SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

Non-Linear Modelling and Seismic Analysis of Masonry Structures
-A Review

Jacob ALEX[1] and Arun MENON[2]

ABSTRACT

The paper reviews current non-linear modelling approaches for investigating the seismic behaviour of masonry. Non-linear analysis of masonry has been in use as a seismic assessment tool. The storey-shear mechanism was the first attempt towards capturing the non-linear behaviour of masonry and it is the basis of analysis in many a modelling tool. However, macro-element modelling (equivalent frame models), which evolved from the storey shear mechanism approach, has consequences in global analysis due to certain assumptions. These issues are identified in this paper in an attempt to increase the efficiency of simple, non-linear tools for seismic assessment of masonry structures.

1. INTRODUCTION

Masonry has been used in a wide variety of forms as a basic construction material for public and residential buildings for thousands of years. Evidently, the existence of well-preserved masonry structures shows that masonry structures can resist loads and environmental impacts quite efficiently.

It is a well-known fact that in the design of any structure, the loads that cause damage are accidental live loads, especially seismic loads. Structural walls, which are the primary seismic load-resisting elements, will be damaged if they are not adequately designed to withstand inelastic deformation and to dissipate energy. The behaviour of the masonry matrix (mortar + unit) is complex and typically, loses its linearity at 30-40% of its elastic limit (Kaushik et al., 2007). Mortar joints, under the action of combined loads exhibit different directional properties making them a distinct plane of weakness. Therefore, a study on the seismic behaviour of masonry structures is not complete without recourse to non-linear analysis.

The role of non-linear analysis for masonry seriously considered since the Friuli earthquake in Italy. In 1978, Tomaževič suggested the use of an equivalent static, non-linear method called the storey shear mechanism (SSM) based on an equivalent frame modelling. This procedure involves the generation of a force-displacement ("pushover") curve that describes the overall behaviour of the structure ranging from its elastic state, degradation or damage and finally to collapse.

The significance of the storey-shear mechanism can be explained from a simple example given below. Consider a two-storied unreinforced masonry structure located in seismic zone V (IS-1893, 2002) in India. The seismic demand on the shorter wall, with a response reduction factor (R) of 1.5, as per an equivalent static analysis is found to be 146 kN. The allowable shear stress calculated using an elastic stress approach as per IS 1905 (1987) obtained was within the permissible limit, $\tau = 0.1 + 0.167 f_d$, where f_d is the axial stress due to dead load. However, a non-linear assessment of the structure using SSM shows that the shorter wall would have a shear capacity of 86 kN making it

[1] Ph.D. Scholar, Dept. of Civil Engineering, Indian Institute of Technology Madras, Chennai 600036, India
[2] Assistant Professor, Dept. of Civil Engineering, Indian Institute of Technology Madras, Chennai 600036, India

unsafe as it is lesser than the computed demand. The wall is expected to have a flexural mechanism of failure with an ultimate drift of 24 mm. However, such an assessment is based on significant assumptions on the diaphragm flexibility, spandrel behaviour and out-of-plane action, explained in detail later, which lead to a certain inconsistencies. This paper reviews the issues seen in the modelling approaches and explores possible solutions by which they could be overcome.

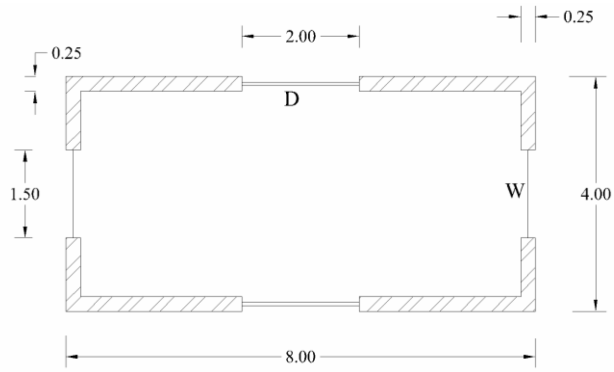

Figure 1: Ground floor and first floor plan of the structure (dimensions in m)

2. REVIEW OF THE MODELLING APPROACHES

The lateral load-resisting elements in masonry structures are the shear walls in the orthogonal directions connected with floors. The strength and ductility of the walls maintain the gravity load bearing capacity as long as its lateral load capacity is not significantly degraded (i.e. a loss less than 20 %). A structure can dissipate seismic energy in two mechanisms: namely the out-of-plane (local mechanism) and in-plane damage (global mechanism) (Magenes and Menon, 2009). In accordance with performance-based earthquake engineering concepts, non-linear static analysis is widely used for the seismic verification. SSM determines the relationship between the resistance and relative storey drift of the ground storey walls. SSM, which forms the basis of several non-linear tools (e.g. TREMURI, SIMPLE POR, RAN) relies on the in-plane resistance of the wall, but neglects the contribution of their out-of-plane capacity. Macro-element modelling is used in these tools where by a wall with openings was divided into panels called piers (*vertical elements*), spandrels (*horizontal elements)* and rigid nodes (*pier-spandrel joint element).*

The in-plane behaviour of masonry piers, initially studied by Turnsek and Cacovic (1971) and Tomazevic and Lutman (1988) and later by Magenes and Calvi (1997), identified that in-plane failure of a wall may be due to flexure mechanism, and shear mechanism (sliding shear and diagonal tension) as seen in Figure 2. The in-plane shear capacity of the wall, $\mathbf{V_u}$ is the minimum of the three closed-form solutions in Table 1.

