Nonlinear analysis of masonry-infilled steel frames with openings using discrete element method

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Abstract

Nonlinear numerical modeling of masonry-infilled frames is one of the most complicated problems in structural engineering field. This complexity is attributed to the existence of joints as the major source of weakness and material nonlinearities as well as the infill-frame interaction which cannot be properly modeled using the traditional finite element methods. Although there are many numerical studies available on solid masonry-infilled steel frames’ behavior, however, few researches have been conducted on infilled frames with openings. In this paper a two-dimensional numerical model using the specialized discrete element method (DEM) software UDEC (2004) is developed for the nonlinear static analysis of masonry-infilled steel frames with openings subjected to in-plane monotonic loading. In this model, large displacements and rotations between masonry blocks are taken into account. It was found that the model can be used confidently to predict collapse load, joint cracking patterns and explore the possible failure modes of masonry-infilled steel frames with a given location for openings and relative area. Results from the numerical modeling and previous experimental studies found in the literature are compared which indicate a good correlation between them. Furthermore, a nonlinear analysis was performed to investigate the effect of door frame on lateral load capacity and stiffness of infilled frames with a central opening.

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1. Introduction

Steel and reinforced concrete framed structures in urban areas are usually infilled with masonry as interior and exterior walls. The resulting system is referred to as an infilled frame, which has high in-plane stiffness and strength. At low levels of lateral forces, frame and infill wall behave monolithically. However, as the lateral forces’ level increases, the frame deforms in a flexural mode while the infill attempts to deform in a shear mode. Interaction between frame and infill panel significantly increases the infilled frame lateral stiffness and drastically alters the expected dynamic response of the structure. However, the effect of masonry-infill panels is often neglected in the analysis of infilled frames by structural engineers, as it is in current practice. Such a neglect may lead to a substantial inaccuracy in predicting the lateral stiffness, strength, and ductility of the frame.

Since 1950s, extensive studies have been performed on lateral load behavior of masonry-infilled frames both experimentally and analytically. Stafford Smith [26,27], Riddington and Stafford Smith [20], Liauw and Kwan [11] and Moghadam et al. [17] have conducted experimental and analytical investigations on the lateral stiffness and strength of steel frames infilled with masonry panels. Comprehensive description of the studies performed until 1987 has been reported in the state-of-the-art report on infilled frames by Moghadam and Dowling [15]. Dawe and Seah [5], Mosalam et al. [19] and Flanagan and Bennett [7] have studied the behavior of masonry-infilled steel frames under lateral in-plane loads. Saneinejad and Hobbs [22] developed an inelastic analysis and design method for infilled steel frames subjected to in-plane forces. The model was later adopted by Madan et al. [13] and implemented in software IDARC for dynamic...
analysis of infilled frames. Mehrabi et al. [14] have proposed a smeared-crack finite element model to study the nonlinear behavior of infilled reinforced concrete frames. Based on minimizing the factor of safety with reference to the failure surfaces in a masonry infill, Moghadam proposed [16,17] a new analytical approach for the evaluation of shear strength and cracking pattern of masonry infill panels.

Nevertheless, the previous numerical modeling experiences have shown that this kind of structures cannot be correctly studied by conventional methods of continuum mechanics like the finite element methods that are used to analyze engineered structures. These methods fail to predict mechanisms where, for example, initially isolated parts react dynamically together. Continuum-based methods can give satisfactory results but generally fail to provide a practical analysis for masonry structures. Although, some finite element and finite-difference programs include interface elements that allow the user to incorporate a few discontinuities, most of these break down when a large number of discontinuities are encountered or when displacements along the discontinuities become too large [8]. As an alternative to the traditional finite element continuum approach, a discrete element method (DEM) can be employed to simulate the nonlinear behavior of masonry-infilled steel frames. In the discrete element method, large displacements and rotations between blocks, including sliding between blocks, the opening of the cracks and even the complete detachment of the blocks, and automatic detection of new contacts as the calculations’ process are allowed.

