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DINAMIC FORCES IN THE ECCENTRICALLY COMPRESSED MEMBERS OF REINFORCED CONCRETE FRAMES UNDER ACCIDENTAL IMPACTS

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Abstract: Interest to solving scientific problems related to the evaluation of facilities' resistance and its protection against progressive collapse increases and attracts more and more attention of specialists in the field of structural analysis and design. Therefore, the article presents the results of a computational analysis of dynamic forces in eccentrically compressed reinforced concrete members of structures under accidental impact such as sudden removal of a load bearing member. Using relations for specific deformation energy and integrating it through the cross-section area, the analytical expressions for dynamic strains and curvatures have been obtained for physically and structurally nonlinear RC frame members under eccentric compression. These expressions in some cases allow symbolic solution, for example, in MathCAD software. In contrary, it can be solved with the approximate iterative method. To assess the reliability and effectiveness of the proposed quasi-static method, the analysis of the cast-in-situ reinforced concrete frame resistance to progressive collapse has been performed. The article also provides comparison of the simulation results of the nonlinear quasi-static analysis and the nonlinear dynamic time-history analysis.

Keywords: reinforced concrete, eccentric compression, accidental impacts, energy dissipation, dynamic effect, quasi-static analysis

ДИНАМИЧЕСКИЕ ЭФФЕКТЫ ВО ВНЕЦЕНТРЕННО СЖАТЫХ ЖЕЛЕЗОБЕТОННЫХ ЭЛЕМЕНТАХ МНОГОЭТАЖНЫХ ЗДАНИЙ ПРИ ОСОБЫХ ВОЗДЕЙСТВИЯХ

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Аннотация: Решение научных задач, связанных с проблемой живучести и защиты зданий и сооружений от прогрессирующего обрушения при особых воздействиях привлекает все более значительное внимание специалистов в области расчета и проектирования строительных конструкций, зданий и сооружений. В рассматриваемой статье приведены результаты расчетного анализа динамических догружений внецентренно сжатых железобетонных элементов конструктивных систем при их структурной перестройке, вызываемой аварийными воздействиями. Для таких физически и конструктивно нелинейных систем, применительно к рассматриваемому напряженному состоянию на энергетической основе путем интегрирования выражений удельной энергии деформации волокна (фибры) по высоте расчетного сечения железобетонного элемента получены аналитические выражения для динамических деформаций и кривизн этого элемента. Структура уравнений в отдельных случаях допускает их решение в символьном виде, например, в ПК MathCAD, либо они решаются одним из приближенных методов. Для оценки достоверности и эффективности предложенного квазистатического метода выполнен расчёт живучести монолитной железобетонной рамы каркаса пятиэтажного здания и приведено сопоставление результатов нелинейного расчетного анализа по квазистатическому методу и методу прямого динамического расчета.

Ключевые слова: железобетон, внецентренное сжатие, особое воздействия, диссипация, динамический эффект, квазистатический метод

INTRODUCTION

Interest to solving scientific problems related to the structures' progressive (disproportional) collapse resistance under special impacts increases and attracts more and more attention of specialists in the field of structural design and analysis as evidenced by the number of research articles published over the past two decades. In many countries including Russia, the theoretical [1-8] and experimental [9-14] studies have been performed that became the basis for development and introduction of new regulatory documents for the facilities' protection against progressive collapse under accidental impacts [15-18]. Currently, the most complex and debatable question in the considered field is the assessment of the dynamic effects in structural systems during forces' redistribution through the alternate load paths, when accidental impact occurred [20].

