

CONFERENCE PAPER

NZAAR-CCS-18-49 CCS 2018

Strength and Stability of Reinforced Beam-Column Concrete Elements under Dynamic Loading with Fire Exposure Conditions

Levon Avetisyan *

Department of Reinforced Concrete and Masonry Structures, Moscow State University of Civil Engineering, 26 Yaroslavskoe Shosse, Moscow 129337, Russia

* Correspondence: avetisyanlevon@inbox.ru

PUBLISHED IN

The Fourth NZAAR International Event Series on Natural and Built Environment, Cities, Sustainability and Advanced Engineering
International Conference on Civil Engineering, Construction Management and Structural Design (CCS 2018)

Pages	60-69 of the volume
Held	27 January 2018, GTower Hotel, Kuala Lumpur, Malaysia
ISSN	2463-5979 (online) 2463-5960 (print)
Publisher	New Zealand Academy of Applied Research Ltd, Auckland

ABSTRACT

This article gives the method of dynamic calculation of reinforced beam-column concrete elements for dynamic loadings varying with time. The results of experimental researches of reinforced concrete elements under dynamic loading in fire exposure conditions are presented, and dependence of the dynamic strengthening coefficient of the elements at a temperature up to $T = 9000\text{ C}$ is obtained. The main scenarios that can arise in high-rise buildings with progressive fracturing are developed. The analytical calculation of the change in the load-carrying capacity of reinforced beam concrete column is carried out for different thermal-force actions. Numerical analysis of a multi-storey reinforced concrete frame in the Sap 2000 software complex is carried out, taking into account the change in the strength and deformation properties of load-carrying elements for different temperature actions by means of the plastic hinge function.

Keywords: reinforced concrete columns; beam-column elements; dynamic strength; fire exposure conditions

HOW TO CITE

Levon Avetisyan (2018). Strength and Stability of Reinforced Beam-Column Concrete Elements under Dynamic Loading with Fire Exposure Conditions. In *The Fourth NZAAR International Event Series on Natural and Built Environment, Cities, Sustainability and Advanced Engineering*, Kuala Lumpur, Malaysia, 27 January 2018 (pp. 60-69). Auckland: New Zealand Academy of Applied Research. ISSN 2463-5979 (online), 2463-5960 (print).

STRENGTH AND STABILITY OF REINFORCED BEAM-COLUMN CONCRETE ELEMENTS UNDER DYNAMIC LOADING WITH FIRE EXPOSURE CONDITIONS

Levon Avetisyan*

Moscow State University of Civil Engineering, Department of Reinforced Concrete and Masonry Structures, 26
Yaroslavskoe Shosse, Moscow, 129337, Russia

* avetisyanlevon@inbox.ru

www.nzaar.com

Abstract

This article gives the method of dynamic calculation of reinforced beam-column concrete elements for dynamic loadings varying with time. The results of experimental researches of reinforced concrete elements under dynamic loading in fire exposure conditions are presented, and dependence of the dynamic strengthening coefficient of the elements at a temperature up to $T = 900^{\circ}\text{C}$ is obtained. The main scenarios that can arise in high-rise buildings with progressive fracturing are developed. The analytical calculation of the change in the load-carrying capacity of reinforced beam concrete column is carried out for different thermal-force actions. Numerical analysis of a multi-storey reinforced concrete frame in the Sap 2000 software complex is carried out, taking into account the change in the strength and deformation properties of load-carrying elements for different temperature actions by means of the plastic hinge function.

Keywords: reinforced concrete columns, beam-column elements, dynamic strength, fire exposure conditions

1. Introduction

One of the most important tasks of buildings' stability is the development of methods for calculating load-carrying structures, which make it possible to most accurately assess their ability to resist various influences. One of such scenarios can be dynamic loading of the reinforced beam-column concrete elements (after removal of one of the columns) in fire exposure conditions, which is the closest to reality scheme for calculating progressive fracturing.

Analysis of possible consequences provides information for designing more stable structures, finding cost-effective solutions, increasing their safety, and strengthening already fractured buildings and structures [1].

Methods of dynamical calculation of reinforced concrete structures beyond the natural elastic limits are mainly based on idealized schemes of rigid-plastic or elastoplastic material, and in the case of high-strength reinforcement - on the models of non-linearly strained systems.

