Deformation and failure of monolithic reinforced concrete frames under special actions

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Abstract. The results of experimental and theoretical studies of crack formation, deformation and failure of monolithic reinforced concrete frames under accidental action caused by the sudden removal of one of columns of the first floor. Experimental studies were carried out on the model of structural fragment of a three-story two-span reinforced concrete frame, designed from fine-grained concrete class B40 with reinforcement A500. The test of experimental structures is executed by the gravitational load, with use of specially designed lever loading device. Special effect in the form of sudden removal of central columns of the frame was modelled using a specially designed mechanism in the form of sudden switching-off device. The obtained experimental values of widths of cracks, deflections, pictures of cracking and failure of experimental structures of the frames before and after beyond-design action. The experimental values of these parameters are compared with the results of calculation by the method taking into account the specifics of static-dynamic loading of physically and constructively nonlinear structures under the considered special actions.

1. Introduction
The solution of the tasks connected with the problem of protection of buildings and structures from progressive collapse in recent years has received increasing attention in both domestic and foreign studies. Nevertheless, the overwhelming number of publications devoted to theoretical studies describing the solutions of individual problems, considering the specifics of deformation and failure of structural systems in out-of-limit states. Among the studies of this direction in reinforced concrete structures, the works [1-13] can be noted. There are very few studies devoted to the experimental determination of deformation parameters and the criteria for failure of reinforced concrete structures under special actions. In Russia, such studies were carried out mainly on models of structural systems (studies by Geniev G.A., Klyueva N.V. [2], Kolchunov V.I., Bukhtiyarova A.S. [6], Fedorova N.V. Korenkov P.A. [7], Demyanov A.I. [4], etc.). In foreign publications, along with structural tests on models, individual studies of full-scale structures are also given (for example: the works of Yu J., Tan K.H. [17], Ahmadi R. [11], Pham A.T. [16]). The main disadvantage of these studies is that the loading of structures was carried out with the help of hydraulic test systems, which did not allow to study the redistribution of force flows after the removal of one of bearing elements from the structure. Experimental studies of structures in the out-of-limit states, with the evaluation of experimental
parameters and criteria of the out-of-limit states for normalization and experimental verification of theoretical solutions, have not been carried out to date.

2. Research methods
Experimental studies were performed by testing reinforced concrete structures of three-story two-span monolithic frames. Tested three series of frames experienced by two samples in each series. Experimental designs of frames of various series were made of concrete class B40, had the same formwork dimensions (figure 1a), the same reinforcement schemes and different longitudinal working reinforcement of the beams (table 1).

Prototypes of frames of the first series РЖ-2Φ8(b) are reinforced in upper and lower zones along cross-section of the beam with working reinforcement in the form of two bars with a diameter of 8 mm of class A500. Prototypes of the frames of the second series РЖ-1Φ8 (s) are reinforced in upper and lower zones along height of cross-section of the beam with one bar with diameter of 8 mm of class A500. Experimental samples of frames of the third series РЖ-2Φ8 (w) symmetrically in height of cross-section of the beam in the compressed and stretched zones by two bars with diameter of 8mm class A500. Transverse reinforcement of the beams of all series accepted wire diameter 2mm in increments of 50mm and 100mm.

![Figure 1. Scheme of formwork (a) and reinforcement (b) of structural frames.](image)

Such reinforcement was adopted based on results of calculation of experimental structures of frame for design test load in the form of concentrated forces $P$ applied in pairs to each beam symmetrically at distance of 350 mm from supports. Beyond-design action was carried out by sudden removal of middle or extreme column of the first floor of the frame. The method of application of such actions is given in [10]. According to the results of the calculation with the adopted reinforcement, specimens of frames of the first series provide implementation of the first criterion of special limit state [9] when the deformations of compressed concrete in the most stressed combinations reach the limit values ($\varepsilon_{c2} \leq 0.0035$) and the failure occurs on compressed concrete. Accordingly, the specimens of frames of the second series are reinforced on the basis of the second criterion of a special limit state, when the deformations of the tensile reinforcement reach the limit values ($\varepsilon_{s2} \geq 0.025$) and the failure of beam of
frame can occur due to the fluidity of the longitudinal reinforcement. The calculated reinforcement of specimens of frames of the third series is taken on the basis of the condition that under special action the failure of compressed and tensile concrete in several cross sections of the beam is allowed, but the third criterion is fulfilled - by the destructive force in the reinforcement when it works as a stretched hanging thread.

