MODELING TECHNIQUES: DEVELOPMENT & IMPLEMENTATION

One of the biggest challenges in modeling infill frames is to capture the interface behavior between the infill panel and the bounding frames. Several investigations have been carried out to develop modeling techniques for infill frames, at both the macro and the micro level.

Analytical models of infill walls can be categorized as micro-models, macro-models, or homogenization models, based upon the simulation technique utilized. Micro-models are more detailed finite element (FE) models, which are often used to conduct detailed analytical studies of the behavior of frames with masonry infills. By contrast, macro-models utilize fewer, larger elements (generally equivalent diagonal struts), and are often used for design and practical assessment purposes. Homogenization models take an intermediate approach. This e-document outlines the development of all three types of analytical models and their implementation in analytical studies, drawing upon literature surveys by Burton (2012) and Mosalam and Günay (2012). Singh and Das (2006) provide added details on early approaches to micro-modeling.

Micro-Modeling

Micro-modeling is based on the Finite Element (FE) method and requires modeling of the masonry units and mortar joints as discrete elements, with proper consideration of the masonry unit/mortar joint interface. Micro-modeling is the most accurate analytical approach, because it can take into consideration the frame/infill wall interface conditions, various material constitutive relationships for the frame and the infill wall materials, and several other modeling parameters.

Micro-models can also be used for walls with openings. Parametric studies of single-bay, single-story infilled frames with different types of openings (Reflak and Fajfar (1991)) demonstrated that a macro-model with an equivalent diagonal strut was inaccurate for infill walls with openings. Reflak and Fajfar’s work built upon research that had been conducted to calculate natural frequencies and the associated mode shapes (Thiruvengadam (1985)), but it adopted an approach to simplify the computation, in which each infill was treated as a substructure, and the degrees of freedom (DOF’s) were reduced to the boundary DOF’s by static condensation.

The use of the FE method for the analysis of infilled frames dates back to 1967, when Mallick and Severn suggested the first finite element approach. In that first application, linear elastic material models (such as linear plane stress elements) were employed. Slip between the frame and the infill panel was accounted for by using a link element to model frictional shear forces in the contact region. Effects of opening size, position and shape were among the parameters investigated using linear elastic models.

Early FE models (e.g., Liauw and Kwan (1982); Dhanasekar and Page (1986)) used three types of elements: linear and nonlinear beam elements to represent frames; plane elements to represent infill; and interface elements or one-dimensional joint elements to capture interface behavior. Several early studies (e.g., Axely and Bertero (1979), Reflak and Fajfar (1991)) suggested condensation approaches be used to reduce the degrees of freedom and the associated computational burden.

Consideration of the material nonlinearity of infill walls in the FE models started with the modeling of crack initiation and propagation, among other sources of material and geometrical nonlinearities. The bracing action provided by the infill wall to the frame produces a high transverse tension field in the infill wall panels. As the URM infill wall material typically has low tensile strength, cracking is an important material behavior to model for infill walls.

In addition to cracking, frame/infill wall discontinuities and interaction, wall bracing effect on the frame, and frame inelastic behavior were considered for dynamic analyses (Rivero and Walker (1982)). One observation from the computational results was that the wall braced the frame after the contact at the opposite diagonal corners. In addition, researchers concluded that it was necessary to model the frame/infill wall discontinuity and gap formation.

Researchers Dhanasekar and Page (1986), Asteris (2000), and Syrmakezis and Asteris (2001), among others, proposed constitutive models for infill brick masonry, which included inelastic stress-strain relationships and failure surfaces. Liauw and Kwan (1982) analyzed frames with square and rectangular infill wall panels, using the constitutive models in combination with one dimensional (1D) interface elements to model frame/infill wall separation and masonry unit/mortar joint cracking. Diagonal cracking and corner crushing failure modes were observed for square and rectangular panels, respectively. Another outcome was the significant effect of the compressive strength of infill wall material on the ultimate strength of the infilled frame, in the case of the corner crushing failure mode. Increasing the infill compressive strength was also found to change the failure mode from corner crushing to diagonal cracking.

Since the early attempts to use FE in the analysis of infilled frames, modeling of the frame/infill wall interface conditions has received significant attention. In the early studies, researchers used short stiff linking members, friction elements, or bar type elements to model the interface between the frame and the infill wall (e.g., Mallick and Severn (1967), Liauw and Kwan (1982), Rivero and Walker (1984), Dhanasekar and Page (1986)).