The normalizing values under dynamic loading are selected in such a way that they can be found by existing calculation methods and that they are convenient for experimental determination.

Since the currently used calculation methods allow one to determine moments of flexion, deflections, expansion angles in the plasticity hinges, the ultimate values of these quantities are taken as the normalizing ones.

For reinforced beam-column concrete elements with large eccentricity, the state 1b is usually used to characterize as the extension angle in the plasticity hinge, in which the strain of the concrete of the compressed zone reach the ultimate values. The researches show that it is more convenient to normalize the ultimate state 1b of beam-column elements by the plastic angles of expansion in plasticity hinges that arise during the construction work only in the plastic stage.

The structural strength condition (with the formation of n-plasticity hinges in it) takes the form:

$$\psi_i \leq \psi_{i,u} \quad (i=1,2,3,\dots,n). \quad (1)$$

Here ψ_i and $\psi_{i,u}$ are the plastic extension angles in the i -hinge of plasticity, obtained from the dynamic calculation in the plastic stage.

The procedure for determining the $\psi_{i,u}$ value for different structures varies, depending on the ultimate strains of the concrete and the length of the plastic zone. Achieving by stresses of the ultimate state does not lead to complete fracture of the material, since the behavior of the concrete and reinforcement under loading is characterized by plastic strains. After reaching the ultimate state, the material of the element is capable of absorbing a part of the loading and fracturing until the fracturing reach an ultimate value, after which a complete shutdown of the structure takes place. Therefore, as a criterion for the fracturing of a reinforced concrete element, it is more expedient to take the ultimate fractures of concrete with compression and tension ε_{cu} , $\varepsilon_{cu,d}$, and reinforcement $\varepsilon_{su,d}$.

The ultimate value of the plastic extension angle in the plasticity hinge for beam-column elements is determined by the formula:

$$\psi_u = \left[\varepsilon_{cu} / \xi_d - k_{sd} R_s / E_s (1 - \xi_s) \right] C_M C_B \left(1 + \frac{N}{2N_u} \right) \left(\frac{l_0}{h_0} \right)^{1/4}, \quad (2)$$

where ε_{cu} - ultimate strain of the edge fiber of the concrete of the compressed zone, depending on a number of factors, including the e_0 value; for practical calculations, it is recommended to take $\varepsilon_{cu} = 320 \cdot 10^{-5}$.

ξ_d - relative height of the compressed zone.

C_M, C_B - coefficients depending, respectively, on the shape of the moment curve and class of the concrete.

l_0 - length of the section of the structure adjacent to the hinge, on which the bending moment keeps the sign.

N_u - ultimate load-carrying capacity of the element under central compression:

$$N_u = k_{d,b} R_b b h + k_{d,s} R_{sc} (A_s + A'_s). \quad (3)$$

2. Literature Review

The stiffness of beam-column elements after the formation of cracks is a variable value that depends on the change in the value of the longitudinal force $N(t)$ in time. Since the analytically constructed strain diagrams (Fig. 1), which have a curvilinear outline, are difficult to express by one common function, a simplified "M-1/r" diagram is used in the form of a broken line consisting of three straight lines (diagram "a", Fig. 1).

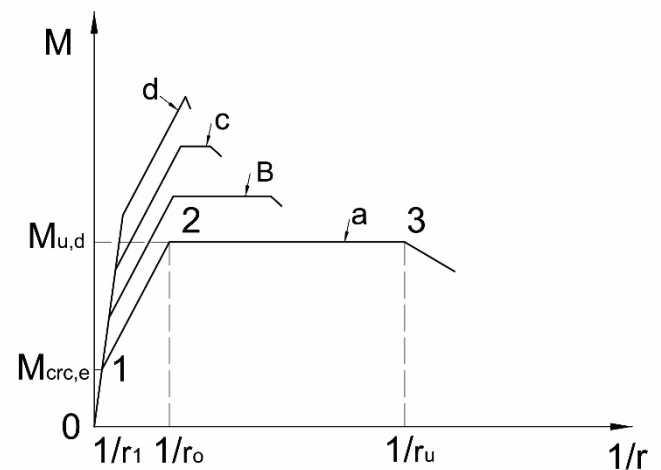


Fig.1. Diagrams "moment-curvature" of beam-column elements reinforced with A500 steel for various eccentricities: "a" – e_{01} ; "b" – e_{02} ; "c" – e_{03} ; "d" – e_{04} ; $e_{04} < e_{03} < e_{02} < e_{01}$

