THE PROGRESSIVE COLLAPSE EVALUATION OF BUILDINGS DUE TO BLAST ATTACKS

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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. The performed experimental test aimed to initiate the collapse by destroying, through explosion, a column situated at the ground level, on the contour of the building, at half the distance of the long side of the construction, using charges placed in boreholes. 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.*

***Keyword**: applied element method, explosive demolition, progressive collapse*

1. Introduction

Progressive collapse became a subject of interest for researchers especially after the partial collapse of the 22-storey tower in Ronan Point and its importance increased significantly after the worldwide escalation of terrorist activities, reaching its climax after the events of 11/2001.

The term progressive collapse [2] is used to describe the propagation, as a chain reaction, of a local failure thus leading to the total or partial crash of the building, the resulted damage being disproportional with respect to the initial cause. 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 [1-7] 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. The experimental studies

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performed by various authors on this subject consisted in the removal through explosion of an exterior column situated on the middle of the longest side of the construction [8], of an interior column [9], or of two exterior adjoining columns [10] and for the static tests mechanical jacks [11-12] or mechanical damages were employed [13].

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 issue of progressive collapse for structures with precast elements is less approached and discussed in the specialized literature. According to the specifications of GSA 2003 [4] the evaluation of the potential of occurrence and propagation of the progressive collapse in irregular structures or in those made of precast elements has to be treated with increased attention. Although progressive collapse may appear in almost all types of structures, failure in the precast structures may occur more often due to the lack of structural continuity in the nodes (in the connection zone of the elements). There are numerous papers which analyze the behavior of the precast elements [17-18] and their connections [19-20] at cyclic loads, fewer papers which investigate the seismic capacity of structures made of precast elements in order to determine their vulnerability [21] and even less which approach the aspects concerning the collapse of such structures [22].

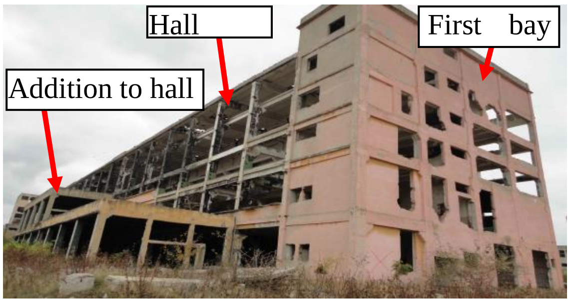


Figure 1 The section on which experimental tests were performed

The current paper approaches, in the first part, the response of a precast industrial structure made of reinforced concrete frames, in the case of the removal through blast of an exterior column situated at ground level on the middle of the longest side of the building.

2. Experimental test

2.1 Building Characteristics

The building used for the present study was part of a group of structures situated on the platform of an old chemical plant, built in 1980 but never put into service. When the tests were performed the entire industrial platform was decommissioned thus facilitating the possibility of conducting all the required experimental activities.

