Infill Walls Contribution on the Progressive Collapse Resistance of a Typical Mid-rise RC Framed Building

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Abstract. This study investigates the effect of the autoclaved aerated concrete infill walls on the progressive collapse resistance of a typical RC framed structure. The 13-storey building located in Brăila (a zone with high seismic risk in Romania) was designed according to the former Romanian seismic code P13-70 (1970). Two models of the structure are generated in the Extreme Loading® for Structures computer software: a model with infill walls and a model without infill walls. Following GSA (2003) Guidelines, a nonlinear dynamic procedure is used to determine the progressive collapse risk of the building when a first-storey corner column is suddenly removed. It was found that, the structure is not expected to fail under the standard GSA loading: DL+0.25LL. Moreover, if the infill walls are introduced in the model, the maximum vertical displacement of the node above the removed column is reduced by about 48%.

1. Introduction

Over the last 50 years several catastrophic collapses took place, such as those of the Ronan Point apartment building (London, 1968), the Alfred P. Murrah Federal Building (Oklahoma, 1995) and the World Trade Center (New York, 2001). U.S. General Service Administration and Department of Defense have developed two guidelines: GSA (2003) [1] and DoD (2005) [2]. Both guidelines provide an independent methodology for minimizing the potential to progressive collapse in the design of new and upgraded buildings, and for assessing the potential to progressive collapse in the existing buildings. In 2013, the new versions GSA (2013) and DoD (2013) were issued.

GSA (2003) Guidelines [1] define the progressive collapse as "a situation where a local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause". In the initial phase of the structural design, it is recommended that the following characteristics be considered: redundancy, continuity and ductility. GSA (2003) Guidelines [1] are based on *Alternate Path Method*, that allows local failure to occur, but provides alternate load path to bridge over the damage and to avoid the collapse. This method is intended to enhance the probability that if localized damage occurs as the result of an abnormal load, the structure does not fail by progressive collapse or the damage will extend disproportionately to the original cause.

There are many experimental [3-8] and numerical [9-16] studies regarding the progressive collapse risk assessment of RC framed structures, but very few of them have considered the interaction between

the frame elements and the infill walls. A complete analysis should include the effect/contribution of the secondary elements, such as infill walls, in the evaluation of the structural response.

Sasani [5] has evaluated experimentally and numerically the response of a six-storey RC infilled-frame structure, following the simultaneous removal of two adjacent exterior columns. The building had infill walls only on the 2nd, 4th, 5th and 6th floors, as part of the demolition procedure, the infill walls in the 1st and 3rd floors were removed. The infill walls were made of hallow clay tiles. Experimentally, Shan et al. [8] have investigated the progressive collapse resistance capacities of two four-bay and two-storey RC frame specimens (with and without infill walls) and studied the role that the infill walls played in the progressive failure process of the RC frames. The experimental results and observations regarding the variation in the resistance force, horizontal displacement of joint, strains in the beams and columns, development of cracks in infill walls, beams and columns, rebar fracture and failure mechanisms are provided.

Tsai and Huang [10] have numerically examined the influence of brick-infill partitions on the progressive collapse resistance of a 10-storey RC building, moment-resisting frame structure with a 2-story basement. Equivalent compression struts are used to simulate the brick-infill panels. Xavier et al. [12] have investigated the influence of unreinforced masonry panels on the robustness of multi-storey buildings under sudden column loss scenarios. Recently, Helmy et al. [14] have studied the contribution of the non-structural infill walls in prevention of the progressive collapse of a 10-storey RC framed structure subjected to the loss of a primary vertical support. In Romania, only Lupoae et al. [15] have performed nonlinear dynamic analyses on a 6-storey RC framed building, with and without brick-infill walls, to highlight their importance in reducing or increasing the potential for progressive collapse.