To construct simplified strain diagrams, let us consider beam-column element loaded with a longitudinal force $N(t)$ with eccentricity e_0 and reinforced with steel with a physical yield point (A500 class) for which the following condition is met:

$$\xi \leq \xi_d \quad (4)$$

If the condition $\xi > \xi_d$ is met, then a brittle fracturing of the structure occurs at the ultimate strain of the concrete of the compressed zone and at the absence of plastic strain in the stretched reinforcement is taking place. The strain diagram of such elements does not have a plastic section and corresponds to the case of brittle fracture (Fig. 1, diagram "d").

To calculate it, two types of dynamic loadings, varying in time are accepted.

1) $N(t) = \frac{N}{(1-t/t_{ef})}$ - the form of a linearly increasing function;

2) $N(t) = N_m e^{-\frac{1}{E} \left(\frac{t-t_{ef1}}{B} \right)^2}$ - the form of exponential function.

3. Research Methods

Let us consider the case of an eccentrically applied longitudinal dynamic loading with eccentricity e_0 .

The equation of motion of the column in the initial-elastic stage, looks like:

$$B_1 \frac{\partial^4 y_1}{\partial z^4} + N(t) \frac{\partial^2 y_1}{\partial z^2} + m \frac{\partial^2 y_1}{\partial t^2} = 0 \quad (5)$$

where:

B_1 - stiffness of the element in the stage without cracks (elastic stage) $B_1 = E_b I_{red}$ (work on the section 0 - 1, Fig. 1);

$N(t)$ - short-term dynamic force applied with eccentricity e_0 ;

m - mass per column;

y_1 - deflection of the column in the stage without cracks;

z - coordinate of the cross section along the length of the column.

The solution of equation (5) can be represented as the sum of the functions of deflections from static action of the longitudinal load y_{st} and the dynamic deflection y_d ;

$$y_1(z; t) = y_{st} + y_d \quad (6)$$

The function y_{st} is found by means of integrating equation

$$B_1 y_{st}^{IV} = 0 \quad (7)$$

Substituting the boundary data according to the conditions of the support fastenings, we find the value of the static deflection.

Dynamic deflection is found by solving the equation:

$$B_1 y_{st}^{IV} + m y_d = 0 \quad (8)$$

where y_d can be represented as the form of a trigonometric series:

$$y_d(z, t) = \sum_{n=1}^{\infty} T_{n1}(t) Z_n(z) \quad (9)$$

where $Z_n(z)$ - deflection function, which is represented in the form:

$$Z_n(z) = \sin \frac{n\pi}{l} z \quad (10)$$

$T_{n1}(t)$ - dynamical function that can be obtained using the Bubnov-Galerkin variational method, depending on the type of loading and the initial conditions.

The deflection and bending moment in the sections of the column from the action of static and dynamic loading is determined [2,3]:

$$y_1(z, t) = \frac{N e_0}{2 B_1} (l z - z^2) \left(1 - \frac{t}{t_{ef}} \right) + \sum_{n=1,3,5,\dots} \sin \frac{n\pi}{l} z \cdot T_{n1}(t) \quad (11)$$

$$M_1 = N e_0 \left(1 - \frac{t}{t_{ef}} \right) + B_1 \cdot \frac{\pi^2}{l^2} \sum_{n=1,3,5,\dots} \sin \frac{n\pi}{l} z \cdot T_{n1}(t) \quad (12)$$

We determine the time of formation of normal cracks from the condition:

$M_1(l/2, t_1) = M_{crc,e}$, here $n=1$ leads to the solution:

$$N e_0 \left[\left(1 - \frac{t_1}{t_{ef}} \right) + \frac{4}{\pi} \left(\cos \omega_1 t_1 - \frac{\sin \omega_1 t_1}{\omega_1} \right) \right] = M_{crc,e} \quad (13)$$

The equation of the column motion in the elastoplastic stage (section 1-2) looks like:

$$B_2 \frac{\partial^4 y_2}{\partial z^4} + m \frac{\partial^2 y_2}{\partial t^2} \left(-\frac{\partial^2 M_1}{\partial z^2} + N(t_1) \frac{\partial^2 y_1}{\partial z^2} \right)_{t=t_1} = 0 \quad (14)$$