In later studies, FE and fracture mechanics techniques were used to consider cracking and separation phenomena between a RC frame and an URM infill wall. Effects of the frame/infill wall contact length and the frame/infill wall relative stiffness on the redistribution of internal straining actions in the frame members and stresses in the infill walls were investigated.

Nonlinear FE analyses were also used to investigate the effect of important variables on the various possible infill cracking patterns. Investigated variables included vertical load, wall slenderness, and stiffness ratios between masonry units and mortar and between the URM infill and the RC frame.

Interface conditions were modeled approximately to permit sliding and separation, together with a simplified micro-model consisting of four plane-stress elements for the infill and inelastic material laws for the RC frame and the infill walls. Zhuge and Hunt (2003) have proposed a Distinct Element approach to model masonry infills. This approach considers masonry as an assembly of discrete blocks. The mortar joints are represented numerically as contact surfaces formed between two block edges. This model can also capture the behavior of separation and sliding. Some researchers investigated the effect of the initial gaps between the frame members and infill walls—which gaps might be present due to shrinkage of the material or due to poor construction—on the behavior of infilled frames (e.g., Rivero and Walker (1982), Riddington (1984), Richardson (1986).). Such studies showed that even relatively small initial gaps significantly reduced the lateral stiffness. However, the effects of these initial frame/wall interface gaps on the cracking pattern or ultimate strength were insignificant.

To conclude, the FE method or the micro-modeling approach is the most appropriate modeling approach to simulate all possible failure mechanisms in infilled frames. This method takes into account the interaction between masonry blocks along the joints, as well as the frame infill interaction. The main drawbacks to this approach are the effort required to construct these complex models and the computational demand to analyze them under various loading conditions. As a result, the FE method is typically used only in research studies.

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Macro-Modeling

Macro-models are based on a physical understanding of the behavior of each infill panel as a whole. The infill panel is typically represented by a single global structural member, most often by equivalent diagonal struts.

The composite action between the infill wall and the bounding frame depends upon the area of contact between these two components. At the macro level, it has been found that the infill panel separates from the surrounding frame at relatively low lateral load levels, after which contact between the frame and infill is limited to the two opposite compression corners (Shing and Mehrabi 2002).

In the typical case of non-integral infill walls (i.e., in the absence of shear connectors), partial composite action occurs between the infill wall and the surrounding frame. Since the first attempts were made to model the response of composite infilled frame structures, experimental and conceptual studies have suggested that a diagonal strut with the appropriate geometrical and material characteristics can be used to represent this partial composite action. This is known as the equivalent diagonal strut approach. Several investigators (e.g., Holmes (1961, 1963), Stafford-Smith (1962, 1966, 1968), Thiruvengadam (1985), Chrysostomou (1991), Hamburger (1993), and El-Dakhakhni et al. (2003)) have proposed variations on the equivalent strut model, with the key parameter being the effective width of the strut.

In the first attempt to use an equivalent diagonal strut (Holmes, 1961), the infill wall was modeled as an equivalent pin-jointed diagonal strut with the same elastic material properties as masonry, the same thickness as the infill panel, and a width equal to one third of the length of the strut. This estimation of the strut width was later modified to take into account the frame/infill wall relative stiffness, dimensions of the infill wall, and the moment of inertia of the columns of the surrounding frame (Mainstone, 1971).

Klingner and Bertero (1976) suggested a model of a diagonal strut with hysteretic behavior, which is able to simulate the stiffness degradation generated by cyclic loading. In addition, equations were developed to calculate the initial stiffness and ultimate strength of the equivalent diagonal struts (Stafford-Smith, 1962, 1966; Stafford-Smith and Carter, 1969; Mainstone, 1971; Hamburger, 1993). The results of such research, which was conducted to study the behavior of infilled steel frames subjected to monotonic loading, are still used due to their simplicity. For example, the current U.S. standard ASCE 41, *Seismic Rehabilitation of Existing Buildings* (ASCE, 2006), uses a compression only strut for modeling infill walls and uses equations similar to those from early research for modeling the stiffness and strength of the diagonal strut. It should be noted that in the equivalent diagonal strut model, only one of the two diagonal struts is effective at any time during dynamic excitation, because the struts are compression-only, and one of the struts is under compression while the other under tension at any time. Alternative proposals for evaluation of the equivalent strut width, considering the slippage along the frame/infill wall interface, have been proposed.