For the assessment of such a dynamic effect in composite nonlinear deformable bars under instantaneous structural transformation, G.A. Geniyev proposed energy approach [21, 22]. Later, V.I. Kolchunov, N.V. Fedorova, N.B. Androsova, P.A. Korenkov and etc. [22-26] developed this approach. Their papers provide solutions for uniaxial compression and tension, as well as in transverse bending. They obtain the upper limit values of the forces and the lower limits of deformations in absolute value. With regard to cases of a more complex stress-strain state, the assessment of dynamic effects during the instantaneous load bearing member removal requires integrating the expressions for the specific strain energy over the cross-section area. However, such an approach may be associated with some computational difficulties when constructing the energy expressions, especially for structures with inelastic second-order effects. Therefore, we consider a simplified practical method for assessment the dynamic effects in physically nonlinear eccentrically compressed (or eccentrically tensioned) reinforced concrete bars. Due to the phenomenon of buckling in such members, the bending moment acting in cross-sections is a function of the axial force. In this case, the ultimate bending moment perceived by the cross-section also depend on the magnitude of the acting axial force. This leads to the fact that the moment vs. curvature interaction diagram of such element changes during loading. In addition, there is a decrease in the magnitude of the ultimate forces in flexible eccentrically compressed reinforced concrete members in comparison with those one for members of zero slenderness ratio, i.e., for the cross-section pure strength [29], that can be explained as inelastic second-order instability. These circumstances must be considered when constructing a simplified method for estimating the dynamic effects in eccentrically compressed members of frame structures under accidental actions.

MATERIALS AND METHODS

Let us consider an eccentrically compressed reinforced concrete element of unit length subjected to the external axial force N_{ext} and the bending moment M_{ext} as *Figure 1,a* shows. Because of external forces, the internal forces arise in the element cross-sections: $N_{int} = N_{ext}$, $M_{int} = M_{ext}$. For practical purposes, we neglect the increments of the axial force and bending moment along the unit length of the bar under consideration since its size dL is sufficiently small.

Considering the strain diagram in *Figure 1,b*, the internal axial force acting in the cross-section is equal to (1):

$$N_{int} = \varepsilon_{av} B_{N0} = \varepsilon_{av} E_b A_{b,red} = N_{ext}, \qquad (1)$$

where ε_{av} is the average strain value within the cross-section, E_b is the tangent modulus of elasticity of normal weight concrete at a stress of σ = 0 and at 28 days; $A_{b,red}$ is the reduced cross sectional area. It should be noted, that $A_{b,red}$ is the function of the axil force N.

If we approximate the moment - curvature interaction diagram with the second order polynomial, the internal bending moment can be obtained from Eq. (2):

$$M_{int} = \chi B_{M0}\left(1 - \frac{\chi}{2\chi_0}\right) = M_{ext}, \qquad (2)$$

where χ is the curvature of the deformed cross-section, $B_{M0} = E_b J_{b,red}$ is the initial (or undeformed) bending stiffness, χ_0 is the curvature corresponding the ultimate moment M_{ult} as *Figure 1,d* shows.

