonry houses built with either burnt clay brick or natural stone masonry. Technically, they are called Unreinforced Masonry (URM) Houses; as they have masonry walls with no steel reinforcement embedded in them to improve their behaviour during earthquakes. During an earthquake, unreinforced masonry (URM) walls are pushed sideways, along their length (in-plane) and thickness (out-of-plane) directions. When shaken along their length, they develop diagonal cracks along their length and/or separate at wall junctions. When walls collapse, they bring down the roof along with them. This is the main reason for large loss of lives during earthquakes that have occurred in different regions of the country. The poor performance of URM structures even under low to moderate seismicity (Sikkim 2011 and Nepal 2015) has seen it use been banned in a few countries (New Zealand) through techno-legal regulations

The other variant of masonry structures involves the introduction of both vertical and horizontal steel. The vertical steel is used to increase the flexural capacity of the wall, while the introduction of horizontal steel increases the shear capacity of the wall. However, the introduction of horizontal reinforcement increases the bed joint thickness that has a detrimental effect on the compressive strength of masonry. Nevertheless, Indian codes (IS-1905, 1987) has not formulated any design or detailing code for reinforced masonry possibly as the quality of bricks in our country, on an average are not suitable for reinforced masonry. Bricks should be of high strength, dense and needs to have low rate of moisture. A high rate of moisture absorption may lead to corrosion of the reinforcement and loss in their strength. In addition, the absence of skilled labour has limited the use of reinforced masonry in India. All the afore-mentioned reasons have limited research on reinforced masonry walls. However, research on RM walls subject to axial compression and shear has gathered momentum since the 1990s.

Reinforced masonry (RM) structural walls form the main lateral load resisting system in low to mid-rise buildings, located in moderate/high seismic regions.

Compared to unreinforced masonry walls, RM walls have higher flexural and shear strengths and are expected to efficiently resist earthquake shaking through inelastic actions. Proper design and detailing of longitudinal and transverse reinforcements helps in achieving the required strength and ductility. Two common ways of detailing longitudinal reinforcement in RC walls are:

(a) Uniform distribution of reinforcement along the length of the wall, and (b) reinforcement concentrated at the two ends of the wall (see Fig 1 A and B).

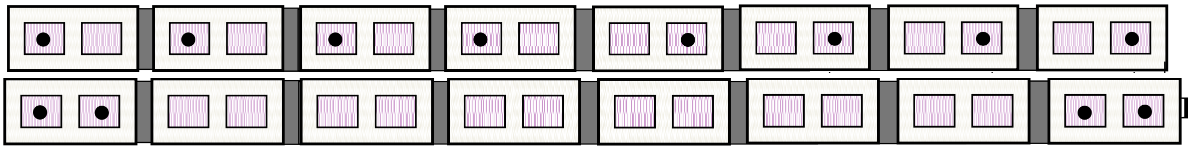

Figure 1: Detailing of longitudinal steel in RM walls

Although, in-plane flexural strength of walls with the two distributions are generally not too different for the same amount of total reinforcement, the former often results in enhanced shear strength and improved shear behaviour (displacement and ductility) as it resists the propagation of diagonal shear cracks in the wall. The flexural strength and curvature ductility of RM walls sections can be determined from their axial load-moment interaction curves and non-linear moment-curvature (*M-Φ)* curves. Curvature ductility is the general measure of ductile response of a section that depends on compressive strain of masonry, compressive strength of masonry, yield strength and strain of reinforcing steel and the level of axial load. Typically sections subject to high axial loads have very low curvature ductility as their failure is due to the crushing of masonry.

The *M-Φ* response curve must represent the effective (cracked) flexural rigidity, flexural strength, and curvature ductility of the RM section. Strain levels in masonry and reinforcement at the onset of critical damage states like cracking of masonry, yielding of reinforcement bars in tension, and compression failure of masonry are used in the estimation of flexural strength and curvature ductility. This paper proposes methods to arrive at idealized multi-linear M-*Φ* curves of RM wall sections with both uniformly distributed reinforcement along the length of the wall and reinforcement concentrated at the two ends of the wall, at different levels of axial stress.