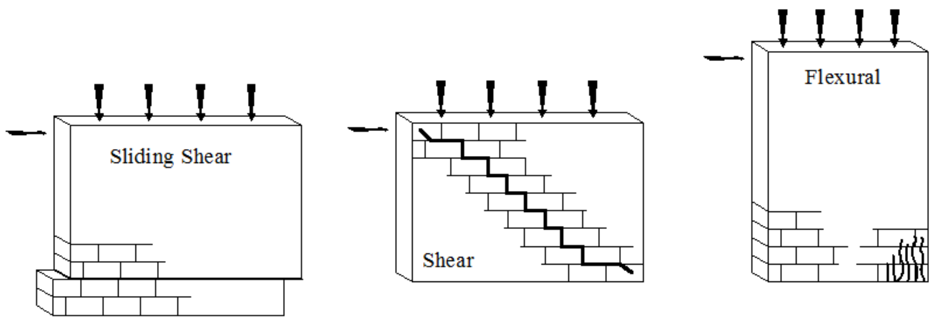

Figure2: In-plane failure pattern of masonry walls (Tomaževič, 1999)

The modelling approach was based on certain assumptions:

- Rigid horizontal floor diaphragm action, ensures that the lateral displacements are same at every point on the diaphragm and the lateral forces are distributed to the walls in accordance to their in-plane stiffness. Axial forces in the piers are not expected to vary under seismic action.

- The deformed configuration of the structure under seismic action is assumed to take the shape of its first mode (in the non-linear range).
- The contribution of a pier to resist lateral forces depends on the expected mechanism in the wall. After its ultimate displacement has been attained the wall does not resist lateral forces.
- The spandrel is assumed as rigid, elastic coupler that undergoes no damage and transfers the moment from the pier. This could lead to the pier having a fixed-fixed boundary condition.
- The out-of-plane capacity of walls is neglected, purportedly on the conservative side.

Table1: In-plane capacity of a masonry pier (Magenes and Calvi, 1997)

FAILURE MODE	EXPRESSION	TERMINOLOGY
Flexure	$V_u = \frac{\sigma l^2 t}{2h}\left(1 - \frac{\sigma}{\kappa f'_m}\right)$ (1a)	σ: axial stress on the pier l: length of the pier t: thickness of the pier f'_m: compressive stress of masonry κ :Stress block factor for converting parabolic stress profile to a rectangular one h: height of the pier f_{tu}: tensile stress of masonry β: aspect ratio (depends on boundary condition) c: cohesion property μ: coefficient of friction α_V: shear span ratio
Principal tension	$V_u = \frac{f_{tu} lt}{\beta}\sqrt{1 + \frac{\sigma}{f_{tu}}}$ (1b)	
Sliding shear	$V_u = lt\left(\frac{1.5c + \mu\sigma}{1 + 3(\alpha_V c/\sigma)}\right)$ (1c)	

These closed-form strength equations provide reliable estimates of in-plane capacity with varying axial load, while depending only on mechanical properties of masonry (as a homogeneous material) and geometry of a wall (Figure 3). There is research interest in developing robust non-linear tools, based on such ultimate strength equations for the seismic evaluation of masonry structures.

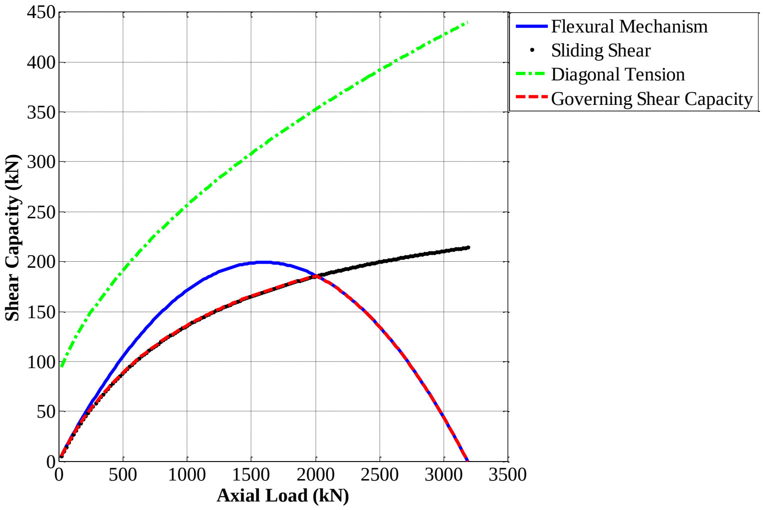

Figure 3: Axial load-shear capacity interactions for a 1.5x3x0.25 m masonry pier

2.1 Non-linear modelling approaches

The modelling approaches adopted generally include (Zucchini and Lourenço, 2004):

Detailed micro-modelling: Brick units and mortar in the joints are represented by continuum elements whereas the unit–mortar interface is represented by discontinuum elements;

Simplified micro-modelling: Expanded units are represented by continuum elements, whereas the behaviour of mortar joints and unit–mortar interface is lumped in discontinuum elements;

Macro-modelling: Units, mortar and unit–mortar interface are smeared out in homogenous continuum.

Micro-modelling studies provide a better understanding about the local behaviour of masonry. The great interest received in the micro-modelling of masonry in recent years derives mainly from the efforts made by an increasing number focus on preserving historical and monumental constructions. Masonry joints act as planes of weakness and the explicit representation of the joints and units in a numerical model is a logical step towards developing a rigorous analysis tool. Finite element methods have provided accurate results not only in the pre-peak and post-peak behaviour but also with respect to force and displacement capacities. But studies by Augenti and Parisi (2009), Calderini and Lagomarsino (2008) and Augenti and Romano (2008) show that these approaches are cumbersome and requires a higher computational effort, limiting their use to component level studies.

Hence, several methods based on the macro-element or equivalent frame approach have been developed, where a homogenised material model is used. The results obtained, were comparable to the sophisticated non-linear finite element analysis, albeit the computational load. Various program and mathematical models that have been developed include Simple Pushover Response (POR) method (Tomaževič, 1978), RAN (Augenti and Parisi, 2009), SAM (Magenes, 2000) and TREMURI (Lagomarsino et al., 2013 and Penna et al., 2013) among others. The macro-element method models a perforated wall by dividing them into bi-dimensional elements called macro elements. The macro-element has (Augenti and Parisi,2009):

1) To adequately represent elastic section and mechanical properties of the masonry panel;
2) To be able to adequately evaluate lateral strength associated with the main failure modes (shear and flexure-rocking);
3) To be able to provide a consistent cyclic response under lateral loading.