The main objective of this study is to assess the influence of the autoclaved aerated concrete (AAC) infill walls on the behaviour of a typical mid-rise RC framed building subjected to the removal of a first-story corner column. The 13-story existing building from Brăila (a zone with high seismic risk from Romania), was designed in 1972 according to the former Romanian seismic code P13-70 [17]. None of the previous investigations focuses on the effect of the AAC infill walls on the progressive collapse resistance of an old RC framed building, designed 45 years ago. Based on the nonlinear dynamic procedure recommended by the GSA (2003) Guidelines [1], two numerical models (a model without infill walls and a model with infill walls) are analysed in the Extreme Loading® for Structures computer software.

2. Building details

The 13-story building erected in 1974-1975 in Brăila (Romania) was designed according to the much more permissive Romanian seismic code P13-70 [17]. The existing structure was "tested" by four major earthquakes that occurred in Romania: in 1977 with $M_w = 7.4$, in 1986 with $M_w = 7.1$, on 30 May 1990 with $M_w = 6.9$ and on 31 May 1990 with $M_w = 6.4$, where M_w is the earthquake moment magnitude. Since 1986 the building was seismically instrumented and its structural response has been closely monitored. The technical reports have shown that the building resisted without significant structural damages.

The 13-story structure consists of five bays of 6.00 m in the longitudinal direction and two bays of 6.00 m in the transverse direction. The total height of the building is 37.45 m. The current floor height is 2.75 m, except for the first two floors which have 3.60 m. The dimensions of the beams and columns are given in table 1. The thickness of the slabs is 15 cm and the total thickness of the AAC exterior walls is 25 cm.

Table 1. Dimensions of the structural elements.

Floor	Transverse beams [mm]	Longitudinal beams [mm]	Columns [mm]
1st, 2nd	350x700	350x650	700x900
3rd, 4th, 5th	350x700	350x650	700x750
6th, 7th, 8th, 9th	300x700	300x650	600x750
10th, 11th, 12th, 13th	300x600	300x550	600x600

In addition to the self-weight of the structural elements, supplementary dead loads (DL) of 2.2 kN/m^2 is considered on the current floor and 2.0 kN/m^2 on the roof floor. The live loads (LL) are: 2.0 kN/m^2 on the current floor and 2.5 kN/m^2 on the roof floor. The building was seismically designed according to the former Romanian code P13-70 [17] and the total seismic force is S = 0.037G, where G is the total weight of the structure.

Beams, columns and slabs are detailed according to the former Romanian code for concrete structures STAS 8000-67 [18]. The concrete class used is B250 and the steel type is: PC52 for longitudinal reinforcement bars and OB38 for stirrups. The reinforcement details of the structural elements are not provided in this paper; these are presented in the author's PhD thesis [19]. The properties of AAC masonry are calculated according to the Romanian design code for masonry structures CR6-2006 [20]. All the material properties used in the numerical model of the structure are presented in table 2.

Table 2. Material properties.

Material	Characteristic	Value
Concrete (B250)	Young's modulus	29 [GPa]
	Tensile strength	1.9 [MPa]
	Cube compressive strength	22 [MPa]
Longitudinal reinforcement (PC52)	Young's modulus	210 [GPa]
	Yield strength	340 [MPa]
	Ultimate tensile strength	520 [MPa]
	Ultimate strain	22 [%]
Lateral reinforcement (OB38)	Young's modulus	210 [GPa]
	Yield strength	260 [MPa]
	Ultimate tensile strength	370 [MPa]
	Ultimate strain	26 [%]
Autoclaved Aerated Concrete masonry	Young's modulus	1879 [MPa]
	Tensile strength	0.16 [MPa]
	Compressive strength	2.21 [MPa]

3. AEM numerical models of the building

In this study, Extreme Loading® for Structures (ELS®) – a very performant specialized software [21], which is based on *Applied Element Method* (AEM) is used. The structure is modelled in ELS®, as an assembly of small elements, which are connected together by three types of springs: one normal and two shear springs. The generation of springs is automatically performed by the ELS® software. These springs are important because they represent the continuity between elements and reflect the material proprieties used in the model. Strains, stresses and failure criteria are calculated using these springs [22].