Estimation of Axial Load, Flexural Capacity and Curvature

The axial load and the flexural strength of RM walls can be estimated from basic principles of mechanics considering equilibrium of forces, compatibility of strains and constitutive relations of materials, given in equations (1) to (3):

$$\Sigma\left(f_{sci} - f_{msci}\right)A_{sci} + f_{m,avg}tx_u - \Sigma f_{sti}A_{sti} = P \tag{1}$$

Compatibility conditions

$$\frac{\varepsilon_{m1}}{x_u} = \frac{\varepsilon_{st4}}{L' - x_u} \tag{2}$$

The design flexural compressive *σ-ε* curve of masonry is parabolic up to an ultimate strain of 0.003 with maximum compressive stress f'_m (Fig. 3.A). Also, strain limits ε_u of 0.003 used are from experimental observations (Kaushik et al. 2007). The value of 0.003 for ε_u corresponds to strain at which vertical cracks are observed in the afore-mentioned experiment. The constitutive relation of masonry is defined by its design stress-strain curve given by:

$$f_m = 0.85 f'_m \left(2\left(\frac{\varepsilon}{\varepsilon_u}\right) - \left(\frac{\varepsilon}{\varepsilon_u}\right)^2 \right) \tag{3}$$

where :

- A_{sti} is the area of reinforcement bars in i^{th} layer under tension
- A_{sci} the area of reinforcement bars in i^{th} layer under compression,

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- L' the distance of the extreme layer of reinforcement from the extreme layer in compression, $f_{m,avg}$ the average compressive stress in masonry
- f_{sti} the stress in i^{th} layer of reinforcement bars under tension
- f_{sci} the stress in i^{th} layer of reinforcement bars under compression (both estimated from stress-strain characteristics of reinforcement bar)
- f_{msci} the stress in masonry at the level of i^{th} layer of reinforcement bars under compression
- x_u the depth of neutral axis
- f'_m the compressive strength of masonry
- ε_u the compressive strain in masonry corresponding to f'_m
- and ε the ultimate strain in masonry at highly compressed edge at peak stress.

The design flexural compressive *σ-ε* curve of HYSD bars (both tensile and compressive) has three parts is described in Fig. 3B. The limiting strain corresponding to yielding of reinforcement bars given by:

$$\varepsilon_y = 0.002 + \left(\frac{0.87 f_y}{E_s}\right) \qquad (4)$$

where:

E_s: Youngs' modulus of reinforcing steel (200 GPa)

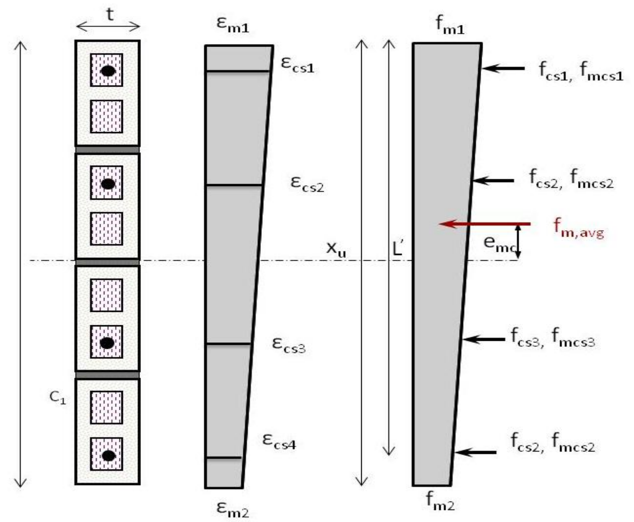
Once the stresses in the reinforcing steel and masonry are obtained, the axial load is calculated as in equation (1). The flexural strength of the RM wall is calculated by considering the moments of the tensile and compressive forces along the centroidal axis. Figure 4 A shows a typical axial load-bending moment *(P-M)* interaction (normalised to their respective capacities) of RM walls. The failure of a RM wall is defined by values of limiting states of strains in masonry and the reinforcing steel. A limit state is said to have reached when one of the following limiting strains is attained:

1. Tensile cracking of masonry: This limit state is reached when the tensile stress in the extreme edge of masonry exceeds the tensile strength of masonry (assumed to be zero in this research).
2. Yielding of extreme of reinforcement on tension side: This limit state is reached when the tensile strain in the outermost layer of reinforcement reaches a value of strain given in equation (4).
3. Crushing of masonry: This limit state is reached when the strain in the extreme layer of compression reaches the crushing strain of masonry ε_u.

Based on the above-mentioned limit states a *P-M* interaction envelope of RM sections under have three distinct regions (see Fig 4 and 5);

- Balance point: It is defined as the point on the *P-M* interaction at which the strains in concrete and steel reach corresponding limiting strains in crushing and tension simultaneously. The axial load at this point is denoted as balanced axial load P_{bal}.
- Compression failure region (above balanced point): Region on the *P-M* interaction where the failure is characterised by the strains in the extreme compressive fibre reaching the crushing strain of masonry (ε_u). The axial load in the region is greater than P_{bal}.
- Tensile failure region (below balance point): It is the region on the P-M interaction where the failure is due to the tensile strain in the extreme layer of reinforcement reaching aits yield. The axial load in this region is lesser than the balanced axial load P_{bal}.

