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PROGRESSIVE COLLAPSE EVALUATION OF AN INDUSTRIAL BUILDING – NUMERICAL APPROACH

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Abstract*: The paper presents the evaluation of the potential of occurrence and propagation of the progressive collapse for an industrial building made of precast and cast in place elements. A model was developed for the numerical evaluation using the method of applied elements and a demolition scenario was used through which the column was instantaneously removed. The potential of occurrence and development of collapse was studied, using scenarios of column removal in according to GSA 2003 as well as scenarios in which two or three columns were removed. The evaluation was based on the determination of the rotations of the beam ends. The obtained results highlighted the importance of the manner in which the structure is designed and built, of the height regime and structural conformity, on the collapse.*

Keywords: applied element method, progressive collapse, precast concrete elements.

1. Introduction

The issue of the progressive collapse for the reinforced concrete structures is well addressed in the specialized literature considering the existing regulations and design codes [1-6] as well as the numerous published papers [8-16]. Under the existing regulations [2] two alternative design methods are provided to ensure the necessary strength of the buildings to this kind of phenomenon: the **direct and indirect** methods. The evaluation of the potential of occurrence and propagation of the progressive collapse, using the direct method [4-5], involves the removal of some of the vertical support elements in order to test the capacity of the structure to redistribute the additional resulting efforts.

Numerical analyses were performed according to the specifications of the existing norms [4-5] taking into account nonlinear dynamic analyses [11-14], dynamic linear and nonlinear analyses [10] and also static and dynamic, linear and nonlinear analyses [15]. The instantaneous removal of the support element according to GSA 2003 does not depend on the type of event which led to its destruction. This approach, although justified from the point of view of the simplification of the analysis has the disadvantage that in the case of an explosion the effects on the structure can be significant and the structural response can be considerably different in comparison to the case of the instantaneous removal.

The current paper approaches the structural response, when one, two or three columns were removed, considering different configurations, was determined using the method of applied elements. The aim was to asses the manner in which the efforts are redistributed mainly through the evaluation of the rotation of the beams.

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2. Numerical simulation

2.1Applied Element Method (AEM)

For the structural model the applied element method was used, which combines features of both the finite element method and the discrete element method. The main advantage of this method is that it can describe the behavior of the structure from the application of the forces, the crack propagation and the separation of the structural elements to the total collapse [17].
The structure is modeled as an ensemble of small elements, with special shape and determined dimensions. These types of elements do not deform, the change in their position being considered as for a rigid medium. AEM elements are connected using the entire surface of the elements, through a series of connecting springs that adopt all the material types and properties. Each group of springs completely represents the stresses and deformations of a certain volume and each element has six degrees of freedom. This modeling method allowed the study of the initiation and propagation of cracks and of the failure of the structure using only one initial model. The location of the failure is determined during the cyclic process.

2.2 Geometrical Model

The geometrical model of the hall is shown in the following figure (Fig. 1). The considered loadings are only those from the self weight of the building and a load of 100 kg/m^2 from the existing finishes.

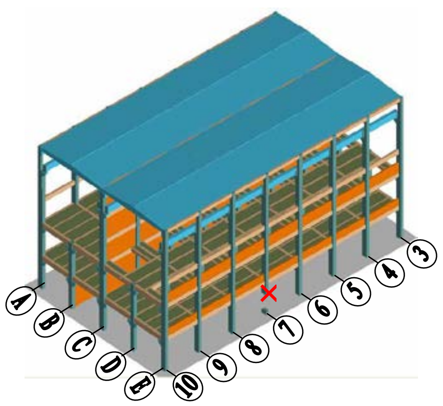

Fig. 1 Geometrical model of the hall

In order to model the longitudinal reinforcing of the beams as well as the rebars which make the connection between the longitudinal and transversal beams and columns, new reinforcing styles were defined for the longitudinal reinforcing within the program. For the longitudinal beams a reinforcing style was defined in which two rebars are lifted at the upper side and are then passing through the column, in the vicinity of the node. For the transversal beams, the lifted rebars are mark 6 and the rebars mark 4 ensure the connection with the reinforcing bars mark 5 from each beam, stopping at the column, Fig. 2.
For the cast in place concrete model a beam without reinforcing bars was defined and the connection with the column was made through the mark 7 stirrups of the haunch.

