13TH CANADIAN MASONRY SYMPOSIUM
HALIFAX, CANADA
JUNE 4TH – JUNE 7TH 2017

BEHAVIOR OF MASONRY INFILL WALLS IN CASE OF FAILURE OF A SUPPORTING COLUMN

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ABSTRACT
The in plane behavior of masonry infill walls that are subjected to lateral loading simulating the effects of earthquakes on buildings has been the subject of many studies. The present work is focused on a problem that has been hardly studied and refers to the vertical action on such walls. In particular, it concerns a vertical action that evolves when a supporting column of a multi-story reinforced concrete frame with infill masonry walls fails. Such failure may happen as a result of extreme loadings for instance a strong earthquake, car impact, or military or terror action in proximity to the column. Without infill walls, the loss of a supporting column may lead to a partial or even full progressive collapse of the bare reinforced concrete frames. The presence of masonry infill walls may restrain the process and even prevent the development of a progressive collapse mechanism. The aim of this study is to look into the role of the composite action of a frame and an infill wall in the event of loss of a supporting column. The study adopts an experimental methodology and numerical methods aiming to evaluate the contributions of the unreinforced masonry infill, to examine its interaction with the frame, and to quantify its contribution to the resistance of the bare frame under such circumstances.

KEYWORDS: *masonry, infill wall, vertical load, extreme event, progressive collapse*

INTRODUCTION
Progressive collapse is a well-known structural failure process that is initiated by local damage and propagates into a major portion of the structural system. Usually, the unreinforced masonry infill walls are considered by the designer as non-structural and they are not taken into account in the structural design. Several experimental and numerical studies aiming at evaluation of the RC building resistance to progressive collapse have been carried out and reported in the literature.

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These studies may be subdivided into two main groups that differ with regard to the infill wall model. In the first group, the infill wall is modelled as a continuum linear shell element [1]. This modeling approach cannot account for cracking and it is suitable for the linear infill wall behaviour only. The second group uses strut elements to model the infill wall. Usually, a single strut that is connected to the beam-column joints [1–3] is used. In other studies, multiple struts are used as well [4]. The single strut approach does not allow evaluating the real bending moment and shear forces in the RC frame as it does not simulate the interaction between the infill wall and the surrounding frame. The multiple strut approach with off-diagonal struts introduce discrete contacts with the RC frame, but this is still a simplified representation with a pre-determined contact with the frame that is not related to the variable continuous contact pressure distribution between the masonry infill and the RC frame. Also, in all these studies, the infill wall properties are taken on the basis of earlier experimental, analytical, or numerical studies of the frame-infill wall composite behaviour under the action of a lateral load rather than of a vertical load. This is mainly due to the open questions regarding the composite wall behaviour under the action of a vertical load that the present paper aims at answering.

Experimental studies aiming at investigating the behaviour of a RC infilled frame under the effect of failure of an interior column are reported in [5–7]. While these studies examined an interior column a peripheral column (Figure 1) is more likely to be damaged, in case of car collision or a nearby explosion in proximity to the building façade. Experimental studies on the composite wall behaviour in the case of a peripheral column loss have not been found.

Many studies have focused on the in plane behavior of masonry infill walls that are subjected to lateral loading simulating the effects of earthquakes on buildings. Most of these studies focused on a typical single bay, single story wall in a building with commonly locally used types of masonry blocks. Extensive studies examined several major governing parameters such as: the wall geometry, window opening in the wall, type of the masonry blocks and their geometry, frame's beam and column stiffnesses, reinforcement details in the RC frame, construction method of the wall, effect of vertical load, etc. [8–14]. Some of the studies were extended to include the overall building parameters such as the number of stories and number of bays [15,16]. On the one hand, these studies shed light on the general behaviour of masonry-frame compositing. On the other hand, the role of those parameters, properties, and features in the event of the vertical response due to failure of a peripheral column has not been investigated.