Table 1. Main characteristics of beams of experimental structural frames.

| № | Series | Cross-section scheme | Reinforcement, № positions in Fig. 1 | Concrete class |
|---|--------|----------------------|--------------------------------------|---------------|
| 1 | РЖ-2Ф8 (b) | 1-1 2-2 3-3 | 1 2 3 4 5 | A500 A500 Bp500 Bp500 |
| 2 | РЖ-1Ф8 (s) | 1-1 2-2 3-3 | 1 2 3 4 5 | A500 A500 Bp500 Bp500 |
| 3 | РЖ-2Ф8 (w) | 1-1 2-2 3-3 | 1 2 3 4 5 | A500 A500 A500 Bp500 |

Details about the calculation of experimental designs of frames, schemes and stages of loading by test load and the scheme of installation of measuring instruments for testing frame are given in works [10, 13].

3. The results of experimental studies and their analysis

Cracking, deformation and failure of experimental structures of frames under the considered special actions in the form of sudden removal of one of the supporting columns has a number of features.

At the first stage of testing of structures under their loading to the total design load \( \Sigma P_{\text{max}} \), the first cracks (Tp-1, figure 2) in the experimental specimens of frames were found in the support zones of the beams at the total load \( \Sigma P_{1} = 17,3 \text{ kN} \) - for the frames of the first series and under load \( \Sigma P_{2} = 16,3 \text{ kN} \) - for the frames of the second series. Accordingly, the moments of formation of these cracks were 0,39 and 0,34 kN. As the load increased, new cracks were formed in other areas of structures of the frame. So, for frame of the first series РЖ-2Ф8(b), with increase in the total load to the level \( \Sigma P_{1} = 20,0 \text{ kN} \), new cracks (Tp-2) appeared in support section of beams above the first floor at the extreme column, and with the total load \( \Sigma P_{2} = 27,7 \text{ kN} \) - cracks in the support zones of the beams above the second floor (Tp-5, Tp-6). At the same loading stage normal cracks formed in the span sections of the beams above the first and second floors (Tp-3, Tp-4, Tp-7).

In the experimental frame of the second series, the picture of formation and development of cracks was similar to the picture of cracks of the first series. The differences were that in this frame, the height of cracks and the width of their opening were significantly greater than in the construction of the frame of the first series at the same level of loading. This is due to the fact that the development of cracks in the height of cross section of the beam and the width of their opening is determined by the height of the compressed concrete, depending on the reinforcement ratio of the section.
Tests according to the indications of strain gauges on the reinforcement showed that in the structures of both series the reinforcement in the entire range of load changes until a special effect (sudden removal of column of the first floor) was worked elastically and therefore the crack opening width in the cross section of the beams as the loading increased in proportion to the deformation of the tensile reinforcement (figure 3). Here are the values of crack opening width calculated by the refined method [3].
The character of the growth of deflections of beam of frame during loading to the design level \( \Sigma P_{\text{max}} \) can be judged by the diagram “total load-deflection \( (\Sigma P - f) \)” built for the experimental structure of frame of the first series (figure 4). Prior to cracking, an almost linear “\( \Sigma P - f \)” relationship was observed. After the formation of cracks and a noticeable decrease in rigidity of cross-sections of beams of the frame, and an increasingly noticeable nonlinear increase in deflections of sections of the frame.

With the same level of loading, the deflection of beams of frames of the first series in the corresponding sections was significantly 17-28% less than the deflections of frames of the second series, the reinforcement intensity of which is much lower.

**Figure 4.** Dependence of the total load-deflection of the experimental construction of the frame of the first series РК-2Ф8 (b): 1- Experimental result; 2- Theoretical result by the method [3]

After beyond-design action in the form of a sudden removal of the middle column of the frame (second stage of testing), the character of stress-strain state of the experimental structures has changed significantly. Due to the structural alteration of the structural system (structural nonlinearity) and the change in force flows in the frame of the first series, the previously formed cracks 1 in the support zone of the beams near the middle column were closed (figure 5b). Cracks 2 received significant increments from dynamic loading of structural system elements, up to crushing of compressed concrete in the zone of these cracks. Cracks 3 developed along the cross-sectional height of the beam and their opening increased. At the same time, a significant number of cracks of type 2 and 3 were formed in all the beams of the studied structure. In the structure of the frame of the second series, the scheme of crack formation and opening after removal of the middle column was similar as in the frame of the first series. At the same time, comparing the overall picture of cracks, it can be seen that in the over-reinforced structure of the first series, a characteristic brittle fracture of the support zones of the beams on the compressed concrete was observed - the first criterion, as expected from the calculation results.