B_2 - the dynamic stiffness of the column in the stage with cracks in the stretched zone,

for simplicity assumed as constant, is calculated by the formula:

$$B_2 = \frac{M_u - M_{crce}}{1/r_0 - 1/r_1} \quad (15)$$

M_u - moment of internal forces in the element when the yield point in the stretched reinforcement is reached (point 2):

$$M_u = k_{bd} R_b b x_d (h_0 - x_d / 2) + k'_{sd} R_{sc} A'_s (h_0 - a') - N(h/2 - a) \quad (16)$$

$1/r_1$ - curvature in the middle axis of the element (in stage I) is determined as:

$$\frac{1}{r_1} = \frac{M_1}{B_1} \quad (17)$$

$1/r_0$ - curvature of the curved axis of the element when the yield point in the stretched reinforcement is reached:

$$\frac{1}{r_0} = \frac{M_s}{B_{0,lon}} - \frac{N}{h_0 E_s A_s} \quad (18)$$

$B_{0,lon}$ - stiffness of the element at the time of yield occurrence in the reinforcement.

The deflection and bending moment in the sections of the column in the elasto-plastic stage (section 1-2) are determined:

$$y(z, t) = y_1(z, t_1) + y_2(z, t), \quad (19)$$

$$M(z, t) = M_1(z, t_1) + M_2(z, t), \quad (20)$$

The solutions of equation (19) are obtained as:

$$y_2(z, t) = \sum_{n=1}^{\infty} T_{n2}(t) \sin \frac{n\pi}{l} z, \quad (21)$$

$T_{n2}(t)$ - dynamic function, determined depending on the type of loading in stage 2 (with cracks), when

$$\omega_{n2} = \frac{n^2 \pi^2}{l^2} \sqrt{\frac{B_2}{m}}, \quad (22)$$

$$M_2(z, t) = B_2 \frac{\pi^2}{l^2} \sum_{n=1,3,5,\dots} n^2 \sin \frac{n\pi}{l} z T_{n2}(t). \quad (23)$$

The bending moment at the end of the elastic stage in the span section $z_0 = l/2$ is equal to:

$$M(l/2, t) = M_1(l/2, t_1) + M_2(l/2, t) \quad (24)$$

The maximum value $T_{n2}(t)$ is defined for $t = t_{m1}$

The plastic stage is formed if $z = z_0$

$$|M(z_0, t_{m1})| > M_u \quad (25)$$

When $z_0 = 0,5l$

$$M(z_0, t_{m1}) = M_{crce} + B_2 \frac{\pi^2}{l^2} \sum_{n=1,3,5,\dots} n^2 \sin \frac{n\pi}{l} z T_{n2}(t) \quad (26)$$

At calculation it is necessary to consider not less than 3 n values.

With the hinged support of the ends of the column, the plastic hinge arises only in the span, therefore, the plastic stage occurs immediately after the elastic one. With rigidly clamped ends, it is necessary to find the transition time from the elastoplastic to the plastic stage.

The time of the elastoplastic stage is determined under the condition:

$$|M(z_0, t_2)| = M_u, \quad (27)$$

The strength of the column is verified from the condition

$$\psi_{ma} \leq \psi_u \quad (28)$$

Dynamic calculation of load-carrying reinforced concrete elements in fire exposure conditions is a difficult task and differs from calculations under static loading.

To determine the influence of fire exposure on the physical and mechanical properties of the concrete and reinforcement under dynamic loading, and to obtain the coefficient of dynamic hardening, concrete cubes and prisms were heated up to 300, 500, 700 and 900°C, and reinforced concrete beams and columns – up to 500 and 900°C in the standard fire mode and they were tested under static and single dynamic loading [4].

The received results have shown that:

- the strain (the value of the relative elongation δ) of reinforcing steel in normal conditions at a pressure up to 700 MPa increased to 9%; after heating up to 500 °C the pressure decreased up to 550 MPa and the stress by 6,8%. After heating up to 900 °C, the pressure decreased up to 400 MPa, and the strain decreased by 5,5%;

- the modulus of elasticity of concrete after heating up to 500 °C decreased by a factor of 3, and after heating up to 900 °C by a factor of 5 compared with its parameters without fire exposures.