In such tools, masonry piers and spandrels are modelled as 2d elements, with eight degrees of freedom (Galasco et al., 2004), idealised with a bilinear force-displacement hysteresis. The resisting mechanism is typically governed by in-plane response of walls (shown in Figure 2). Collapse mechanisms due to out-of-plane response are neglected in all these tools. The spandrel elements are modelled as pier elements rotated 90 degrees. The ultimate displacements of the pier and spandrels are empirically determined as 0.4-0.8% for a shear mode of failure and 0.8-1.2% for a flexure mode. The tools also assume that a rigid diaphragm action prevails. In TREMURI, however it is possible to vary the in-plane stiffness of the diaphragm (Penna et al., 2013). The assumption of rigid diaphragm action may hold true for new masonry structures with RC floors. Nevertheless, for existing structures with timber or timber-masonry floors, such an assumption will not hold.

3. ISSUES IN NON-LINEAR MODELLING TOOLS

From the above, it becomes evident that there are possible shortcomings in the modelling tools used in the context of assessment of existing structures, particularly:

- Neglecting out-of-plane failure (treated often through separate local mechanism analysis by limit methods) and the effect of biaxial demand in walls (in-plane and out-of-plane),
- Inadequate representation of spandrel behaviour,
- Not considering suitable diaphragm stiffness or reliance on rigid diaphragm assumption.

3.1. Biaxial Effects in Masonry Walls.

A major drawback while using these approaches for seismic assessment is that only the pure in-plane strength of the wall is considered, despite the fact that masonry structures are notorious for out-of - plane failure under seismic action. While it is possible to obtain the seismic demand on the wall by considering load combinations (e.g. EL_x+0.3EL_y or EL_y+0.3EL_y), a rational method has not yet been proposed to describe the out-of-plane effects on the in-plane capacity of a member under seismic response. Post earthquake inspections of unreinforced masonry structures have shown that out-of-plane mechanisms are very common place, and may compromise the global capacity.

Theoretical studies by Doherty et al. (2002) have determined the out-of-plane resistance of a wall subjected to an out-of-plane bending. A tri-linear relationship was developed to characterize the non-linear force displacement relation of an unreinforced brick masonry wall. The initial bi-linear model developed was found to be an over estimation of the out-of-plane capacity. The non-linear bending of the mortar reduced the capacity. However, at large deformations little variation from the bi-linear behaviour was seen.

Theoretical studies developed a way to represent the out-of-plane resistance of an unreinforced masonry wall as the minimum of two kinematic mechanisms (Felice and Giannini, 2001) that include,

1. Detachment of the facade from the transversal wall due to a vertical crack at the connection;
2. Collapse due to the opening of a diagonal crack on the transversal walls.

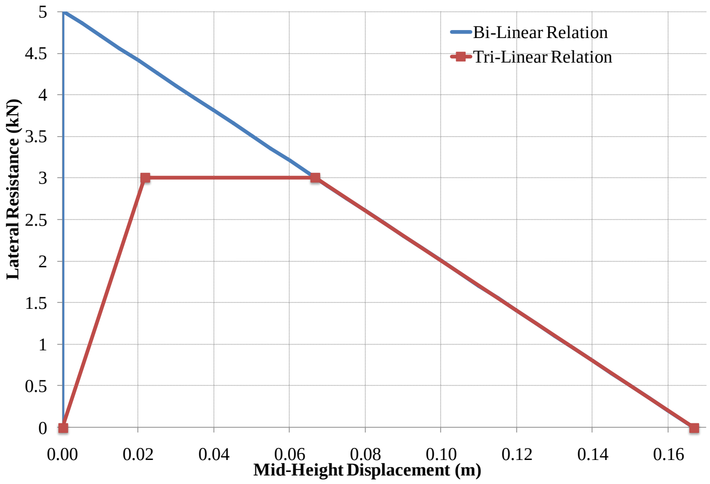

Figure 4: Tri-linear out-of-plane force-displacement relation (Doherty et al., 2002; Griffith et al., 2003)

This phenomenon addresses the behaviour of walls in upper stories. The ease of use and its accuracy has lead to its introduction in TREMURI. But this method could only be used in structures with regular plan and elevation, with a particular failure pattern (Lourenço et al., 2011).

Several bidirectional tests by Flanagan and Bennet (1999) including out-of-plane test with uniform load, in-plane loading followed by out-of plane uniform load, out-of-plane drift followed by in-plane loading and combined out-of-plane air bag and in-plane loading were performed on structural clay tile infilled steel frames. A significant out-of-plane stability under combined loading was noticed due to arching mechanism. But in traditional load bearing masonry walls, this increase in stability may not be seen due to the absence of rigid supports that facilitate arching action.

Studies by Najafgholipour et al. (2013) developed an interaction surface between the in-plane and out-of-plane capacities of masonry walls using both force-based experimental and theoretical studies. To obtain the interaction, further tests were conducted in which an in-plane diagonal test of the wall was conducted at varying percentages of out-of-plane capacity. However, the in-plane capacity of the wall specimen (0.6x0.6x0.1 m) was obtained by conducting an in-plane diagonal compression test. This ensured that the wall would fail in shear by developing diagonal cracks. However, a wall of aspect ratio '1.0' is more often than not expected to fail in either in flexure or by bed joint sliding. In addition, it remains to be seen how effective an in-plane diagonal test of a wall is when it comes to simulating seismic behaviour when compared to a lateral load test under axial load.