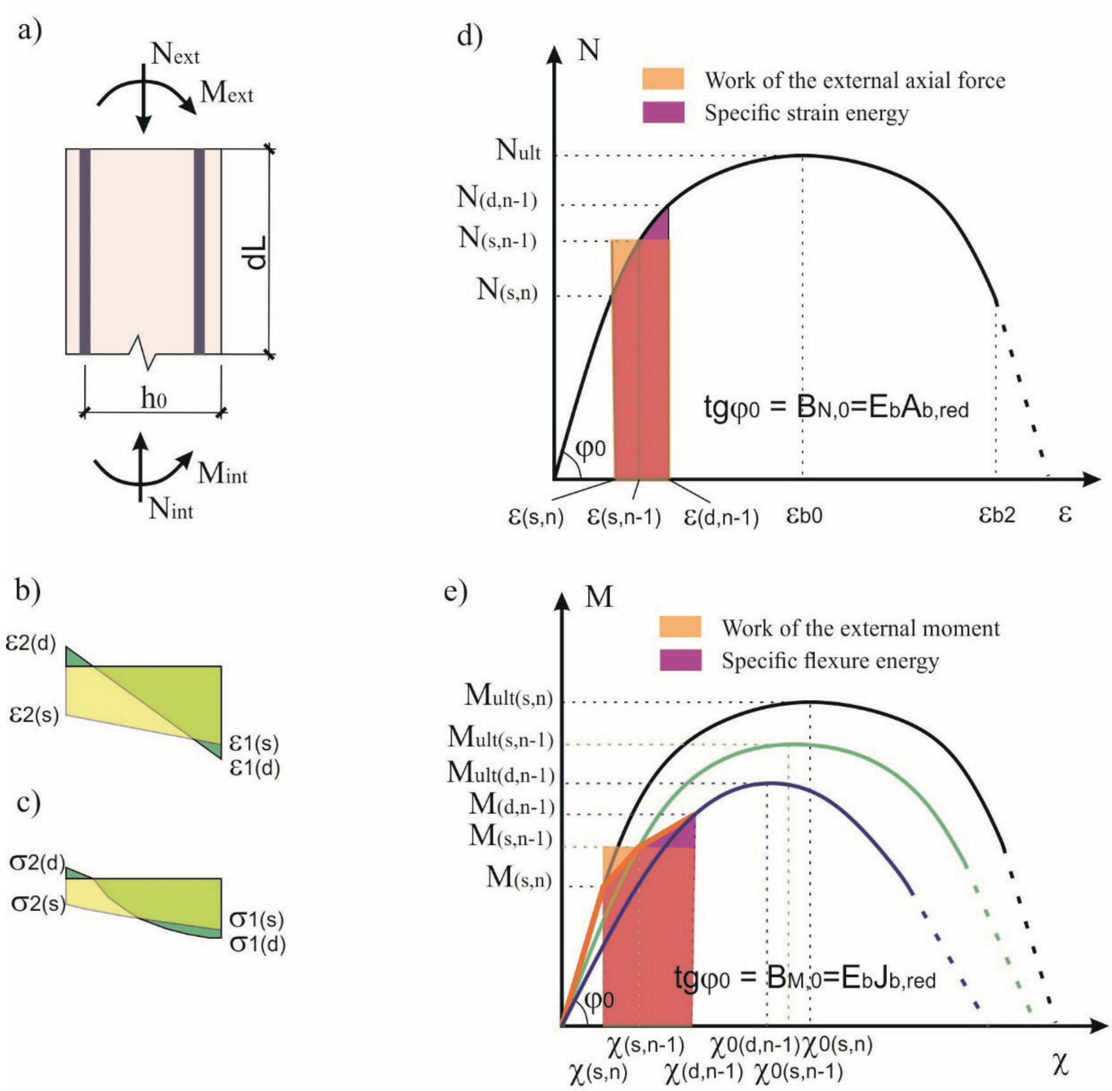

Figure 1. Eccentrically compressed reinforced concrete element: a) design scheme; b) strain diagram; c) normal stresses' diagram; d) scheme of the axial forces vs. strain diagram; e) moment vs. curvature diagram

In the first approximation, we find the curvature χ_0 using the expression for the ultimate bending moment of an eccentrically compressed element, according to SP 63.13330:

$$\chi_0 = \frac{0.8\varepsilon_{b2}}{x}, \qquad (3)$$

where ε_{b2} is the ultimate compressive strain in concrete, x is the neutral axis depth for rectangular compressive stresses diagram in concrete in accordance with 8.1.14 SP63.13330:

$$\begin{cases} x = \dfrac{N + R_{sn}A_s - R_{scn}A'_s}{R_{bn}b} \; for\ \xi \le \xi_R, \\ x = \dfrac{N + R_{sn}A_s \dfrac{1+\xi_R}{1-\xi_R} - R_{scn}A'_s}{R_{bn}b + \dfrac{2R_{sn}A_s}{h_0(1-\xi_R)}} \; for\ \xi > \xi_R. \end{cases} \tag{4}$$

In contrast to SP 63.13330, the formulas (4) adopt characteristic strength of materials in regard with SP 385.132580 requirements for a special limit state criterion. Characteristic compressive strength of steel reinforcement $R_{sc,n}$ corresponds to strain ε_s = 0.0035 which is restricted by ultimate compressive strain in concrete. $\xi = x/h_0$ is the relative depth of the neutral axis, ξ_R is the boundary relative depth of the neutral axis.