For the calculation of the reinforced concrete structures for dynamic effects, it is necessary to know the determining values of the coefficients of dynamism of materials K_d for compression and tension, determined only experimentally [5,6].

4. Analysis & Results

The values of the dynamic hardening coefficient of concrete and reinforced concrete samples as a function depending on the loading time in normal conditions and at different temperatures are presented in Table. 1.

Table 1

The dynamic hardening coefficient of dynamic hardening (K_d) of concrete cubes in normal conditions and under temperature influences.

Sample series №	Temperature, °C	Loading, kN			Dynamic hardening coefficient, K_d	
		static	dynamic		0,07sec	0,1 sec
			0,07sec	0,1 sec		
1	20	351,72	424,46	423,67	1,206	1,204
2	300	335,02	323,97	322,37	0,967	0,962
3	500	282,84	238,06	236,93	0,841	0,837
4	700	79,63	46,83	45,90	0,588	0,576
5	900	39,72	19,42	18,78	0,489	0,473

For the dynamic hardening coefficient of concrete prisms at the temperature of $300^{\circ}\text{C} \leq t \leq 900^{\circ}\text{C}$, the following dependencies are obtained.

– at a loading time of 0,07 sec.:

$$K_d = 1,303 + 1,9375 \cdot 10^{-7} t^2 - 0,001117 t \quad (29)$$

– at a loading time of 0,1 sec.:

$$K_d = 1,2413 + 1 \cdot 10^{-7} t^2 - 0,000938 t \quad (30)$$

Based on the analysis of the experimental researches' results, it is established:

1. In normal conditions, with a decrease in the loading time, the dynamic hardening coefficient (K_d) reaches up to 1.2-1.3.

2. When heated up to 500 °C, the concrete strength decreases sharply, especially the dynamic strength, which was equal to the static, and with increasing temperature up to 900 °C the dynamic strength decreased by a factor of 2 compared with the static strength.

3. The modulus of elasticity was 35 000 MPa and decreased up to 15 000 MPa and at 500 °C - up to 9 000 MPa.

4. The stress and modulus of elasticity of the reinforced bars after heating up to 900 °C turned out to be 1.4 times less than in the reinforcement not subjected to fire exposure.

5. The dynamic hardening coefficient of concrete cubes and prisms when heated up to 500 °C reached a value of 0,84, after heating up to 900 °C - up to 0,48.

Figure 2 shows schematically the change in the carrying capacity of a structure under dynamic loading.

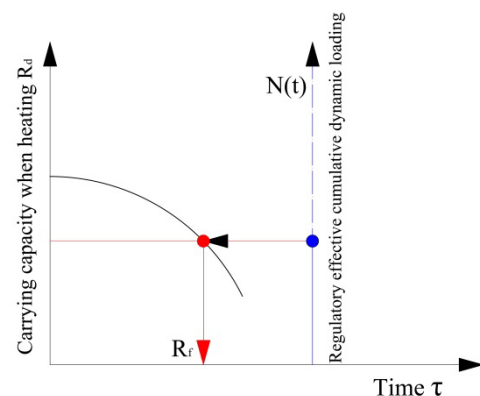


Fig. 2. Fire resistance of the structure according to the carrying capacity R under dynamic loading

Figure 2 shows that the dynamic effect on the construction reduces the fire resistance of the structure. The reduction depends on the type of dynamic loading.

Calculation of the reinforced concrete column is made as for a dynamic effect of the type

$$N(t) = N_m e^{\frac{1}{E} \left(\frac{t - t_{ef1}}{B} \right)^2}, \text{ when } t \geq t_1 \quad (31)$$

where: $E(t_1)$ - coefficient showing the intensity of the load increase, determined during the calculation

$$E(t_1) = 0,489 \left(\frac{t_1 - t_{ef}}{B} \right) \quad (32)$$

$$B = \frac{1}{\sqrt{2\pi N_m}}, \quad (33)$$

t_1 - the time of formation of normal cracks.

5. Discussion & Conclusion

Let us consider the main scenarios that can arise in high-rise buildings with progressive fracturing.

1. The explosive effect, which leads to the instant removal of one carrying element (the removal of the column of the 1st floor is considered as the worst case), as a result of which the remaining columns operate in normal temperature conditions with dynamically increasing loading. The dynamic factor of the loading varies $K_0, -1,2, \dots, 2$ depending on the number of stories of the building. This case is accepted as the main one in all calculations for progressive fracturing.