Therefore, it is necessary to develop a better theoretical understanding as to how the in-plane seismic behaviour of a masonry wall may be affected in the presence of an out-of-plane displacement**.** This interaction will be useful as the closed form solutions use to determine the capacity in Table 1 is valid only while considering pure in-plane action. The first step to this was to develop bi-axial interaction surface of masonry according to Bresler and Pister (1958). Figure 5 drawn as per Eq. 2 shows the biaxial interaction:

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha} + \left(\frac{My}{M_{uy1}}\right)^{\alpha} = 1 \qquad (2)$$

where M_{ux}: Design flexural load in the x direction under bi-axial loading
M_{uy}: Design flexural load in the y direction under bi-axial loading
M_{ux1}: Design flexural strength in the x direction under pure axial loading
M_{uy1}: Design flexural strength in the y direction under pure axial loading
α: factor depending on the load capacity

Tomaževič (1999) showed that there was a reduction of the axial load bearing capacity of the structure due to secondary displacements, a phenomenon that is seen due to bending at large axial loads. This was an important relationship as most of the works have identified the out-of-plane displacement as an effect of the inertial load (e.g. Flanagan and Bennet, 1999; Najafgholipour et al., 2013). Tomaževič derived the relation for a wall supported on both sides. It was seen that walls with low strength wall failed due to crushing rather than by secondary effects. In this paper, the effect of secondary displacements was examined for a wall that was free at the top. The effect of secondary displacements is more in a cantilevered wall (Figure 6a and 6b).

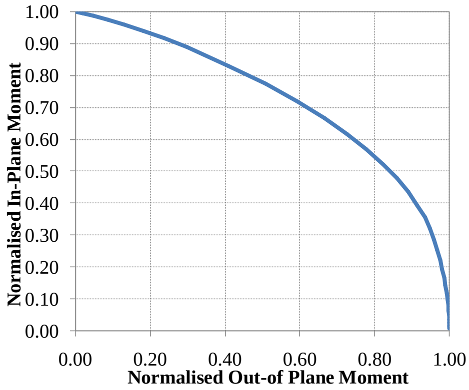

Figure 5: Bi-axial moment interaction

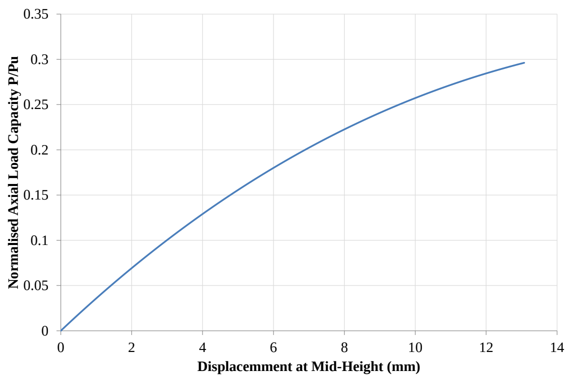

Figure 6a: Secondary effects in a fixed-fixed Wall (Failure due to crushing)

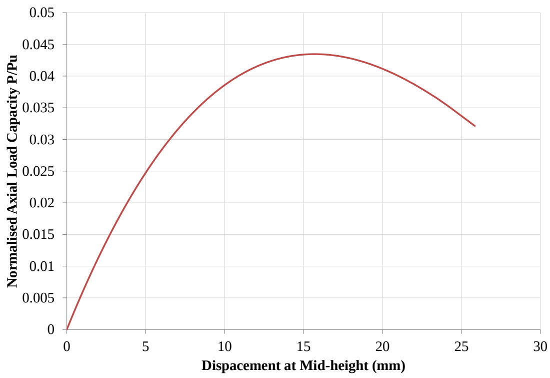

Figure 6b: Secondary effects in a cantilevered wall (Failure due to secondary effects)

It is seen that the in-plane closed form solutions in equations (1a)-(1c) are directly proportional to the axial load bearing capacity and determining this drop in the axial capacity to account for the effect of out-of-plane displacement is a possible step. However, this paper accounts for the out-of-plane displacement in differently. The effect of out-of-plane eccentricity is considered. The effective thickness of the wall changes as the eccentricity of the axial load is sequentially increased.

Effective thickness, $t_c = 3\left(\frac{t}{2} - e\right)$, if $e \geq \frac{t}{6}$ (3a)

and $t_c = t$ if, $e \leq \frac{t}{6}$ (3b)

where t is the thickness of the wall

After determining the effective thickness, the variation of the capacity at a given axial load can be determined with varying eccentricity using equations 1a-1c. The effect of increasing eccentricity on the variation of the capacity at an axial load of 210kN for aspect ratios of 1.5, 1.0, and 0.75 is determined. The shear capacity does reduce with an increasing eccentricity (Figure 7a-7c). The in-plane flexural and shear capacity drops to zero at eccentricities almost equal to half the thickness.

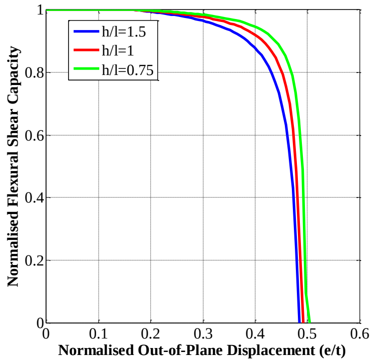

Figure 7a: Reduction of flexural capacity with out-of-plane displacement

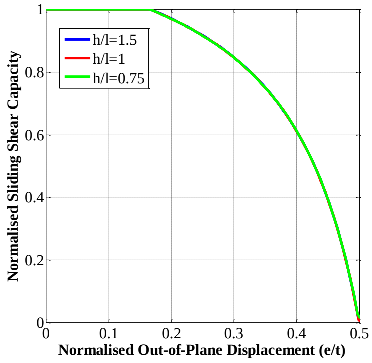

Figure 7b: Reduction of sliding-shear capacity with out-of-plane displacement

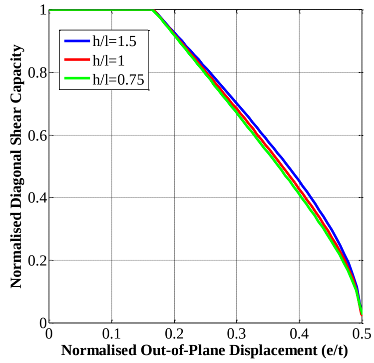

Figure 7c: Reduction of diagonal shear capacity with out-of-plane displacement

3.2 Spandrel Strength and Stiffness

In order to obtain reliable information about the global seismic resistance it is important that the model selected be capable of representing the basic features of the behaviour at both structural and component level. Masonry shear walls, solid or pierced with openings represent the basic structural elements of a masonry structure resisting seismic loads. In masonry walls, spandrels connect the walls and transfer seismic forces. Depending on the size and proportions of openings and on the mechanical properties, either the spandrel is stronger than the pier as the case in any unreinforced masonry structures or vice versa. It is in this regard that the spandrel is classified as a secondary element and the storey-shear mechanism assumes that the spandrel acts as a rigid coupler.