The coefficient 0.8 adopted in formula (3) reflects the ratio between the depth of the neutral axis for the rectangular distribution of normal stresses in the compressed zone and the actual depth of the neutral axis. It should be replaced with 0.7 for concrete of compressive strength classes B70-B100. Popov D. [28] provides the values of this coefficient for corrosion-damaged reinforced concrete elements under dynamic loading.

Following the approach proposed by G.A. Geniyev [20], we accept the principle of constancy of the total specific strain energy. In addition, we introduced the assumption that this principle meets to compression and bending separately with an enough accuracy for practical purposes. As a result, we obtain a system of equations in which the first one can be resolved with respect to the strain $\varepsilon_{d,n-1}$, and the second one resolved with respect to the strain $\varepsilon_{d,n-1}$ and curvature $\chi_{d,n-1}$:

$$\begin{cases} \Phi(\varepsilon_{d,n-1}) - \Phi(\varepsilon_{s,n}) = N_{s,n-1}(\varepsilon_{d,n-1} - \varepsilon_{s,n}); \\ \Phi(\chi_{d,n-1}) - \Phi(\chi_{s,n}) = M_{s,n-1}(\chi_{d,n-1} - \chi_{s,n}), \end{cases} \tag{5}$$

where $\Phi(\varepsilon_{d,n-1})$ is the specific strain energy for uniaxial dynamic compression caused by sudden load bearing structural member removal and determined from Eq. (6):

$$\Phi(\varepsilon_{d,n-1}) = B_N \int_0^{\varepsilon_{d,n-1}} \left(\varepsilon - \frac{\varepsilon^2}{2\varepsilon_{b0}}\right) d\varepsilon = \\ = \frac{E_b A_{b,red} \varepsilon_{d,n-1}^2}{2} \left(1 - \frac{\varepsilon_{d,n-1}}{3\varepsilon_{b0}}\right). \tag{6}$$

$\Phi(\varepsilon_{s,n})$ is the specific strain energy for uniaxial compression with service load before sudden load bearing structural member removal and determined from Eq. (7):

$$\Phi(\varepsilon_{s,n}) = B_N \int_0^{\varepsilon_{s,n}} \left(\varepsilon - \frac{\varepsilon^2}{2\varepsilon_{b0}}\right) d\varepsilon = \\ = \frac{E_b A_{b,red} \varepsilon_{s,n}^2}{2} \left(1 - \frac{\varepsilon_{s,n}}{3\varepsilon_{b0}}\right) \tag{7}$$

$\Phi(\chi_{d,n-1})$ is the specific flexure energy for uniaxial dynamic compression caused by sudden load bearing structural member removal and determined from Eq. (8):

$$\Phi(\chi_{d,n-1}) = B_M \int_0^{\chi_{d,n-1}} \left(\chi - \frac{\chi^2}{2\chi_0}\right) d\varepsilon \\ = \frac{E_b J_{b,red} \chi_{d,n-1}^2}{2} \left(1 - \frac{\chi_{d,n-1}}{3\chi_{0,d,n-1}}\right). \tag{8}$$

$\Phi(\chi_{s,n})$ is the specific flexure energy for uniaxial compression with service load before sudden load bearing structural member removal and determined from Eq. (9):

$$\Phi(\chi_{s,n}) = B_M \int_0^{\chi_{s,n}} \left(\chi - \frac{\chi^2}{2\chi_0}\right) d\varepsilon = \\ = \frac{E_b J_{b,red} \chi_{s,n}^2}{2}\left(1 - \frac{\chi_{s,n}}{3\chi_{0,s,n}}\right). \tag{9}$$

$N_{s,n-1}(\varepsilon_{d,n-1} - \varepsilon_{s,n})$ is the specific work under compression after the sudden load bearing structural member removal,

$M_{s,n-1}(\chi_{d,n-1} - \chi_{s,n})$ is the specific work under flexure after the sudden load bearing structural member removal.

