CHAPTER III

IDEALIZATION OF MASONRY INFILL WALLS AND BEHAVIOR MODES

3.1 Introduction

Masonry is a term covering a very wide range of materials such as adobe, brick, stone and concrete blocks and each of these materials in turn varies widely in the form and mechanical properties. It is a non-homogenous and anisotropic composite structural material. Masonry may also be used with or without reinforcement or in conjunction with other materials. Various construction forms include from non-mortared stacked stone blocks to carefully mortared walls designed for ductile response under seismic attack.

Masonry structures of substantial size can be designed to perform adequately under major earthquakes, provided that careful design and detailing requirements are followed. As masonry is a comparatively brittle material, it is generally necessary to design for higher seismic forces than that required for other materials. Its behavior is not perfectly elastic even in the range of small deformations. Even when lateral deformation of the wall is kept constant during a given time interval, changes in resistance and crack distribution can be observed during the test in the non linear range. Behavior of infill frame systems subjected to in-plane lateral forces is influenced by mechanical properties of the frame and infill materials, stress or lateral deformation levels, existence of openings in the infill and the geometrical proportions of the system. Existence of an initial gap between the frame members and the infill also influences the behavior of the system. The infills do not participate as part of the primary structure on the assumption that, precautions are taken to avoid load being transferred to them. It is evident from the frequently observed diagonal cracking of such infill walls that the approach is not always valid. Hence such cases require incorporation of modified mode of behavior for the frame and design of walls.
3.2 Components of Infill Wall

Infill frame elements are made up of infill-panel and frame components. Infill panels are categorized according to the material and geometric configuration. Clay brick is the most common and traditional type of infill. Most often it is un-reinforced except in modern buildings where it may be reinforced, at still a relatively modern form of un-reinforced masonry infill construction hollow clay tile (HCT) is used. Hollow concrete block laid up with mortar is also commonest form of using the concrete masonry unit (CMU). CMU may be left hollow or filled with grout. Concrete with usual minimal reinforcement is also used as infill.

Infills have a wide variety of geometric configurations. Aspect ratios (length/height) for infill panels usually vary from approximately 1:1 to 3:1 with most ranging from 1.5:1 to 2.5:1. For the sake of partition and/or facade suiting, infills can be configured in varied forms. There may be eccentricity between the frame components and infill axes. Based on geometric configuration there are two categories for infill panel components - solid panels and panels with openings. Doors and windows are the two most prevalent opening types.

The frame components are categorized primarily by material. Concrete frames are most common forms of construction: they may be classified as either ductile or non-ductile. Ductile detailing requires closely spaced transverse hoops in the beams, columns and connections. When infills are present, shear force demands are considerably higher leaving the beam or column vulnerable to shear failure. The presence of masonry infill affects the seismic behavior of buildings in the following ways (Penelis and Kappos, 1997) -

- As a consequence of increase in stiffness of buildings, the fundamental period is decreased and the base shear is increased.
- The lateral stiffness in plan and elevation is modified.
- The structural system is relieved of seismic action as part of the load is carried by the infills.
- Energy dissipation capacity of the building is substantially increased.
The more flexible the structural system, the greater the above effects of the infills. It is a common misconception that masonry infill in structural steel or reinforced concrete frames can only increase the overall lateral load capacity. Earthquake damage can be traced to structural modification of the basic frame by so-called nonstructural masonry partitions and infill panels. Masonry infill can drastically alter the intended structural response, attracting forces to parts of the structure that have not been designed to resist them (Paulay and Priestley, 1992).

The high degree of uncertainties on the analysis of buildings due to effects of infill includes -

- The variability of their mechanical properties and hence the low reliability in their strength and stiffness.
- Tightness when connected to the surrounding frame (wedging condition).
- The potential modification of their integrity during the use of the building.
- The non-uniform degree of their damage during the earthquake.

Thus the safety of the structure cannot rely, not even partly, upon the infills and only their probable negative influence is taken into account (Penelis and Kappos, 1997).

