547. Steel frame response due to extreme dynamic load

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Abstract. Dynamic response of a rigid frame on a steel columns is investigated. Free motion of the structure when deformations in the columns are elastic is discussed. Motion of the structure after horizontal impact and blast loading is investigated and axial and transverse velocities after the impact are assessed. Interconnection of the axial and the transverse column forces in the elastic and elasto-plastic deformation regions are considered.

Keywords: steel frame, impact, elaso-plastic deformations.

Introduction

Inelastic static analysis has become almost routine in the design practice, but dynamic analysis remains a challenge. The answer, given by Elnashai [1] to the question do we really need inelastic dynamic analysis, is: there will always be a domain where dynamic analysis is necessary, but the "necessity domain" is ever diminishing. Or may be the static domain is ever increasing, and this conclusion differs from the former. In plastic hinge theory it is assumed that the plastic deformation is concentrated on the two ends of a beam elements [2]. In the nonlinear dynamic analysis of frames subjected to distributed loads the moving node strategy is presented by Yan and Au [3]. A push-over analysis procedure based on continuous non-linear post-elastic material model is developed by Hasan, Xu and Grirson [4]. The technique provides the ability to monitor the progressive plastification of steel frame elements and structural systems under increasing intensity of earthquake ground motion. Interaction of column axial force and bending moment must be taken into account to predict axial displacements. Como, De Stefano and Ramasco [5] point out the progressive axial shortening of adjacent columns and remarkable amplification in beam plastic rotations as inelastic seismic response of steel frames. Both statical and kinematical approach are investigated by Benfratello, Cirone, Giambanco [6] in optimal design of steel frames subjected to cyclic loads. The typical Bree-like diagram discloses dependence of elastic shakedown, plastic shakedown or incremental collapse, instantaneous collapse dependence on fixed load versus cyclic load values.

A second-order plastic-zone formulation for the non-linear analysis and design of steel frames is presented by Alvarenga, Silveira [7]. A beam-column finite element model based on Bernoulli-Euler theory is applied. The axial force iterative integration process is developed at elemental level. Aim of this process is to catch axial force balance and more closely follow the plasticity spread in a beam-column member. The behavior of structure due to blast loading is examined by Fischer, Haring [8]. A single degree of freedom (SDOF) model is accepted, a method to determine the best parameters of typical structural resistance functions for SDOF is introduced. The equivalent ersatz-system is realized with a lumped mass and idealized load resulting in one-dimensional deflection. Impact effect of sudden fracture at steel frame connections under severe earthquake or other extreme loads is presented by Chen, Bian, Liao [9]. Response model also is SDOF. If fracture in the frame occurs, the constraint condition of

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connection can change immediately from completely rigid to ideal hinge. A new design-oriented methodology for progressive collapse assessment of floor systems within multi-storey buildings subjected to impact from an above failed floor is proposed by Vlassis et all [10]. The two extreme impact possibilities of fully rigid and fully plastic impact are determined. The strength, ductility supply and energy absorption capacity of the lower impacted floor are investigated. Some blast-resistant steel-framed buildings, anchored or free to slide, are examined by Summers [11]. Dynamic model and energetic fracture criterions during impact loading are investigated by Žiliukas, Gintalas [12].

In this paper response of a rigid frame on steel columns is investigated. The transverse and the axial displacements of the columns are considered, therefore a three-dimensional motion in a plane should be investigated. The distinctive feature of the structure is the substantial difference in rigidity to vertical and horizontal displacements. Influence of the rigidity and damping in the elastic region and possible extension to the elasto-plastic regions of deformation are discussed. Dependence of the horizontal force F and the vertical axial force N of the column on both horizontal w and vertical u displacements in the elasto-plastic region is a characteristic feature of the solution [13].

