SHEAR IN REINFORCED AND UNREINFORCED MASONRY: RESPONSE, DESIGN AND CONSTRUCTION

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ABSTRACT

This paper describes the vulnerability of masonry under shear; first the mechanisms of in-plane and out-of-plane shear performance of masonry are reviewed; both the unreinforced and lightly reinforced masonry wall systems are considered. Factors affecting the response of unreinforced and reinforced masonry to shear are described and the effect of the variability of those factors to the failure mode of masonry shear walls is also discussed. Some critique is provided on the existing design provisions in various masonry standards.

Keywords: masonry, unreinforced, reinforced, shear, in-plane, out-of-plane, vulnerability.

1. INTRODUCTION

Masonry is widely used in buildings either as load bearing walls or cladding/ infill walls. In most applications masonry is seldom properly designed; the construction is also not well supervised. Whilst masonry in buildings do not exhibit measurable distress under gravity loading, its failure often is widely reported even under moderate lateral loading; such failures occur both in the load bearing and in the cladding masonry elements causing significant loss of lives and damages to properties. Inappropriate interaction between the cladding walls and structural framework causes damage to building elements, especially under lateral loading. With the increase in the occurrence of extreme lateral loading events, especially in recent times, it becomes more pressing to understand the structural performance of masonry under shear than ever before. Furthermore, in spite of its popularity, many nations do not have proper masonry design standard (eg., Shaffi et. al 2001, IS1905 1987) and most universities do not teach masonry design to their graduating engineering cohorts (Hendry 2001, Drysdale and Hamid 2005). As a result of this, often load bearing and cladding masonry are detailed by draftspersons and constructed under limited engineering supervision.

With the realisation of the more frequent occurrence of extreme events, loading codes of most nations demand adequate detailing of the structural and nonstructural (including fittings) elements against the lateral loading. Unfortunately, in the absence of masonry design standards, engineers in

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many nations are left with no guidelines for the provisions of lateral restraints and their appropriate design as demanded by their respective loading standards. The purpose of this paper is to raise the awareness of the importance of proper design of masonry for lateral loading to improve the safety of buildings in general. This paper focuses on the proper understanding of the performance of masonry under shear because masonry is vulnerable to lateral loading (FEMA307 1998).

2. UNREINFORCED MASONRY

Shear in unreinforced masonry is largely resisted due to deformation in mortar bed joints and is defined conveniently by Mohr-Coulomb failure envelop that relates the shear strength τ_u to the shear bond strength τ_0, an apparent friction coefficient μ and vertical stress f_v in the joint as shown in equation (1); this linear equation is usually applicable to a small range of f_v, typically 2MPa or lower reflecting the significant nonlinearity the vertical stress can cause to masonry.

$$\tau_u = \tau_0 + \mu f_v \tag{1}$$

Where any insert is provided in masonry, such as flashing or damp proof course, τ_0 will vanish and the shear capacity of such elements is solely dependent on μ and f_v (Dhanasekar 1998). The value of μ for cyclic loading is critical in earthquake design of partition or cladding walls as μ may not have been mobilised and is usually quite low compared to its static counterpart.

In spite of its simplicity, equation (1) provides a reasonable conservative estimate of shear capacity of URM walls and hence is adopted in the European (BS5628 1985, BS-EN1986 2005) and the Australian standards (AS3700 2001). The American (MSJC 2008) and Canadian standards (CSA-S304.1 2004) relate the shear strength of masonry to the square-root of its characteristic compressive strength $\sqrt{f_m^{'}}$; the proportionality constants vary between the standards. In the MSJC(2008) the nominal shear strength of masonry is provided as $\left(3.8\sqrt{f_m^{'}} \leq 300\right)$ in the US units and in the CSA304(2004) the same is provided as $\left(0.16\sqrt{f_m^{'}} \leq 0.40\sqrt{f_m^{'}}\right)$ in the SI units. For the inplane shear strength the Canadian standard introduces $\left(0.16\left(2-\frac{M}{Vd}\right)\sqrt{f_m^{'}}\right)$ a moment-to-shear ratio $\left(0.25 \geq \frac{M}{Vd} > 1.0\right)$ in recognition of the presence of moment with shear.