2. The explosive impact, leading to the removal of one vertical carrying element in time. The remaining columns work in normal temperature conditions. The recording of the removal time of the column in comparison with the "instant" removal has a number of specific features that directly affect the value of the natural frequencies, periods, and forms of vibrations of the remaining columns.

3. The explosive impact, leading to the instant removal of one vertical carrying element. The remaining columns work

under fire action in conditions of dynamically increasing loading.

4. The explosive post-inflammatory impact, leading to the instant removal of one vertical carrying element, which leads to a change in the strength and strain properties of the constituent materials and to a decrease in the dynamic response of the constituent materials.

The remaining scenarios can be a different combination of these options, taking into account the type of dynamic loading, duration and type of fire (standard, real).

For comparison, let us determine the load-carrying capacity of the reinforced concrete samples under static and dynamic loading in the conditions of a standard fire at a time of 1 000 seconds. Initial data: section $100 \times 100 \text{ mm}$; estimated length $L_0 = 600 \text{ mm}$; class of concrete B30, medium density in dry condition (granite crushed stone) $\rho_0 = 2485 \text{ kg/m}^3$, wet weight $w = 4,6\%$, reinforcement $4\phi 8 \text{ A500}$, thickness of the protective layer $\alpha_0 = 12 \text{ mm}$

Eccentricity $e_0/h = 0,35$, the dynamic load is determined by (31) at $N_m = 61 \text{ kN}$.

The results of the calculation show that the time for the formation of the first cracks in the fire exposure conditions is 3,4 times faster than in normal conditions. In normal conditions, the stiffness of the element in the stage without cracks is 1,45 times greater than in the fire exposure conditions. The stiffness of the element in the second stage when calculated in normal conditions is 1,2 times greater than in the case of fire exposures [2].

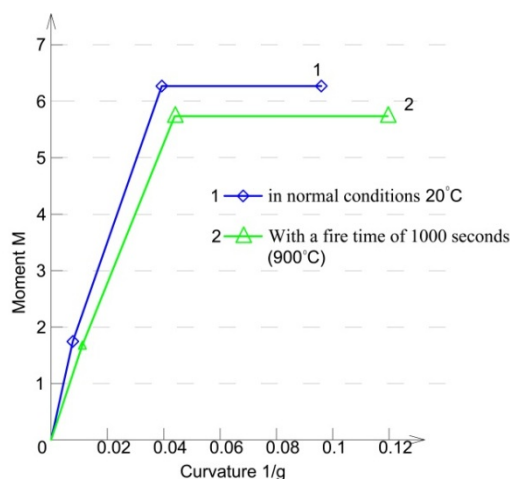


Fig. 3. The dependence "moment-curvature" in different work conditions of the element

For the column in question, the ultimate value of the extension angle is $\psi_u = 0,0172539$. Dynamic calculation of the column shows that in normal conditions the strength of the column is ensured, since the condition (28) is kept. In fire exposure conditions, the extension angle of the plastic hinge $\psi_{\max T} = 0,0454083$ is much larger than the maximum extension angle ψ_u , which indicates the fracturing of the element.

The calculation makes it possible to construct a "moment-curvature" diagram of an element operating in normal conditions and under fire exposure (see Fig. 3).

Further, rupture loads were calculated at eccentricities $l_0/h = 0,3; 0,35; 0,4$. Calculated and experimental results obtained for reinforced concrete columns under dynamic loading are shown in Table. 2.

Table 2

The results of the dynamic calculation of the reinforced concrete column in a standard fire, $\tau = 1000\text{sec}$.

Parameter	According to the experiment				By calculation			
	Central loading	Eccentricity			Central compression	Eccentricity		
		$l_0/h = 0,3$	$l_0/h = 0,35$	$l_0/h = 0,4$		$l_0/h = 0,3$	$l_0/h = 0,35$	$l_0/h = 0,4$
Rupture load (kN)	367,94	237,90	178,00	161,20	352,1	234,6	176,9	158,49
Deflection (mm)		3,54	3,71	4,02		3,31	3,39	3,9

The discrepancy between the experimental and calculated results does not exceed 5%, which indicates a sufficient accuracy of the calculation technique.