Free motion of structure

The two columns $B_0 B$ and $D_0 D$ support a rigid structure, therefore the equal horizontal displacements u of the hinges B, D, positive vertical displacements w_B, w_D are depicted in Fig. 1. The angular velocity of the rigid structure $\omega = (\dot{w}_B + \dot{w}_D)/l$. The cendroid C velocity components are $\dot{w}_{CZ} = (\dot{w}_B + \dot{w}_D)/2$, $\dot{u}_{CX} = \dot{u} + n_0(\dot{w}_B + \dot{w}_D)$, where $n_0 = h_0/l$, h_0 is the centroid height (Fig. 1). If rigidities of the columns $B_0 B$, $D_0 D$ are equal approximate values of the natural frequencies are

$$\omega_1 = \sqrt{\frac{2k_x}{m}}(1 - n_0 \varepsilon), \ \omega_2 = \sqrt{\frac{2k_z}{m}}, \ \omega_3 = \frac{l}{i_C}\sqrt{\frac{k_z}{2m}}, \tag{1}$$

where m is mass of the structure, k_x, k_z are stiffnesses of the columns; $i_C = \sqrt{I_C/m}$, I_C – moment of inertia of the structure with respect to the centroid C_v (Fig. 2). The axial stiffness k_z significantly exceeds the lateral stiffness k_x:

$$\varepsilon = 2n_0 \frac{k_x}{k_z} \ll 1. \tag{2}$$

The frequency ω_1 in (1) is determined neglecting ε^2, while frequencies ω_2, ω_3 are expressed neglecting ε.

If U, W_B, W_D are amplitudes of the oscillations then the first natural frequency ω_1 corresponds to the eigenmode $\varepsilon U = W_B = W_D$: that is rotation around the point C_ω, and the distance $h_\omega = C_0 C_\omega = l/2\varepsilon$. As the distance $h_\omega \gg l$ displacements of B and D slightly deviate from the horizontal line BD. Nevertheless this deviation is important and approximation of the frequency ω_1, presented in (1), is essential. The second eigenmode is vertical translation of the structure $U = 0$, $W_B = -W_D$. The third eigenmode is rotation around the centroid C

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$U\frac{l}{2}h_0 = -W_B = -W_D$. If $i_C > l/2$ the frequency $\omega_3 < \omega_2$ and the order of frequencies should be changed.

By assuming that displacements $u_0 = w_{B0} = w_{D0} = 0$ when time $t = 0$ and initial velocities are $\dot{u}_0$, $\dot{w}_{D0} = \dot{w}_{B0} = \dot{w}_0$, the displacements can be deduced:

$$\begin{cases} u = \dfrac{\dot{u}_0 + 2n_0\dot{w}_0}{\omega_1}\sin\omega_1 t - \dfrac{2n_0\dot{w}_0}{\omega_3}\sin\omega_3 t, \\[3mm] w = \varepsilon\dfrac{\dot{u}_0 + 2n_0\dot{w}_0}{\omega_1}\sin\omega_1 t + \dfrac{\dot{w}_0}{\omega_3}\sin\omega_3 t. \end{cases} \tag{3}$$

Assumption $\dot{w}_{D0} = \dot{w}_{B0} = \dot{w}_0$ implies initial angular velocity $\omega_0 = 2\dot{w}_0/l$ and vertical translation velocity $\dot{w}_{CZ} = 0$.

For column connected to structure with a hinge $k_x = 3EI_A/H^3$, $k_z = EA/H$, where E is Young's modulus, I_A – moment of inertia, A – cross-section area. For a double-tee cross-section shape

$$\frac{k_x}{k_z} = \left(\frac{h}{H}\right)^2 \frac{1+5q}{2},$$

where $2h$ is the web depth (Fig. 2), the shape parameter $q = 2A_1/A$, $2A_1$ is the area of the flanges. Taking $n_0 = \dfrac{h_0}{l} = \dfrac{1}{u}$, $\dfrac{h}{H} = \dfrac{1}{30}$ it can be solved from (1), (2)

$$\varepsilon = \frac{1+5q}{1800} = \frac{1}{1800} \div \frac{1}{7200}, \quad q = 0 \div 0.8,$$

$$\frac{\omega_1}{\omega_3} = 2\frac{i_C}{l}\sqrt{\frac{k_x}{k_z}} = \frac{1}{30}\sqrt{\frac{1+5q}{4.8}} \approx \frac{1}{66} \div \frac{1}{29},$$

