Investigation of monolithic frame with initial punching damage under accidental impacts

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Abstract. According to regulations and standards, all buildings and structures must fulfill the requirements for mechanical safety. At the same time, the safety of structures must be ensured throughout their entire service life. The annual increase in the number of accidents due to terrorist acts, domestic gas explosions, collisions of vehicles with load-bearing structures, etc. has led to the creation of normative documents on the protection of buildings from progressive collapse. At the same time, the issue of buildings sustainability, which were designed without taking into account these requirements and which may have suffered localized damage during operation, remains open. Such buildings include monolithic houses with flat slabs, which have been erected since the beginning of the XXI century. The aim of the work is to study the resistance of these frames to accidental impacts and the influence on the survivability of structures of local defects in the most critical zone of the column-slab junction. Calculations on stability assessment in case of emergency of a fragment of a 9-storey monolithic building have been performed and it has been established that at certain combinations of damages in the support zones, progressive destruction of neighboring structures may occur.

Keywords: stability, accidental effect, beamless frame, punching, dynamic analysis, progressive collapse

1 Introduction

When designing buildings and structures, it is necessary to ensure mechanical safety throughout the entire life cycle of the object. However, during the operation of buildings, the structure may be affected by various unfavorable factors that may lead to its local weakening. These factors may be related to unfavorable environmental effects: fire, seismic effects on structures and local mechanical loads of high intensity. In addition, there is a risk of errors in design or construction. As a result of one of these factors, defects may occur in local sections of the structure, which cannot always be controlled due to the difficulty of access for inspection, but which may affect the performance of structures, especially in case of emergencies. One of the basic principles in the design of buildings and structures with regard to protection against progressive failure is to localize the emergency area, with the exception of the scenario of sequential destruction of the system, by developing uniform plastic deformations in the nodes.

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At the same time, the question of ensuring the survivability of structural systems arose in the early 70-ies of the last century and is actively developing in today's realities [1]. One of the most critical places of the monolithic frame is the column-slab interface, the failure of which is usually brittle and can lead to significant consequences up to the chain failure of neighboring structures [2-5]. Both domestic and foreign researchers have paid special attention to this node [6-9]. At the same time, a significant part of the research was devoted to the operation of the node to prevent the realization of the mechanism. At the same time, a significant part of their work is devoted to the study of the stress-strain state under central load application without the presence of eccentricities and bending moments. Less attention has been paid to the performance of the node after the onset of punching [10-11]. However, through these works, it has been found that the interface node can remain in operation, taking less load and allowing the full realization of the operation of monolithic frames with the transition of the slab operation as a hanging slab "cable-stayed". To ensure the spatial work of the frames, requirements for hidden connections have been introduced.

A significant part of buildings with normal level of responsibility does not meet the requirements of modern normative documents on protection from progressive destruction and in case of emergency situation can be in the risk zone. The performance of the structures of these buildings has received less attention, there is no data on the consideration of these systems taking into account various defects that can significantly affect the performance of the system in case of emergency.

2 Materials and Methods

The main objective of the work is to determine the influence of damage in the support zone on the bearing capacity of the node and on the overall stability of the system in case of emergency. To achieve this goal, it is necessary to solve 2 tasks: to analyse the stress-strain state of the column-slab junction at different degrees of damage. Taking into account this evaluation, to study the performance of the system as part of the monolithic frame of the system under emergency conditions.

To obtain the required solutions, an explicit solver was used, which allows to significantly increase the calculation speed when solving quasi-static problems by direct integration method (Explicit solver: Abaqus/Explicit).

To model the artificial material (concrete), the damage plasticity model [12] was used. This plasticity model takes into account the formation and development of cracks in tension, compression and shear. Two main failure mechanisms are considered - crack formation in tensile concrete and crushing of compressed concrete. The performance of concrete was modelled using the concrete performance diagram according to national regulations.

When the material reaches the ultimate stresses or strains, the elements worked on a downward branch with the appearance and distribution of cracks in the volume. The stresses in the elements are found using the concrete damage coefficient, full and plastic deformations (see Figs. 1, 2). The concrete body in all schemes was modelled by volumetric cubic elements of C3D8R type with simplified integration and distortion control.