According to the calculation, the fire resistance of the reinforced concrete column at a standard load of 161,2 kN in a standard fire is 50 minutes. Taking into account, for example, that the dynamic effect is applied to the 30th minute from the start of the standard fire, and knowing the value of the dynamic factor for the material, we can construct Fig.4

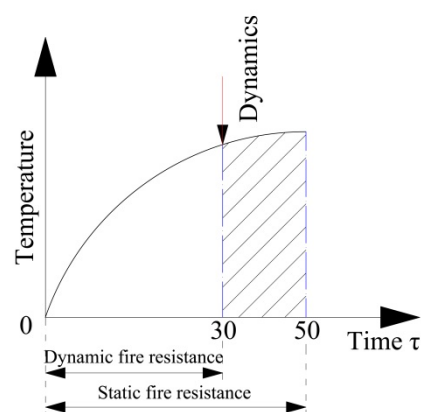


Fig.4. Change in calculated fire resistance under dynamic loading

Hitching marks dangerous zone, it shows that, after a dynamic effect, the reinforced beam-column concrete element ceases to meet the required fire resistance limit.

This stage presents the results of the calculation of reinforced concrete columns in the software complex Ansys Civil FEM, which accurately allows to determine the temperature in the section of the reinforced concrete column at any time during a standard fire.

During the calculation, changes in such properties of reinforced concrete as heat conductivity and heat capacity from temperature as well as coefficients $K_{s,T}$, $K_{d,T}$, were taken into account.

Calculation of the reinforced concrete column was carried out with quadrilateral heating of the column under static and dynamic loadings. Table 3 shows the results of the load-carrying capacity of the reinforced concrete column, calculated in the software complex Ansys 14.0.

Table 3

The results of the dynamic calculation of the strength of reinforced concrete columns in fire exposure conditions at an eccentricity of 3,5 cm

Temperature (°C)	With static loading and fire (kN)		With dynamic loading and fire (kN)	
	In the program Ansys 14.0	By means of the allowed methodology	In the program Ansys 14.0	By means of the allowed methodology
0°C	185,55	190,60	218,95	219,45
500 °C	145,40	146,80	135,70	135,40
900 °C	92,50	93,40	64,32	63,15

The calculation shows that under static loading, the reduction in the load-carrying capacity of the reinforced concrete column, with four-sided heating of the column, is 21,6% at 500 °C, and 50,1% at 900 °C. Under dynamic loading, it decreases: at 500°C - by 38,02%, at 900°C - by 70,6%. Let's research the behavior of the 25-storey monolithic frame building before and during fire exposures at different fire temperatures under dynamic loading by means of the Sap2000 SC version 12, which allows to solve a non-linear dynamic problem using the Newton-Raphson technique [7].

The purpose of the function Hinge - hinge for reinforced concrete elements has a very effective value, it allows to identify both static nonlinear reserves, fracturing of elements, and dynamic. As a result it reveals the reserve of carrying capacity of

reinforced concrete elements before the stage of fracturing.

Figure 5 shows the behavior of the plastic hinge from the load value according to FEMA 356 [8], which can be monitored by using the Sap2000 software complex.

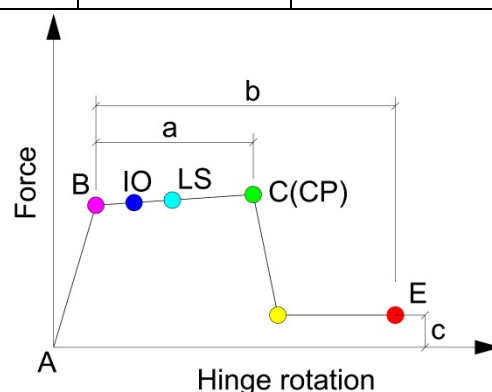


Fig. 5. Model of the behavior (work stages) of the plastic hinge

The stages of development of the plastic hinge are divided into:

- 1) $A > B$ - elastic stage;
- 2) $B > C$ - plastic stage; the loading resulting in the formation of a plastic hinge can not lead to fracturing depending on:
 - 2a) IO – constant loading;
 - 2b) LS – safe stage;
 - 2c) CP — prevention of fracturing;

3) $C \triangleright D$ – residual reserve;

4) $D \triangleright E$ – complete fracturing.

