# Finite element analysis of building collapse during demolition

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## 1. Introduction

Thousands of precast concrete panel buildings constructed in the past five decades throughout Europe are nowadays attaining their intended lifetime. For example, only in eastern Germany there are about one million unoccupied apartments – most of them in precast concrete panel buildings. Although large sums of money have been expended on retrofitting these houses, German ministry of construction recommended demolition of as many as 350 thousands of apartments for which retrofitting is not feasible. Expected costs of the demolition run at € 350 million, with some experts putting the figure even ten-times higher. To reduce these costs, new deconstruction methods are being developed. Due to their cost and time efficiency, demolition methods employing controlled explosions receive much attention. In order to safely and successfully perform the demolition, it is essential that appropriate sizes, placement and timing of charges are determined. To date, this process mostly relies on simplified mechanical analysis [4], empirical formulas, experience of demolition engineers, and verification on simple physical models. At the same time, experience with demolition of precast concrete panel buildings is rather limited and its full-scale experimental investigation is very costly. Furthermore, the traditional procedures of demolition design do not provide tools for its conscious optimization.

In order to facilitate a conscious design of safe and efficient deconstruction procedures, a methodology for FEM-based simulation of collapsing precast concrete buildings has been proposed recently [3]. In the next section we review the basic ideas and features of this methodology. The main objective of the present paper is to demonstrate the use of this approach to check and tune up the deconstruction design of an 8-story building.

## 2. Computational tool for simulation of demolition process

### *2.1 Basic strategy*

In a contrast to the standard structural analysis, when we want to simulate building demolition, the main interest is prediction of mechanical behavior of the structure during the phase when it disintegrates and loses static stability. The mechanical phenomena to be dealt with include material fracturing and yielding on one hand, and dynamic motion (finite displacements and rotations) and interaction of debris on the other. Since even separate numerical analysis of each of these phenomena presents a complicated task, when we have to consider them simultaneously, a suitable computational strategy has to be employed.

A typical precast concrete building consists of relatively stiff reinforced concrete members (panels), which are interconnected by rather weak joints. Structural failure in such a system usually occurs at or in the vicinity of the joints. The failure usually has a localized character and involves cracking and crushing of concrete and yielding and rupture of steel reinforcement. A detailed simulation of these phenomena generally requires two- or three-dimensional FE analysis with solid elements and nonlinear material models*). Despite ever-increasing computational power, such an analysis is feasible at the level of an individual structural element, but performing this way a geometrically nonlinear transient-dynamic analysis of an entire building would be too costly. On the contrary, the latter can be efficiently analyzed using beam or plate elements. Thus, we model the whole structure as an assembly of deformable beam elements interconnected by fracturing joints**).

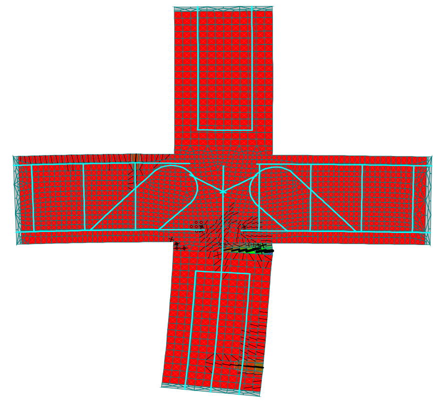

Figure 1: Typical failure of a joint predicted by meso-level analysis [3]

### *2.2 Fracturing joint and panels model*

In order to construct appropriate models of joints to be used on the macro-level, first, a series of meso-level analyses of typical structural joints and adjacent panels under various loading conditions was conducted. Since detailing of the joints as well as of the precast panels used to be standardized for each structural system, only a few analyses had to be done. Concrete and mortar were modeled using a fracture-plastic model for cementitious materials [1], which utilized the smeared crack approach to represent fracture. Reinforcement was assumed as elastic-plastic. The joints' and panels' details as well as material characteristics were obtained from ref. [6], design code [7], [2], and other period documents. The analyses were performed assuming 2-D stress state using commercial code ATENA.

The results revealed that localized cracks within and in a close vicinity of joints (Figure 1) dominated the structural response. It was concluded that to model the joint failure, it was sufficient to represent the cross sections weakened by localized cracks. When the cracks were bridged by reinforcement, it yielded in the vicinity of the damaged zone, and concrete crushing occurred near the compressed surface opposite to the crack. In addition, both wall and floor panels exhibited distributed cracking within their spans, which also affected their behavior.

On the basis of the meso-level results, simplified mathematical model that related the joint or panel overall loading (bending moment $M$ and axial force $N$) and deformation (curvature $\kappa$ and axial strain $\varepsilon$) was formulated. In an analogy with the smeared crack concept, the model was based on the consideration that cracking and reinforcement yielding occurred over a zone of finite width $w$. Since the outer surfaces of the fractured zone were assumed to remain planar, the overall strain within the fractured zone was linear along its height. After introducing appropriate nonlinear stress-strain relations for concrete and reinforcement, the $\{M, N\}$ vs. $\{\kappa, \varepsilon\}$ relations were derived. Note that the relations were coupled, nonlinear and covered the complete range of hardening and softening up to the complete separation of the fractured section. Consequently, these moment vs. curvature and axial force vs. strain

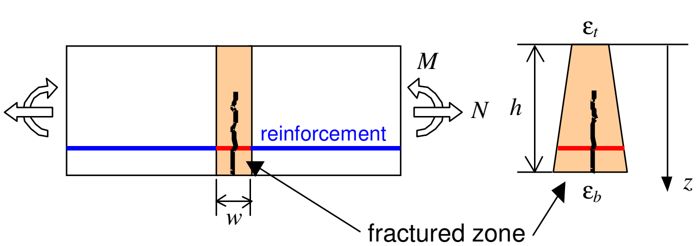


Figure 2: Geometrical assumptions of the cracked section model

*) We call this analysis on a *meso-level.*

**) This level of analysis will be referred to as *macro-level*.

Figure 3: The building to be deconstructed

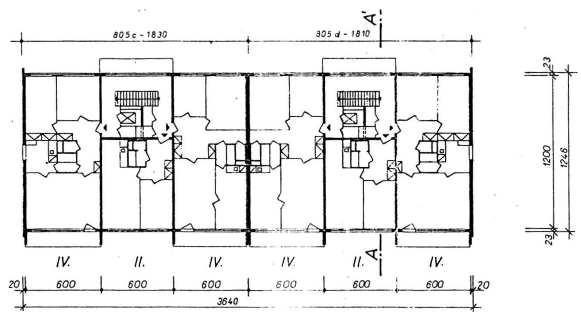


Figure 4: Typical story plan

relations were implemented into beam elements of the general purpose FE program ADINA®, which would be eventually used to analyze the structure on the macro-level.

## 3. Deconstructed building characteristics

The computational tool outlined in Section 2 has been used to verify and tailor demolition procedure for an abandoned building (Figure 3) located in the housing estate Chanov in northern Bohemia. The 8-story, 6-section building was designed and constructed in late nineteen seventies according to a unified structural system T-08BU, which used to be widely used in the Czech Republic for residential buildings construction. It was a precast reinforced concrete panel system with transversal bearing walls spaced at 6 meters and story height of 2.8 meters. Figure 4 shows the plan of a typical story. Longitudinal strengthening was provided by precast stairwells and stair flights. All other structures, including partitions and front/back facade panels were not part of the load-carrying system.

The bearing wall panels were made of lightly reinforced concrete (class B250), while prestressed concrete (also class B250) was used for the floor panels. The floor panels contained longitudinal voids to reduce weight. Figure 1 shows a typical joint of the transversal wall and floor panels. The reinforcement of the floor panels and that of the bottom wall panel was interconnected, but the connection to the top wall panel was unreinforced. The joint was filled with mortar and concrete class B250. The facade panels were attached to the transversal bearing walls. Further details can be found, e.g., in ref. [6].

## 4. Deconstruction process

### *4.1 Concept*

The deconstruction of the building is scheduled to take place in three phases:

a) removal of non-bearing structures, windows, technical equipment etc.,
b) demolition of the bearing structure,
c) crushing and recycling of the torn-down debris.

Since phases a) and c) will be performed in a traditional way, we pay attention mainly to phase b). Due to the presence of high-voltage power lines and other buildings next to the demolished object, it is desirable that the debris' spread is limited. To this end, demolition by means of a controlled vertical collapse is chosen. The easiest "brute force" way to cause the vertical collapse would be by blasting of all inner and outer transversal bearing walls on several floors. However, this solution would require the use of a large amount of explosives and consequently it would cause undesirable pressure wave and scatter of debris, mainly form the top floors. Thus, a neater approach is proposed as follows.