3. REINFORCED MASONRY

Shear in reinforced masonry is largely resisted due to aggregate interlock in the grout, dowel action of the flexural reinforcement (perpendicular to the direction of shear) and the tensile action of shear reinforcement (parallel to the direction of shear) with no significant effect of mortar joint (Kumar and Dhanasekar 1995, Brummer and Shing 1996). Walls subjected to vertical out-of-plane bending will invariably be provided with vertical steel; horizontal shear in such walls would only derive marginal benefit of the vertical steel but will derive excellent benefit from horizontal reinforcement where provided. If the same wall is subjected to vertical shear, the vertical reinforcement will prove to be quite effective. Shear behaviour of reinforced masonry can be inferred from the theory of reinforced concrete as long as the masonry is fully grouted and reinforced - for example, reinforced masonry beams (Dhanasekar and Wong 2001). For the out-of-plane design, the

Australian, European and British Standards provide a singly reinforced formula by either limiting the maximum effective tensile reinforcement or by specifying the neutral axis parameter be less than a conservative limit far below the balanced limit (for example, 0.40 in AS3700 2001). Masonry is seldom designed as doubly reinforced as restraining compression steel is always a challenge. The North American standards define the nominal shear strength of reinforced masonry as a function of moment-to-shear ratio$\left(0.16\left(2-\frac{M}{Vd}\right)\sqrt{f'_m}\right)$ in SI units or in Imperial units as$\left(4.0-1.75\left(\frac{M}{Vd}\right)\sqrt{f'_m}\right)$ in (MSJC 2008). The CSA-S304(2004) perhaps is the only standard that includes the size effect in reinforced masonry beams; the nominal shear strength is defined for fully grouted and reinforced masonry beam is defined as as $\left(0.16\left(1.0-\frac{d-400}{2000}\right)\sqrt{f'_m}\right)$.

Unfortunately reinforced masonry is designed in many varied forms; some with fully grouted and fully reinforced in the vertical and horizontal directions at close spacing (eg. 800mm) and others with only selected cores of hollow block masonry wall is grouted and reinforced in the vertical direction with low horizontal reinforcement. Whilst fully reinforced masonry can be regarded behaving similar to reinforced concrete under shear (both inplane and out-of-plane), the partially grouted masonry behaviour under inplane shear is not well understood (Dhanasekar 1992, Dhanasekar et. al., 2010). Designers may conceive such walls as ‘mixed’ system consisting of reinforced core elements (Dhanasekar 2004, Dhanasekar and Shrive 2002 and Dhanasekar et. al. 1997) and unreinforced masonry panels spanning between the cores by naming them as “partially grouted reinforced masonry”. The wall may also be conceived as a reinforced masonry in which the spacing of walls is spaced wider than the fully reinforced masonry (for example, spacing of vertical reinforcement up to 2m) and are called as “wide spaced reinforced masonry” (Haider and Dhanasekar 2004, Dhanasekar and Haider 2004, Dhanasekar et. al. 2008 and Shrive et. al. 2009).

The construction of the reinforced masonry should be supervised and care should be taken in mixing and placing of high-slump grout into clean hollow cores provided with reinforcement tied to the started bars (Shanmugasundaram et. al. 1992, Hendry 2001 and Henderson et. al. 2006 and Dhanasekar 1992). As air could be entrapped within the grouted core space, pouring the grout slowly allowing time for escape of air from the cavity is absolutely essential. Bad practices noted at site include pushing of vertical reinforcing bars after grout is poured; such practice will effectively make the steel ineffective as the bars might not interact with the starter bars. The horizontal bars are better positioned at mid thickness of wall along the web shell as these shells are generally non-load bearing and un-mortared. Where horizontal bars are placed in the bed joint, care must be taken to sandwich them into the mortar with effective mortar cover on all sides to avoid development of stress concentration at steel – face shell interface (in cases if the bars are directly laid on face shell prior to mortaring).

4. INFILL MASONRY

Unreinforced wall panels are typically used as infill walls in flexural framed buildings; structural frame is first built with the masonry walls constructed later leaving some gaps between the framed members and the wall. In these applications, masonry is regarded as mass with its stiffness disregarded in the analyses and neither the frame nor the wall is designed for their potential interaction. Where out-of-plane loads dominate on these infill walls, they fail prematurely, potentially leaving the framed structure (where designed properly for seismic action effects) with minimal damage. However, if the seismic direction coincides with the inplane direction of the wall, the wall will not fail early, causing potential interaction with the flexible frame. Due to the high stiffness, the wall will generate higher seismic forces for which the building would not have been designed, causing significant damage to framed structures with potential for collapse of the whole building. Several failures are reported in the literature as case studies and theoretical analyses (Chiou et. al. 1999, Pujol et. al. 2008, Mehrabi et. al. 1996). Realising the vulnerability of these common buildings, most design standards now require all components, structural and non-structural, to be detailed and tied and ties designed to avoid premature failure. Where proper detailing is achieved, RC frames containing masonry infill walls perform superbly (Pujol et al 1996).