The Eqs. (10)…(14) allow determining geometric parameters for the above mentioned relation:

$$A_{b,red} = b \int_{-0.5h}^{0.5h} \left(1 - \frac{\varepsilon}{2\varepsilon_{b0}}\right) dy + \frac{E_{s1}}{E_b} A_s + \frac{E_{s2}}{E_b} A'_s, \tag{10}$$

Where

$$\varepsilon = \varepsilon_{av} + \chi \cdot y, \tag{11}$$

$$S_{b,red} = b \int_{-0.5h}^{0.5h} \left(1 - \frac{\varepsilon}{2\varepsilon_{b0}}\right) y dy - \frac{E_{s1}}{E_b} A_s(0.5h - a) + \frac{E_{s2}}{E_b} A'_s(0.5h - a'), \tag{12}$$

$$y_{g.c.} = \frac{S_{b,red}}{A_{b,red}}, \tag{13}$$

$$J_{b,red} = b \int_{-0.5h+y_{g.c.}}^{0.5h+y_{g.c.}} \left(1 - \frac{\varepsilon}{2\varepsilon_{b0}}\right) y^2 dy + \frac{E_{s1}}{E_b} A_s(0.5h + y_{g.c.} - a)^2 \\ + \frac{E_{s2}}{E_b} A'_s(0.5h - y_{g.c.} - a')^2. \tag{14}$$

Eqs. (10) … (14) apply the following symbols:
y is the current depth from geometrical cross sectional gravity center in Eqs. (10), (12) and from physical cross sectional gravity center in Eq. (14);
ε_{b0} is the compressive strain in concrete at the ultimate compressive stresses $\sigma = R_{b,n}$;
E_{s1}, E_{s2} are moduli of deformation for reinforcement steel in tension and in compression at current strain values in accordance with stress vs. strain curve;
A_s, A'_s are the cross sectional areas of reinforcement steel in tension and in compression respectively;
b is the cross-sectional width out of bending moment plane;
h is the overall cross-sectional depth in the bending plane;
a, a' are the distances from gravity centers of reinforcement in tension and compression respectively to the concrete surfaces;
$S_{b,red}$ is first moment of area for reduced cross-section measured from geometrical gravity center;
$y_{g.c.}$ is the distance between geometrical and physical gravity centers of the cross-section when the positive direction is to the most compressed face;
$J_{b,red}$ is the moment of inertia of the reduced cross-section with respect to physical gravity center.

After substitutions (1), (6), (7) and mathematical operations, the first equation of system (5) transforms into a 4th-oreder polynomial with respect to $\varepsilon_{av(d,n-1)}$ that is the average cross-sectional strain under dynamic loading (15):

$$\begin{gathered}\varepsilon^4_{av(d,n-1)}\frac{b\cdot h}{12}-\varepsilon^3_{av(d,n-1)}\left(\frac{b\cdot h}{6}+\frac{b\cdot h}{4\varepsilon_{b0}}+\frac{A_s\cdot E_{s1}}{6E_b}+\frac{A'_s\cdot E_{s2}}{6E_b}\right)+\\ +\varepsilon^2_{av(d,n-1)}\left(\frac{b\cdot h}{2}+\frac{A_s\cdot E_{s1}}{2E_b}+\frac{A'_s\cdot E_{s2}}{2E_b}\right)-N_{s,n-1}\cdot\varepsilon_{av(d,n-1)}+\\ +\varepsilon_{av,s,n}\cdot\left(N_{s,n-1}+\frac{E_b\cdot A_{b,red(s,n)}\cdot\varepsilon_{av,s,n}}{6\varepsilon_{b0}}\right)=0.\end{gathered}\qquad(15)$$