3.3 Behavior and Failure Modes of Infills and Frames

Behavior of infill frame systems subjected to in-plane lateral forces is influenced by mechanical properties of the frame and infill materials, stress or lateral deformation levels, existence of openings in the infill and the geometrical proportions of the system. Existence of an initial gap between the frame members and the infill also influences the behavior of the system.

Under small deformations the stiffness behavior is dominated by the panel stiffness characteristics (FEMA 306, 1998). Hence the events that define the shape of the force deformation curve are bed-joint sliding, diagonal tension, corner crushing, general shear failure and out-of-plane failure. As the deformation increases, the panel characteristics will be a function of its element properties. Stair-stepped pattern of cracks through head and bed joints will result when the masonry units are strong
relative to the mortar. When the mortar is stronger than the units, rather a rare case, cracks will develop through the units as well as the mortar. With the stair-stepped cracks, shear can continue to be resisted after cracking by the development of a compressive stress normal to the bed joints, characterized as a compression strut. If the mortar is weak relative to the units, an infill panel may crack along the bed joints instead of along the diagonal. When the infill panel is sufficiently strong in shear, the compressive stress at the compression corners will fail in crushing. The large forces generated in this mode will be distributed to the beam and column members and may result in either column or beam shear failures (Meharbi et al., 1996).

The following are failure modes recognized in masonry infill frames -

3.3.1 Bed-Joint Sliding Shear Failure

Sliding shear failure through bed joint of a masonry infill is associated with infill with weak joints and strong members. Masonry infill cracks along horizontal mortar joints and separates into several parts. Bed-joint sliding is likely to occur when the bounding frame is strong and flexible. Separated parts of masonry infill permit free deformation of columns, ultimately resulting in plastic hinging of columns at the joints. Short column effects may takes place if the mortar beds are relatively weak compared to the adjacent masonry units, especially bricks. A plane of weakness forms usually near the mid height level of the infill panel. There is really no limit to the displacement capacity of this behavior mode (Paulay and Priestley, 1992).

![Fig.3.1 Sliding shear failure of masonry infill (Paulay and Priestley, 1992)](image-url)
3.3.2 Diagonal Cracking

This usually occurs if the masonry is strong and the contact between the masonry and frame is good. A windward column, supported by the infill, fails in shear. Transverse to the principal compression formed across the diagonal of an infill strains are the tension strains. Diagonal cracks are formed as a result of the tensile strain exceeding the cracking strain of the infill panel material. These cracks commence in the center of the infill and run parallel to the compression diagonal. The cracks tend to propagate until they extend from one corner to the diagonally opposite corner. Diagonal cracking behavior usually signals the formation of a new diagonal strut behavior mode.

![Image of diagonal cracks](image.png)

**Fig. 3.2** Masonry Failure with X-shaped cracks due to the R/C frame interstorey drift (Penelis and Kappos, 1997)

3.3.3 Corner Compression Failure

This is because of the high stress concentrations at each corner of the compression diagonal. Corner crushing is located over a relatively small region for strong/stiff columns and beams; whereas for weaker frames, especially concrete frames, corner crushing is more extensive and the damage extends into the concrete frame itself. This mode of failure is gradual and is accompanied by a rapidly increasing rate of deflection, therefore, the collapse may be assumed to be due to a plastic type of infill failure.
3.3.4 Out of Plane Failure

The failure is due to ground shaking transverse to the plane of a wall. Out-of-plane failure may occur in the upper storeys of high-rise buildings, where the floor accelerations are basically resonance amplifications of prominent sinusoidal ground motions. In lower storeys, when out-of-plane shear is combined with high in-plane storey shears, infill panels tend to progressively ‘walkout’ of the frame enclosure.

3.3.5 Premature Failure of Frame Elements or Connections

Failures of columns, beams and connections due to compressive ‘strut’ are reactions imparted to them by the masonry infill. Another mode of failure of frame elements is the failure of the tension or compression chords of the infill frame acting as a monolithic flexural element. The latter is predominant in case of slender infill frames.

3.4 Modeling of Masonry Infill Walls

Unless masonry infill walls are adequately isolated from the concrete frame members and the floor above, masonry elements shall be included in the model of the physical structure. The mathematical model of the physical structure shall represent the spatial distribution of mass and stiffness of the structure to an extent which is adequate for the calculation of the significant features of its dynamic response.