5. CONFINED MASONRY

Confined masonry is built as a system of unreinforced masonry walls confined with tension columns and beams. The columns and beams are poured after the masonry is constructed – to maximise the interaction between masonry and the surrounding concrete members. As the concrete members are non-flexible with limited reinforcement, these systems always include the wall stiffness in the seismic load calculation as such can be potentially designed appropriately avoiding unexpected catastrophic failure. As the concrete elements contain limited steel, these systems are quite economical and are widely adopted in the Central American countries; confined masonry is highly recommended in India and Indonesia where earthquakes cause significant losses fairly frequently in recent times (Brezev 2007, Franch et. al. 2008). The partially reinforced masonry system constructed of hollow concrete blocks can be regarded as a form of confined masonry.

6. RESPONSE OF MASONRY TO SHEAR

Design provisions for out-of-plane shear response are fairly non-controversial and can be designed using the Coulomb’s failure surface or using the theory of under-reinforced concrete beam shear failure for the unreinforced and reinforced walls respectively (Dhanasekar and Wong 2001).

Design of the unreinforced and reinforced walls for inplane shear is more complex; the provisions in various standards can at best be described as reflecting the poor state-of-the-art in this field. Walls under inplane shear loading are said to exhibit three modes of failure, namely, (1) diagonal shear (2) sliding shear and (3) flexural rocking. It is claimed that the diagonal shear is more brittle, although several test results exist on walls that exhibit diagonal shear failure capable of being laterally drifted beyond 1%. Generally the reinforced and unreinforced masonry shear walls exhibit mixed modes of failure. Often another potentially brittle mode of failure, corner crushing, is not considered in the design. Some older, yet unpublished, data are shown in Figures 1 and 2.

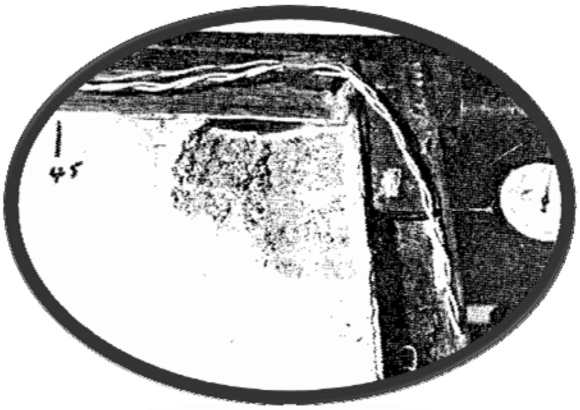

Figure 1: Corner Crushing in an Infill Shear Wall (Dhanasekar 1985)

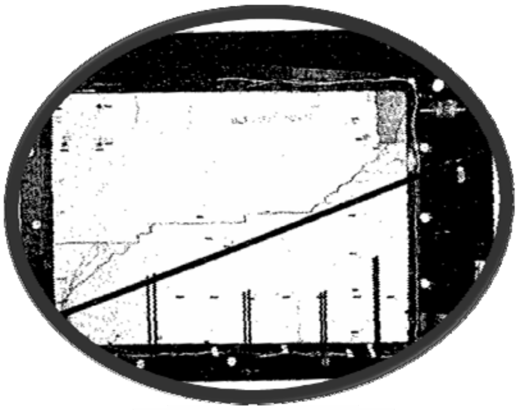

Figure 2: Repaired Infill Wall under Diagonal Shear (Dhanasekar 1985)

Figures 1 and 2 show a bone-dry infill shear panel that failed due to corner crushing and then when repaired with higher strength material (Figure 2), exhibited a combined shear failure involving diagonal and sliding shear mechanisms. Further examination of the strength properties of masonry (tensile bond, shear bond and compressive) affected by the effect of drying has revealed significant (75%) increase in the tensile and shear bond strengths whilst the compressive strength remained relatively unaffected due to drying (Table 1). The examination involved testing of 24 triplets for shear bond strength, 16 couplets for direct tensile bond strength and 24 four high prisms for compressive strength. Specimens were moist cured for 28 days in an environmental chamber and then left to dry at ambient conditions in the lab. Four to six specimens were tested at the age of 14 days, 28 days, 77days and 105 days.