In few cases, Eq. (15) allows a solution in symbolic form, for example, in the MathCAD Software. However, as a rule, it is more convenient to find its solution using one of the approximate methods. In this case, the solution should correspond to the physical essence of the problem (16):

$$\begin{cases}\varepsilon_{av(d,n-1)}>\varepsilon_{av(s,n-1)},\text{если }\varepsilon_{av(s,n-1)}>\varepsilon_{av(s,n)};\\ \varepsilon_{av(d,n-1)}<\varepsilon_{av(s,n-1)},\text{если }\varepsilon_{av(s,n-1)}<\varepsilon_{av(s,n)},\end{cases}\qquad(16)$$

and provides the minimum value of the strain energy among all physically feasible solutions.
Substituting (8), (9), (14) into the second equation of system (5) with respect to (2), (3), (4) and the value $\varepsilon_{av(d,n-1)}$, we obtain a 7th-order polynomial with respect to $\chi_{d,n-1}$ that is the curvature of an eccentrically compressed element from the axis passes through the physical gravity center:

$$\begin{gathered}a_1\chi^7_{(d,n-1)}+a_2\chi^6_{(d,n-1)}+a_3\chi^5_{(d,n-1)}+a_4\chi^4_{(d,n-1)}+a_5\chi^3_{(d,n-1)}+a_6\chi^2_{(d,n-1)}+\\ +a_7\chi_{(d,n-1)}+a_8=0,\end{gathered}\qquad(17)$$

where a_1, a_2, …, a_8 are the constants determined from the Eqs. (1) – (15) for given stress-strain state of the primary and secondary design schemes under static loading.
The solution of the polynomial (17) can be found approximately using the MathCAD Software. In this case, the obtained solution should satisfy to the physical essence of the problem (18):

$$\begin{cases}\chi_{(d,n-1)}>\chi_{(s,n-1)},if\ \chi_{(s,n-1)}>\chi_{(s,n)};\\ \chi_{(d,n-1)}<\chi_{(s,n-1)},if\ \chi_{(s,n-1)}<\chi_{(s,n)},\end{cases}\qquad(18)$$

and provides the minimum value of the strain energy among all physically feasible solutions.
If we know the average cross-sectional strain $\varepsilon_{av(d,n-1)}$ and the curvature $\chi_{d,n-1}$ relative to the physical gravity center of the cross-section under dynamic loading, then it is easy to determine the stress - strain state at any point of the cross-section from (11), (13) and evaluate criteria of the special limit state in accordance with SP 385.132580. Expressions (1) and (2) allow determination of the acting in the cross-section.

RESULTS

To evaluate the proposed method, we carried out analysis resistance to progressive collapse of the 5-storey reinforced concrete frame (Figure 2, a) when the outer column on the first floor in the B-1 axes failured. The spans of the frame are 6 m, the floors' height is 3.3 m. The frame material is concrete of B30 compressive class. The cross-sectional dimensions of the columns are of 300x300 mm (*Figure 2,b*). For the girders, it is of 300x450 mm (*Figure 2,c,d*). A500 was adopted as a longitudinal reinforcement steel bar, and stirrups were made of A240 reinforcement steel. Reinforcement parameters have been choosen in accordance with design requirements of SP 63.13330 and result of structural analysis for combination of characteristic loads according to SP 20.13330 that included death and long-term loads, short-term loads on floors with a characteristic value of 1.5 kN/m^2, snow load with a characteristic value of 1.5 kN/ m^2, wind load for I wind region, terrain type A.