Holmes (1961) suggested that the effective width of an equivalent strut depends primarily upon the thickness and aspect ratio of the infill. Stafford-Smith and Carter (1969) have proposed that the equivalent strut width is not a constant value, but varies with the applied loading and the relative properties of the frame and infill. However, Mehrabi et al. (1994) have found that the lateral stiffness of the infilled frames using Stafford-Smith and Carter’s equivalent struts is consistently underestimated by a factor of two when the bending stiffness for uncracked RC sections is used.

Three of the basic failure modes of masonry-infilled frames are:

1. Compression failure of the diagonal strut, which occurs as crushing of the infill in at least one of its corners. This type of failure takes place when the infill material has low compressive strength.

2. When the compressive strength of the masonry is high, the forces transferred from the infill wall to the surrounding frame result in damage to the frame members before infill damage occurs. This might result in shear damage in the columns, due to the additional horizontal forces transferred from the infill wall. In addition, plastic hinges may form in columns, beams, or beam-column joints. In rare cases, tension failure of columns may occur in infilled frames with high aspect ratio (wall height to length ratio), due to the additional vertical forces transferred from the infill walls.

3. Failure occurs due to out of plane effects, where damage takes place in the central region as a result of the arching action of the infill wall.

The single equivalent diagonal strut models mentioned above are concerned chiefly with the first failure mode. Regarding the second failure mode, several researchers (e.g., Reflak and Fajfar (1991), Saneinejad and Hobbs (1995), Mosalam et al. (1997) Buonopane and White (1999), and El-Dakhakhni et al. (2003)) have suggested that the bending moments and shear forces in the frame members cannot be replicated using a single diagonal strut connecting the load at two corners. These researchers have proposed more complex macro-models, using more than one diagonal strut to produce better estimations of the bending moments and shear forces in the frame members.

Thiruvengadam (1985) proposed a multiple strut model of an infill panel considering reciprocal stiffening effects. The model consists of a moment resisting frame with a number of pin-jointed diagonal struts in both the directions.

El Dakhakhni et al. (2003) noted that the equivalent strut is not simply a line connecting the corners of the surrounding frame but rather, is a stressed region of the panel that connect parts of the frame in the vicinity of the two loaded corners. They suggested that the panel be modeled using three parallel struts in each direction, with the off-diagonal struts being positioned at critical locations along the frame members. Mohebkhah et al. (2007) proposed a modified version of El Dakhakhni's three strut model that accounts for the presence of central window openings. Other researchers (e.g., Chrysostomou et al. (2002), Hashemi and Mosalam (2007), Kadyiewski and Mosalam (2009)) have studied multiple-strut approaches that use as many as eight struts.

Hashemi and Mosalam (2007) proposed the use of a three-dimensional strut and tie model that captures the interaction of in-plane and out-of-plane strength under bi-directional loading. The model consists of eight compression struts with a tension tie at the center of the infill panel. They have shown that the out-of-plane (OOP) capacity of an infill wall decreases with the existence of in-plane forces, whereas the in-plane capacity of an infill wall decreases when OOP forces are present.

As the macro-model becomes more complex, the number of parameters involved increases. For example, the proportions of stiffness and strength between the struts need to be calibrated for the three strut model. However, the simplicity of the single diagonal strut model can be maintained by using shear springs at the column ends: the shear spring force represents the lateral force transferred from the infill wall to the columns. In this way, the potential for brittle shear failure of the columns can be simulated without losing the simplicity of the single diagonal strut model.

One of the failure modes of masonry-infilled frames is out-of-plane failure. Several analytical models have been introduced to represent the OOP behavior of infill panels. The simplest models assume that the infill panel acts as an elastic plate and apply the classical theory of elasticity solution, as derived in Timoshenko and Woinowsky (1959). Considering two-way inelastic behavior, models based on the modified yield-line theory have also been developed (e.g., Drysdale and Essawy (1988), Haseltine (1976), Haseltine et al. (1977), Hendry (1973), and Hendry and Kheir (1977)); these models depend upon the tensile capacity of the masonry to carry OOP forces, which assumption is valid before cracks develop in the URM. However, such models predict the OOP response poorly, after cracking of the masonry has occurred.

Most experimental data suggest that after the initial cracking of the URM infill wall, the OOP strength depends upon the compressive strength of the masonry, not upon its tensile strength, due to the arching action of the infill wall. Analytical methods based on this arching action have been developed by McDowell et al. (1956), Angel and Abrams (1994), Klingner et al. (1996) and Dawe and Seah (1990), to name a few. The current U.S. standard ASCE 41, *Seismic Rehabilitation of Existing Buildings* (ASCE, 2006), considers arching action if certain conditions are present: for example, the panel should be in full contact with the surrounding frame components, and the frame members should have sufficient strength to resist thrusts from arching of the infill panel. Tensile and compressive strengths are used for calculation of the OOP capacity in the absence and the presence of arching action, respectively.