The failure of the frames of the second series was characterized by sharp increase in the number of cracks in the tensile zone along the entire lower surface of the beams and cracks in the support tensile
zones of the beams adjacent to the extreme columns of the frame. Due to this character, the failure of the frames of this series was “soft” character, accompanied by significant deflection of the beams.

Figure 5. Picture of the formation and opening of cracks of all types after beyond-design action of the frame of the first (a, b) and second (s, e) series: a, c - Theoretical result b, d - Experimental result
The analysis of the quantitative change of the width of crack opening in beams of frame at the design load and after beyond-design action (table 2) showed that cracks in the area where the beam adjoins the extreme column (Tp-2 and Tp-6) received maximum increments. Using these data, the coefficient of dynamic loading of cross-sections of beams (\( \theta_{ds} \)) from the considered beyond-design action was calculated as the ratio of the maximum crack opening width before and after beyond-design action.

**Table 2.** Crack widths in the experimental structures of the frames before and after beyond-design action

| № series of frame | Crack number according to the scheme of figure 2 | Crack opening width before shutdown of connection \( a_{cr,c} \) | Crack opening width after shutdown of connection \( a_{cr,d} \) | Coefficient of dynamic loading \( \theta_{ds} \) |
|-------------------|-----------------------------------------------|-----------------------------------------------|-----------------------------------------------|-----------------------------------------------|
| РЖ-2Ф8(b)         | 1                                             | 0,13                                          | Closing                                       | -                                             |
|                   | 2                                             | 0,10                                          | Failure                                       | -                                             |
|                   | 3                                             | 0,06                                          | 0,25                                          | 4,2                                           |
|                   | 4                                             | 0,07                                          | 0,20                                          | 2,9                                           |
|                   | 5                                             | 0,08                                          | Closing                                       | -                                             |
|                   | 6                                             | 0,08                                          | Failure                                       | -                                             |
| РЖ-1Ф8(s)         | 1                                             | 0,21                                          | Failure                                       | -                                             |
|                   | 2                                             | 0,16                                          | 1,06                                          | 6,6                                           |
|                   | 3                                             | 0,09                                          | 0,30                                          | 3,3                                           |
|                   | 4                                             | 0,11                                          | 0,11                                          | 6,1                                           |
|                   | 5                                             | 0,12                                          | Closing                                       | -                                             |
|                   | 6                                             | 0,12                                          | 1,00                                          | 8,3                                           |

In order to assess the deformation criterion of special limiting state that limits the maximum permissible relative deflection of beams of the structural system [9], experimental values of relative deflections of elements of experimental structures of the frame before and after beyond-design action were calculated (table 3).

Analyzing these data, it can be seen that, during beyond-design action in the structures of the frames of the first and second series, it leads to the achievement in them of the deformation criterion of a special limiting state.

**Table 3.** Relative deflections of beams of the experimental structures before and after beyond-design action

| № series of frame | № of deflection meters according to the scheme of Fig. 3 | Relative deflection before shutdown of connection \( (f/l)_a \) | Relative deflection after shutdown of connection \( (f/l)_d \) |
|-------------------|--------------------------------------------------------|--------------------------------------------------------|--------------------------------------------------------|
| РЖ-2Ф8(b)         | П1                                                      | 1/2611                                                | 1/18,8                                                |
|                   | П2                                                      | 1/1736                                                | 1/17,1                                                |
|                   | П3                                                      | 1/910                                                 | 1/32,4                                                |
|                   | П4                                                      | 1/1002                                                | 1/16,4                                                |
|                   | П1-Са                                                   | 0                                                      | 1/13,0                                                |
| РЖ-1Ф8(s)         | П1                                                      | 1/2914                                                | 1/56,2                                                |
|                   | П2                                                      | 1/1950                                                | 1/56,3                                                |
|                   | П3                                                      | 1/940                                                 | 1/101,4                                               |
|                   | П4                                                      | 1/1050                                                | 1/54,7                                                |
|                   | П1-Са                                                   | 0                                                      | 1/47,1                                                |

\(^a\)Deflection meter at the point of support of the beam on the removed column

For the frames of the first series, the exhaustion of bearing capacity in the out-of-limit state is caused by the failure of compressed concrete (Figure 5a). For the frames of the second series, the
exhaustion of bearing capacity was characterized by a significant deflection exceeding the criterion [9] 1/50 span (Figure 5b).