where the moment of inertia of the structure is assumed $I_C = \dfrac{m}{12}\left(l^2 + 4h_0^2\right)$.
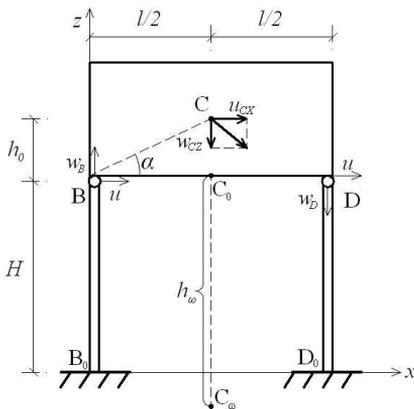
Fig. 1. Displacements of structure   **Fig. 2.** Cross-section of column
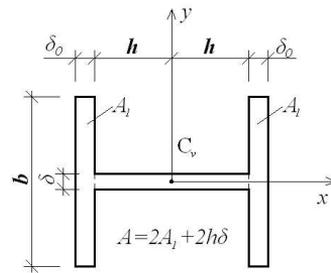
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Considering that $\omega_3 \gg \omega_1$, decrease of the third eigenmode amplitude will be very high when compared to the first eigenmode. Only the terms with $\sin \omega_1 t$ in (3) are significant after 10 or something like that natural periods $T_3 = 2\pi/\omega_3$. Really, ratio of the first mode amplitudes after a half of period $T_1/2$ is $\dfrac{b_1'}{b_2'} = e^{\pi\zeta_1}$, but ratio of the third mode amplitudes after the same interval of time $T_1/2$ is $\dfrac{b_1''}{b_2''} = e^{\pi\zeta_3 \omega_3/\omega_1} \gg e^{\pi\zeta_1}$. In addition the damping ratio of the third mode ζ_3 can be more than ζ_1 [5]. So, the system of equations (3) can be approximated

$$\begin{cases} u = \dfrac{\dot{u}_0 + 2n_0\dot{w}_0}{\omega_0}\sin\omega_1 t, \\ w = \varepsilon\dfrac{\dot{u}_0 + 2n_0\dot{w}_0}{\omega_0}\sin\omega_1 t. \end{cases} \tag{4}$$

Horizontal impact and blast loading

Motion of the structure after impact by body of mass m_e and velocity $\dot{u}_e$ (Fig. 3) can be deduced applying principle of conservation of linear momentum and angular momentum. From the conservation principle for z axis $\dot{w}_{CZ0} = (\dot{w}_{B0} - \dot{w}_{D0})/2 = 0$, so the velocities $\dot{w}_{D0} = \dot{w}_{B0} = \dot{w}_0$. The other two equations can be presented:

$$\begin{cases} (m_e + m)\dot{u}_0 + 2\dfrac{h_e m_e + h_0 m}{l}\dot{w}_0 = m_e(1 + e_E)\dot{u}_e, \\ m_e\dot{u}_0 + 2\dfrac{h_e m_e + h_E m}{l}\dot{w}_0 = m_e(1 + e_E)\dot{u}_e, \end{cases} \tag{5}$$

where $h_E = i_c^2/(h_e - h_0)$, $h_e \neq h_0$, e_E is coefficient of restitution. The final velocities after the impact $\dot{u}_0$, $\dot{w}_0$ can be solved from (5). Relation of these velocities is

$$\frac{\dot{u}_0}{\dot{w}_0} = \frac{2}{3}\frac{(l/2)^2 + 4h_0^2 - 3h_0 h_e}{(h_e - h_0)l}. \tag{6}$$

If $h_e = h_0$ the impact is direct central and $\dot{w}_0 = 0$. Dependence of the ratio $\dot{u}_0\big/\sqrt{\dot{u}_0^2 + \dot{w}_0^2}$ on the distance ratios h_0/l, h_e/l is in Fig. 4. It should be noted that the ratio does not depend on the coefficient of restitution, but either of the two solutions $\dot{u}_0$, $\dot{w}_0$ of the system (5) depend on e_E and mass ratio m_e/m.