Masonry exhibits distinct directional properties due to the influence of mortar joints acting as planes of weakness. Depending upon the orientation of the joints to the stress directions, failure can occur in the joints alone or simultaneously in the joints and blocks. The great number of the influencing factors, such as dimension and anisotropy of the bricks, joint width and arrangement of bed and head joints, material properties of both brick and mortar and quality of workmanship, make the simulation of plain brick masonry extremely difficult. The following two levels of refinement for masonry models can summarize the analytical procedures.
• Micro-modeling (Masonry as a multi-phase material)

According to this procedure the masonry units, the mortar, and the interface, are modeled separately. While this leads to more accurate results, but as the level of refinement is high the analysis will be computationally intensive, and so will limit its application to small scale laboratory specimens (Asteris et al. 2003).

• Macro-modeling (Masonry as one-phase material)

According to this procedure no distinction between the individual masonry units and joints is made, and masonry is considered as a homogeneous, isotropic or anisotropic continuum. While this procedure may be preferred for the analysis of large masonry structures, it is not suitable for the detailed stress analysis of a small panel, due to the fact that it is difficult to capture all its failure mechanisms. The influence of the mortar joints acting as planes of weakness cannot be addressed. (Reinhorn et al.1994).

The macro-model of an infill frame structure shall include the stiffness effects of the infill as a pair of diagonal strut in the bays of the frame. The diagonal struts shall be considered as resisting only compression axial loads. Their lines of action shall intersect with the beam-column joints. The secant stiffness of the force-displacement relationship shall be used to determine the effective area of the diagonals.

The load resisting mechanism of infilled frames was idealized as a combination of a moment resisting frame system formed by the frame and a pin-jointed truss system formed by the infill panel. The analysis requires the determination of the geometry and hysteretic rule parameters from theoretical or empirical models (Stafford Smith 1962).

The method developed by Saneinejad and Hobbs (1995) takes into account the elastoplastic behavior of infill frames considering the limited ductility of infill materials. Various governing factors such as the infill aspect ratio, the shear stresses at the infill-frame interface and relative beam and column strengths are accounted for, in this development. However, the formulation furnishes only extreme or boundary values for design purposes.
The control parameters to calibrate a macro-model are obtained using experimental data or micro-models which would simulate the real behavior. For the analysis where the emphasis is on evaluating the overall structural response, macro-models can be substituted for micro-models without substantial loss in accuracy and with significant gains in computational efficiency.

3.5 Control Parameters of the Model

Numerous parameters that govern the behavior of infills can be grouped into mechanical, geometrical and empirical parameters to represent the masonry. (Reinhorn et al. 1997)

The mechanical parameters include the strength of masonry units, elastic modulus of masonry and strains in the masonry panels.

The geometrical parameters include determination of the width of the diagonal strut, contact length of the frame and infills and the area of the strut.

3.5.1 Compressive strength and Stress Strain Behavior of Masonry

Masonry is typically a non-elastic, non-homogeneous and anisotropic material composed of two materials of quite different properties, viz. stiffer bricks and relatively softer mortar. Under lateral loads, masonry does not behave elastically even in the range of small deformations. Masonry is very weak in tension because it is composed of two different materials distributed at regular intervals and the bond between them is weak (Hendry et al. 1996). The compressive strength \( f_m \) is the parameter that mainly controls the resistance of the strut and has to be distinguished from the standard compressive strength of the masonry by taking into account the inclination of the compression principal stresses and the mode of failure expected in the infill panel. Considering the shear and normal stresses in the bed joint and neglecting the axial stress parallel to the bed joints the following equation was given by Crisafulli, (1997) so as to determine the compressive strength.

\[
\bar{f}_c = f_m \sin^2 \theta
\]  