Figure 6. General view of failure of experimental structures of the first (a) and second (b) series.

4. Conclusions
The application of beyond-design action to the loaded statically indefinite frame structure in the form of the sudden removal of one of the supporting column causes a dynamic loading of all elements of such structural system.

The quantitative values of dynamic loads of forces arising in the frame elements and the character of failure of elements of experimental structures of the frames depend on the level of the applied design load, the location of the element in the structural system in relation to the removed structure, the intensity of reinforcement of elements of the frame.

The obtained experimental data can be used for verification of analytical dependences on determination of criteria of the special limit state of reinforce-concrete elements of the structural systems in their protecting from progressive collapse.

References
[1] Androsova N B and Vetrova O A 2019 The analysis of studies and requirements for the protection of buildings and structures against progressive collapse in regulatory documents of Russia and the European Union Building and reconstruction 1(81) 85
[2] Geniev G A and Klyueva N V 2000 Experimental-theoretical studies of continuous beams in case of accidental shutdown of individual elements News of higher educational institutions. Construction 10(502) 21
[3] Golyshev A B and Kolchunov V I 2009 Resistance of reinforced concrete (Kiev: Osnova) p 432
[4] Demyanov A I and Alkadi S A 2018 Experimental-theoretical studies of static-dynamic deformation of a spatial reinforced concrete frame with complex-stressed beams of solid and composite cross-Sections Industrial and Civil Engineering 6 68
[5] Zenin S A, Sharipov R Sh, Kudinov O V, Shapiro G I and Gasanov A A 2016 Methods of Calculating of Large-Panel Buildings: How to Prevent Progressing Collapse Academia. Architecture and Construction 4 109

[6] Klyueva N V, Kolchunov V I, Rypakov D A and Bukhtiyarova A S 2015 Durability and deformability of precast-cast-in-place frameworks for residential buildings with low material consumption at beyond-design-basis impacts Industrial and Civil Engineering 1 5

[7] Klyueva N V and Korenkov P A 2016 Method of experimental determination of parameters of survivability of reinforced concrete frame-rod structural systems Industrial and Civil Engineering 2 44

[8] Kodysh E N, Trekin N N and Chesnokov D A 2016 Protection of multiistory buildings from progressing collapse Industrial and Civil Engineering 6 8

[9] SP 385.1325800.2018 Protection of buildings and structures against progressive collapse. Design code. Basic statements (Moscow) p 20

[10] Fedorova N V, Korenkov P A and Vu N T 2018 Experimental method of research of deformation of monolithic reinforced concrete building under accidental actions Building and reconstruction 4(78) 42

[11] Ahmadi R, Rashidian O, Abbasnia R, Nav F M and Usefi N 2016 Experimental and Numerical Evaluation of Progressive Collapse Behavior in Scaled RC Beam-Column Subassemblage Shock and Vibration

[12] Alogla K D, Weekes L and Augusthus Nelson L 2017 Theoretical assessment of progressive collapse capacity of reinforced concrete structures Mag. Concr. Res. 69(3) 145

[13] Fedorova N V, Vu N T and Korenkov P A 2019 Deformation and failure of a monolithic reinforced concrete frame under accidental actions Journal of Physics: Conference Series

[14] Jian H and Zheng Y 2014 Simplified models of progressive collapse response and progressive collapse-resisting capacity curve of RC beam-column substructures J. Perform. Constr. Facil. 28(4) 04014008

[15] Mohajeri Nav F, Nima U and Abbasnia R 2018 Analytical investigation of reinforced concrete frames under middle column removal scenario Adv. Struct. Eng. 21(9) 1388

[16] Pham A T and Tan K H 2017 Experimental study on dynamic responses of reinforced concrete frames under sudden column removal applying concentrated loading Eng. Struct. 139 31

[17] Yu J and Tan K H 2013 Experimental and numerical investigation on progressive collapse resistance of reinforced concrete beam column sub-assemblages Eng. Struct. 55 90