In the following paper, *P-M* interaction and non-linear *M- Φ* curves are developed for two reinforced masonry walls (wall A with uniformly distributed steel and wall B with reinforced concentrated at the two ends of the walls. The curvature ductility of the walls has been examined at various levels of axial stress.



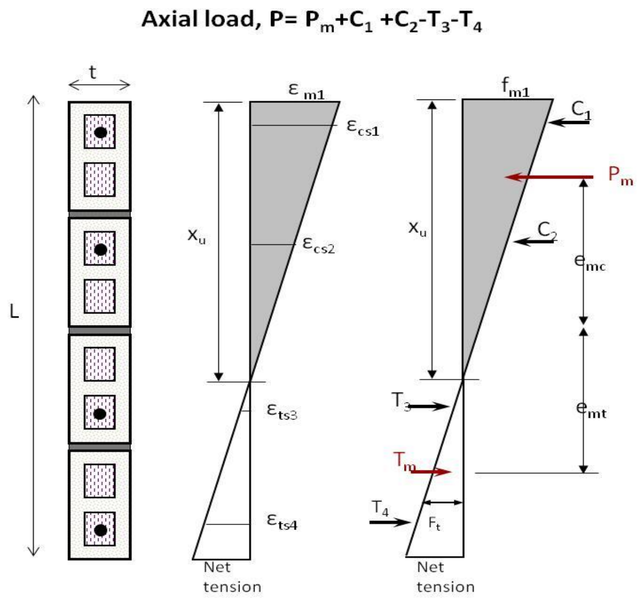


Fig. 2. Typical stress and strain distributions across rectangular RM walls A) Entire section under compression and B) Once tensile cracking commences