(3.2)
Under the lateral loads, masonry does not behave elastically even in the range of small deformations. Masonry is very weak in tension because it is composed of two different materials distributed at regular intervals and the bond between them is weak. Therefore, masonry is normally provided and expected to resist only the compressive forces. During compression of masonry prisms constructed with stronger and stiffer bricks, mortar of the bed joint has a tendency to expand laterally more than the bricks because of lesser stiffness. As mortar is confined laterally at the brick-mortar interface by the bricks because of the bond between them, the shear stresses at the brick-mortar interface result in an internal state of stress that consists of triaxial compression in mortar and bilateral tension coupled with axial compression in bricks that lead to the failure of the prisms. Since masonry is an assemblage of bricks and mortar, it is generally believed that the strength and stiffness of masonry would lie somewhere between that of bricks and mortar.
Fig. 3.4 Stress-strain behavior of masonry prism (Paulay and Priestley, 1992)

The combined effects of lower modulus of elasticity and higher Poisson’s ratio result in a tendency for lateral mortar tensile strains to greatly exceed the lateral masonry unit strains. (Paulay and Priestley, 1992)

Friction and adhesion at the mortar masonry interface constrains the lateral strains of mortar and masonry unit to be equal. There will be set up of self-equilibrating lateral forces between compression in the mortar and tension in the masonry unit (Fig.3.5).

Fig. 3.5 Failure mechanisms for masonry prisms (Paulay and Priestley, 1992)

The resulting tri-axial compressive stress state in the mortar enhances its crushing strength, while the combination of longitudinal compression and lateral biaxial tension in the masonry unit reduces its crushing strength and induces tendency for vertical splitting. The strength of the confined mortar may be approximated by
following equation (Paulay and Priestley, 1992) -

\[ f_{c,j} = f'_{cb} + 4.1f_l \]  \hspace{1cm} (3.3)

Where,

- \( f'_{c,j} \) - Compressive strength of confined mortar,
- \( f'_{cb} \) - Uniaxial compressive strength of masonry unit,
- \( f_l \) - Lateral compressive stress developed in the mortar.

Using linear failure criterion (after Hilsdorf, 1969)

The strength of the masonry is given by -

\[ \frac{f_x}{f'_{cb}} + \frac{f_y}{f'_{cb}} = 1 \]  \hspace{1cm} (3.4)

Where,

- \( f'_{cb} \) - Biaxial tensile strength of masonry unit,
- \( f_x \) - Lateral tensile stress at failure,
- \( f_y \) - Axial compressive stress at failure.

![Mohr’s failure criterion for a masonry unit](image)

Fig.3.6 Mohr’s failure criterion for a masonry unit (Paulay and Priestley, 1992)
By considering the transverse equilibrium requirements of a mortar joint of thickness $t$, a tributary height of masonry unit equal to one-half a masonry unit above and below the joint in conjunction with equations (3.3) and (3.4), the longitudinal stress $f_p'$ that causes failure is found to be (Paulay and Priestley, 1992) -

$$f_p' = f_y = \frac{f'_{cb} (f'_{ub} + \alpha f'_{ub})}{U_u (f'_{ub} + \alpha f'_{cb})}$$

(3.5)

$$\alpha = \frac{j_i}{4.1h_u}$$

(3.6)

Where,

- $f'_j$ - compression strength of mortar bed,
- $h_u$ - height of the masonry unit,
- $U_u$ - Stress non-uniformity coefficient ($U_u = 1.5$ Hilsdorf, 1969).

**Fig.3.7 Transverse Equilibrium of masonry unit and mortar in prism**

(Paulay and Priestley, 1992)

It is not always feasible to conduct the compression testing of masonry prisms to obtain the actual prism strength, which is the basic structural property for design of
masonry. On the other hand, compressive strength of bricks \( (f_b) \) and mortar \( (f_j) \) can be easily evaluated by the test on bricks and mortar using the empirical relations. The following empirical relation was suggested by (Dayratnam 1987; Kaushik et al. 2007) for Indian burnt clay bricks -

\[
\begin{align*}
F'_m &= K f_b^\alpha f_j^\beta \\
\end{align*}
\]

Where, \( K, \alpha \) and \( \beta \) are constants.

\[
\therefore F'_m = 0.63 f_b^{0.49} f_j^{0.32}
\]

In above equation \( \alpha \) must be higher than \( \beta \) because \( F'_m \) was found to depend more upon the strength of bricks than mortar.