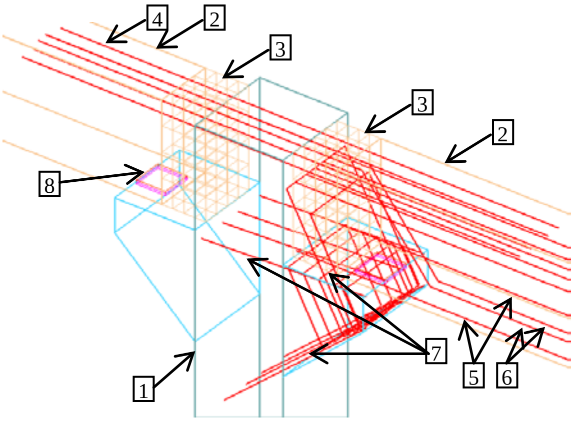

Fig. 2 Defining a new reinforcing style for the transversal beams: 1-column, 2- transversal beams; 3-cast in place concrete; 4- longitudinal connection reinforcing, top; 5,6-beam rebars; 7-haunch rebars; 8-metallic plates.

The inclination of the stirrups occurred as a result of the transformation of a beam into an element with 8 nodes in order to obtain the shape of the haunch. Also, the connection between the transversal beam and the haunch was made using metallic plates (8) arranged both on the beam as well as on the haunch.

2.3 Column Removal Scenarios

The demolition scenario was used as a column removal scenario, being predefined within the program. This scenario is frequently used for the demolition with explosives and for the progressive collapse cases when the user knows the elements which are to be destroyed. During this demolition scenario both the elements which are to be destroyed as well as the time when they will be instantaneously removed are specified. The column removal was instantaneously done at the time t=0.00s. The advantage of using this method consists in the reduction of the computation time compared to the explosion solution.

Because the studied construction has a reduced height regime and was designed to withstand high loads, other scenarios of removal of the vertical supports were used besides those specified in GSA 2003, Table II. The column removal according to scenarios 1-4 was performed according to GSA 2003 (exterior column on the long side, short side and corner of interior column) and the other scenarios were based on 1-4 but were improved by removing other support elements as well. The notations from the column "Positioning" are considered according to Fig. 1.

Table II Column removal scenarios

Scenario	No. of removed columns	Layout	Positioning
1.	1	Exterior - long side	E-7
2.	1	Exterior - short side	C-10
3.	1	Exterior - corner	E-10
4.	1	Interior	C-7
5.	2	Exterior - long side Exterior - corner	E-9 E-10
6.	2	Exterior - short side Exterior - corner	D-10 E-10
7.	2	Exterior - long side Exterior - long side	E-7 E-6
8.	2	Interior Interior	C-7 D-7
9.	3	Exterior - corner Exterior - long side Exterior - short side	E-10 E-9 D-10
10.	3	Exterior - corner Exterior - long side Exterior - long side	E-10 E-9 E-8
11.	3	Exterior - long side Exterior - long side Exterior - long side	E-8 E-7 E-6

3. Results and discussions

3.1 E-7 Column Removal

The graphical representation of the vertical displacement of node E7, situated above the damaged column, obtained from the numerical simulation, Fig. 3, indicates a maxim value of 9.41 mm, different than the value of -9.61 mm resulted from the experimental measurements (a difference between them of 2%).

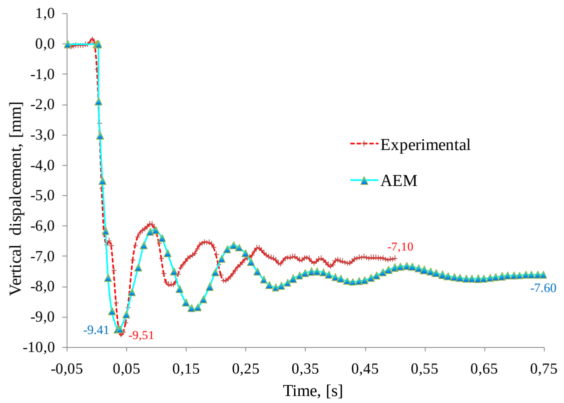

Fig. 3 Experimental and analytical vertical displacements of joint E7 for the second floor

There also appears a difference between the times at which the maximum values are recorded, namely 0.035s for the numerical solution and 0.041s for the experimental one. Also, the numerical results show a final displacement of 7.60 mm which is higher by 7% in comparison to the final experimental displacement of 7.10mm. It can be seen that the oscillations of the structure in the experimental case are damped faster than the ones from the numerical simulation.

After the removal of the column E7 a change in the shape of the bending moment occurs for the transversal or longitudinal beams connected to the axis of the removed column, Fig. 4. This change in the shape of the bending moment corresponds to the behavior of the structure which develops mechanisms capable of redistributing the additional efforts occurring as a result of the removal of a vertical support element.