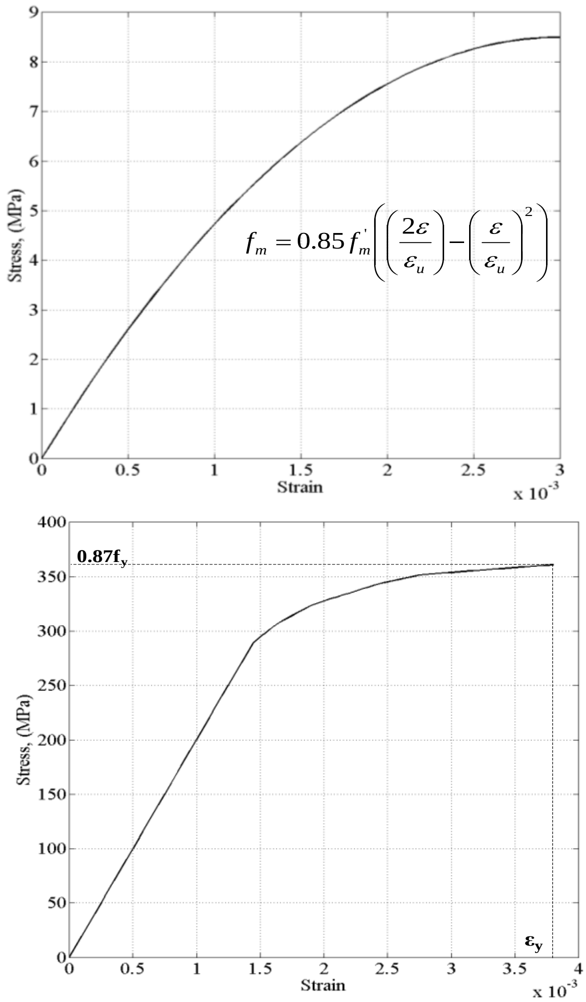


Fig. 3: Design stress-strain relation for A) masonry and B) reinforcing steel

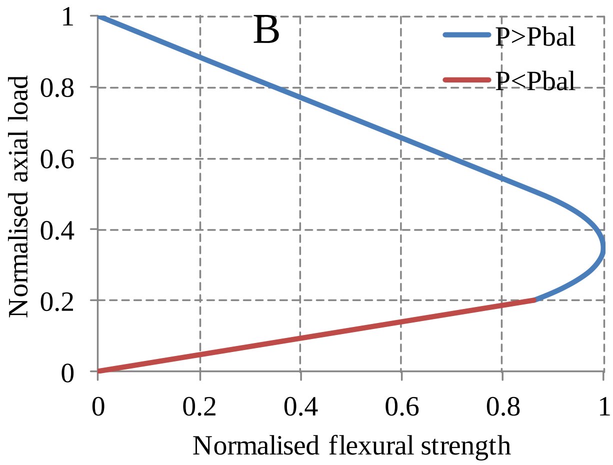


Fig. 4: Typical axial load-bending moment interaction of RM walls

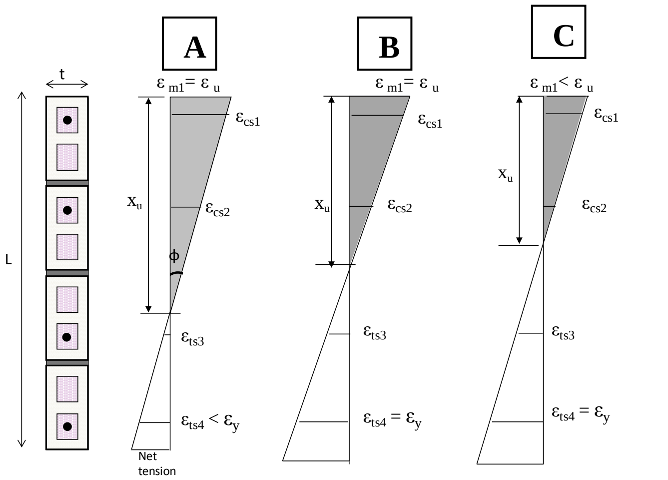


Fig. 5: Strain variations across RM sections under axial loads A) Above balance point B) At balance point C) Below axial load

Axial Load-Flexural Capacity (*P-M*) Interaction and Moment-Curvature (*M- Φ*)

The *P-M* interaction and *M-Φ* curves of two reinforced masonry walls with reinforcing steel detailing as given in Figure 1 is provided in this sections. The curves are determined from simple principle of mechanics considering equilibrium of forces, compatibility of strains and constitutive relations of materials. As per SP-7, the minimum reinforcement in each orthogonal should be at least 0.07% of the gross area of the wall. In this paper the *P-M* interaction and *M- Φ* curves of a wall with dimensions of 3270*3000*200 mm (l*h*t) are examined. The wall is constructed with concrete masonry blocks (CMU) with block dimensions

400(l)*200(w)*100(h). The area of vertical reinforcing steel is fixed as 0.1% of the gross cross-sectional area of the wall. The reinforcing steel is detailed in two ways:

a) Uniform distribution of reinforcement along the length of the wall (Wall A, see Fig. 6A), and

b) Reinforcement concentrated at the two ends of the wall (wall B, see Fig 6 B).

The *P-M* interaction of walls A and B is shown in Fig. 5. From Fig.5 and table 1, one observes that the detailing of the reinforcement does not have an effect on the axial load capacity of the wall. However, there is a 7.5% increase in the flexural strength of the wall with reinforcement concentrated at the corner.

The *M-Φ* curve of walls A and B subject to different levels of axial stresses are also investigated. In this regard, *M-Φ* curves of the walls at an axial load of $0.21P_u$ ($=P_{bal}$), $0.4P_u$ ($>P_{bal}$) and $0.1P_u$ ($<P_{bal}$) are studied and reported in Fig. 7 and table 2. The idealized bi-linear curve is obtained by equating the energy dissipation capacity of the non-linear and idealized bilinear *M- Φ* curve. So the area below the non-linear and idealized bilinear *M-Φ* curve must be the same. The moment capacity in the bilinear *M-Φ curve* does not imply the design value. An estimate of curvature ductility (μ_{Φ}) and effective flexural rigidity (defined as the initial slope of the bi-linear curve) of walls A and B are studied and reported in Table 2. The flexural rigidity is expressed as a function of the uncracked flexural rigidity. The afore-mentioned parameters are estimated at axial load demands of 0.4 P_u (above balance point), $0.21P_u$ (at balance point) and $0.1P_u$ (and below balance point). Results from fig. 6-7 and table 2 indicate that:

- Curvature ductility of walls A and B subject to high axial loads ($>P_{bal}$) are low since their failure is characterised by the compressive failure (crushing) of masonry rather than yielding of steel.
- A drop in the effective flexural rigidities is also observed at higher axial loads since the failure of walls at high axial is characterised by crushing of masonry rather than yielding of steel.
- Results also indicate that there is no effect of the reinforcement detailing on either the curvature ductility or the effective flexural rigidity.