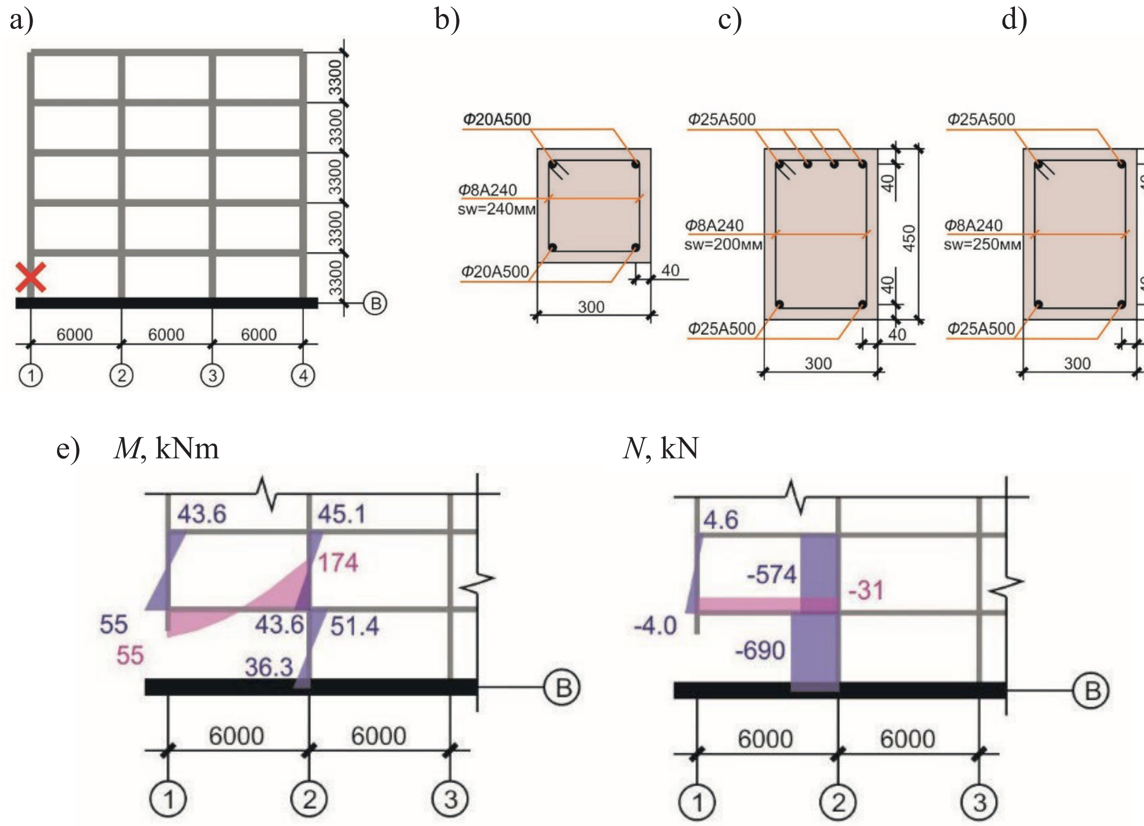

Figure 2. 5-storey reinforced concrete frame: a) primary design scheme; b) cross-section and reinforcement scheme of the columns; c) the same for the supporting sections of the girders; d) the same for girders in the middle of the span, e) the results of a nonlinear dynamic analysis for elements adjacent to the area of initial local failure for the time t = 0.18 s after the impact

We performed the assessment of the frame stress-strain state according to the primary and secondary design schemes [7] after decay of oscillations using nonlinear quasi-static analysis. To assess the reliability of the proposed method, we additionally perform a nonlinear dynamic time-history analysis of the frame. Using modal analysis for secondary design scheme of the reinforced concrete frame (Fig. 2, a), we obtain the period of vibration T = 0.97s for the lower mode which is most similar to the assumed deformed state of the frame after accidental impact. Thus, the time of redistribution of the reaction of the removed column was accepted t = 0.1T = 0.097 s. For comparison, the upper cross-section of the column on the first floor in the B-2 axes was chosen.