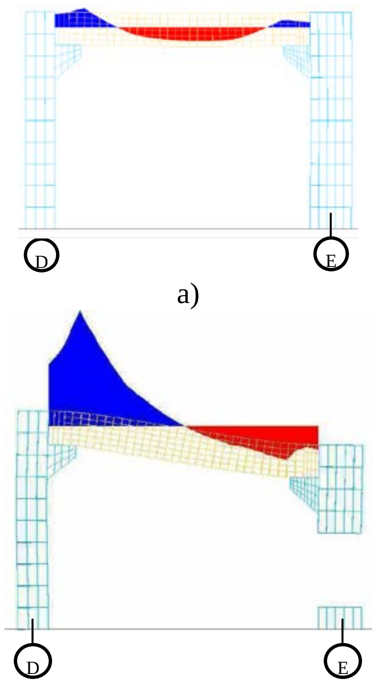

Fig. 4 Bending moment distribution in the transversal beam D7-E7 on the second floor before a) and after b) the column removal (not to scale)

Such mechanisms which can increase the capacity of the structure to resist failure (collapse) include: catenary action of slab and beams allowing the gravity load to span to adjacent elements; b) Vierendeel action from the moment frame above a damaged column and c) gravity load support provided by the nonstructural elements such as partitions and infills.

The catenary action in beams involves large deformations and utilizes tensile forces to balance the amplified gravity loads due to the doubling of the span (associated with the loss of a middle column) and the dynamic effect (associated with the sudden loss of the supporting force).

The Vierendeel action can be characterized by the relative vertical displacement between beam ends and the double curvature deformations of beams and columns. Such a deformed shape (Fig. 4, after the column removal) provides shear forces in beams in order to redistribute the vertical loads following the column removal.

An important observation is related to the shape of the moment around the support (the haunch) of the transversal beams, Fig. 4. It can be seen that the maximum moment is outside the support area and thus also outside the zone where the elements are working together. An immediate effect is represented by the cracking mechanism of the concrete for the beam D7-E7, Fig. 5. Thus, the cracking of the beam is more pronounced outside the support zone of the beam (on the haunch) and continues on the connection zone of the beam to the column but at a reduced level.

According to UFC [5] the beam and column rotations after the removal of a vertical supporting element must be checked. The rotation of the beams and columns is computed by dividing the maximum deformation Δ to the length L of the element (the length of the beam on the longitudinal direction or the distance between the support elements of the transversal beams), Fig. 5.

The limit value of the rotation angle for the beams according to table 4-1 from UFC [5] is 3.61 degrees. Similar values are presented in [18] as well: the average yield rotation is 0.34° and the average ultimate rotation of a plastic hinge is 3.4°, respectively. The difference between the maximum rotations of the two beams occurs due to the free length of the beams. If for the longitudinal beam this value is of 5.225 m (the distance between the columns E7 and E8) for the transversal one it is 4.175 m (the distance between the haunches of the columns E7 and D7). The maximum values of these rotations (0.10 degrees for the longitudinal beam and 0.12 degrees for the transversal beam) are under the limit values established in UFC.

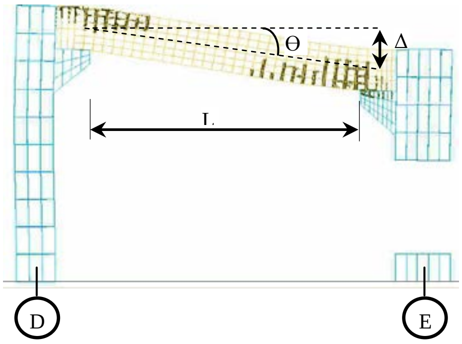

Fig. 5 Cracking state of the beam D7-E7 corresponding to the maximum bending moment (not to scale)

The graphical representation of the rotations of the longitudinal and transversal beams which connect in joint E7, above the damaged column, is shown in Fig. 6.

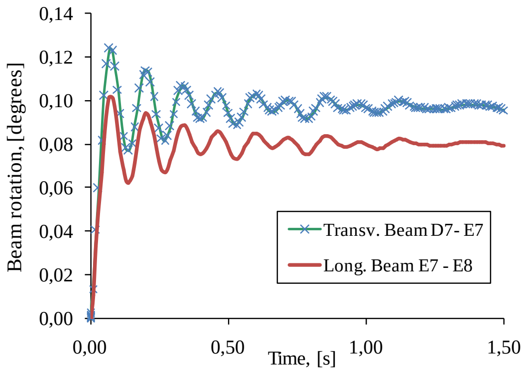

Fig. 6 The time variation of the rotation of the longitudinal and transversal beams after the removal of the column E7

3.2 Column Removal According to Scenarios 2-11

After the column removal according to the scenarios from Table I, the rotations shown in Table III were obtained. By comparing the values of the beam rotations from Table III with the limit value indicated in UFC it is found that only in scenario no. 10 the beam rotations exceed the failure limit of these elements. Thus, the collapse of the frame whose columns were destroyed occurs, Fig. 14.