Kadyiewski and Mosalam (2009) proposed an alternate model that also considers in-plane and out-of-plane behavior consisting of two beam column elements with a node at mid-span used to account for out-of-plane inertial forces.

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Homogenization Modeling

Instead of modeling the masonry units and mortar joints as discrete elements, one can derive a representative behavior of masonry from the material behavior of those individual components, by providing constitutive relationships in terms of averaged (homogenized) stresses and strains from the constitutive relationships of the components.

Research has been conducted on this homogenization modeling technique, which might be considered as an intermediate level between micro-modeling and macro-modeling. For this purpose, smeared cracking approach has been investigated and utilized by researchers (e.g., Lotfi and Shing (1991), Lotfi and Shing (1994), and Mosalam (1996)). Smeared-crack approaches have also been used to evaluate the shear resistance of masonry panels in infill frames (Shing, and Lotfi (2002)), in an effort to account for the fact that multiple cracks could develop over the area of an element. Mehrabi et al. (1994) modeled infill frames using a combination of smear cracks to capture diffused cracking, and discrete cracks to capture the dominant shear cracks in non-ductile concrete members. Stavridis and Shing (2008) developed a systematic calibration methodology for FE models of infilled reinforced concrete frames, which also combined the smeared crack approach with discrete crack elements.

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Reliability Studies

One methodology proposed by Dymiotis et al. (1999) for reliability studies and probabilistic assessment involves defining random variables and limit-state criteria and subsequently, deriving vulnerability (or fragility) curves for an adequate number of input motions, while accounting for the uncertainty in failure criteria at both the member level and the structure level.

Despite the relatively large number of seismic reliability studies that have been conducted and reported on in the literature, few early researchers have dealt with infilled frames. Variability in masonry parameters was introduced by Hwang and Huo (1994) and Mosalam et al. (1997). However, both of these studies assumed concrete block infills, not unreinforced clay-brick masonry. Colangelo et al. (1995) used a classic ‘‘pilotis’’ frame (all bays infilled except those of the ground story) as an example of an irregular structural system in their study. A discussion of issues involved in infilled-frame modeling was provided by the Comité Euro-International du Béton (CEB) (1994). Dymiotis et al. (2001) assessed the performance of infilled frames under earthquake loading, using a probabilistic methodology, paying as much attention to the variability of masonry walls as to that of the RC members.

A case study by Hashemi and Mosalam (2007) used the first-order reliability method or FORM (Haldar and Mahadevan 2000) and the program FERUM (Haukaas and Der Kiureghian 2003) to obtain the probability of failure and the reliability indices for the three limit-state functions—in-plane (IP), out-of-plane (OOP), and combined failures—for URM infill walls. FORM uses linear approximation of the limit-state function close to the design point in the normal space of random variables.

Hashemi and Mosalam (2007) considered a five-story RC structure with URM infill walls as a case study to set a framework for reliability analysis of such buildings. The computational model of the structure, using a diagonal strut model to represent the URM infill walls, was subjected to a series of ground motions scaled to different hazard levels for a range of spectral accelerations (SA). The URM infill wall IP and OOP demand forces and their statistical properties at different seismic hazard levels were determined. It was observed that the IP forces at the first floor were greater than at the third floor and roof levels, while the OOP forces at the third floor and roof were greater than at the first floor. These findings are consistent with field evidence from 2009 L’Aquila (Italy) earthquake, in which damage to upper story infill walls of buildings was observed.

The statistical properties of the IP and OOP capacities of the URM infill wall were calculated based on a recent U.S. pre-standard for the seismic rehabilitation of buildings (FEMA 356, 2000). The interaction curve for the IP and OOP capacities was based on the interaction curve obtained from FE analysis of the URM infill wall from the researchers’ shake table test specimen. Defining failure as the loss of infill walls, researchers determined limit-state functions corresponding to reaching the IP, OOP, and combined bidirectional capacities of the URM infill wall. Researchers used FORM to calculate the probabilities of failure at each seismic hazard level corresponding to each limit-state function for the URM infill walls at the first, third, and roof levels of the building. Based on the fragility functions for IP, OOP and combined failure for the different floor levels, researchers concluded that the failure probability of URM infill walls due to the OOP forces alone was negligible for the first floor and small for the third floor and roof. However, these forces had significant effects on the combined failure probability and became increasingly important to consider, the higher that walls were in the structure.

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