A plasticity model with two linear regions, the elastic work zone and the yield area, was used to model the rebars. The rebars were modelled by two-node truss-type bar elements (B31). It is worth noting that the loss of bond between reinforcement and concrete is not considered at this stage. It is accounted for indirectly in the concrete model by some elongation of the downward branch section. The relationship between the parameters indicated in the diagram can be described in the form of formulas (1) and (2).
The contact between the steel plates and the concrete surface was modelled using the hard contact tool. The coefficient of friction between concrete and metal plate was assumed to be 0.4 according to similar works [13]. Also, in order to more closely simulate the performance of the node in real tests, a function was used to account for small displacements and surface uplift of the concrete over the steel support. The finite element size was 25 mm.

Having analysed the works on the failure mechanisms of these nodes, it is proposed to identify 3 main damages, in which further operation of structures is possible without taking into account the occurrence of emergency situations (see Fig. 3).

That is, for the 1st level roughly corresponds to the load of 25-30% of the ultimate destructive load with the development of tangential and normal cracks on the tensile surface of the specimen. The 2nd level corresponds to a load of 30-50% of the ultimate load with

$$\sigma_c = (1-d_c) \cdot E_0 \cdot (\varepsilon_c - \varepsilon_{c_{\|^p}})$$

$$(1)$$

$$\sigma_t = (1-d_t) \cdot E_0 \cdot (\varepsilon_t - \varepsilon_{t_{\|^p}})$$

$$(2)$$

**Fig. 1.** Concrete deformation diagram under longitudinal compression

**Fig. 2.** Concrete deformation diagram under longitudinal tension

**Fig. 3.** Level of punching damage of slab-column joint
the development of tangential, normal cracks and radial cracks. The 3rd level corresponds to a load of 50-85% of the ultimate load with the development of tangential and radial cracks. The failure mechanism for penetration damage is shown in Fig. 4.

![Fig. 4. Surfaces of volume punching area](image)

On the basis of the accepted damage levels of the interface nodes it is proposed to assess the load-bearing capacity of a fragment of a 9-storey spatial monolithic frame (see Fig. 5).
The storey height was taken as 2.7 m. It is suggested to consider the most unfavourable, but still quite common frames with asymmetric spans and monolithic pylons. The thickness of the slab is 200 mm, with a protective layer of 25 mm. A uniformly distributed load of 200 kg/m² was applied to the frame, taking into account the self-weight load.

To reduce the computational capacity, the spanning sections of the floor slabs and the central sections of the pylons at the level of 1-3 floors were replaced by 4-node shell elements (shell S4R type elements) (see Fig. 4, white area). The slab dimensions of the volume piercing part did not exceed the length of 5 h (where h is the thickness of the slab), taking into account the possibility of operation of the piercing sections after the occurrence of push-through. In the properties of these elements the parameters for the rebar layer were specified. Three scenarios of emergency situation connected with removal of pylon in axes B/2 were considered:

1) Without local damage
2) In the presence of local damages of the 3rd level in the area of pylon B/1 (Variant 1).
3) In the presence of localized damage of the 3rd level in the area of pylons B/1 and A/3 (Variant 2). The bottom and top reinforcement of the slab was taken from the conditions of normal operation.

At the initial stage, in order to obtain a balanced state of the system, the support connection of pylon B/2 was replaced by a system of equal forces with an appropriate schedule of variation in time by analogy with [14-16].

3 Results

The results of calculation of different FE models without and with the presence of various levels of damage are presented. In this case, two factors served as criteria for failure of the specimens: occurrence and development of ultimate tensile deformations of concrete on the compressed surface of concrete, as well as a sharp jump in the displacement of the support part of the column through which the load was transmitted. Images of the development of deformations in the specimen as well as the load-displacement diagram of the support assembly are shown below (see Figures 6-9).

At the time moment t=4 seconds the removal of the pylon in axes B/2 takes place. Then the system is stabilised and the data is recorded for 15 seconds. For convenience of further analysis, the data are plotted by groups of pylons: corner, centre and end pylons.