The response of spandrel elements has not been studied in great detail despite the fact that they affect the seismic behaviour of the structure. Firstly, damage in the spandrel contributes to a significant energy dissipation of the structure. It also alters the boundary conditions of the piers, which will alter the global response of the structure. International codes adopt simplified models (weak spandrels-strong piers and strong spandrels-weak piers) that make the modelling of spandrels superfluous. The hypothesis of strong and rigid spandrels may be true in the case of new structures. However, examination of existing structures shows that spandrels undergo damage (Lourenço et al., 2011). Even in more complex modelling techniques (equivalent frame idealisation), the spandrels are modelled as rotated piers adopting the same failure criteria. This leads to a predominance of flexural failure (Cattari and Lagomarsino, 2008). The strength of the spandrels in that case is given by equations (4a) and (4b).

$$V = \frac{N l_{sp}}{2}\left(1 - \frac{N}{\kappa f_{cu} t l_{sp}}\right) \qquad (4a)$$

$$V = \frac{H_{sp} l_{sp}}{2}\left(1 - \frac{H_{sp}}{0.85 f_{hu} t l_{sp}}\right) \qquad (4b)$$

where σ_{sp} : is the axial stress on the spandrel;
l_{sp}: is the length of the spandrel;
t: thickness of the spandrel;
f_{cu}: vertical compressive strength of masonry;
f_{hu}: horizontal compressive strength of masonry;
N: Axial load acting on the spandrel element.
H_p: Minimum of the axial load provided by a tensile resistant element and $0.4 h_{sp} t_{sp} f_{hu}$
Equation (4b) is used in the presence of a tensile resistant element like an R.C tie beam.

Post-earthquake observations show that the main failure modes of spandrels include: i) rocking and ii) diagonal cracking. Cattari and Lagomarsino (2008) developeda model based on the in-plane failure pattern of spandrel. They concluded that the failure was due to the shear failure of the horizontal mortar joints and developed a relation: $M_u = f\left(N, \eta, \mu_t, \mu_c\right)$. (4c)

Where M_u = Ultimate moment capacity of the spandrel
N = Axial load of the spandrel.
η = Ratio of tensile strength to compressive strength
μ_t,μ_c = Ductility in tension and compression in the micro-model considered.

The use of this model showed an increase in the global capacity of the structure than when the strength of the spandrel was determined as per equation (4a). This was due to the fact that when the strength was determined as per equation (4a), the moderate axial stress on the spandrel leads to poor coupling of the piers. The model developed was able to simulate the shear failure of the spandrels.

A review of the existing spandrel strength formulations by Beyer and Mangalathu (2012) concludes that experimental force-deformation relationships of spandrels showed two different levels of strength; a peak strength and a residual stength. They also concluded that the existing formulae addressed only plane spandrels and more research was required to quantify the contribition of lintels and arches as they affected the residual strength of the spandrel.

Table 2: Closed form strength solutions for spandrel as per (Beyer and Dazio, 2012)

FAILURE MODE	EXPRESSION	TERMINOLOGY
Flexure	$V_u = (f_t + p_{sp})\dfrac{h_{sp}^2 t_{sp}}{6}\dfrac{2}{l_{sp}}$ (5a)	p_{sp}: axial stress on the spandrel l_{sp}: length of the spandrel t_{sp}: thickness of the spandrel f_t: tensile strength of head joints h_{sp}: height of the pier f_{bt}: diagonal tensile stress of masonry α_v: shear span(depends on boundary condition) c_p: cohesion property μ_p: coefficient of friction
Shear cracking through joints	$V_u = \dfrac{f_{bt} h_{sp} t_{sp}}{1+\alpha_v}\sqrt{1+\dfrac{p_{sp}}{f_{bt}}}$ (5b)	
Shear cracking through the units	$V_u = \dfrac{2}{3}\left(c_p + \mu_p p_{sp}\right)h_{sp} t_{sp}$ (5c)	

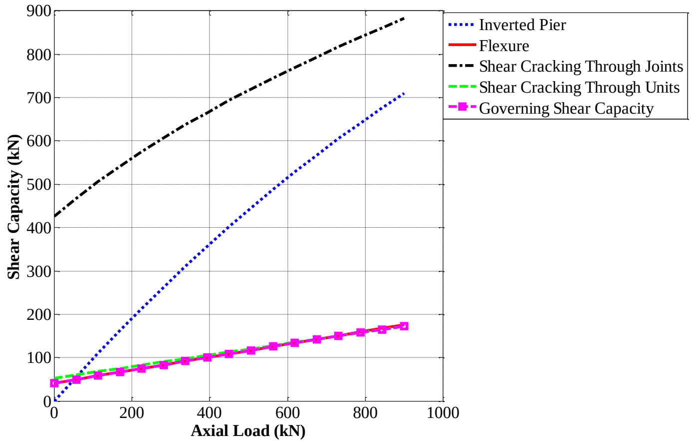

Figure 8: Comparison of the spandrel strength solutions in Eq. (4a) and (5a)-(5c)

Experimental studies by Beyer and Dazio (2012) on four spandrels developed relationships for the force-deformation relationships of masonry spandrels subject to a deformation limit. These tests helped to obtain ceratin closed-form solutions that described both the shear as well flexural failure of masonry spandrel. This paper reviews the closed form solutions given in (4a) and (5a-5c) for a spandrel of dimension 1.5x0.9x0.25 m. The closed-form solution in (4b) is not taken into consideration as this paper does not analyse structures with a tensile resistant element. From the graphs (Figure 8), it can be seen that consideration of the spandrel as a rotated pier leads to overestimation of the capacity of the spandrel especially at moderate to high axial loads. There is nearly a variation of two times at an axial load of 200kN and this goes on increasing with the axial load. This uncertainty regarding the behaviour of the spandrel element is an area of further research.

