Structures 10 (2017) 1–12

Contents lists available at ScienceDirect

Structures

journal homepage: www.elsevier.com/locate/structures

Finite Element Study of the Effect of Interfacial Gaps on the in-Plane Behaviour of Masonry Infills Bounded by Steel Frames

Xi Chen, Yi Liu *

Dept. of Civil and Resource Engineering, Dalhousie Univ., Halifax, NS B3J 1Z1, Canada

ARTICLE INFO

Article history:
Received 23 March 2016
Received in revised form 28 September 2016
Accepted 10 November 2016
Available online 11 November 2016

Keywords:
Masonry infills
Steel frames
Interfacial gaps
Stiffness and strength
Reduction factor
Finite element study

ABSTRACT

A finite element study was conducted to investigate the in-plane behaviour and strength of concrete masonry infills bounded by steel frames with the focus on the effect of interfacial gaps. Various gap locations and sizes were studied. The results showed that the top beam-infill gap was more detrimental to the stiffness and strength of infill than the column-infill gaps. Gap(s) had less effect on infills bounded by weak frames than strong frames. Different arrangements of gap locations at the column-infill interface affected the behaviour of infilled frames only when the friction coefficient exceeded 1.0. The provisions contained in the American masonry standard MSJC 2013 for design of infilled frames with gaps were examined. It was found that in most cases, the reduction factor for the gap effect specified in the MSJC 2013 produced conservative estimates for both the stiffness and strength. Based on the finite element results, a set of lower bound expressions for stiffness and strength evaluation over a wide range of gap size and locations were proposed.

1. Introduction

Masonry walls built inside concrete or steel frames are commonly known as masonry infills. When the infilled frame is subjected to in-plane lateral loading, the infill with inherently large in-plane stiffness, will attract large forces to the frame region and change the lateral load distribution of the structure. The understanding of interaction between the masonry infill and the surrounding frame is thus crucial in determination of infill contribution to the stiffness and strength of the infilled system. Considerable amount of experimental research has been conducted to investigate the interaction between the infill and its bounding frame in an effort to quantify the infill contribution to the system stiffness and strength [1–14]. These studies have contributed to the development of the so-called "diagonal strut" method. This method replaces the infill with an equivalent diagonal strut connecting two loaded corners to resist the lateral loading. Once the strut width is known, the stiffness of the system can be determined through a frame analysis while the strength of the infill can also be formulated as a function of the width. Due to its simplicity, the "diagonal strut" concept has been adopted in various international standards [15–17] for design of infills, albeit with different forms of strut width equation. It should be pointed out though, that the diagonal strut method contained in these standards is only applicable to the so-called participating infills where the interfacial gaps between the infill and the bounding frame are zero or within a specified limit. For example, both the Canadian masonry design standard S304-14 [15] and Eurocode 8 [17] require that no interfacial gaps be existent for infills to be considered as participating infills while no guidelines are provided for the treatment of interfacial gaps in either document. The American masonry standard MSJC 2013 [16] states that infills can be considered as participating provided that the top beam-infill gap is <9.5 mm (3/8″), but in such a case, a factor of 0.5 must be applied to the stiffness and strength of the infill. However, there was no sufficient background information provided on the given gap size limit or the reduction factor. In practice, initial gaps between the infill and the bounding frame are common occurrence due to shrinkage and settlement of the infill or defects in masonry workmanship. These gaps could be located at either the beam-infill interface, or column-infill interface, or both. Since the total elimination of gaps is not realistic, it is thus important to evaluate the effect of gap on the infill contribution to the system behaviour and strength.

Despite a large amount of research available to address the infill behaviour in general, studies focusing on the effect of gaps were limited. Within the few available studies on gaps, the detrimental effect of gaps on both the strength and stiffness of the infilled system was reported [2,5,8,18]. However, the range of reduction observed in these studies was scattered and due to limited data points, no correlation between the gap and the infill stiffness and strength was defined. For example, the experiment program of Yong [2], and Dawe and Seah [5] showed that a 20 mm gap at the top beam-infill interface reduced both the crack load and ultimate load of the infilled steel frame by 50 and 60% respectively. Flanagan [6] reported that a 25.4 mm gap at the column-infill interface of infilled steel frame resulted in nearly no reduction in the ultimate capacity but a nonsymmetrical cracking pattern where more

* Corresponding author.
E-mail address: yi.liu@dal.ca (Y. Liu).

http://dx.doi.org/10.1016/j.istruc.2016.11.001
2352-0124/

cracks developed in the lower portion of the panel. The study conducted by Kadir [19] showed that a top beam-infill gap of 1.6 mm did not significantly affect the frame ultimate load but it reduced its cracking load. In the experimental study conducted by Ng'andu [20] on infilled steel frames, a 12 mm gap at the top beam-infill interface was found to reduce the system stiffness by about 20% but had no significant effect on the cracking load or ultimate load. Nazief [21] conducted a finite element study on infilled steel frames where the effect of beam-infill gap and full separation gap (around the three infill-frame interfaces) was studied. Results showed that a top beam-infill gap or a full separation gap up to 5 mm did not have a significant impact on the infill ultimate strength while the lateral stiffness was reduced by around 30%. A top beam-infill gap of 10 to 15 mm resulted in a reduction in the ultimate strength of 24% while a full separation gap of the same size resulted in a reduction of 50%.