### 3.5.2 Elastic Modulus

The elastic modulus \( E_m \) represents the initial slope of the strain-stress curve and its value exhibits a large variation. Different approaches can be found in the literature for the calculation of \( E_m \) (Kaushik et al. 2007). Since masonry is a composite material consisting of bricks and mortar which have distinct properties, several researchers assumed linear elastic behavior for both materials. The sum of deformation of the bricks and mortar joints is equal to the compressive deformation of masonry. Some other researchers related the modulus of elasticity of masonry walls with the compressive strength of the material. These empirical equations suggests a range of values between \( 250 f_{m\%} < E_m < 1000 f_{m\%} \) (Kaushik et al. 2007).

- **Paulay and Priestley (1992)**
  \[
  E_m = 750 f_m \quad \text{MPa} \quad (3.9)
  \]

- **Drysdale (1990)**
  \[
  E_m = 550 f_m \quad \text{MPa} \quad (3.10)
  \]

- **Sinha and Pedreschi (1983)**
  \[
  E_m = 1800 f_m^{0.83} \quad \text{MPa} \quad (3.11)
  \]

- **Hendry (1990)**
  \[
  E_m = 2116 f_m^{0.50} \quad \text{MPa} \quad (3.12)
  \]

An average value of \( E_m \sim 550f_m \) was found to be in line with internationally accepted documents and codes like FEMA306, IBC2003, Euro code 6 etc. (Kaushik et al. 2007).
3.5.3 Geometrical Parameters

The geometrical parameters of the model are the thickness of the panel, the contact length with the beam and column and the area of the strut.

a) Contact Length

The contact length $z$, as defined by Stafford Smith (1966), who introduced the dimensionless relative stiffness parameter $\lambda$, is given by-

$$z = \frac{\pi}{2} \lambda$$  \hspace{1cm} (3.13)

$$\text{Where, } \lambda = \left[ \frac{E_c t \sin 2\theta}{4E_m I_c h_m} \right]^{1/4}$$  \hspace{1cm} (3.14)

$h$ = column height between centre lines of beams,

$h_m$ = height of infill panel,

$E_c$ = expected modules of frames material,

$E_m$ = Modulus of infill materials,

$I_c$ = moment of inertia of column,

$d_m$ = diagonal length of infill panel,

$t$ = thickness of infill panel,

$\theta$ = $\tan^{-1}(h_m / l_m)$. 
b) Width and Area of Strut

The area of strut $A_m$ is defined as the product of the panel thickness and the equivalent width of the strut $b_w$, which normally varies between 10% and 25% of the diagonal of the infill panel, as Stafford Smith (1962) concluded based on experimental data and analytical result. There are also numerous empirical expressions by different authors for the evaluation of the equivalent width, presented hereinafter. Holmes (1961) suggested that:

$$b_w = \frac{d_w}{3} \quad (3.15)$$

Mainstone (1971) obtained a set of equations for different levels of performance and his equation was included in FEMA 274 (1997) for the analysis and rehabilitation of buildings.

$$b_w = 0.175(\lambda h)^{-0.4} d_w \quad (3.16)$$

Liauw and Kwan (1984) presented Equation taking $\theta$ equal to 25° and 50° in order to represent the commonest cases in practical engineering.

$$b_w = \frac{0.95 h_w \cos \theta}{\sqrt{\lambda h}} \quad (3.17)$$

Finally, Paulay and Priestley (1992) gave a conservative value for the estimation of $b_w$, which is useful for the design purpose.

$$b_w = \frac{d_w}{4} \quad (3.18)$$
3.6 Load Effects on Seismic Behavior - Frame Elements

The analysis of a structure should include the simultaneous effects of gravity and lateral loads. Gravity loads should include dead loads and likely live loads. The nonlinear response of a structure to lateral loads depends (in a nonlinear way) on the gravity loads present at the time of lateral loading. Considering a beam (Fig.3.9a), the effect of light gravity load is to reduce the reserve moment and shear strengths at the right end and increase the reserve strengths at the left end (reserve strength is defined as the difference between the total strength and the resistance used up by gravity load). Therefore, for a given lateral drift, the gravity load will increase the inelastic rotation demands at the right end of the beam and decrease them at the left end. For larger gravity loads, the effects are increased, and the inelastic mechanism may shift from beam hinging at the ends to hinging along the beam span.