The purpose of this paper is to investigate the nonlinear in-plane behavior of masonry-infilled steel frames with openings. For this purpose, a two-dimensional discrete element model (at a semi-detailed level) is developed using the specialized discrete element software UDEC (2004) [8] for the nonlinear inelastic static analysis of infilled steel frames subjected to in-plane monotonic loadings. This model is used to investigate the effect of door opening positions on the lateral stiffness, ductility and collapse load of such frames. Also, a further analysis is performed to investigate the effect of door frame’s stiffness on the lateral load behavior of these kinds of frames with a central opening.

### 2. Discrete element method

Discrete element methods were initially developed for the study of jointed and fractured rock masses in 1971 by Cundall [23]. Due to their capability to explicitly represent the motion of multiple, intersecting discontinuities, these methods are particularly suitable for the analysis of the masonry structures in which a significant part of the deformation is due to relative motion between the blocks. So far various discrete element applications to masonry structures have been reported for both static and dynamic analysis [2,3,9,10,24,25].

A. Mohebkah and Tasnimi [18] developed a two-dimensional DEM model to investigate the seismic behavior of confined and reinforced brick masonry walls. All the analyses in this study were performed using the specialized discrete/distinct element software UDEC (Itasca, 2004). The Universal Distinct Element Code (UDEC) is a two-dimensional numerical program based on the distinct element method for discontinuum modeling. UDEC simulates the response of discontinuous media subjected to either static or dynamic loading. The discontinuous medium is represented as an assemblage of discrete blocks. The discontinuities are treated as boundary conditions between blocks; large displacements along discontinuities and rotations of blocks are allowed. In other words, the joints are viewed as interfaces between distinct bodies. Individual blocks behave as either rigid or deformable material. Deformable blocks are subdivided into a mesh of finite-difference triangular elements, and each element responds according to a prescribed linear or nonlinear stress–strain law. The formulation of these elements is similar to the constant strain triangle (CST) finite element formulation. The disadvantage of this element is that, in case of complicated deformation problems such as beam/column components’ behavior in an infilled frame, the number of triangular elements in which the area has to be discretized may become very large.

In order to model the mechanical interaction between blocks, it is assumed that the blocks are connected by normal and shear elastic springs. The relative motion of the discontinuities is also governed by linear or nonlinear (as appropriate) force–displacement relations for movement in both the normal and transverse (tangential) directions. The formulation used in the program permits both geometric and physical nonlinearities of the intact material to be modeled. UDEC has several built-in material behavior models, for both intact blocks and the discontinuities, which permit the simulation of response representative of discontinuous materials. The program is based on a “Lagrangian” calculation scheme that is well-suited to model the large movements and deformations of a blocky system. In this program, the explicit solution procedure is used, in which motion equations are setup for each time increment. This takes place for each block.

The original UDEC program was based on the plane strain situation. In the UDEC 4.0 version it is possible to give the
stress perpendicular to the plane of the structure a constant value. The plane stress situation is then obtained by giving the stress perpendicular to the plane of the structure a value of zero. This is encountered, for example, in masonry structures loaded only in the plane of the structure. The calculations performed in the discrete element method alternate between the application of a force–displacement law at all contacts and Newton’s second law at all blocks or nodes [8]. The force–displacement law is used to find contact forces from known displacements. Newton’s second law gives the motion of the blocks resulting from the known forces acting on them. Mechanical damping is used in the DEM to solve both static and dynamic solutions. For each class, a different type of damping is used. For static analysis, an approach similar to dynamic relaxation technique is employed. In this technique, the equations of motion are damped to reach the equilibrium state as soon as possible.

3. DEM modeling of masonry-infilled steel frames

The described discrete element method is employed here to study the nonlinear lateral load behavior of some concrete masonry-infilled steel frames tested at the University of New Brunswick during a period of time in the past by Dawe et al. Detailed experimental results of the specimens have been summarized in Dawe and Seah [5]. Among the 28 large-scale specimens tested under racking load in their program, specimens WC7, WC3, WC5 and WC6 were chosen. These four specimens are identical single panel concrete masonry-infilled steel frames with different masonry strength and infill panel geometrical characteristics. All specimens were 3600 mm long by 2800 mm high. Panels consisted of $200 \times 200 \times 400$ mm concrete blocks placed in running bond within a surrounding moment-resisting steel frame fabricated using W250 × 58 columns and a W200 × 46 roof beam. The first specimen is a solid infilled frame while the last three specimens have $0.8 \times 2.2$ m door openings in different horizontal positions in the panels. Lintel beams consisting of two 15M deformed bars placed in grouted bond beam blocks were used to span openings. In each case, a horizontal load applied at an upper corner of the frame was gradually incremented up to the interstory drift ratio of 1%. Table 1 summarizes the characteristics of each specimen. The typical test setup including the loading system and the dimensions is shown in Fig. 1 [5].