In the above mentioned studies, the gap scenarios were often focused on the top beam-infill [2,5,19,20] and with limited range of sizes. The obtained information is not sufficient to define the effect of gap location or size on the infill stiffness and strength in relation to the infilled system. In light of this, this study was then motivated to further investigate the effect of gaps on the behaviour of infills bounded by steel frames. The numerical modeling technique was used where a finite element model of the infilled frame subjected to in-plane lateral loading was developed. The model was validated against experimental results reported in the literature for infilled frames with or without gaps. The objective of the study was to determine correlations between the gap size and location and the lateral stiffness and strength of the infilled frame and to assess the validity of design provisions on gapped infilled frames contained in the American design standard MSJC 2013. To this end, various gap locations, sizes and arrangement scenarios were considered as main parameters. The effects of bounding frame stiffness and friction coefficient were also included in the study.

2. Finite element model

A simplified micro-modeling technique encoded in the software ANSYS was used where the infill was modeled using homogeneous continuum elements and the mortar effect was modeled using a surface-based cohesive contact model. The masonry unit was expanded in each directions by the half mortar thickness and the expanded mortar units interact with each other through the interface. The masonry units were meshed into 2 × 2 plane-stress elements and placed in the running bond. Details of the model is available in a previous paper published by the authors [22]. For the interest of completeness and easy reference, the key components of the modeling are summarized in the following.

2.1. Constitutive models for masonry and steel

The constitutive model for masonry in compression was based on the experimentally calibrated model proposed by Priestley and Elder [23] and it is expressed in Eqs. (1a)–(1c) where f'_m is the compressive strength of masonry. An elastic perfect plastic material model was used for frame members where the elastic modulus and the yield strength of steel were taken as 200 GPa and 350 MPa, respectively.

$$\varepsilon \le 0.0015, \qquad \sigma = \frac{f'_m}{0.9375}\left(\frac{2\varepsilon}{0.002} - \left(\frac{\varepsilon}{0.002}\right)^2\right) \tag{1a}$$

$$0.0015 \le \varepsilon \le 0.0025, \quad \sigma = f'_m(1 - z_m(\varepsilon - 0.0015)) \tag{1b}$$

$$\text{where } z_m = \frac{0.5}{\frac{3 + 0.29 f'_m}{145 f'_m - 1000} - 0.002} \tag{1c}$$

2.2. Interface model

The interface was modeled using surface-based cohesive contact pairs to capture the cracking and sliding failure of the mortar joints. Each contact pair consists of one zero-thickness surface-to-surface cohesive zone interface element, CONTA171, and one zero-thickness target element TARGE169 where CONTA171 was used to detect contact or separation between "target" surfaces (TARGE169) and a deformable surface defined by CONTA171 element. This interface element was also used for the contact between the infill and the frame members. A schematic view of the modeled infilled frame is shown in Fig. 1(a). The geometry and configuration of inter-blocks contact pairs are shown in Fig. 1(b). Cracking and sliding failure of mortar joints were monitored using the linear elastic traction-separation laws as shown in Fig. 2. Represented by the bilinear cohesive zone material model of the CONTA171 element, separation initiates when the maximum normal or shear stress in a contact pair reaches the corresponding strength. The shear stress and the relative slip behaviour is plotted in Fig. 2(a) which shows separation begins at point A where the shear strength (τ_t) is achieved and is completed at point C when the shear stress reaches zero. The softening stage provides mortar joints with a residual shear strength to account for the frictional resistance of the joint after shear crack occurred. For tension separation, a similar bilinear constitutive relationship but with a cut-off was adopted. As Fig. 2(b) shows, once the tension stress in the contact pair reaches the tensile strength (f_t) of mortar joint, the contact pair is considered separated to represent the tensile cracking of mortar joint. The normal and shear stiffnesses, k_n and k_s, were determined based on the following expressions suggested by Lourenço [24]:

$$k_n = \frac{E_u E_{mortar}}{t_m(E_u - E_{mortar})} \tag{2}$$

$$k_s = \frac{G_u G_{mortar}}{t_m(G_u - G_{mortar})} \tag{3}$$

where E_u and E_{mortar}, and G_u and G_{mortar} are the Young's moduli and shear moduli of masonry and mortar respectively; and t_m is the thickness of mortar joints. In lieu of experimental data, G_u and G_{mortar} can be taken as $0.4E_u$ and $0.4E_{mortar}$ respectively.

2.3. Failure criteria

A Hill type yield criterion (f_1) proposed by Lourenço and Rots [25] was adopted to monitor the compression failure of masonry infill. This failure surface was also shown by Dawe et al. [12] to produce results in reasonably good agreement with experimental data.

$$f_1 = \frac{\sigma_x^2}{f_{mx}^2} + \beta\frac{\sigma_x\sigma_y}{f_{mx}f_{my}} + \frac{\sigma_y^2}{f_{my}^2} + \gamma\frac{\tau_{xy}^2}{f_{mx}f_{my}} - 1 \le 0 \tag{4}$$

where subscripts x, and y indicate the directions parallel and perpendicular to the bed joint. Hence, σ_x and σ_y, and τ_{xy} are the nodal normal stresses in the x- and y-direction, and the shear stress, respectively; f_{mx} and f_{my} are the uniaxial compressive strength of infills in the x- and y-direction, respectively. As the masonry infill was assumed orthotropic, the compressive strength of masonry in head joint direction (x-direction) was taken as 0.7 times that in bed joint direction (y-direction) [16]. The factor γ accounts for the shear stress contribution to compression failure while the factor β controls the coupling between normal stresses in x- and y-directions. In lieu of experimental data, as suggested by Dawe et al. [12] for masonry commonly used in practice, the value of β and γ can be taken as −1.0 and 5.5 respectively.

The damage initiation criterion is defined using a Mohr-Coulomb friction yield surface combined with a tension cut-off and elliptical compression cap as shown in Fig. 3 where f_t is the tensile strength of mortar