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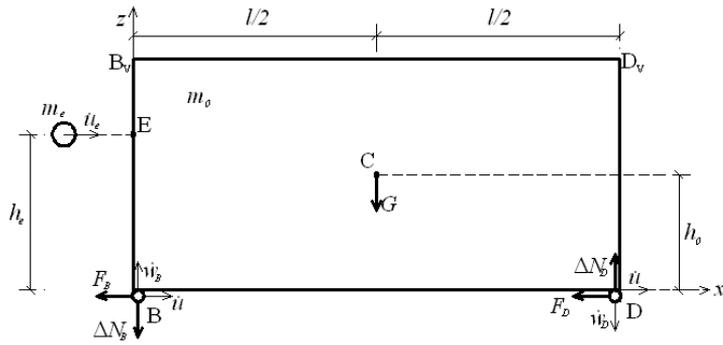
Fig. 3. Impact on the structure
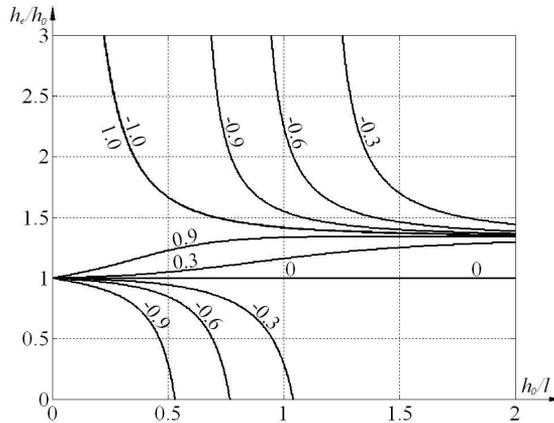
Fig. 4. Ratio of the velocities $\dfrac{\dot{w}_0}{\sqrt{\dot{u}_0^2 + \dot{w}_0^2}}$ due to horizontal impact

A blast loading on the structure face wall can be assumed as distributed linear momentum or linear impulse during the time interval t_+. Loading on the horizontal plane $B_V D_V$ (Fig. 3) is a travelling shock wave and, if the distance l exceeds significantly the positive pressure wave length λ_{ex}, the resultant should be approximated by force R_{zex} moving at a velocity $V_{ex} \approx 350 \ m/s$. In Table 1 Δp is the maximal pressure at the front of a shock wave, t_+ – time of positive pressure in a fixed-point at a distance b_{ex} from the blast of C_{ex} trotyl (or other equivalent explosive) [14]. The time it takes for the force R_{zex} to move the distance l is $t_{ex} = l/V_{ex} = 20/350 = 57 \cdot 10^{-3} \ s$ – approximately 10 times exceeds the time t_+ if $l = 20 \ m$.

Table 1.

C_{ex}	b_{ex}	Δp	t_+	V_{ex}	λ_{ex}
kg	m	MPa	s	m/s	m
1	10	0.0070	$4.7 \cdot 10^{-3}$	350	1.6
1	20	0.0050	$6.7 \cdot 10^{-3}$	347	2.3
0.5	10	0.0087	$4.2 \cdot 10^{-3}$	347	1.5
0.5	5	0.0230	$3.0 \cdot 10^{-3}$	371	1.1

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Fundamental translation frequency of buildings $f_1 \approx 1 \div 4 \ Hz$ [15]. So the structure can be assumed as acted upon by a force of a short duration compared with a natural period: t_+ is equal to one tenth of the period T_1 or less. The time t_{ex} in many cases fulfils this requirement also.

The total pressure, acting on the face wall, is $2\Delta p$ due to wave reflection.

Axial and transverse forces of columns

When the structure is subjected to some impact or blast loading the transverse forces F_B, F_D and the axial force increments ΔN_B, ΔN_D are exerted (Fig. 3).

If displacements $u_A = u_B = u$ and no plastic deformations take place in the columns the transverse forces $F_B = F_D = F$ (Fig. 1). Only horizontal translation and rotation around the centroid C is considered, therefore $w_B = w_D = w$ and

$$\begin{cases} F = -\dfrac{m}{2}\ddot{u} - m\dfrac{h_0}{l}\ddot{w}, \\ \Delta N = -m\dfrac{h_0}{l}\ddot{u} - 2m\dfrac{i_C^2 + h_0^2}{l^2}\ddot{w}. \end{cases} \tag{7}$$

When equations (4) are applied and condition $\varepsilon \ll 1$ is taken into account the dependence $\Delta N = 2n_0 F$ can be deduced. The axial forces of the columns are

$$\begin{cases} N_D = \dfrac{1}{2}G + 2n_0 F_D, \\ N_B = \dfrac{1}{2}G - 2n_0 F_B. \end{cases} \tag{8}$$

The dimensionless parameters are determined:
$\beta_D = \dfrac{M_D}{hN_Y} = \dfrac{H}{h}\dfrac{F_D}{N_Y}$, $\beta_B = \dfrac{H}{h}\dfrac{F_B}{N_Y}$, $\alpha_D = \dfrac{N_D}{N_Y}$, $\alpha_B = \dfrac{N_B}{N_Y}$, where $N_Y = A\sigma_Y$,
A – cross-section area of a column, σ_Y – yield stress [13]. Equation (8) then

$$\begin{cases} \alpha_D = \alpha_G + C_H 2\beta_D, \\ \alpha_B = \alpha_G - C_H 2\beta_B, \end{cases} \tag{9}$$

where $C_H = \dfrac{h_0}{l}\dfrac{h}{H}$, $\alpha_G = \dfrac{G}{2N_Y}$.