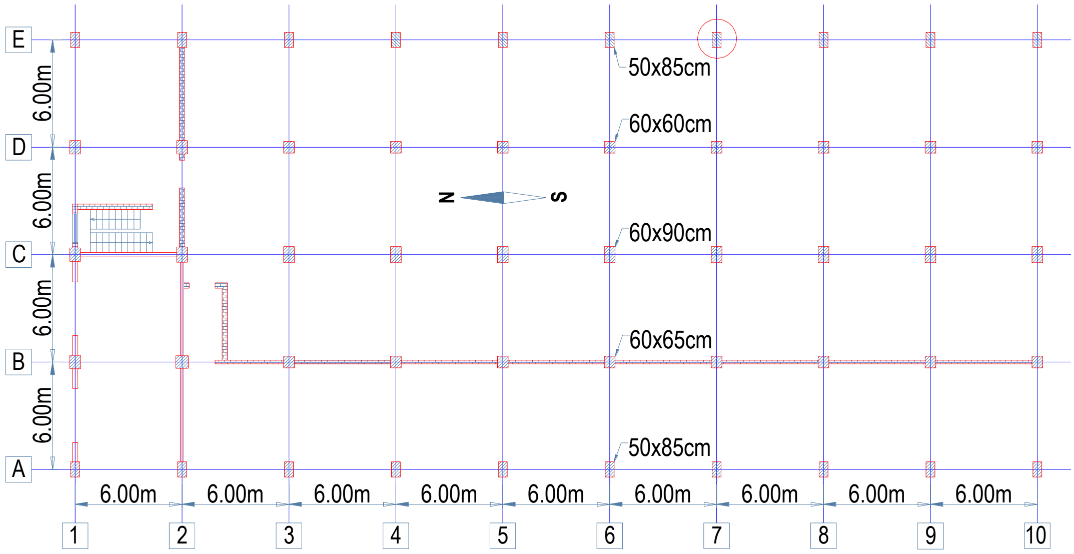


Figure 2 Axes plan of the investigated section of the building

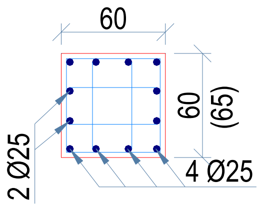
The building was made of four sections, designed as reinforced concrete structures. The main elements (columns, transversal beams, roof elements) were precast and the longitudinal and secondary beams, the intermediate floors and the access stairs were cast in place. The further description of the building will concentrate on the first section, Figure 1, the one subjected to the experimental study.

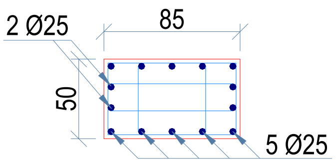
The structure of the section was regular in plan, with 9 bays and 4 spans, each having 6m, Figure 2. The first bay had the height regime G+5S, whereas for the other bays the height regime was G+2S (the last floor was built on the height of the 2nd, 3rd and 4th floors and on the first bay), Figure 1. The last level of the structure, between axes 2 – 10, was designed as a production hall with a high clear height (11.30m), with two spans each of 12m, with running beams placed on the cantilevers of the columns at the elevation of +17.75m, where gantries necessary for carrying out the production processes were installed. The intermediate floors were made of cast in place reinforced concrete (RC) elements having the thickness of the concrete slab of 20 cm for the first level and 15 cm for the second level. In order to efficiently withstand and redistribute the live loads, the floors had intermediate secondary beams (GS-25x50cm), arranged at a distance of 2m between the axes, on the longitudinal direction of the building. The precast RC columns were designed with cantilevers on which the transversal beams were supported. The geometrical dimensions and the reinforcing details of the columns are presented in Table I.

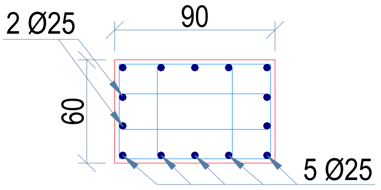
The transversal beams (A-E direction) were precast and supplementary concrete was cast in place over them, on the cantilevers of the columns, to ensure the appropriate structural behavior (all elements working together as a whole), Figure 3.

Table I Geometry and reinforcing details of the columns

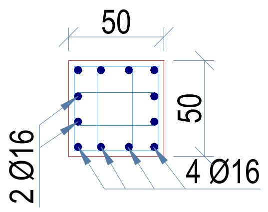
Columns for the ground, first and second level

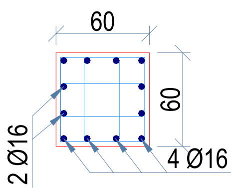






Columns for the third level





All the stirrups for the columns are Φ10/10cm

Two of the types of the longitudinal reinforcing bars of the beams (marks 3 and 4) stopped at the column and the rebars mark 5 along with the rebars mark 6 ensured the connection between the beam and the column at the upper part. Also, in order to withstand the shearing forces with high values at the beam ends, in the area of the supplementary cast in place concrete, the longitudinal rebars exiting the beam and the column were enclosed with stirrups Φ10/10cm, fixed on the cantilevers of the columns and locked in place after the assembly of the longitudinal rebars. At the lower part of the beam the connection with the column was made through metallic plates fixed both on the haunch of the column as well as on the lower part of the beam. The monolithic floors were cast in place after the fixing of these transversal beams which were designed to have at the upper part, on their entire length, elements ensuring their working together with the RC slab.

The longitudinal beams (3-10 direction) were cast in place together with the reinforced concrete slab and their working together with the column was ensured by the passage of some of the rebars of the beam through holes especially left in the column. Thus, at the bottom side, the reinforcing bars stopped at the contact with the column and the connection with it was made through other rebars, which passed through the column.

The slabs were cast in place together with the longitudinal principal and secondary beams. Reinforcing bars Φ10/15 cm were used on both directions, top and bottom. In the slab above floor 1, multiple holes having a technological purpose were made, with the dimensions 4.00x4.00m, which were bordered by secondary beams having the cross-sectional dimensions 25x50cm.

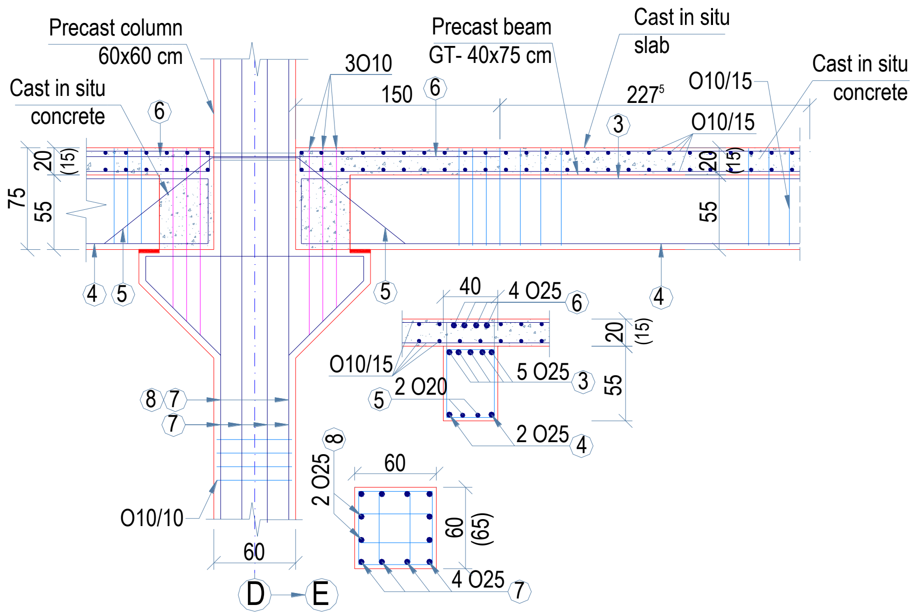


Figure 3 Transversal precast beam - precast columns joint details

2.2 Test setup

The aim of the experimental tests was the measurement of the vertical displacement of the node located directly above column E7, Figure 2. According to the GSA analysis scenarios for the occurrence and propagation of the collapse, a column situated at the ground level, in the middle of the longest side of the building, was removed. The removal was done instantaneously, through explosion. This kind of removal was possible because the construction was going to be demolished. For the instantaneous removal, explosive charges placed inside several boreholes made directly into the column were used, Figure 4.

a) The layout of the explosive charges network

b) Concrete damage and rebars bending after the detonation of the charges

Figure 4 The column before and after the detonation of the explosive charges

The employed method is the closest to the recommendations of GSA, namely the instantaneous removal of a support element, regardless of the type of threat. The energy produced after the detonation of the explosive charges was mostly consumed in the fragmentation and throwing of the concrete pieces, Figure 4b, and thus the aerial shock wave resulted from the blast had very little values and did not influence the behavior of the structure after the column removal.

Measuring the absolute displacement of the structure on the vertical direction was facilitated by the existence of an auxiliary structure. The gap between the two buildings was 5cm. For the displacement measurements an inductive displacement transducer was used. The body of the transducer was fixed on the column of the adjacent structure (the fixed column) by means of a steel plate whereas the plunger of the transducer was fixed using a hot rolled steel profile at 15cm above the slab of the first floor on the E7 column. The head of the plunger was fixed with a bolt nut on the hot rolled steel profile. Thus, the head of the plunger could move together with the structure of the hall after the destruction of the ground floor column.

3. Results and discussions

Following the detonation of the explosive charges placed in the boreholes made in the column E7 the concrete was thrown almost entirely, except in the zone of the column of the neighboring construction, Fig. 4b. The stirrups were straightened and some were thrown out of the column and the longitudinal reinforcing bars were bent. The bending of the longitudinal rebars occurred mainly due to the action of the shock wave and due to the propulsion of the concrete fragments and not as a result of the vertical displacement of the structure after the column destruction.

The vertical displacement of the node located above the blast damaged column is shown in Fig. 5. Immediately after the detonation of the explosive charges the sensor measured a displacement along the positive direction of the axis Z of approximately 1 mm as a result of the action of the shock wave on the metallic support on which the transducer was installed. After the damage of the column the structure did not directly displace until the maximum value was reached. It had a first displacement until the value of -6.5 mm at t = 0.02s, followed by a light displacement along the positive direction of the axis Z and then a downward displacement occurred until the maxim value of -9.51 mm at time t = 0.044s. As a consequence of the redistribution of the efforts between the structural elements, the structure then oscillated around the permanent value of the vertical displacement, respectively -7.1 mm. It is thus observed that the permanent final displacement represents 66% of the maximum recorded displacement. The maximum vertical displacement of the node located directly above the one destroyed by the blast is comparable in terms of size with the ones obtained by Sasani et al. [8] and and Sasani and Sagiroglu [9], for constructions with different height regimes and structural layouts.

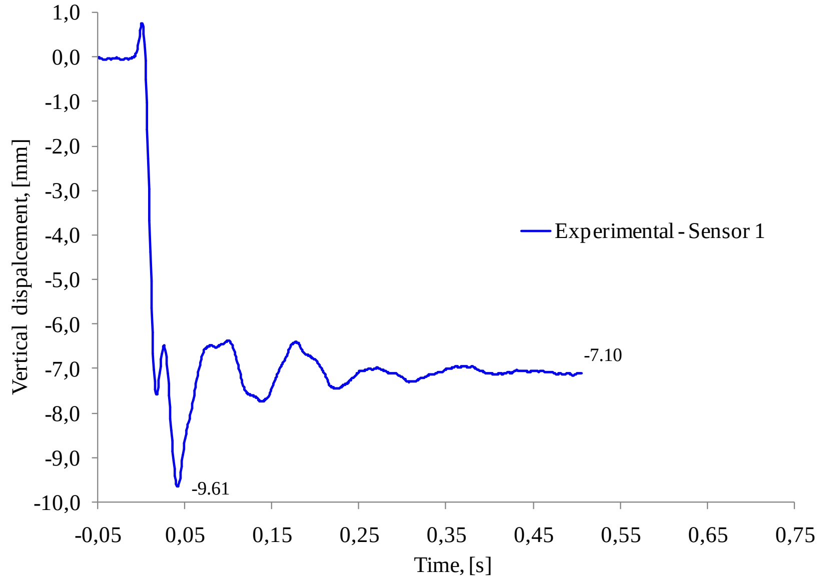


Figure 5 The vertical displacements of joint E7 for the second floor

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 experimental test.

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).

Based on the vertical displacement of the joint above the removed column it can be validated a numerical model of the building and after that a numerical evaluation of the progressive collapse under different scenarios can be performed.

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