3.3 Diaphragm Flexibility

A fundamental assumption in the storey shear mechanism is that the walls are suitably anchored to the floor elements. The primary function of a floor slab is to (1) support and transfer gravity loads to the load bearing elements and (2) to connect all the vertical elements at the floor level and transmit lateral forces (wind/earthquake) to them in proportion to their stiffness. This is called diaphragm action. The European Macroseismic Scale (EMS) identifies rigid diaphragm action as one of the factors to be satisfied for the ideal behaviour of the structure. A rigid diaphragm holds together the walls when subjected to ground shaking there-by improving its seismic behaviour and preventing out-of-plane failure. Flexible diaphragms deform in-plane when subjected to lateral loads and are incapable of transmitting torsional forces. The distribution of lateral loads to vertical wall elements takes place in proportion to the tributary area associated with each wall element for vertical loads distribution. The presence of a flexible diaphragm is identified as an important weakness from post earthquake surveys around the world (Senaldi et. al, 2012). This section reviews the effect of diaphragm flexibility from works conducted earlier and examines the effect of diaphragm flexibility using the non-linear tool 3MURI. The closed form equations (1a – 1c) given in Table, 1 only hold good as a meaningful assessment tool if the diaphragm is rigid enough to provide box action. However many prevailing masonry structures have semi-rigid diaphragms, that may or may not be well connected to the walls and the equations may not be a proper representation of the pier capacities in that case.

A survey in 1994 Cohen et al. (2004) concluded the chief deficiency in nearly 4500 seismically vulnerable buildings (mainly URM structures) was the flexible diaphragm. The storey drift can only be used as a damage potential index for buildings with a rigid diaphragm. Rather, for buildings with flexible diaphragms, it is related to the in-plane drift of the diaphragm. So the concept of drift ratio was introduced for the seismic evaluation of masonry structures with flexible diaphragms (Cohen et al., 2004). The diaphragm drift ratio was given by,

$$DDR = \frac{\Delta_{diaph}}{\frac{L}{2}} \qquad (6)$$

Where Δ_{diaph} is the in-plane deflection of the diaphragm; and

L is the plan length of the diaphragm.

The DDR was used for the development of a 2-DOF analysis tool to evaluate the response of the low rise masonry structures with flexible diaphragms (Cohen et al. ,2006). Experimental studies conducted by Abram and Tena-Colugna (1996) proved that the effect of the flexibility of the diaphragm did alter the dynamic properties of structures. Tests were conducted on three structures i.e.

a. Two-storied box structure with flexible roof displacement;
b. Two-storied office building with the roof diaphragm is more flexible than the first floor diaphragm; and
c. A seven storied hotel.

A MDOF system based on the linear behaviour of masonry was developed to estimate the dynamic response of structures with and without rigid diaphragm. The studies proved that the lateral displacements did decrease by 1.55 times with increase in the floor rigidity. It was also found that the lateral accelerations dropped (drop of 54 %) with increasing flexibility of the floor element. What was interesting to note was the diminishing effect of the torsional mode as the flexibility increased.

A very recent work by (Lourenço et al.,2011) explored methods of analysing masonry structures without box action using the micro-model approach for modelling. Analysis of three heritage structures was conducted to determine the most appropriate analysis method that would consider the effect of flexible diaphragm. It is seen the limit analysis was an effective tool to determine capacities of structure with regular plan and elevation and a recognizable damage pattern. Results from tests conducted also showed that the non-linear static method results did not conform to the non-linear dynamic results that simulated the damage sustained in earlier earthquakes. Another alternative was to conduct a simultaneous adaptive pushover analysis in the longitudinal and transverse direction in the relation of 100% and 30%. This, however, lead to concentration of damage in the lintels. It was concluded that neither the static or dynamic non-linear methods simulated the effects out-of-plane displacement properly.

Analytical works (Giongo et al., 2012) show that the diaphragm stiffness does not influence the force and displacement capacities for regular 2 storied structures. The diaphragm was modelled as per the ABK formulations developed in 1984 and then its shear stiffness was determined, while masonry is modelled as an elastic-no-tension model. A non-linear static analysis of the same structures concluded that the effect of roof flexibility was significant when an "additional eccentricity of 2 m" to the mass centre was provided.

3MURI allows a user to account for flexible diaphragms floor elements. The floor elements are identified by a principal direction young's modulus E_1, young's modulus E_2 in the perpendicular direction, Poisson ratio υ and G_{12} the shear modulus. The modulus of elasticity E_1 and E_2, account for the degree of connection between the walls and the diaphragm and the shear modulus influences the tangential stiffness and the horizontal force transferred among the walls (Lagomarsino et al., 2013). The orthotropic matrix for the floor element is determined as shown,

$$D = \begin{bmatrix} \frac{E_1}{1-e\nu^2} & \frac{e\nu E_1}{1-e\nu^2} & 0 \\ \frac{e\nu E_1}{1-e\nu^2} & \frac{eE_1}{1-e\nu^2} & 0 \\ 0 & 0 & G_{12} \end{bmatrix} \quad (7)$$

Where $e=E_2/E_1$

A non-linear static analysis of two structures (symmetric and un-symmetric, wall thickness 250 mm) with a rigid and timber floor is done in 3MURI. The value of *e* is varied from 0.1 to 1 to see the difference in response.