In the case of column (Fig.3.9b), variations in gravity load produce variations in column axial force, with consequent changes in both column strength and deformability. Increases in axial load invariably decrease flexural deformability. Increases in moment strength result in increased shear demands and may result in shear failure that would not be expected at lower axial loads.

(a) Beam Elements (ATC-40, 1996)
During an earthquake, acceleration-induced inertia forces will be generated at each floor level, where the mass of an entire storey may be assumed to be concentrated. The summation of all the floor forces, $f_i$ (Fig.3.10), above a given storey will then locate the position of the resultant force $V_i$ (base shear) within that storey. In reality, during earthquake, buildings are generally subjected to large inertial forces, which cause members of buildings to behave in a non-linear manner i.e. stress does not remain proportional to strain.

Fig.3.10 Effects of Lateral Forces on a Building

Fig.3.11 illustrates a generalized load-deformation relation appropriate for most concrete components. The relation is described by linear response from $A$ (unloaded

Ref: (b) Column Elements (ATC-40, 1996)
component) to an effective yield point B, linear response at reduced stiffness from B to C, sudden reduction in lateral load resistance to D, response at reduced resistance to E, and final loss of resistance thereafter (Fig.3.11.a). Deformations beyond point E are not permitted because gravity load can no longer be sustained (Fig.3.11.b). In some cases, initial failure at C will result in loss of gravity load resistance, in which case E is a point having deformation equal to that at C and zero resistance (ATC-40, 1996). The above main points are shown below in the load-deformation relation figure.

![Load Deformation Relations for Non-degrading Components](image)

**Fig.3.11 Load Deformation Relations for Non-degrading Components**

Where,

- $Q_c$ - Refers to the strength of the component,
- $Q$ - Refers to the demand imposed by the earthquake.

Lateral loads should be applied in predetermined patterns that represent predominant distributions of lateral inertial loads during critical earthquake response. Lateral loads commonly may be lumped at floor levels if the floor diaphragms are sufficiently rigid in their plane. The taller a building, the more significant the effect of lateral forces will be. As a structure is displaced laterally, its lateral load stiffness usually decreases with increasing lateral displacement.

For the same maximum displacement at roof level, the overall ductility demand in terms of the large deflection is much more readily achieved when plastic hinges develop in all the beams (Fig.3.12b) instead of only in the soft-storey column (Fig.3.12c). The column sway mechanism, also referred to as a soft-storey, may impose plastic hinge rotations, which even with good detailing of affected regions, would be difficult to accommodate (Paulay and Priestley, 1992).
3.7 Out of Plane and In Plane Loading Masonry Walls

Wall will be subjected to simultaneous vertical, in plane, and out of plane (face) load, because of the multiracial nature of ground shaking under seismic forces. The in-plane response will primarily be a result of the resistance of the wall to inertia forces from other parts of the structure, such as floor masses. The out-of-plane response will be due to the inertia mass of the walls themselves responding to the floor-level excitation. The design envelope of in-plane moments exceeds those resulting from the code distribution of forces at levels above the base due to higher-mode effects (Paulay and Priestley, 1992). Magnitude of out-of-plane moments will be larger in the upper than in the lower storeys.

Fig.3.13 Response of a Masonry Wall to Biaxial Excitation
Due to in-plane action, tension cracking will reduce the ability of the walls to provide restraining moments at floor levels. It should be noted that maximum in-plane moments occur at the base of wall while maximum out-of-plane moments occur in the top storey, where in plane moments are low.

3.8 Summary

In this chapter study is made on the failure modes of brick masonry infill panels as well as finding the strength of brick work. It is observed from the study that the strength of bricks as well as brick work varies from region to region.

The value of the width of the diagonal strut also varies from researcher to researcher. There is no universal formula available which predicts the exact width of diagonal strut, as it depends upon various parameters.

In this chapter modeling aspects as well as control parameters of the infill modeling as diagonal strut were dealt.