The specimens were modeled at a semi-detailed level (the so-called micro-modeling strategy) using discrete element method. This implies that the joint is modeled as an interface with zero thickness, in analogy with the discontinuum finite element modeling [12]. In this approach, fictitious expanded block dimensions are used that are of the same size as the original dimensions plus the real joint thickness as shown in Fig. 2(b). The interface’s stiffness is deduced from the stiffness of the real joint. The inelastic, isotropic model is used for the behavior of the blocks. The blocks are considered fully deformable, thus allowing deformation to occur both in the blocks and joints and a better simulation of crack propagation and sliding in the joints. The analysis was carried out sequentially. First, each model was brought to equilibrium under its own dead weight. In order to determine a collapse load, it is often better to use displacement-controlled boundary conditions rather than force-controlled. Therefore, an
incremental horizontal displacement was applied at the top left corner of the models.

3.1. Elastic parameters

In order to generate a DEM micro-model based on interface elements with zero thickness, the size of the blocks has to be expanded by the mortar thickness \( h_m \) in both directions. It follows that the elastic properties of the expanded block and the interface joint must be adjusted to yield correct results. Due to the relative dimensions of mortar and block, it is assumed that the elastic properties of the blocks remain unchanged. Then, under the assumption of stack bond and uniform stress distributions both in the block and mortar, the components of the elastic stiffness are evaluated as follows [21]:

\[
\begin{align*}
 k_n &= \frac{E_b E_m}{h_m (E_b - E_m)} \\
 k_s &= \frac{G_b G_m}{h_m (G_b - G_m)}
\end{align*}
\]

where \( E_b \) and \( E_m \) are the Young’s moduli, \( G_b \) and \( G_m \) are the shear moduli respectively, for block and mortar and \( h_m \) is the actual thickness of the mortar. The accuracy of this methodology has been verified by Lourenço [12] using some detailed discontinuum finite element analyses.

3.2. Inelastic parameters and constitutive criteria

There are many parameters that affect the behavior of the model. The most fundamental parameter is the constitutive model chosen to represent the material behavior. UDEC version 4.0 has some built-in constitutive material models such as: isotropic elastic, Drucker–Prager, Mohr–Coulomb, strain-hardening/softening, etc. In the case of very low stress levels, a linear-elastic model is sufficient. However, for high stress levels a nonlinear model which can simulate crack formation, shear or crushing is needed. The nonlinear post-peak behavior of stone-like materials such as masonry is characterized by softening which determines the way in which crack formation propagates within a block [21]. It has been shown that when the shear displacement increases, the masonry block cohesion does not suddenly, but more or less gradually decreases to zero. Hence, the concrete blocks were built using a strain-hardening/softening material model. This model is based on the UDEC Mohr–Coulomb model with tension cut-off in conjunction with non-associated shear and associated tension flow rules. The difference, however, lies in the possibility that the cohesion, friction, dilation and tensile strength may harden or soften after the onset of plastic yield. In the Mohr–Coulomb model those properties are assumed to remain constant [8]. But here, the user can define the cohesion, friction and dilation as piecewise-linear functions of a hardening parameter measuring the plastic shear strain. In other words, the post-failure behavior in a strain-softening model is dictated by values for angle of internal friction and cohesion, which decrease with increasing percent strain after failure.

Since the steel frame components in the model were expected to behave inelastically at ultimate state of loading, a Von-Mises material model was chosen to represent the steel frame behavior. The Von-Mises criterion is not available in UDEC. However, the Drucker–Prager criterion can be degenerated into the Von-Mises criterion for \( \phi = 0 \) [8]. Although the steel frame components are made up of steel I-sections, they were modeled as solid blocks of steel with equivalent elastic and inelastic mechanical properties. The average compressive and tensile strength of the concrete masonry blocks are 31 and 1.0 MPa, respectively [5].

Strain-softening model for the blocks is described by two parameters \( c \) and \( \phi \). Angle of internal friction can be calculated only if a series of uniaxial and triaxial tests are performed on samples of the material in question. Tasnimi and Farzin [28] based on some tests and nonlinear finite element analyses derived the following equations for the angle of internal friction and cohesion of concrete in terms of concrete compressive strength \( f'_c \):

\[
\phi = 0.145 f'_c (\text{MPa}) + 49.71 \tag{3}
\]

\[
c (\text{MPa}) = 0.1065 f'_c (\text{MPa}) + 0.531.
\]

Hence, the angle of internal friction and cohesion calculated from Eq. (3) for \( f'_c = 31 \) MPa are 54.2° and 3.83 MPa, respectively. These parameters are required to capture the inelastic behavior of concrete masonry blocks using the strain-softening model.

For the joints, simulating the characteristics of the mortar, a Mohr–Coulomb slip model is employed. The average angle of internal friction and cohesion of the mortar joints \((\phi_j, c_j)\) are 35° and 0.6 MPa, respectively [5]. Another required inelastic parameter is the dilation angle. Dilatancy is a measure of the change in volume that occurs when shear stress is applied to a material. This change is characterized by a dilation angle, \( \psi_j \), which measures the uplift upon shearing. It has been shown [1,19] that dilatancy must be taken into account in the...
analysis of confined masonry structures such as infilled frames. In this paper, a typical dilation angle of $12^\circ$ [4] was chosen for the joints.

The coefficient of friction between steel components and masonry panel has an influence on the behavior of the model and must be defined. Hence, a coefficient of friction equal to 0.25 was assumed for the frame-to-panel interfaces [6].

4. Analysis and comparison with test results

The two-dimensional discrete element model developed in this study was used to investigate the behavior of concrete masonry-infilled steel frame specimens WC7, WC3, WC5 and WC6. The lintel beam reinforcements in specimens with openings are modeled using the local reinforcement element available in UDEC. This element considers the local effect of reinforcement where it passes through existing discontinuities (joints). This element exploits simple force–displacement relations to describe both the shear and axial behavior of reinforcement across discontinuities.

Fig. 3 illustrates the comparison between the load–displacement diagrams of all the experimented specimens, and that of the numerical analysis, up to a deformation in which the failure mechanism is formed. The local peaks in the diagrams corresponded to the state at which a new joint crack occurs or plastic behavior takes place in the blocks. Also, Table 2 presents the difference between the numerical and experimental collapse loads. The agreement between experimental and numerical responses is satisfactory with a maximum error of 20% for specimen WC5 and an average error for the three specimens of 1%. The difference in the peak load value of specimen WC5 could be due to necessary differences in the material properties of the experiment and discrete element model. In the report of Dawe and Seah [5], only the average compressive and tensile strength of the blocks used in all the 28 tested specimens were given not those of the material used in the specimen construction. The actual blocks with low compressive strength placed at the top compressive corner may have had some influence in the experimental results. In other words, the weak blocks may have induced a premature corner crushing leading to a low ultimate load. Not enough information concerning the actual inelastic parameters of the specimen was found in the report. Nevertheless, up to the peak load point (14 mm), there is reasonable good agreement between the numerical and experimental results.

Together with the global load–displacement response, a comparison in terms of the deformed geometry, cracking pattern and the failure mechanism is necessary to assess the validity of the numerical results. In Fig. 4, DEM qualitative results of specimen WC3 at the time of ultimate capacity (lateral displacement equal to 30 mm) are shown. As the load increases, cracks occur at the horizontal and vertical interfaces. Then, some stepwise cracks are propagated off-diagonally at the bottom right corners of the piers. There are some plastic indicators in the program that can be used to assess the state of nonlinear blocks in the numerical model for a static analysis. Such an indication usually denotes that plastic flow is occurring, but it is possible for a block mesh element simply

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental (kN)</th>
<th>Numerical (kN)</th>
<th>Ratio (Num./Exp.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WC7</td>
<td>534</td>
<td>501</td>
<td>0.94</td>
</tr>
<tr>
<td>WC3</td>
<td>285</td>
<td>297</td>
<td>1.04</td>
</tr>
<tr>
<td>WC5</td>
<td>245</td>
<td>295</td>
<td>1.20</td>
</tr>
<tr>
<td>WC6</td>
<td>365</td>
<td>368</td>
<td>1.01</td>
</tr>
</tbody>
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to sit on the yield surface without any significant flow taking place. Fig. 4(a) shows the collapsed interfaces as well as the plastic behavior in the blocks. The failure situation of the elements is shown in this figure using three distinct symbols: ×, * and ○ indicating “yielding in past”, “at yield surface” and “tensile failure”, respectively. The situation “yielding in past” indicates the unloaded yielding elements so that their stresses no longer satisfy the Mohr–Coulomb yield criterion. As well, the situation “at yield surface” illustrates the actively yielding elements which are important to the detection of a failure mechanism. The experimental failure and crack patterns of the specimen have not been given in Ref [5] to be compared with the numerical results. Because of the low cohesion available at the bottom pier-to-frame interface and lack of any restraint, right pier starts to move left horizontally. This movement makes the pier ineffective to some extent in resisting the lateral load. In other words, most of the infill resistance to the lateral force is provided by the left pier shear-arching action. This phenomenon is more evident in Figs. 4(b), (d) and 5. Fig. 4(b) shows the displacement vectors in which the horizontal movement of the right pier is obvious. Also, in this figure it is seen that the little detached triangular part of the left pier (in its bottom right corner) does not take part in the lateral load-resisting piers’ system and the left pier rotate in relation to its bottom left corner.

Fig. 5 shows the variation of piers average shear stress at top horizontal section (Sec. A-A in Fig. 4(a)) versus lateral displacement of the frame. The piers shear stress variations have been monitored up to failure point as the numerical analysis progressed. It can be seen that, up to lateral displacement of 0.5 mm, both piers contribute to the lateral-resisting system equally. From this point, the value of right pier shear stress increases gradually indicating the horizontal movement of the pier. Finally, at lateral displacement of 9 mm, the pier yields in shear and therefore the shear stress is not increased anymore. In other words, at this point a shear-slip crack occurs at its bottom level. Sliding shear occurred for the left pier top level at lateral displacement of 20.5 mm as shown in Fig. 5. The horizontal displacement contours of the specimen are also shown in Fig. 4(c). This figure shows again that the left
pier undergoes more severe damage than the right one due to the above-mentioned reason. Fig. 3(d) shows the magnified picture of the deformation mechanism of specimen WC3 in which the inclined crack patterns as well as the frame-to-panel interface separation in the vicinity of the windward column is clearly observed. The obtained results reveal the capacity of the DEM to model masonry-infill walls’ behavior.

Fig. 6 shows a representation of the principal stress tensors for the WC3 model at ultimate capacity. In this figure, principal stresses’ directions show the activated compressive parts orientation of the infill panel subjected to lateral loading. Due to the presence of opening, the infill panel is forced to act as three distinct parts as shown in Fig. 6 with shaded areas including one horizontal and two activated inclined compressive components.

The horizontal part connects the windward column top to two inclined compressive struts to resist the lateral force. The development of a multi-strut macro-model to simulate the nonlinear lateral load behavior of thes frame structures is the subject of an ongoing study which is currently being undertaken by the authors at Tarbiat Modares University. Fig. 7 shows DEM qualitative results of specimen WC6 at the time of ultimate capacity state (lateral displacement equal to 30 mm). It can be seen that in this specimen, the right pier undergoes severe

Fig. 6. Principal stress tensors for specimen WC3 infill wall (magnification factor = 10).

Fig. 7. DEM results of specimen WC6 at a horizontal displacement of 30 mm: (a) blocks’ failure points and crack patterns of the joints; (b) displacement vectors; (c) horizontal displacement contours (m); and (d) deformed geometry (magnification factor = 10).

Fig. 8. Experimental cracking and failure patterns of specimen WC6 [5].
damage due to the formation of a broad diagonal strut. This apparent diagonal strut results in a higher ultimate load than that obtained for specimens WC3 and WC5. In this case, contrary to specimen WC3, due to the presence of higher normal stress resulting from the activated compressive struts, no significant separation or movement takes place in the right pier bottom level. The experimental cracking pattern of the specimen at a horizontal displacement of 22 mm is shown in Fig. 8. It is evident that the pattern of cracking and diagonal tension failure (right pier) as predicted by the numerical model (Fig. 7(a)) is consistent reasonably with laboratory experimental observations. The diagonal tension failure points of the right pier in Fig. 7(a) have been distinguished from the others by the symbol ◦ parallel to the diagonal strut.

5. Effect of door frame on infilled frame capacity

As it is in common practice, the opening is bounded by an internal frame to restrain the blocks from falling into the opening. The effect of such frames on the lateral load capacity of masonry-infilled frames with openings has not been investigated. Therefore, using the aforementioned two-dimensional discrete element model, one more analysis is performed on specimen WC3 considering the door frame around the central opening. To this end, the door frame members were
assumed to be rectangular in cross-section (box steel section 120 × 60 mm) with rigid corner connections. It is assumed that the door frame is a moment-resisting frame.

Structural beam elements are provided in UDEC to simulate supporting members [8]. In this work, this structural element is employed to simulate the supporting effect of the door frame in specimen WC3, to enhance the lateral load capacity and lateral stiffness of the infilled frame. UDEC program uses an explicit formulation in analyzing the behavior of a support structure composed of beam elements and interface stiffness. In this formulation, local stiffness matrices are used following division of the structure into segments with the distributed mass of the structure lumped at nodal points [8]. Forces generated in support elements are applied to the lumped masses which move in response to unbalanced forces and moments in accordance with the equations of motion. This formulation has the following desirable characteristics:

- Slip between support and opening periphery is modeled in a manner identical to block interaction along a discontinuity.
- Large displacements with nonlinear material behavior are readily accommodated.

The material behavior model associated with this element formulation can simulate both elastic and inelastic behavior. Elastic modulus, yield stress and Poisson’s ratio of the frame ($E_s$, $F_y$, $\nu$) was set to 210 GPa, 300 MPa and 0.3, respectively. Fig. 8 shows the effect of door frame confinement on the capacity and stiffness of specimen WC3. It can be seen that due to the presence of support frame around the opening, both stiffness and lateral load capacity of the specimen are increased. In this example, an increase of 28% in lateral load capacity was observed. This is because; the use of door frame enhances the behavior of right pier. Hence, the pier can sustain more lateral shear load before failure. In this case, contrary to that with no door frame, a broad inclined compression strut is formed. This behavior is qualitatively shown in Fig. 9 which is related to the state of ultimate capacity and corresponded to 23 mm lateral displacement. It is seen that contrary to specimen WC3 without door frame, this pier undergoes severe damage (crushing) along its diagonal.

6. Conclusion

A two-dimensional discrete element model developed for the inelastic nonlinear analysis of masonry-infilled steel frames has the ability to consider both geometric and material nonlinearities. To model the masonry-infill panel, a micro-modeling strategy at a semi-detailed level was adopted in which the joint is modeled as an interface with zero thickness. The blocks are considered fully deformable, thus allowing deformation to occur both in the blocks and joints and a better simulation of crack propagation and sliding in the joints. The model has adequate capability to include the effects of many variables such as gap between infill panel and frame members, panels with or without openings, joint reinforcement, etc.

The model was used herein to simulate the lateral load behavior of a few masonry-infilled steel frames with openings available in the literature. It was found that the numerical model is applicable to a detailed simulation of the response of such frames throughout the loading process leading to failure. The prediction of collapse loads and the evolution of the deformations were both in accordance with the experiments. It was shown that the method can simulate correctly the failure mechanisms based on joint separation and sliding. Besides the numerical simulation of the experimental specimens, a further analysis was performed to investigate the effect of door frame confinement on lateral capacity of a specimen with central opening. The analysis showed that, due to the use of a conventional door frame, the lateral load capacity of the infilled frame specimen is increased up to 28%.

It should be realized that, all material properties required by the DEM model were not available and consequently, as pointed out previously, some properties were estimated from reliable literature resources. Authors believe that, better correlation between the DEM model and experimental results can be obtained using appropriate material properties.

References