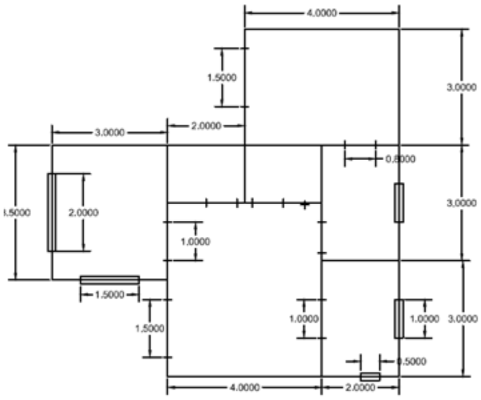

Figure 9a: Model A (asymmetric)

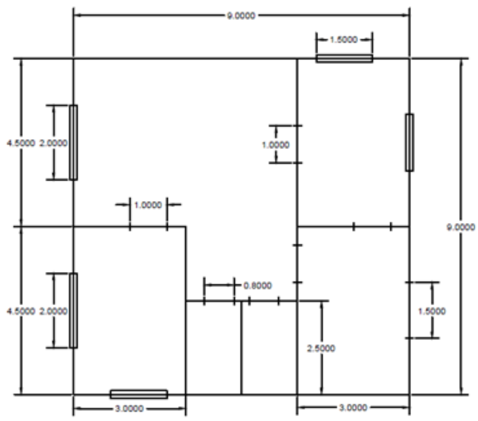

Figure 9b : Model B (symmetric)

From the results shown, it is seen that the effect of changing the value of *e* is more evident in unsymmetrical structures. There is an increase in both the shear and displacement capacities with the increase of *e* from 0.1 to 1 for asymmetric structures with rigid floor. However, in an asymmetric structure with timber floor while the shear capacity increases with *e,* the displacement capacity decreases. This trend however is not seen in symmetric structures where both the capacities remain the same. However, the difference in both shear and displacement capacities in all the 4 cases were not significant (< 10%). Nevertheless, in a structure with a flexible roof such as timber, out-of-plane displacements cause local failure which compromises the global capacity. The analysis results obtained from 3MURI do not capture this reduction of the global capacity and the treatment of the flexible diaphragm is an area of future research.

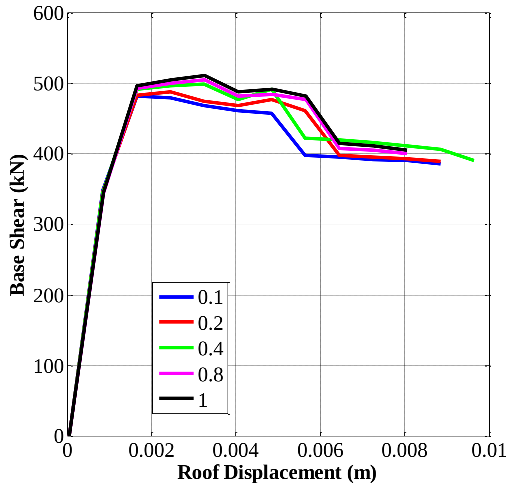

Figure 10a: Model A with timber floor

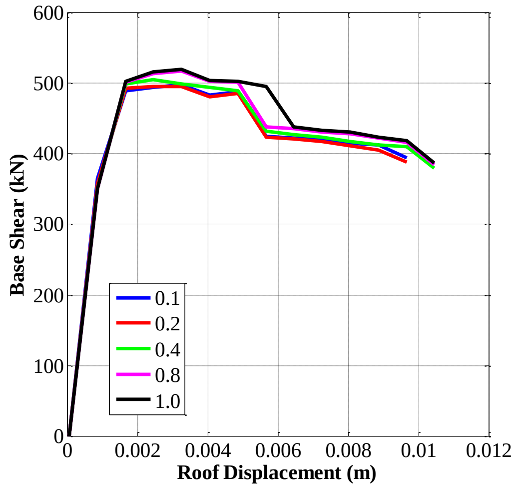

Figure 10b: Model A with rigid floor

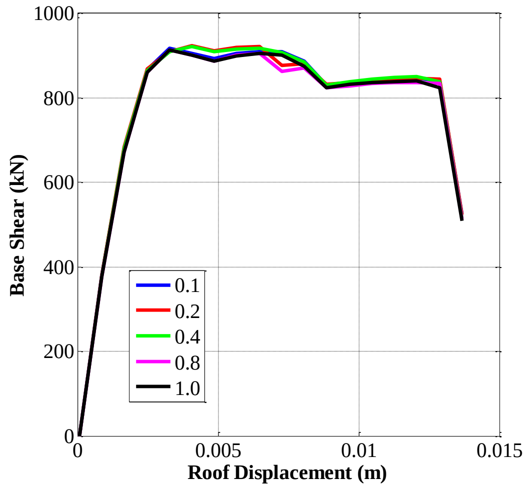

Figure 10c: Model B with timber floor

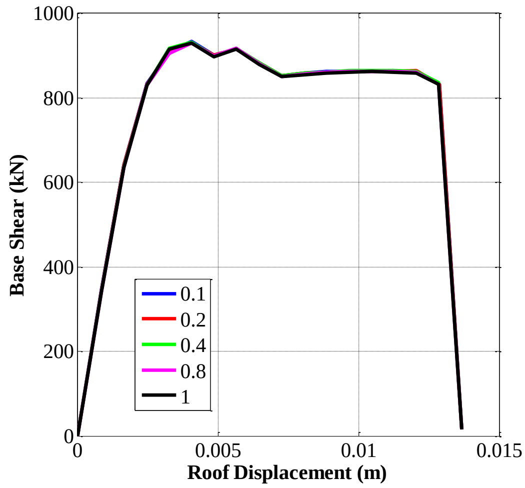

Figure 10d: Model B with rigid floor

4. CONCLUSIONS

This paper reviews the modelling tools available for non-linear analysis of masonry structures. From research works conducted in the past it is concluded that the assumptions made about the effects of out-of-plane displacements, spandrel behaviour and diaphragm flexibility may not be completely acceptable and leads to inconsistencies in estimation of the global capacity.

This work presents a relation between the out-of-plane displacement and the in-plane shear capacity at a given load. From the results obtained the in-plane capacity of the structure will drop with increasing out-of-plane eccentricity and becomes zero at an eccentricity equal to half the thickness. However, the out-of-plane displacement is represented as the varying eccentricity and there is scope for further research.

The closed-form solutions for estimating the spandrel strength have also been reviewed. Comparison between the two approaches shows that there is an overestimation in the capacity of the spandrel in the approach that is used in the analysis tools (TREMURI, RAN etc).

The analysis of a four-storied structure using 3MURI does show a change in both shear and displacement capacities of the structure with varying diaphragm stiffness. However, as the change in the capacities was seen to be below 10 %, the treatment of the flexible diaphragm problem in 3MURI is not fully captured and this is an area that requires further research.

5.REFERENCES

Abram Daniel P and Tena-Colugna Arturo (1996) “Seismic Behaviour of Structures with Flexible Diaphragms”,*Journal of Structural Engineering* 122(4): 439–450

Augenti, N, and F Parisi (2009) “New Tools for Non-Linear Analysis of Masonry Buildings”, In *14th European Conference of Earthquake Engineering*, Ohrid

Beyer, K, and A Dazio (2012) “Developing Force-Deformation Characteristics of Brick Masonry Spandrels in Historic Buildings”, In *15th World Conference on Earthquake Engineering*, Lisbon

Bresler, B, and K Pister, (1958) “Strength of Concrete Under Combined Stresses ”,*Journal of the American Concrete Institute*,321-345

Calderini, Chiara, and Sergio Lagomarsino, (2008) “Continuum Model for In-Plane Anisotropic Inelastic Behavior of Masonry”,*Journal of Structural Engineering* (February): 209–220

Cattari, S, and S Lagomarsino, (2008) “A Strength Criterion for the Flexural Behaviour of Spandrels in Un-Reinforced Masonry Walls,” In *The 14 World Conference on Earthquake Engineering*, Beijing

Cohen, Gregory L, Richard E, Klingner, John R, Hayes, and Steven C, Sweeney, (2004) “Seismic Evaluation of Low-Rise Reinforced Masonry Buildings with Flexible Diaphragms: I, Seismic and Quasi-Static Testing”,*Earthquake Spectra* 20(3): 779–801

Cohen, Gregory L,, Richard E, Klingner, John R, Hayes, and Steven C, Sweeney, (2006), “Seismic Evaluation of Low-Rise Reinforced Masonry Buildings with Flexible Diaphragms: III, Synthesis and Application”,*Earthquake Spectra* 22(2): 329–47,

Doherty, K, M, C, Griffith, N, Lam, and J, Wilson, (2002), “Displacement-Based Seismic Analysis for out-of-Plane Bending of Unreinforced Masonry Walls”,*Earthquake Engineering and Structural Dynamics* 31(4): 833–50

Felice, Gianmarco De, and Renato Giannini, (2001), “Out-of-Plane Seismic Resistance of Masonry Walls”,*Journal of Earthquake Engineering* 5(2): 253–71

FEMA 306, (1998), *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings*

Flanagan, R.D., Bennett, R.M,"Bidirectional behavior of structural clay tile infilled frames", *Journal of Structural Engineering*, 125(1999), 236-244

Giongo, I, M Piazza, A Rizzardi, C Rodegher, and R Tomasi, (2012), “Seismic Evaluation of URM Buildings with Flexible Diaphragms , Proposal of a Simplified ‘ ENT ’ Method,” In *15th World Conference on Earthquake Engineering*", Lisbon

Graziotti, F., Magenes, G., Penna, A., & Fontana, D. (2011), "Experimental cyclic behaviour of stone masonry spandrels", In *15th World Conference on Earthquake Engineering,* Lisbon

Bureau of Indian Standards 1893(PART 1):2002, (2002), *Criteria for Earthquake Resistant Design of Structures*

Bureau of Indian Standards 1905:1987, (1987), *Code of Practice for Structural use of Un-Reinforced Masonry*

Lagomarsino, Sergio, Andrea Penna, Alessandro Galasco, and Serena Cattari, (2013), “TREMURI Program: An Equivalent Frame Model for the Nonlinear Seismic Analysis of Masonry Buildings”,*Engineering Structures* 56: 1787–99

Magenes, Guido, (2000), “A Method for Pushover Analysis in Seismic Assessment of Masonry Buildings”, In *Twelfth World Conference on Earthquake Engineering*, Wellington, 1866–75

Lourenço, P. B., Mendes, N., Ramos, L. F., & Oliveira, D. V. (2011), "Analysis of Masonry Structures Without Box Behavior", *International Journal of Architectural Heritage*, *5*(4-5), 369–382

Magenes, Guido, and Gian Michele Calvi, (1997), “In-Plane Seismic Response of Brick Masonry Walls”,*Earthquake Engineering and Structural Dynamics* 26(February): 1091–1112

Magenes, Guido, and Arun Menon, (2009), “A Review of the Current Status of Seismic Design and Assessment of Masonry Buildings in Europe”,*Journal of Structural Engineering* 35(6):247-256

N, Augenti and A, Romano, 2008, “Seismic Design of Masonry Buildings Using Macro-Elements”, In *14th International Brick and Block Masony Conference*, Sydney, 38–47

Najafgholipour, M A, Mahmoud R Maheri, and P B Lourenço, (2013), “Capacity Interaction in Brick Masonry under Simultaneous in-Plane and out-of-Plane Loads”,*Construction and Building Materials* 38: 619–626,

Penna, Andrea, Sergio Lagomarsino, and Alessandro Galasco, (2013), “A Nonlinear Macroelement Model for the Seismic Analysis of Masonry Buildings”,*Earthquake Engineering and Structural Dynamics*:1787-1789

Senaldi, I, Magenes, G, Ingham, J M, (2012), "The seismic performance of unreinforced stone masonry buildings during the 2010-2011 Canterbury earthquake sequence", In *15th World Conference on Earthquake Engineering,* Lisbon

Tomaževič, Miha, (1999), Earthquake Resistant Design of Masonry Structures, Series on , Imperial College Press, pp 179-200