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EXTENDED ABSTRACT

Analysis of Behavioral Damped Outrigger in Tall Structures by Fourier Method in Hilbert Space

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1. Introduction

Achieving efficient methods to protect the structure against forces such as wind and earthquake is one of the first steps in the design of structures and has led to the provision of structural control systems. Structural control systems include three systems, active control, passive control and semi-active control. (Spencer Jr et al. 2003), (Mulligan et al. 2007). The outrigger system with the viscous damper, was proposed by, (Gamaliel, 2008) and its effect on tall structures was investigated, (O'Neill, 2006) showed that the use of damper and increasing damping in the outrigger system in proportion to increasing the stiffness and dimensions of the structure. (Farzad et al. 2019) has also used ultrasonic algorithms to determine the optimal position of outrigger system in tall steel frames. Experimental and analytical researches also show that the use of outrigger system is effective in reducing the lateral displacement of tall structures. (Tan et al. 2012), (Deng et al. 2014).

(Jovanovich, 2011) Used the Fourier series method in Hilbert space to investigate the transverse vibrations of the beam with boundary conditions of linear viscosity.

In this paper, the vibrations of the structure and the effect of central core system with the damped arm brace using axial load (due to the mass of the central mass) in the control of lateral displacement due to harmonic loading are investigated. Previous studies have not considered the effect of perimeter columns stiffness and the effect of axial force on frequencies and lateral displacement of the structure, and for solving the partial differential equation governing the problem, the Fourier series method is used to define the differential operator in Hilbert space.

2. Methodology

2.1. Central core system with outrigger and viscous damper with axial load effect

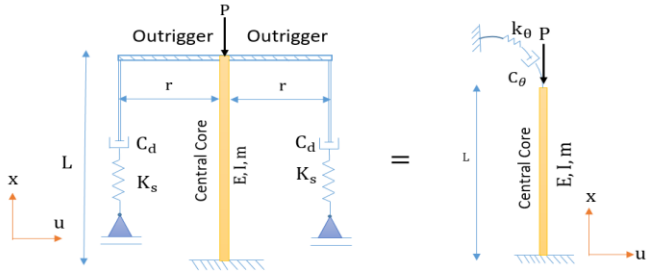

Fig 1. Central core system with outrigger and viscous damper with axial force effect

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$$\frac{\partial^2 u(x.t)}{\partial^2 t} + k^2 \frac{\partial^2 u(x.t)}{\partial^2 x} + c^2 \frac{\partial^4 u(x.t)}{\partial^4 x} = q(x.t) \quad (1)$$

Equation (1) is the differential equation of buckling of a console-beam in dynamic mode with axial load effect. In relation (1) (t) represents time, (u) is the displacement of the core, $q(x,t)$ is the shear force per unit on the core .

In the equation (1) $k^2 = \frac{P}{m}$, $c^2 = \frac{EI}{m}$.

The boundary conditions of the problem are as follows:

$$u(0.t) = 0. \frac{\partial u(0.t)}{\partial x} = 0. \frac{\partial^2 u(L.t)}{\partial^2 x} = -h_2 \frac{\partial u(L.t)}{\partial x} - h_1 \frac{\partial^2 u(L.t)}{\partial x \partial t}. \frac{\partial^3 u(L.t)}{\partial^3 x} = -h_3 \frac{\partial u(L.t)}{\partial x} \quad (2)$$

In the equation $h_1 = \frac{k_\theta}{EI}$, $h_2 = \frac{C_\theta}{EI}. h_3 = \frac{p}{EI}$, $C_\theta = 2r^2 C_d. K_\theta = 2r^2 K_s$. According to Riley's theory: $C_d = 2M\omega\xi$. The initial conditions of the problem are as follows:

$$u(x.0) = f(x). \frac{\partial u(x.0)}{\partial t} = g(x) \quad (3)$$

Using the variable separation technique, differential equation (1) is written in the form of equation (4) and boundary conditions (2) are written in the form of equation (5).

$$c^2 \frac{d^4 \varphi(x)}{d^4 x} + k^2 \frac{d^2 \varphi(x)}{d^2 x} + \lambda^2 \varphi(x) = 0 \quad (4)$$

$$\varphi(0) = 0. \frac{d\varphi(0)}{dx} = 0. \frac{d^2 \varphi(L)}{d^2 x} = (-h_1 \lambda - h_2) \frac{d\varphi(L)}{dx}. \frac{d^3 \varphi(L)}{d^3 x} = -h_3 \frac{d\varphi(L)}{dx} \quad (5)$$

The roots of the equation (relation (6)) of the linear differential are calculated.

$$f_{1.3} = \pm \sqrt{\frac{-k^2 + \sqrt{k^4 - 4c^2 \lambda^2 i}}{2c^2}}. f_{2.4} = \pm \sqrt{\frac{-k^2 - \sqrt{k^4 - 4c^2 \lambda^2 i}}{2c^2}} \quad (6)$$

The relation of (7) eigenvectors is a problem and is established for different eigenvalues.

$$\varphi(x) = C_4 (D_1 e^{f_1 x} + D_2 e^{-f_1 x} + D_3 e^{f_2 x} + e^{-f_2 x}) \quad (7)$$