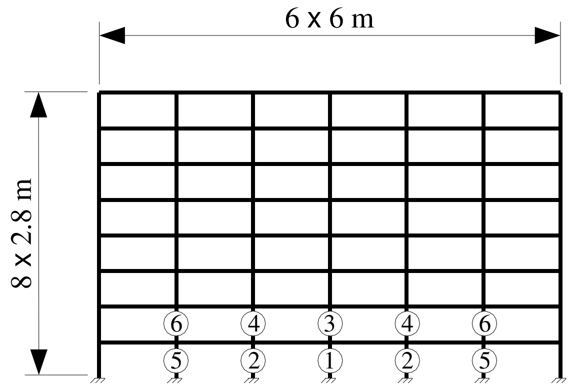

Figure 5: Vertical section of weakened structure with blasting locations

First, the structure is weakened by removal of the longitudinal strengthening system: the stairwells and stair flights. These structures are loosened by small charges, which do not cause global collapse of the building. Next, the inner transversal bearing walls are blasted on the ground and first floor, while the outer transversal walls remain intact and serve as a natural barrier against scatter of debris. The inner walls are removed in the order marked by numbers ① thru ⑥ in Figure 5. The order is determined so that the walls, weakened at their abutment, gradually get in vertical motion, tearing with them the adjacent floor panels. Since the demolition starts with the central wall and symmetrically evolves toward both ends of the building, the joints connecting the floor slabs to the walls are expected to undergo significant bending and fracture, which will ensure disintegration of the structure.

### *4.2 Computational model*

The expected failure mechanism as well as the appropriate timing of the charges are tested by the macro-level finite element analysis. Since the longitudinal strengthening structures have been removed, the building is modeled in 2-D. The walls, floors and joints are modeled by beam elements with rigidity relations discussed in paragraph 2.2. The mesh density is apparent from Figure 6*a*; only elements at joints are 2 cm long.

During construction of the building, the floor panels were placed on the walls first and consequently the joints were realized. To represent this process, the structural model is initially loaded by self-weight while the floor panels are attached to the walls through hinges. In the forthcoming steps these hinges are replaced by beam elements with joint properties.

The blasting-off of the transversal walls is represented by instant removal of the appropriate wall elements. The structure then becomes statically unstable and its consequent deformation and motion are solved as a transient dynamic problem in the finite displacements and rotations range. Damping is introduced through Rayleigh coefficients. The coefficients values, $\alpha = 0.923$ and $\beta = 0.000124$,

Table 1: Analyzed cases of charges timing

| Case | Timing (ms) at location | | | | | |
|---|---|---|---|---|---|---|
| | ① | ② | ③ | ④ | ⑤ | ⑥ |
| I. | 0 | 17 | 25 | 34 | 42 | 50 |
| II. | 0 | 100 | 200 | 300 | 400 | 500 |
| III. | 0 | 300 | 600 | 900 | 1200 | 1500 |

*a*) *b*)

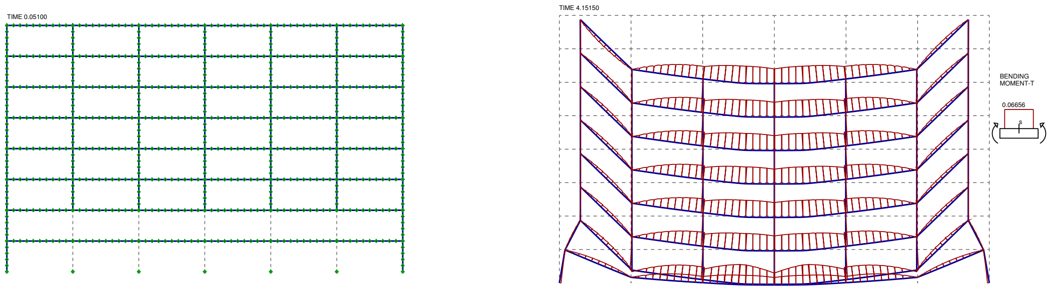


Figure 6: Predicted displacement and bending moment – timing case I (displ. mag. factor 1.0).