The software complex Sap2000 makes it possible to limit or expand the maximum extension angle of the plasticity of the elements, respectively, depending on the geometric dimensions, strength and fracture characteristics and work conditions of the elements.

When analyzing the frame of a building up to 500 °C with the static bending moments and transverse forces, the formation of unacceptable extension angles of the plastic hinge is absent - the safe stage "LS" in Fig. 5 (see Fig. 6a)).

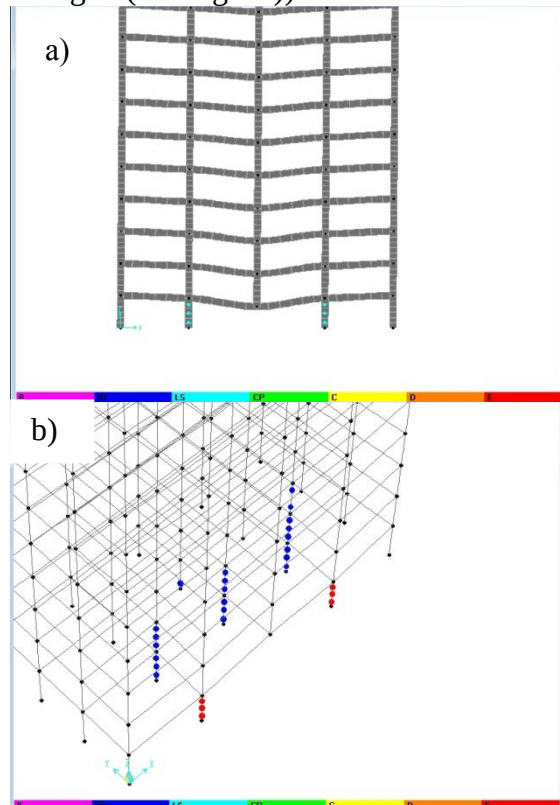


Fig.6. The state of the plastic hinges a) at a temperature 500°C, b) at a temperature 900°C

The performed nonlinear dynamic analysis at $T = 900^\circ\text{C}$ shows that the resistance to the progressive fracturing of this reinforced concrete frame is not ensured that is evidenced by the presence of unacceptable strains in the plastic hinges of the carrying elements if the frame - stage "E" in Fig. 5 (see Fig. 6b).

Thus, a detailed analysis of the development scenarios possible with the

progressive fracturing of the building, together with the change in the physicommechanical characteristics of the materials of the carrying vertical structures, both in dynamic power and in different temperature modes, allows more accurately assess the resistance of buildings to progressive fracturing, which increases safety and reliability of the designed buildings.

6. References

- [1] Quiel S., Marjanishvili S. (2011), Progressive Collapse Mitigation in Multistory Tilt-up Structural Systems, Applied Mechanics and Materials, Vol. 82 : pp. 698-703.
- [2] Tamrazyan A.G. (2013), Reduce the impact of dynamic strength of concrete under fire conditions on bearing capacity of reinforced concrete columns, ICSMIM 2013 2nd International. Conference on Sensors, Measurement and Intelligent Materials.
- [3] Wei-Ming Lin, Lin T. D., Powers-Couche L. J. (1996), Microstructures of fire-damaged concrete ACI Materials Journal, Vol. 93.
- [4] Tamrazyan A.G., Avetisyan L.A. (2016), Experimental and theoretical study of reinforced concrete elements with different characteristics of loading at high temperatures, Procedia Engineering, 153, pp: 721 – 725.
- [5] Tamrazyan A.G., Avetisyan. L.A. (2016), Comparative analysis of analytical and experimental results of the strength of compressed reinforced concrete columns under special combinations of loads. The 5th International Scientific Conference "Integration, Partnership and Innovation in Construction Science and Education", volume 86.
- [6] Tamrazyan A.G., Avetisyan. L.A. (2014), Estimation of load bearing capacity of eccentrically compressed reinforced concrete elements under dynamic loading in fire conditions, Applied Mechanics and Materials, pp:638-640.

[7] Kokot S., Anthoine A., Negro P. and Solomos G. (2012). Static and dynamic analysis of a reinforced concrete flat slab frame building for progressive collapse, *Engineering Structures*, Vol. 40: pp 205-217.

[8] FEMA 356(2002), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency.