Progressive Collapse of flat slab structures – key elements and connections

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Abstract. The loss of even one vertical support member can lead to catastrophic consequences and the progressive collapse of big portion or even the entire structure of the building. This report presents a computational study of a single-story, non-scaled 3D fragment of a high-rise concrete flat slab structure building with peripheral beams along the facade in case of dynamic failure of a column. The main objective is to examine alternative load paths for the transfer of stress from the peripheral beams to the columns, across the slab and its punching areas. In case of accidental failure of an outer column, the facade beams serve as prevention which reduces the redistribution of loads to the interior, i.e. to the columns with critical slab punching areas. Research on the performance of peripheral beams and the key role of corner columns is the focus of this report.

1. Introduction

Accidental actions are extremely short-lived and unlikely to occur, but their consequences could be catastrophic. For example, a progressive collapse with a high rate of destruction. Examples for accidental situations are explosions, vehicle impacts, floods, landslides.

In accidental design situations, limited damage to structural elements is possible and allowed. In these cases, instead of the criterion of ultimate limit state, the requirement for robustness is introduced: the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause [1].

Dozens of examples of disproportionate (or even total) collapse of buildings result from terrorist attacks or unintentional human errors, lead to comprehensive experimental and computational research in recent years worldwide.

The approach to survey the robustness of structures in case of accidental actions through scenarios of local failure of a single element and the search of alternative load paths for transferring the stress is considered to be an universal approach, and is applied in experimental research and in applied problems solving computational studies [3].

Critical for flat slab structures is the "fragile" punching shear phenomenon in the areas of the slab-column connections. Punching shear resistance has been studied in detail under static load and is established with sufficient certainty by modern norms and standards. The experience with accidents involving these punching areas in cases of seismic impacts, especially in irregular structures with significant rotation, is one of the reasons for the construction of beams in the peripheral of the building, especially when the slabs are without cantilever parts.
In any case, flat slab structures are most sensitive to accidental actions and the control over horizontal and vertical progressive collapse process is most challenging. (see figure 1). In this regard, the study of the above-described modified flat slab structures, set in a situation of a dynamic failure scenario of a penu-ultimate-external column, caused (for example) by external explosive impact, can be assessed as relevant.

Figure 1. The stages of the “pancake”-type progressive collapse

Modern research on critical dynamic punching shear areas of flat slabs is extremely insufficient, and also any kind of experimental or computational survey on parts of flat slab structures is lacking. This report presents a computational study of a single-story, non-scaled 3D fragment of a high-rise concrete flat slab structure building with peripheral beams along the facade after a dynamic failure of a column. The main objective is to examine alternative load paths for stress transfer from the peripheral beams to the columns, across the slab and its punching areas.

2. Experimental Program and FEM analysis
This research is a part of a broader program of computational analyses examining the behaviour of flat slab structures in case of dynamic failure of a column. The research is analytical and is necessary due to the extremely complex nature of the phenomenon of progressive collapse. This requires an explicit nonlinear dynamic analysis of the intended tasks - finite element modelling is applied using the explicit reinforcement contact model in ANSYS Autodyn [4].

Input values for concrete properties were those of the Riedel-Hiermaier-Thoma (RHT) concrete model with compressive strength class is C30/37. The failure mode of the concrete model was selected as a tensile failure to simulate the behaviour of a real concrete specimen. The tensile strength was assumed 10% of the concrete compressive strength [7]. The material model of structural steel is elastoplastic with linear hardening and strength class B500C [2].

The considered problems are tested in the conditions of modelled earth gravity. This acceptance aims to take into account simultaneously both the distributed permanent loads and the normalized quasi-constant part of the other variable actions.

The speed of implementation of the external force and displacement in the segment of local damage (failure of reinforced concrete column) is consistent to the fragile nature of elements under pressure.

3. Progressive collapse analysis - objective and models

3.1. Objective
In case of accidental failure of an outer column, the facade beams serve as prevention that reduces the redistribution of loads to the interior, i.e. to the columns with critical slab punching areas. The focus of this experimental program is to examine the behavior of peripheral beams and the key role of corner columns in increasing the robustness of the whole structure.

3.2. Problems
The model is a vertical part of reinforced concrete structure of an 8-story administrative building with flat slabs. The spans are regular, 6.0 m in both directions and the floor height is 3.60 m (see figure 2). The slabs are without cantilever parts and the beams are distributed along the facade.
Figure 2. Details of model frame (unit: cm). (a) Plan of the model; (b) Column sections

The model reviews the full height of a vertical fragment, with two intermediate axes from the corner of the structure, mentally separated from it by conservative cantilever parts with a length of 1.40 m (see figure 2a).

The design of the structure takes under consideration the requirements of the Eurocode standards system. Without having to participate in the seismic protection, the frame elements respond to structural requirements of EN 1998-1.

The columns above the first floor, except for those that are subject to a dynamic failure, are loaded with the calculated values of the vertical forces, determined for accidental design situation of 7 floor levels. The aim is for the columns, which are expected to be subjected to redistribution of forces, to already have objective values of the pressure forces. The failure of the column is modeled by increasing dynamic force from 0 to 500 kN, with speed of dynamic failure about 110 kN/s (see figure 3).

Model 1-C: Simulates failure of the corner column C9.

Model 1-0: Simulates failure of column C8 on the first floor. In this model, the corner column C9 has a minimum admissible cross-sectional size of 30/30 cm.

Model 1: The main model, in which the datum from Model 1-0 has led to the experiment performed with an L-shaped corner column C9 (see figure 2b).
Figure 3. Analysis model

4. Results

4.1. Model 1-C: failure of the corner column C9

The goal is to analyze the magnitude of this accidental action in comparison to the failure of the adjacent penultimate-external column C8.

The simulation has a vertical dynamic action increasing from zero, (see point 3.2), applied in C9.

![Potential destruction at the node beam - column C8 at 3.94s; (b) Condition of the node with C9, where the force is applied at 3.94s](image)

Critical for the structure is the moment 3.94s right before the failure of longitudinal reinforcement in the facade beam. The load is 458 kN with a displacement of 455 mm in the spot of the removed column C9. The deformed state of the beam and the connections with the columns (see fig.4) define the shape of the next failure. The critical areas are the columns C8 and C6 (they are symmetrical), which support the cantilever part, formed after the failure of column C9.
The total collapse of the model's angle at moment 4.5 s is with relatively very large value of the dynamic force, simulated in the corner (see figure 5).

**Figure 5.** Final condition (Model 1-C)

4.2. Model 1-0: failure of the penultimate-external column C8 with a cross-sectional size of column C9 -30/30 cm

The objective is to establish the effectiveness of the constraints in the angle of the structure. The corner column has to ensure the alternative load paths in the peripheral beams.

The deformation of the beam and the connections with the columns (see figure 7) define the manner of the next collapse. The catenary action in large deformations of the beam "retracts" column C9 to the inside. (see figure 6a). This bending, accompanied by high pressure from the upper levels, destroys the corner column before the failure of longitudinal reinforcement in the facade beam (see figure 6b). Such behaviour can be defined as fragile. The scenario of dynamic failure of column C8 and the subsequent collapse of column C9 is a typical case of chain breakdown (or progressive collapse).

**Figure 6.** Column C9 failed before (a) the longitudinal reinforcement in the beam reaches ultimate tensile stress at the connection with C8 (b).
4.3. Model 1: failure of the penultimate-external column C8 with a L-cross section of column C9 - 30/50/30cm

Critical for the structure is the moment 4.1 s, right before the failure of longitudinal reinforcement in the facade beam. The load is 400 kN with a displacement of 235 mm in the spot of the removed column C8. The failure of longitudinal reinforcement is at 4.19 s (see figure 8) with vertical deformations 320 mm.

![Figure 7. Final condition (Model 1-0)](image)

**Figure 7.** Final condition (Model 1-0)

![Figure 8. Longitudinal reinforcement of the beam reaches ultimate tensile stress at 4.20 s.](image)

**Figure 8.** Longitudinal reinforcement of the beam reaches ultimate tensile stress at 4.20 s.

![Figure 9. Beam - column details at 4.20 s: (a) column C8; (b) column C9.](image)

**Figure 9.** Beam - column details at 4.20 s: (a) column C8; (b) column C9.
Figure 10. Tensile stress in longitudinal reinforcement of the beam.

The stress distribution in the longitudinal reinforcement (see figure 10) in the beam demonstrates its combined action - vierendeel action in the beam-column connections in combination with the catenary action of the frame beam [6]. Plastic hinges are formed in the middle near column C8 and at the end of the beam near column C9.

Figure 11. Longitudinal bottom reinforcement in the slab (a) tensile stress; (b) deformation.
The deformed scheme of the bottom longitudinal reinforcement of the slab between columns C5 and C8 shows larger displacements at the connection with the peripheral beam, but they are local. The stresses in this area are limited (see figure 1). Near the column, the vertical displacements of the slab and the reinforcement have induced significant tension stress in the longitudinal reinforcement, typical for a failure of transverse punching shear reinforcement. This is a display of alternative load path behaviour of the longitudinal reinforcement [5].

5. Conclusions
The manner of destruction shown in Model 1 can be assessed as ductile. The catenary action in the beam and the virendeel action in the beam-column connections are established and evident. Those alternative load paths ensure the robustness of the structure in case of dynamic failure of a column and are an effective prevention against the redistribution of the large forces to the punching shear areas in the flat slabs.

The analyzes show that the corner column is a key element that guarantees the alternative load paths in the beams, and the L-shaped column is one of the most proper solutions.

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