In ELS®, the overall equilibrium set of equations in the dynamic problem is given by the relation (1). The solution of the equilibrium equations is solved using either a direct solver (Cholesky upper-lower decomposition) or iterative solver. The solution for dynamic problems adopts the step-by-step integration method (Newmark-beta).

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + [K]\{X\} = \{f\} \qquad (1)$$

where: [M] is the mass matrix, [C] is the damping matrix, [K] is the stiffness matrix, {f} is the external load vector and {X} is the displacement vector.

The constitutive models of materials used in the ELS® computer software are presented in figure 1. For concrete, the Maekawa compression model [23] is adopted. The relation between shear stress and shear strain is assumed to be linear until the cracking of the concrete. Then, the shear stresses drop down. The level of drop depends on the aggregate interlock and friction at the crack surface. For reinforcement, the model proposed by Ristic et al. [24] is used.
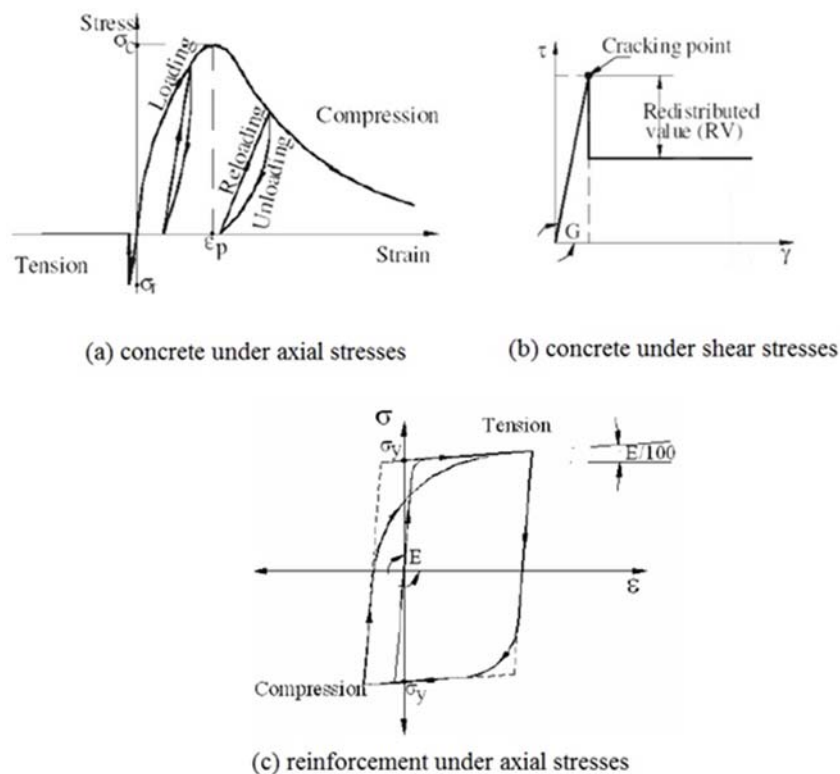

(a) concrete under axial stresses   (b) concrete under shear stresses

(c) reinforcement under axial stresses

Figure 1. Constitutive models for concrete and reinforcing bars used in ELS® [22].

Two three-dimensional models of the 13-story RC framed building are generated in the ELS® computer software: a model without infill walls (frame structure) and a model with infill walls. In figure 2 are comparatively illustrated the two AEM numerical models of the 13-story RC building. In the first model (figure 2a), the infill walls are considered only as uniform distributed dead loads on the exterior beams: 6.5 kN/m for the first-floor exterior beams and 5.0 kN/m for the rest of the exterior beams. In the numerical model, all the reinforcement details of the structural elements are explicitly introduced. The beams are considered as T or L sections to include the slab effect. In accordance with ACI 318-11 [25], the effective flange width on each side of the beam is taken by four times the slab thickness, value adopted also by Sasani et al. [4].