The calculation of the system with damage according to variant 2 was terminated by the programme at time t=7.929 seconds. Therefore, the time interval from 0 to 10 seconds was considered when comparing the results with the frame without damage. According to the results of the monolithic frame calculation, the graphs of pylon support displacement in B/2 axes depending on the degree of damage of nodes at the supports were obtained.

Analysing the data obtained as a result of finite element calculations of the monolithic frame, it can be stated that the local damage in the column zone after the emergency situation affects the redistribution of force flows of the structure. It can also be noted that as a result of the span asymmetry there was overloading of some piers and their operation after the limit stage (pushover failure) (see Figs. 10-12).

Analysing the graphs of support reactions of the system without and with the presence of damage for the 1st variant, it can be noted that local damage to the node nearest to the removed pylon leads to additional loading of the node opposite. The general mechanism of deformation of the system from the point of view of the slab operation is quite similar when analysing the scheme without damage and in the presence of damage under variant 1:
sagging of the floor slab in the central part (in the area of pylon B/2, B/3) on the surrounding structures.

At the same time, in the presence of damage in variant 1, we note higher values of additional loads in the corner columns and a drop in support reactions in the end columns (see Fig. 6-9), which may indicate that the slab begins to sag uniformly along the B-axis.

**Fig. 6.** Ultimate load-bearing capacity and load schedule of the node fragment displacement without damage ($P_{ult} = 204.8$ kN, $\Delta = 2.17$ mm)

**Fig. 7.** Ultimate load-bearing capacity and load-displacement diagram of the node fragment in case of level 1 damage ($P_{ult} = 204.8$ kN, $\Delta = 2.52$ mm)
**Fig. 8.** Ultimate load-bearing capacity and load-displacement diagram of the node fragment in case of level 2 damage ($P_{ult} = 170.2 \text{kN}, \Delta = 2.66 \text{ mm}$)

**Fig. 9.** Ultimate load-bearing capacity and load-displacement diagram of the node fragment in case of level 3 damage ($P_{ult} = 154.5 \text{kN}, \Delta = 2.47 \text{ mm}$)

**Fig. 10.** Diagrams of pylon support displacement in B/2 axes for various levels of damage

**Fig. 11.** Time plot of the support reactions of the centre pylons for the system without damage and with damage under variant 1
Fig.12. Time plot of the support reactions of the centre pylons for the system without damage and with damage under variant 2

4 Discussion

Based on the results of calculations of the node fragments carried out in the FE formulation, as well as according to the analytical methodology, taking into account the proposed damage levels, a comparative table of results was drawn up. (see Table 1) The values of bearing capacity for the analytical methodology are taken at an angle of inclination of the punching pyramid of 24°, which is in general agreement with the results of similar tests.

Table 1. Comparative analysis of the load-bearing capacity for the junction fragment with regard to different damage levels

<table>
<thead>
<tr>
<th>Damage level</th>
<th>Regulation approach</th>
<th>Numerical approach [17]</th>
<th>This study</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N</td>
<td>N</td>
<td>N / N_{dam}</td>
</tr>
<tr>
<td>No damage</td>
<td>253,5</td>
<td>249,3</td>
<td>1</td>
</tr>
<tr>
<td>Damage level 1</td>
<td>-</td>
<td>231,6</td>
<td>0,93</td>
</tr>
<tr>
<td>Damage level 2</td>
<td>-</td>
<td>211,9</td>
<td>0,85</td>
</tr>
<tr>
<td>Damage level 3</td>
<td>-</td>
<td>182,2</td>
<td>0,74</td>
</tr>
</tbody>
</table>

The variation of the proposed methodology considering damage and FE model is observed at the 1st level of damage. Further both methods give similar results. It should be noted that the results for finite element modelling of local nodes should be verified with the test data. This will make it possible to correct the adopted approaches in FE modelling, as well as to evaluate the levels of damage of the node from punching proposed in this paper.
5 Conclusion

Damage in the slab-column junction zone affects the redistribution of force flows in monolithic frames in emergency situations. At the same time, at certain combinations of damage for systems designed without accounting for protection against progressive collapse, a complete loss of their overall stability in emergency situations is possible. Additional research is needed to analyze the effect of damage in other combinations, as well as when considering the removal of corner or end columns.

References