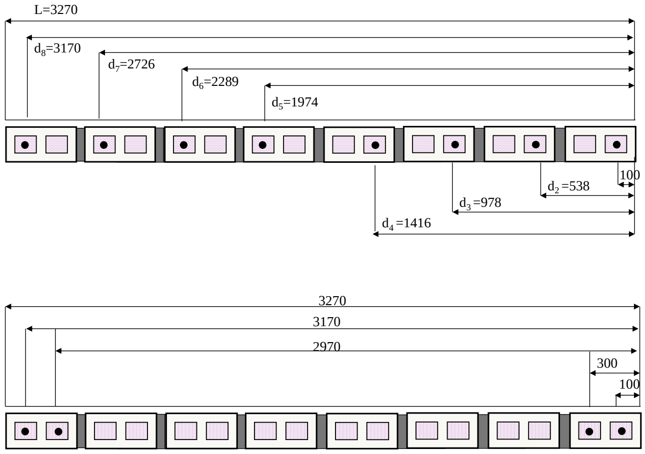


Fig. 6: Cross section of wall A uniformly distributed steel and wall B steel at corners

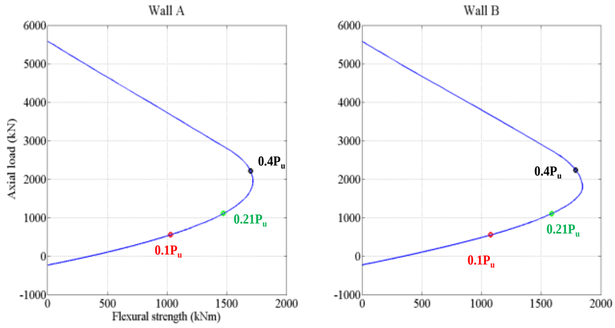


Fig. 7: Axial load –bending moment interaction diagram for the wall A and wall B

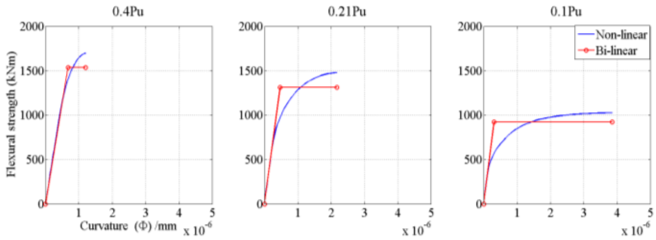


Fig. 8: Moment curvature relationship of wall A

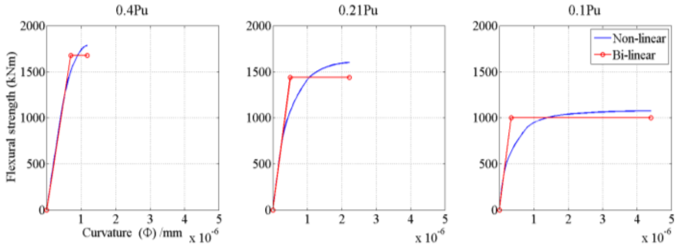


Fig. 9: Moment curvature relationship of wall B

Table 1: Axial load and bending moment capacity of walls A and B

Wall #	Axial load capacity, P_u (kN)	Bending moment capacity, M_u (kNm)	Balanced axial load, P_{bal} (kN)
Wall A	5593	1717	1174
Wall B	5589	1845	1173

Table 2: Curvature ductility and effective flexural rigidity of walls A and B

Axial load demand	Curvature ductility, (μ_Φ) (Wall A)	Curvature ductility, (μ_Φ) (Wall B)	Effective flexural rigidity, (EI_{eff}) (Wall A)	Effective flexural rigidity, (EI_{eff}) (Wall B)
0.4 P_u	1.83	1.69	0.71	0.75
0.21 P_u	4.57	4.38	0.86	0.88
0.1 P_u	12.3	12.84	0.92	0.91

Conclusions

The significant conclusions of this study are enumerated below:

- Results from this paper indicate that the detailing of flexural reinforcement in a masonry wall affects neither the curvature ductility nor the initial flexural rigidity.
- RM walls subject to low axial loads demonstrate high curvature ductility thus making a suitable substitute for RC frame elements.

References

1. Bureau of Indian Standards, Draft National Building Code – SP 7 (September 2015)
2. H.B.Kaushik, D.C.Rai and S.K. Jain, "Uniaxial compressive stress – strain model for clay brick masonry", *Current Science*, Vol. 92, No. 4, pp. 497–501, 2007.
3. Sunitha, P., Goswami, R. and Murthy, C.V.R (2016), "Idealized moment-curvature curves of RC section for the pushover analysis of R.C. frame elements", *The Indian Concrete Journal*, Vol. No 90.