Table 1: Effect of drying on strength properties of masonry

Age/ Moist/ Dry		*Compressive Strength (MPa)*	*Shear-bond Strength (MPa)*	*Tensile-bond Strength (MPa)*
14 days (Moist)	Mean	8.57	0.75	0.23
	CoV	17%	31%	31%
28 days (Moist)	Mean	10.3	0.84	0.35
	CoV	15%	23%	17%
77 days (Dry)	Mean	8.99	1.47	0.61
	CoV	15%	27%	22%
105days (Dry)	Mean	10.5	1.49	0.62
	CoV	5%	27%	26%

From Table 1 it can be inferred that the strength properties of the type of masonry (clay brick & 1:1:6 mortar) that is well moist cured prior to drying has exhibited no major effect of age, but the effect of drying after proper curing is readily visible on the two bond properties (tensile & shear).

The compressive strength, whose failure depends on the crushing / cracking of fired clay bricks, has not shown any significant effect of drying / age. The masonry failure surface, defined by these three strength properties of masonry, usually consider 28 day moist strength properties; no one to date have carefully considered the effect of drying to the properties of masonry and that of the masonry shear walls. When the shear and tensile bond strength values are increased by 175% whilst the compressive strength are retained at original level, it was possible to predict the localised compression failure at the corners as the mid zone of the wall was able to resist higher lateral load due to increased shear bond strength.

The design standards reflect the poor state-of-the-art of the shear wall failure mechanisms as they provide widely differing design clauses with extremely unconservative/ unsafe provisions in many places and too conservative/ uneconomical clauses in others. Through a comparative study of various masonry standards using 75 test data on fully reinforced masonry shear walls reported in the literature, Dickey and Lissel (2009) determined that the North American codes outperform the Australian, European and British codes in terms of economic design – whilst all standards were conservative. The situation becomes rather more complex when partially grouted/ wide spaced reinforced masonry walls are considered. With limited data on 12 full-scale wide spaced reinforced masonry shear walls, Dhanasekar et al (2008, 2010) have shown that the provisions in CSA-S04 and AS3700 are unsafe. It is therefore clear that whilst unreinforced and fully reinforced masonry walls can be designed safely using the provisions in most design standards, it appears safe design may not be achieved through standard provisions for the partially reinforced/ wide spaced reinforced masonry walls. Further research is urgently needed to examine the inplane shear behaviour of partially reinforced masonry walls; this is especially so because these walls are quite economical and can prove to be an adequate structural system for those countries that experience moderate, intra-plate type earthquakes and/ or cyclonic wind. These walls can be analogised to the much celebrated confined masonry system that has large unreinforced masonry panels surrounded by tension columns and beams (that confine the masonry).

7. CONCLUSIONS

Shear behaviour of masonry has been broadly reviewed in this paper in the context of existing design provisions and construction techniques. The following conclusions have emerged:

- Unreinforced and fully reinforced masonry behaviour under out-of-plane shear is reasonably well understood and can be designed with confidence.
- Inplane shear behaviour of masonry in general is not fully well understood; all masonry walls are thought to fail due to diagonal shear/ sliding shear/ rocking flexure, of which the diagonal shear is believed to be more brittle than the other two modes of failure – although contradicting results are reported in the literature.
- Inplane behaviour of fully grouted and reinforced (both vertically and horizontally at close spacing – eg., 800mm) walls can ignore the effect of mortar joints and be designed using the

principles of reinforced concrete. Most design standards predict the behaviour of such walls conservatively, with the North American standards (MSJC 2008; CSA-S304 2004) exhibiting more economical outcomes.

- Inplane behaviour of partially grouted and reinforced masonry walls is poorly understood; some of the design provisions in AS3700 (2004) and CSA-S304 (2004) are unsafe. As this form of construction is quite economical, there is a pressing need to urgently revise the current provisions so that safer designs can be achieved.
- Confined masonry, reported to have been performed well in moderate earthquakes, can be regarded as a form of the partially reinforced masonry; its behaviour require further study. Any new design provisions for this form of masonry system can improve the safety of the existing buildings of this form of masonry construction.
- Shear and tensile bond strengths can increase dramatically when masonry is bone-dry whilst its compressive strength will remain relatively unaffected. This can cause highly brittle corner crushing mode of failure – especially if masonry walls are designed to transfer concentrated lateral loading at corner rather than distributed by the rigid diaphragm.

8. ACKNOWLEDGMENTS

The author acknowledges a range of undergraduate and research students who have contributed to the development of the ideas/ data reported in the paper. Recent deliberations with Professors Page (Newcastle, NSW), Shrive (Calgary, Canada) and Drysdale (McMaster, Canada) as well as industry partners Alan Pearson (CMAA) and Wayne Holt (AdBri Masonry) are also fondly acknowledged.

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