In the ELS® computer software there are two ways to define the masonry structures:

1) the real simulation – in which the bricks can be simulated as built in reality (in a staggered pattern, connected by mortar);

2) the macro-simulation – in which the wall is represented with relatively larger elements composed by bricks and mortar, and adopts models representing the average mechanical properties of both of them.

In this study, for the model with infill walls (figure 2b) the macro-simulation is used. The material properties of AAC infill walls introduced in the AEM numerical model are given in table 2.
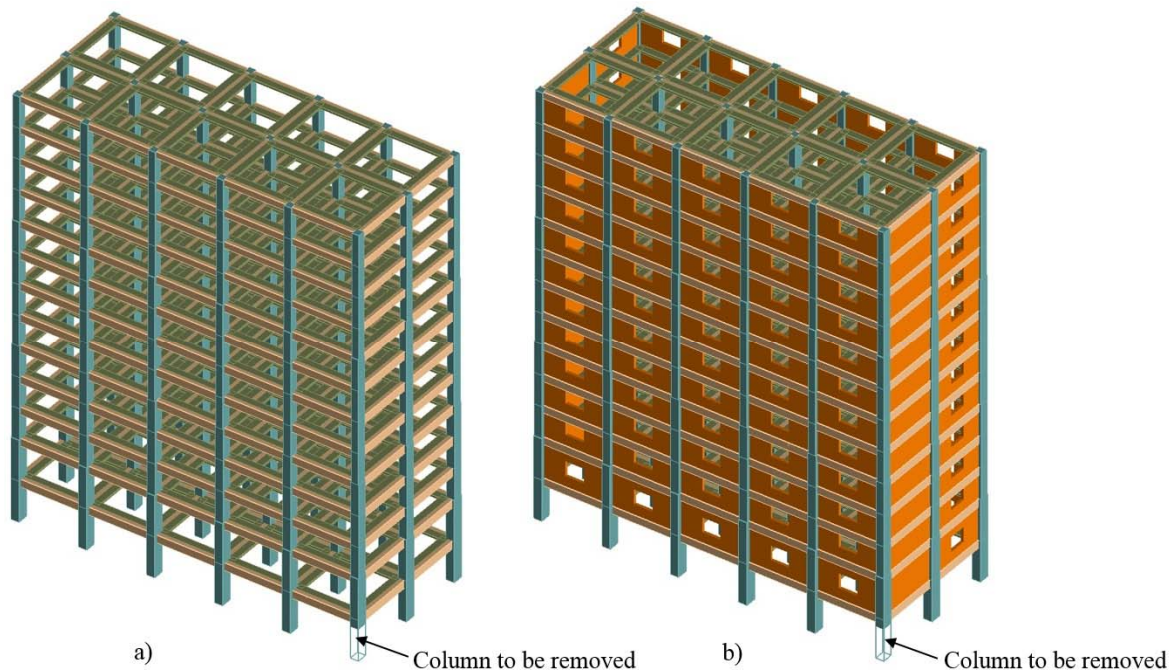

Figure 2. AEM numerical models of the 13-story building: a) without infill walls; b) with infill walls.

4. Progressive Collapse Analysis and Results

In order to evaluate the progressive collapse potential of a structure, GSA (2003) Guidelines [1] allows different analysis procedures: Linear Static (LSP), Nonlinear Static (NSP) and Nonlinear Dynamic (NDP). As nonlinear dynamic analysis gives better and more accurate results, in this study only this procedure is used. The NDP involves the following steps:

- Step 1: the gravity loads that shall be applied downward to the structure under investigation are:

$$Load = DL + 0.25LL \qquad (2)$$

  where, DL is dead load and LL is live load.

- Step 2: a first-story column is suddenly removed from the building. GSA (2003) Guidelines [1] recommends four damage scenarios: case C_1 – the removal of an exterior column located at or near the middle of the short side, case C_2 – the removal of an exterior column located at or near the middle of the long side, case C_3 – the removal of a corner column and case C_4 – the removal of an interior column. In the ELS® computer software, the time removal is set to t_r = 0.005 s. According to GSA (2003) Guidelines [1] the column must be removed in a time less than 1/10 by the period associated to the structural response mode for the vertical motion of the bays above the removed column. A damping ratio of ξ = 5% is used, value also adopted by Sasani et al. [4], Tsai and Lin [9] in the progressive collapse analysis of RC structures. A time step of t_s = 0.001 s is defined in the numerical model. To determine the expected material strengths, GSA (2003) Guidelines [1] recommends to increase the concrete compressive strength, respectively the yield strength and ultimate tensile strength for steel by a strength-increase factor of 1.25.

- Step 3: the response of the structure with respect to the vertical displacement of the node above the removed column is determined in a time span t = 3 s.

The 13-story frame model (without infill walls) is subjected to progressive collapse analysis in all four damage scenarios recommended by the GSA (2003) Guidelines [1]. The results obtained from the

nonlinear dynamic analysis regarding the vertical displacement of the node above the removed column over a time span of t = 3 s are displayed in figure 3. It is observed that, the structure is not expected to fail under the standard GSA loading = DL+0.25LL if a first-story column is suddenly removed.
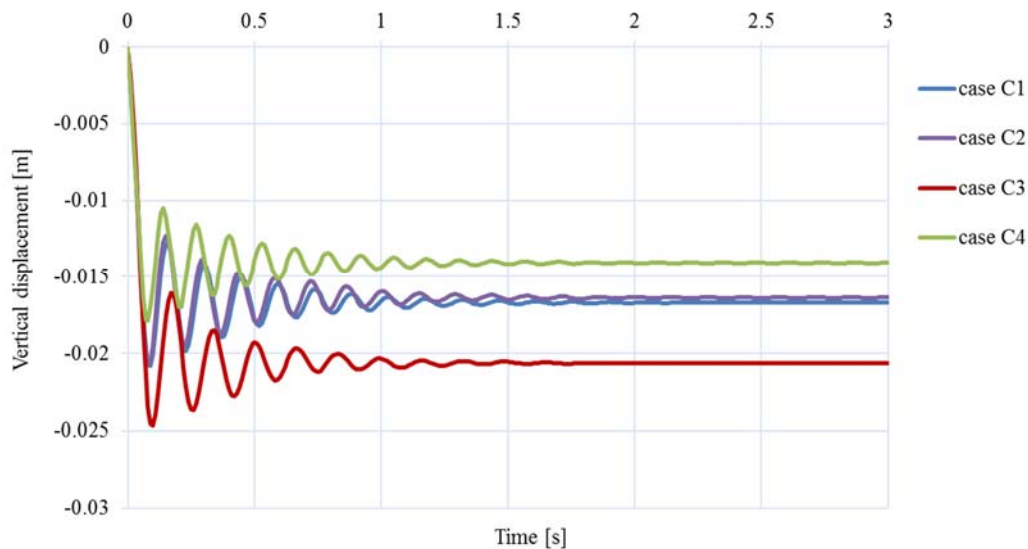

Figure 3. Time-vertical displacement curves obtained for the frame model (without infill walls).

The maximum vertical displacement is obtained for the damage case C_3, when a first-story corner column is suddenly removed from the structure. To assess the influence of the AAC infill walls on the progressive collapse resistance of the 13-story building, the numerical model with infill walls is subjected only to the removal of a first-story corner column. The response of the two numerical models (the model with infill walls vs. the model without infill walls) is presented in figure 4.
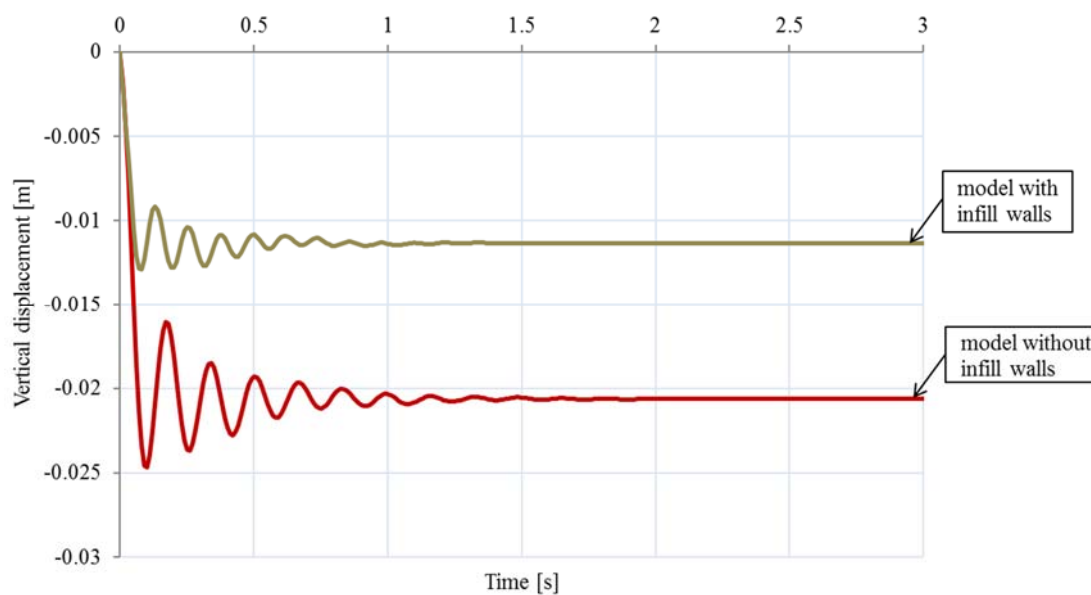

Figure 4. Time-vertical displacement curves: model with infill walls vs. model without infill walls (case C_3).

The results have shown that, if the AAC infill walls are introduced in the numerical model, for the damage case C_3 the maximum vertical displacement is reduced by about 48% (from 2.467 cm to 1.287 cm). Under this level of loads in the ELS® computer software was observed only some cracks in the beams and in the infill walls adjacent to the removed column.

In the authors' previous paper [26], the ultimate load-bearing capacity of the 13-story building model without infill walls was evaluated. It was found that, under the gravity loads 1.65(DL+0.25LL) the structure fails through progressive collapse when a first-story corner column is suddenly removed. If in the numerical model with increased loads 1.65(DL+0.25LL) would be introduced the AAC infill walls, it is very possible that the 13-story structure does not fail under this level of loads. In a future research, the authors will investigate the ultimate load-bearing capacity of the structural model with infill walls.

5. Conclusions

In the present paper, it was investigated the effect and contribution of the AAC infill walls on the progressive collapse resistance of a mid-rise RC framed building. The existing 13-storey structure was designed 45 years ago, according to the former Romanian seismic code P13-70 [17]. Following the methodology provided by the GSA (2003) Guidelines [1], a nonlinear dynamic procedure is performed for the four damage cases, when a first-storey column is suddenly removed.

The results obtained in the ELS® computer software, which is based on AEM method, have shown that the 13-story building is not expected to fail under the standard GSA loading = DL+0.25LL. For the frame model (without infill walls), the maximum vertical displacement of the node above the removed column was obtained in the damage case when a corner column is removed from the structure (case C_3). The model with infill walls was analysed only in this damage case, and the results were compared with those obtained for the model without infill walls.

It was proved that the existence of the exterior AAC infill walls in the AEM numerical model significantly affects the response of the structure following the removal of a column. The AAC infill walls, considered as secondary structural elements, lead to an increase in the strength and rigidity of the structure. Therefore, for more accurate results in the progressive collapse analysis and especially in the assessment of the robustness index of a structure, it is recommended to introduce in the numerical model not only the beams, columns and slabs, but also the existing infill walls as well.

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