When distances BC and CD are equal (Fig. 3) then $N_D = N_B$, $\alpha_D = \alpha_B$ if $F_D = F_B = 0$, $\beta_D = \beta_B = 0$. In general case, when $BC \neq CD$, the constants in equations (9) α_{GD}, α_{GB} will not coincide. However in the perfectly elastic domain of deformations from $u_D = u_B = u$ it follows that $\beta_D = \beta_B$.

The linear dependences (9) of the dimensionless axial forces α on the transverse forces β are presented in Fig. 5 where $\alpha_G = 0.4$ is assumed. The straight line $\beta_1 = C_q(1-\alpha)$,

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$C_q = \dfrac{1+2q}{3}$ depicts the elastic domain. The elasto-plastic domain of column deformation is in between the line β_l and β_r. If angle $\psi = \arctan 2C_H$ is small (the angle ψ_1 in Fig. 5) the lines of the columns B and D are close and the points of intersection with the line β_l,

$$\beta_{D1} = C_q \frac{1-\alpha_{GD}}{1+2C_H C_q}, \quad \beta_{B1} = C_q \frac{1-\alpha_{GD}}{1-2C_H C_q}$$ can be assumed as approximately equal. If

$\alpha_{GD} = \alpha_{GB} = 0.4$, $h_0/l = 1/4$, $h/H = 1/30$, $q = 0.8$ then $C_H = 1/120$ and $\beta'_{D1} = 0.513$, $\beta'_{B1} = 0.528$. But if $h_0/l = 5$ then $C_H = 1/6$ and $\beta''_{D1} = 0.454$, $\beta''_{B1} = 0.608$. It can be assumed approximately that $\beta'_{D1} \approx \beta'_{B1}$ and columns enter elasto-plastic domain both together in the first case.
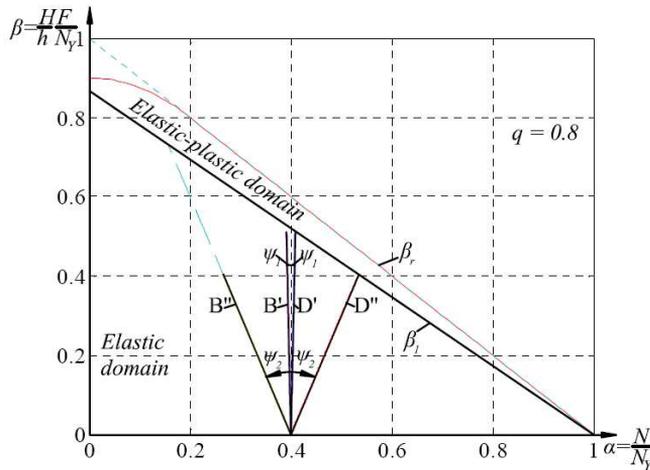
Fig. 5. Dependences of the transverse forces on the axial forces

In the elasto-plastic domain the axial force and the transverse force depends on the axial displacement w and transverse displacement u. If C_H is not small then strain in column B can stay on in the elastic domain while rigidity of the other column suffers degradation. In any case for one or for both columns the explicit dependences of the forces F and N on the displacements w and u are required, as presented in [13], [16].

When $\alpha_{GB} \neq \alpha_{GD}$ the two lines start from the different points on the α axis in Fig. 5 and their relative positions can be different.

The real dependences in the $\alpha - \beta$ plane (Fig. 5) can deviate from the straight lines at the beginning of motion when the third natural mode of a high frequency ω_3 is not reduced by damping.

Conclusions

Motion of a structure subjected to extreme load can be approximated by the first natural frequency and mode. This approximation is satisfactory for solution near the elastic and the elasto-plastic deformation region border.

When deformations of one or more columns are in elasto-plastic deformation region interconnection of the axial and the transverse forces is essential. Both the axial and the transverse forces depend on the axial and transverse displacements.

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