*a*) *b*)

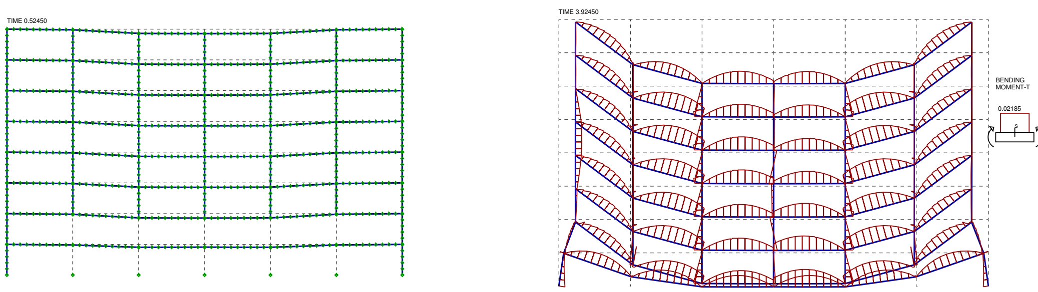


Figure 7: Predicted displacement and bending moment – timing case II. (displ. mag. factor 1.0)

*a*) *b*)

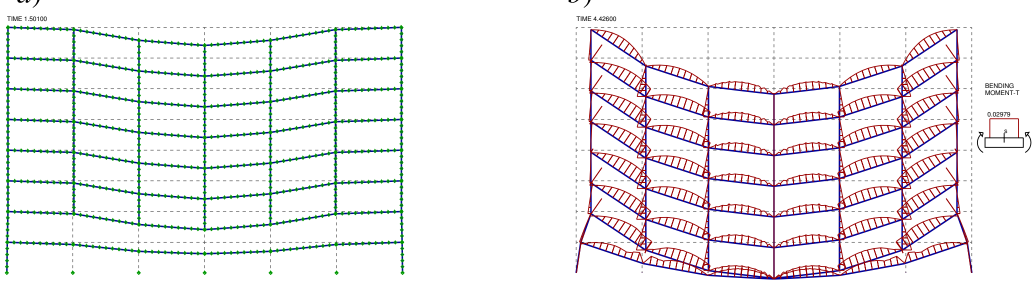


Figure 8: Predicted displacement and bending moment – timing case III. (displ. mag. factor 1.0)

have been determined from experimental data of the 1-st and 2-nd mode damping measured on cracked reinforced concrete beams [5]. Time integration is performed using the implicit Newmark method with time step $\Delta t = 0.0005$ s.

### *4.3 Numerical results*

Three cases of charges timing (Table 1) are analyzed. Timing of case I. is determined from the empirical notion, that material motion starts approximately 50~100 ms after the blast. To ensure that falling debris does not interrupt the ignition lines, the latest charge is detonated 50 ms after the first one. Figure 6*a* shows the original (dashed) and displaced FE mesh immediately after removal of the last walls no. ⑥. The structure exhibits almost no visible displacements, which means that the assumed time of initial material motion is too conservative. Figure 6*b* shows the distribution of bending moments on the displaced mesh when the lowest ceiling hits the ground. It is obvious that with exception of the outmost sections, the ceiling joints are still capable of transmitting bending moment and thus have not completely fractured yet. Therefore, it is concluded that the desired disintegration of the structure cannot be guaranteed.

In order to allow fragmentation of the ceiling structure, the time intervals between the charges have to be longer. We consider two more cases with intervals of 100 ms and 300 ms (Table 1). Figure 7*a* and Figure 8*a* show the state of the structure just after the last blast, while figures *b* depict the displacements and bending moments around the instant when the lowest ceiling hits the ground level. From figures *a* it is obvious that even with the longer intervals, the collapsing structure is not likely to damage the ignition lines. In both cases II. and III. the pull of the inner part causes the outer walls to fail and fall inward. Figure 7*b* indicates that in case II., the floor panel joints on the symmetry plane still exhibit some bending resistance. On the contrary, Figure 8*b* shows a complete loss of flexural strength of ceiling joints starting from the symmetry plane. Referring to paragraph 4.1, it is desirable that the structure starts to collapse from inside to ensure its complete disintegration. Thus, use of the timing scheme III. is recommended.

## 5. Concluding remarks

A computational strategy that employs a multi-level approach to model the physical phenomena that occur during a structural collapse was used to simulate demolition of a multi-story precast building. It is noted that due to numerous uncertainties in the input data (material properties, actual quality of workmanship, etc.) and complexity of the physical problem, the analytical results should be interpreted rather qualitatively than quantitatively. Nevertheless, the presented approach appears as an efficient way to verify whether the intended collapse mechanism takes place. By simulating various blasting scenarios, it is possible to optimize the demolition procedure.

## 6. Acknowledgements

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