The collapse does not propagate outside the frame on the one hand due to the transversal dimensions of the beams and columns adjoining the removed elements and on the other hand due to the structural conformity at the upper part, namely the lack of connection elements (slabs and beams).

Table III Rotation values for the beam ends considering the analyzed scenarios

Scenario	Transversal Beams		Longitudinal Beams	
	Notation[1]	Rotation[2] [degrees]	Notation [1]	Rotation [2] [degrees]
1.	D7-E7	0.12	E7-E8	0.10
2.	B10-C10	0.0276	C9-C10	0.0127
3.	D10-E10	0.203	E9-E10	0.138
4.	D7-C7	0.0085	C8-C7	0.0056
5.	D10-E10 D9-E9	0.79 0.51	E8-E9 E9-E10	0.14 0.35
6.	C10-D10 D10-E10	0.30 0.07	D9-D10 E9-E10	0.17 0.30
7.	D7-E7 D6-E6	0.30 0.27	E8-E7 E5-E6	0.17 0.19
8.	B7-C7 E7-D7	0.03 0.10	C8-C7 D8-D7	0.012 0.038
9.	C10-D10 D10-E10 D9-E9	0.75 0.64 0.97	E9-E10 D9-D10	0.29 0.50
10.	D10-E10 D9-E9 D8-E8	57 57 57	-	-
11.	D8-E8 D7-E7 D6-E6	0.51 0.82 0.52	E9-E8 E5-E6	0.38 0.37

[1]) The value of the rotation was determined for the end of the beam indicated in the second term of the notation (e.g.: D7-E7 – the rotation is determined for the end of the E7 beam).
[2]) The limit value of the rotation angle for the beams, according to UFC [5], is 3.61 degrees.

For the scenarios performed according to GSA 2003 (scenarios 1-4) it is found that the highest rotations are those corresponding to the corner column removal E10 (0.203 degrees for the transversal beam and 0.138 degrees for the longitudinal beam), followed by those corresponding to the removal of the exterior column located in the middle of the longest side E7 (0.12 degrees for the transversal beam and 0.1 degrees for the longitudinal beam). For scenarios 1 to 4 the values of the rotations are smaller than the yielding limit of the reinforcing steel bars, 0.34 degrees according to [18].

For scenarios 5 to 11, the highest rotations of the beams are found is the 5th scenario, corresponding to the removal of the corner column E10 and of the adjacent one on the longest side E9. In this case the rotations of the transversal beams (0.79 for beam D10-

E10 and 0.51 for beam D9-E9) are larger than the plastic hinge development limit.

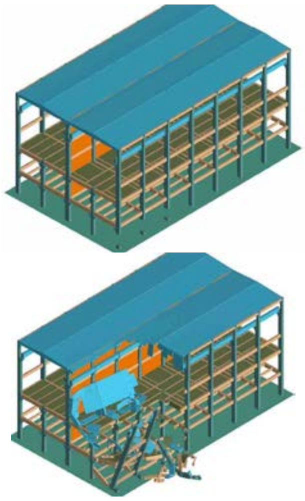

Fig. 7 Collapse of the frame E-D-10-9-8

The rotations of the longitudinal beams (0.14 for beam E8-E9 and 0.35 for beam E9-E10) are however lower than the corresponding yielding limit of the reinforcing steel bars. The final effect is: failure to initiate the collapse of the frame which had its columns removed.

4. Conclusion

The evaluation of the potential of occurrence and propagation of the progressive collapse for an industrial hall made of precast elements was performed by using numerical simulations. The developed numerical model, using the applied elements method, was validated by the experimental study. The numerical analyses performed by increasing the number of removed columns, starting from one column according to the GSA 2003 recommendations, and ending with three columns, led in just a single case to the initiation of the collapse. Even in this case, however, the collapse was limited to the frame whose columns were initially removed. No deterioration of the joints beam-column was observed in any of the analyzed cases due to the manner in which the connection of the precast elements was made (using cast in place concrete).

It was found that in most of the cases the displacements of the nodes located above the removed columns produced only the cracking of the reinforced concrete elements without leading to the yielding of the rebars.

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