Table 1 provides the computation results of the analysis according to the proposed nonlinear quasi-static and dynamic time-history method. In addition, it provides a comparison of the obtained dynamic amplification factors k_d and coefficients of dynamic overloading θ_d.

Table 1. Comparison of the computation results for nonlinear quasi-static and dynamic time-history analysis

Parameter	Units	Quasi-static analysis	Dynamic time-history analysis	Difference, %
Initial data				
$N_{s,n}$	kN	333	333	-
$\varepsilon_{av,s,n}$	-	$0.118\cdot10^{-3}$	$0.118\cdot10^{-3}$	-
$M_{s,n}$	kNm	0.794	0.794	-
$\chi_{av,s,n}$	m^{-1}	$3.3\cdot10^{-5}$	$3.3\cdot10^{-5}$	-
$N_{s,n-1}$	kN	597.3	597.3	-
$M_{s,n-1}$	kNm	30.4	30.4	-
Computation results for dynamic effects				
$N_{d,n-1}$	kN	882	690	27.8
$\varepsilon_{av,d,n-1}$		$0.29\cdot10^{-3}$	$0.25\cdot10^{-3}$	16
$M_{d,n-1}$	kNm	72.5	51.4	41
$\chi_{av,d,n-1}$	m^{-1}	$3\cdot10^{-3}$	$2.59\cdot10^{-3}$	15.8
$k_{d,N}=\frac{N_{d,n-1}}{N_{s,n-1}}$	-	1.48	1.15	-
$k_{d,M}=\frac{M_{d,n-1}}{M_{s,n-1}}$	-	2.38	1.69	-
$\theta_{d,N}=\frac{N_{d,n-1}}{N_{s,n}}$	-	2.65	2.07	-
$\theta_{d,M}=\frac{M_{d,n-1}}{M_{s,n}}$	-	91.31	64.74	-

DISCUSSION

Analysis of the data presented in Table 1 indicates that the average cross-sectional strain according to the proposed method is of 16% higher than the value by nonlinear dynamic analysis. Curvatures shows the difference of 15.8%. However, difference between dynamic axial force and bending moment increase to 27.8% and 41% respectively. This indicates a significant influence of the form of stress vs. strain curve. The 2^{nd} order polynomial approximation was adopted in the proposed method, the approximation of the concrete state diagram by a parabola was used. And the nonlinear dynamic analyses applied the exponential approximation of the piece-wise linear diagram. Also, the discrepancy is due to the fact that the dynamic analysis has been performed for certain time, when the proposed method corresponds to instantaneous impact. At the same time, the excess of the stress-strain state parameters for proposed method provides the additional margin of safety of the bearing element. Besides, it allows assessing the special limit state of the reinforced concrete structures using the results of nonlinear static analysis for the primary and secondary design schemes eliminating nonlinear dynamic analysis. This is especially useful when one conducts structural analysis for the complex structural systems.

CONCLUSION

1. The article provides an approach based on energy relations that allows assessing the bearing capacity of eccentrically compressed physically nonlinear structural elements under accidental impacts caused by sudden failure of a structural member of the facility. The approach uses the results of a nonlinear quasi-static analysis according to the primary and secondary design schemes and eliminate nonlinear dynamic time-history analysis.
2. Structural analysis of 5-storey reinforced concrete frame for progressive collapse resistance shows that the difference between the computational results according to the proposed method and the nonlinear dynamic analysis does not exceed 16% for average strains and curvatures and 41% for internal forces. This discrepancy provides the addition margin of safety.
3. The revealed discrepancy between the computational results is due to the fact that the dynamic analysis has been performed for certain time, when the proposed method corresponds to instantaneous impact, as well as various approximation of stress vs. strain and moment vs. curvature curves adopted in the methods considered in the article.

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