The aim of this study is therefore to investigate the composite action of a frame and an infill wall in the event of loss of a peripheral supporting column. The study aims to achieve this goal by a combination of an experimental methodology and numerical analysis.

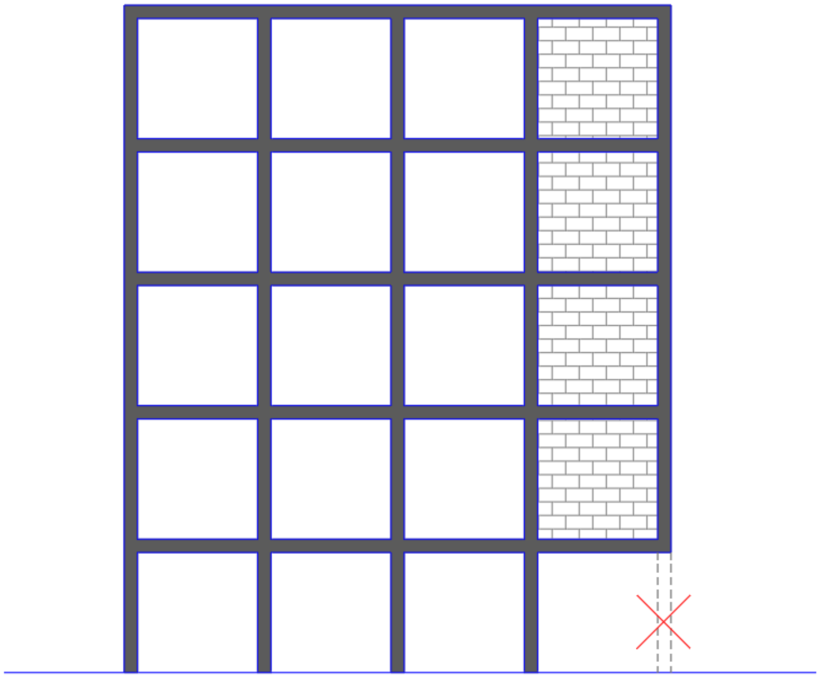

Figure 1: Building facade

EXPERIMENTAL WORK

The experimental setup, materials and results are described in detail in Ref. [17]. The experimental setup focuses on a scaled down (1:2) single bay RC infilled frame. Four design parameters were examined in the experimental study [17]. Beam to column relative stiffness, reinforcement details, block type, and the presence of column shear keys. Seven specimens were tested with various combinations of the design parameters. The test setup is shown in Figure 2. It included hydraulic actuators on both columns. The left column base was supported in the horizontal and vertical directions. These supports represent the lower floor column and the horizontal elements at the adjacent span. The top of the column was supported in the horizontal direction. These supports do not allow the supporting column to rotate. Thus, the frame action was taken into consideration. The left actuator labeled “AC1” applied a constant vertical load of 130kN representing the axial force in the upper floor column. The force was measured by load cells “LC1”. A monotonic vertical load applied by the actuator labeled “AC2” at the top of the right column (the “loaded column”). Four LVDTs labeled “LV1” - “LV4” measured the horizontal displacements and the changes to the length of the diagonals. Linear potentiometer labeled “LP” was connected to the loaded column basis.

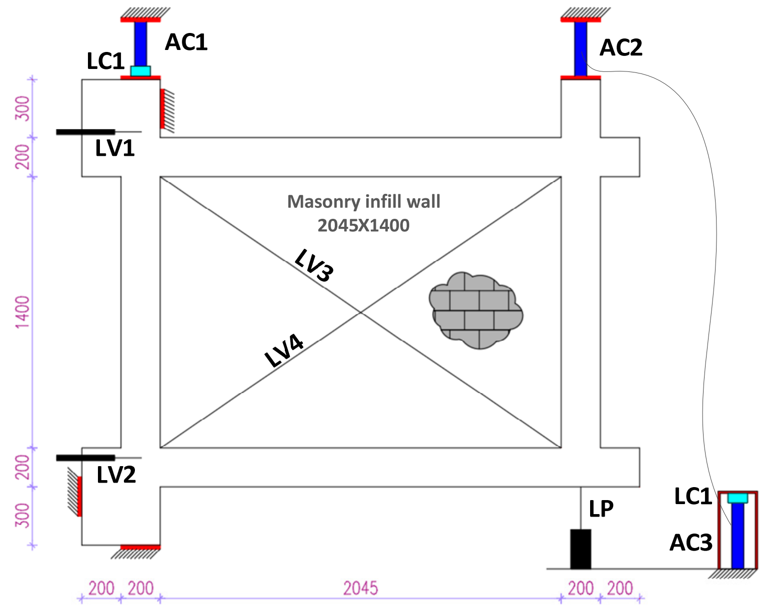

Figure 2: Experimental setup: specimens with stiff columns [17]

Material tests were conducted on the concrete frame, the blocks, the mortar, and the reinforcement. The properties are listed in Table 1.

Table 1: Material Properties

	Compressive strength [MPa]	Dimensions [mm]	Mean elongation at rupture [%]	Ultimate strength [MPa]	Mean yield strength [MPa]
Concrete frame	26.6-28.3	-	-	-	-
AAC concrete blocks	3.1	250x150x100	-	-	-
Hollow concrete blocks	10.9	260x100x100	-	-	-
Mortar	14.2	t=7	-	-	-
Bar size 3mm	-	-	23.0	390	220
Bar size 6mm	-	-	2.75	695	602
Bar size 8mm	-	-	9.03	816	613

EXPERIMENTAL RESULTS

The concrete compressive strength, the initial stiffness, and the ultimate resistance of each specimen are discussed in [17] and for reference they are summarized in Table 2. The specimen with improved reinforcement details shows significant increase of the ultimate load. All the results can be divided into three groups based on the main failure mechanism. The first two depend on the infill wall and surrounding RC frame relative strength: specimens with weak infill wall consisting of AAC blocks (specimens 4, 5 and 6) and specimens with strong concrete blocks (specimen 7). The third failure mechanism is typical to specimens with shear keys and hollow concrete blocks (specimens 1 and 3). In all specimens the frame failure dictated the maximal vertical resistance.

Table 2 : Test results and specimens properties (following [17])

No.	flexible / stiff columns	reinforcement details	Masonry blocks type	construction method	Concrete compressive strength [MPa]	Initial stiffness [kN/mm]	Ultimate resistance [kN]
1	flexible	Regular	Hollow concrete	integral	27.4	27.1	71.1
2	stiff	Regular	Hollow concrete	non-integral	28.3	34.5	77.1
3	stiff	Regular	Hollow concrete	integral	28.3	15.4	72.1
4	stiff	Regular	AAC	non-integral	27.0	6.8	44.5
5	stiff	Regular	AAC	integral	26.8	31.0	71.0
6	stiff	Improved	AAC	non-integral	28.2	27.4	88.5
7	flexible	Improved	Hollow concrete	non-integral	26.6	139.9	122.0

NUMERICAL ANALYSIS

In order to gain insight into the behaviour of the infill-RC assembly and into the failure mechanisms 3D analysis is carried out with ATENA, a non-linear Finite Element (FE) software, and compared with the experimental results. The numerical study is focused on the behaviour of the hollow concrete infill wall. Two groups are modeled and investigated; integral infill walls, i.e. specimens with shear keys, and non-integral infill walls.

The concrete material is simulated with a non-linear model. The uniaxial stress-strain law of the concrete material is shown in Figure 3, where f_c and f_t are the concrete compressive and tensile strengths, and ε_c and ε_t are the strains at the peak stress. The end point of the softening curve (ε_d) is defined by means of the plastic displacement. This value is based on experiments carried out by Van Mier [18]. The unloading is described by a linear function that passes through the origin. The biaxial failure function for concrete is shown in Figure 4. This concrete model includes reduction of compressive strength and shear stiffness after cracking, hardening and softening in compression, and a biaxial strength failure criterion; A detailed description of the constitutive model can be found in ATENA Program Documentation [19]. For the steel reinforcement, a multi-linear model that was fitted the test result given in Table 1 was adopted.

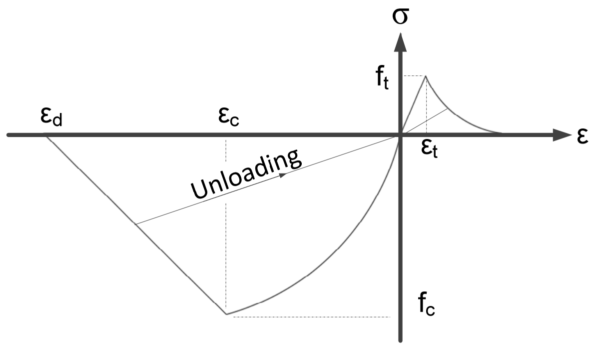

Figure 3: Uniaxial stress-strain law for concrete

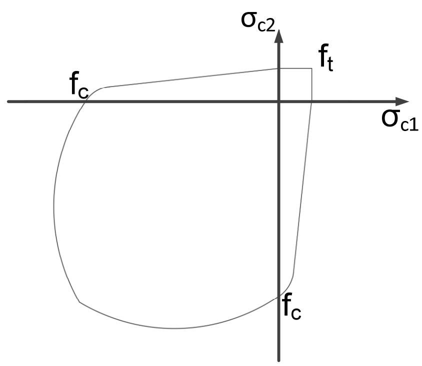

Figure 4: Biaxial failure function for concrete

Interface elements were placed between the concrete frame and the infill wall, as well as between the blocks. The interface mortar material satisfies the Mohr-Coulomb criterion with an ellipsoid in the tension regime as shown in Figure 5. The constitutive relations for the general three-dimensional case are given in terms of tractions on the interface planes and relative sliding and opening displacements in the following form:

$$\begin{Bmatrix} \tau_1 \\ \tau_1 \\ \sigma_1 \end{Bmatrix} = \begin{bmatrix} K_{tt} & 0 & 0 \\ 0 & K_{tt} & 0 \\ 0 & 0 & K_{nn} \end{bmatrix} \begin{Bmatrix} \Delta v_1 \\ \Delta v_2 \\ \Delta u \end{Bmatrix} \tag{1}$$

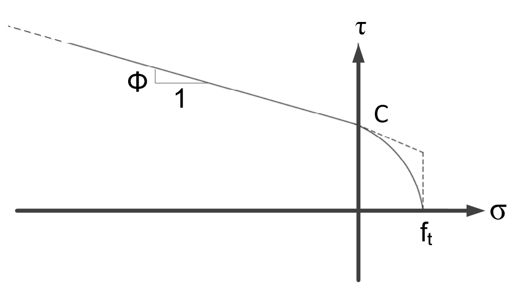

Figure 5: Failure surface for interface element

where K_{tt} and K_{nn} denote the initial elastic normal and shear stiffnesses respectively. Figure 6 shows the interface model behavior in shear and tension. After the interface failure, the interface stiffness should be zero (continuous line), which would mean that the global stiffness will become indefinite. Therefore, for numerical purposes, after the failure of the element a minimal stiffness is implemented (dashed line) in order to preserve the positive definiteness of the global system of equations.

The mechanical properties used in the FE model are listed in Table 1 and

Table 3. The numerical solution of the FE model uses the Newton-Raphson method with the tangent stiffness updated after each iteration.

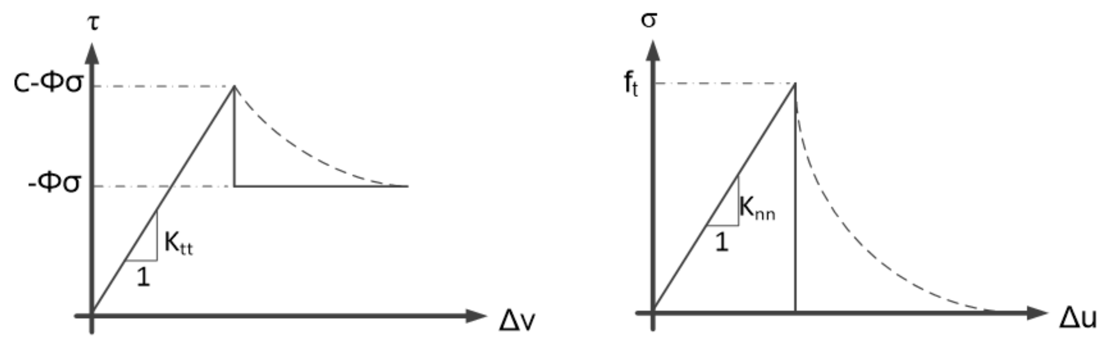

Figure 6: Interface model behavior in shear (left) and tension (right)

Table 3: FE model material properties

Concrete properties [17]	Elastic modulus	30.3	$[GPa]$
	Poisson ratio	0.2	$[-]$
	Tensile strength	2.32	$[MPa]$
	Compressive strength	-25.5	$[MPa]$
	Critical compressive displacement	-5.0E-04	$[mm]$
Interface properties	Normal stiffness [19]	2.0E+08	$[MN/m^3]$
	Tangential stiffness [19]	2.0E+08	$[MN/m^3]$
	Tensile strength [17]	1.3	$[MPa]$
	Cohesion [20]	0.92	$[MPa]$
	Friction coefficient [21]	0.4	$[-]$

Non-integral (without shear keys) infill with hollow concrete blocks

Figure 7 compare the FEA with the experimental results of specimen 2 having non-integral hollow concrete blocks infill wall with regular reinforcement details. The comparison is presented in terms of load-displacement curves. The FEA results show a good agreement with the experimental ones in terms of the initial stiffness, the ultimate resistance, and the displacement at the ultimate resistance. The final damage state of specimen 2 is shown in Figure 8, as well as the FE cracking results. The masonry wall crushing under compression at the upper right side is well captured by the FEA, while the vertical separation between the infill wall and the supported column does not appear in the FEA.

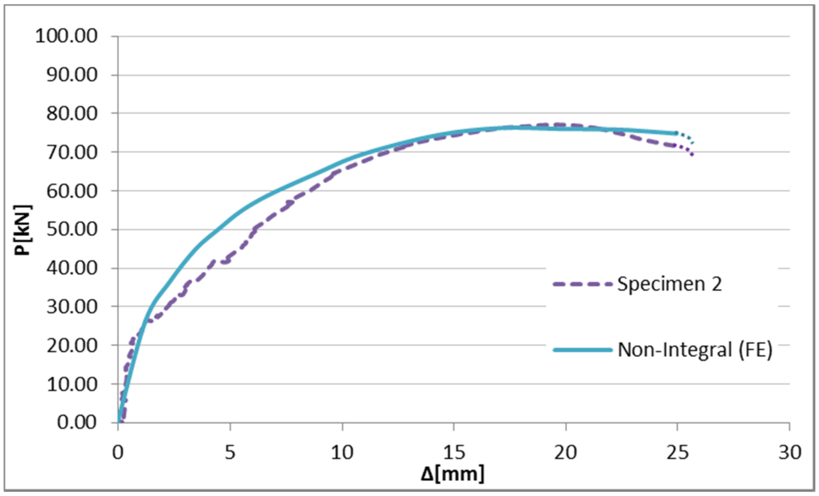

Figure 7: load-displacement curve of specimen 2 [17] **with non-integral infill wall**

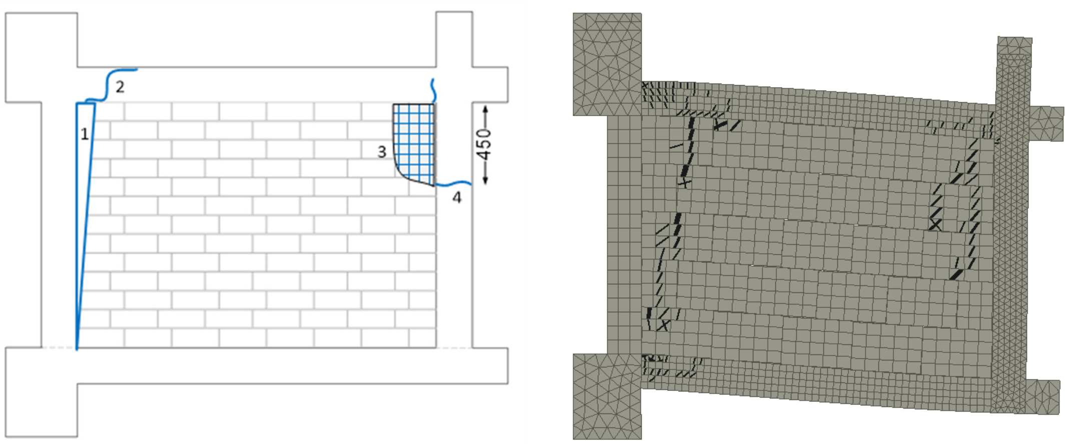

Figure 8: Cracking pattern in specimens with non-integral infill wall. Specimen 2 [17] **(left) and FE model (right).**

Integral (with shear keys) infill with hollow concrete blocks

The main feature of the response of an integral wall is a single vertical crack that evolves in the infill along the end of the shear keys. This is followed by a shear crack at the upper beam. With increasing the load, the infill masonry vertical crack and the beam shear crack expand until failure occurs in the upper beam hoops. This mode of damage is observed in specimens 1 and 3 reported in [17].

Figure 9 compares the numerical and experimental load-displacement curves for the specimens with the integral hollow concrete blocks infill and regular reinforcement details. The numerical results show a good agreement with the experiment ones in terms of the initial stiffness, but the ultimate resistance of the FE model is somewhat lower than the experimental values. In Figure 10, a comparison is made in terms of the cracking pattern of specimen 3 and its FE model. The two patterns are in good agreement.

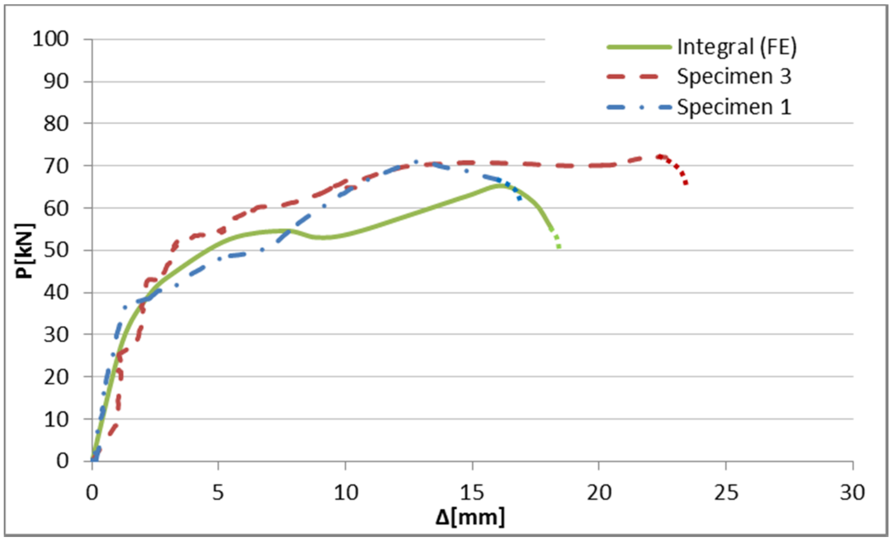

Figure 9: load-displacement curve for specimens with integral infill wall

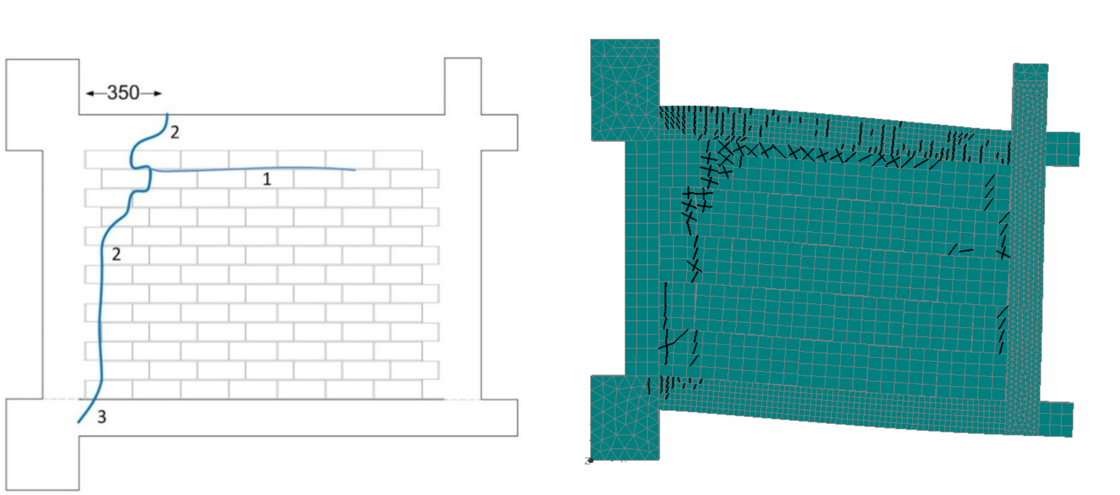

Figure 10: Cracking pattern in specimens with integral infill wall. Specimen 3 [17] **(left) and FE model (right).**

Bare frames

To evaluate the contribution of the infill wall to the bare frame resistance to the action of a vertical load in the case of loss of a supporting column, additional analysis was carried out on the bare RC frame with no infill wall. The results indicate that the resistance of a frame with flexible columns is 24.1kN, while the resistance of a frame with stiffer columns is 30.6kN. Comparisons of the bare frame resistance performance with the overall composite wall resistance (Table 2) demonstrate the significant contribution (up to 500%) of the infill masonry wall to the resistance of the composite wall action.

CONCLUSIONS

This paper compares the results of an experimental investigation and numerical analyses of the response of a RC frame with unreinforced infill walls to loss of a supporting column. The comparative investigation contributes to the understanding of the wall's behaviour, the damage evolution, and its ultimate resistance. The major observations are: (1) infill walls increase the frame resistance. (2) The failure modes of an infill wall in vertical displacement is different than the known failure modes in lateral displacement. (3) The frame failure dictates the maximal vertical resistance. These phenomena, which have been observed in the experiment, have been well captured by the numerical analysis. The comparison has also shown that the interaction between the infill wall and the surrounding frame is a critical feature that defines the infilled frame ultimate resistance. This study and further investigations may contribute to improved structural analysis tools as well as design guidelines that will take into account the interaction between the frame and the infill wall. A sound consideration of this interaction effect may contribute to the design of more robust buildings with improved ability to respond to the event of loss of a supporting column.

ACKNOWLEDGEMENTS

The research for this paper was financially by The Israeli Ministry of Science, Space and Technology (MOST), grant no. 2021954.

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