Equation (8) defines $T = \begin{pmatrix} 0 & -c^2 \frac{\partial^3}{\partial^3 x} - k^2 \frac{\partial}{\partial x} \\ \frac{\partial}{\partial x} & 0 \end{pmatrix}$ as the orthogonal operator and $T^* = \begin{pmatrix} 0 & -\frac{\partial}{\partial x} \\ c^2 \frac{\partial^3}{\partial^3 x} + k^2 \frac{\partial}{\partial x} & 0 \end{pmatrix}$ as the orthogonal operator conjugate in Hilbert space and the principle of interval multiplication of vectors .The coefficient C_4 is calculated to calculate eigenvectors for eigenvalue.

$$C_4 = \frac{1}{\sqrt{\int_0^L \left[\phi_n^2(x) \lambda_n^2 + \left(c^2 \phi'''_{1.n}(x) + k^2 \phi'_{1.n}(x) \right) \left(\phi'_{1.n}(x) \right) \right] dx}} \quad (7)$$

Equation (9) is the lateral displacement equation of the central, and the first expression is the response of the system to the free vibration of the initial excitation is the response to the forced vibrations due to harmonic loading.

$$u(x.t) = \sum_{r=-\infty}^{+\infty} \{ \int_0^L [g(\xi) + \lambda_r f(\xi)] \, u_{1.r}(\xi) d\xi \} \frac{u_{1.r}(x) e^{\lambda_r t}}{\lambda_r} + \sum_{r=-\infty}^{+\infty} \frac{u_{1.r}(x)}{\lambda_r} \int_0^t e^{\lambda_r (t-\tau)} \int_0^L Q(\xi.\tau) u_{1.r}(\xi) d\xi d\tau \quad (9)$$

3. Results and discussion

3.1. Investigation of forced vibrations caused by harmonic loading

Here, by presenting a numerical model, the forced vibrations of a 40-story building and the height of each floor are 3 meters, under the harmonic load as follows.

$$Q(x.t) = A cos(\varpi t) \delta(x - x_f). A = m \times \frac{1}{2} g. \varpi = 3\ Hz\ . x_f = 20\ m \quad (10)$$

3.1.1. Effect of axial load due to mass of central ore and perimeter columns on forced vibrations due to harmonic load

In this section, the effect of axial force on relative displacement is investigated. The weight force, which consists of the central core and the surrounding columns, is calculated in this way and the effect is applied as an axial load on the roof on the structure.

Table 1. Model specifications for different outrigger lengths

For: $\xi = 10\%.\ \omega = 2.37\text{Hz}.\ m = 23116\frac{\text{kg}}{\text{m}}.\ M = 32.36 \times 10^5\text{kg}$ $.\ p = 3.4947 \times 10^7\text{N}$			
	$r = 11.76\text{m}$	$r = 16.6\text{m}$	$r = 23.47\text{m}$
C_d	$15.36 \times 10^6 \frac{\text{Ns}}{\text{m}}$	$15.36 \times 10^6 \frac{\text{Ns}}{\text{m}}$	$15.36 \times 10^6 \frac{\text{Ns}}{\text{m}}$
C_θ	$42.48 \times 10^8\text{Nsm}$	$84.65 \times 10^8\text{Nsm}$	169.21×10^8 Nsm
$N = \frac{E_c I}{2A_c E_s r^2}$	2	1	0.5

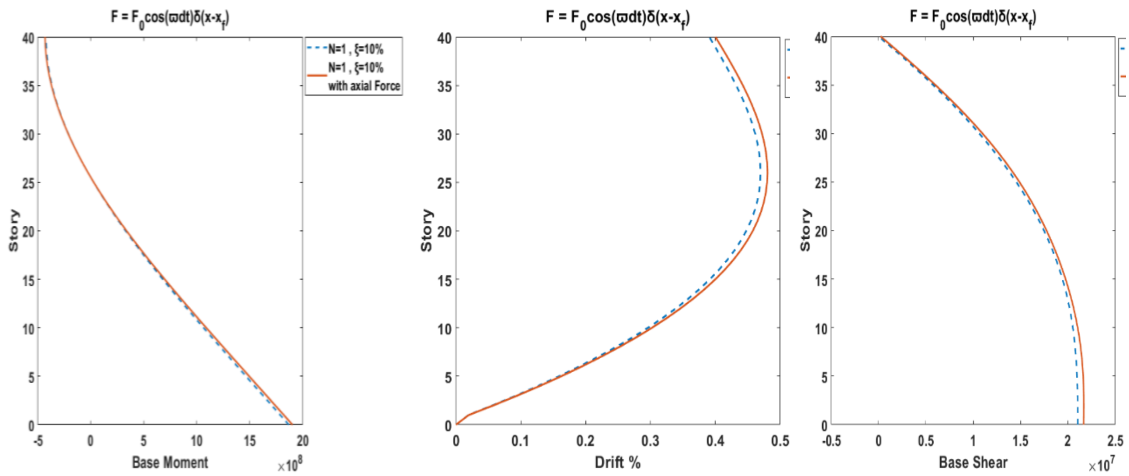

Fig. 1. Comparative displacement and moment and shear diagrams for damping 10% and flexural ratio N=1 with axial force effect

Table 1. Compression of maximum shear and moment and comparative lateral displacement of the structure in axial load effect mode under loading

For $N = 1.\ \xi = 0.1$	$Drift_{max}$	V_{max}	M_{max}
No axial load (P=0)	0.469%	21.03MN	1.869 GN
With axial load	0.481%	21.67MN	1.903 GN
Percentage difference	2.49 %	2.95%	1.79 %

According to the above figure and table, it is clear that considering the effect of axial force has increased the maximum values of comparative displacement and moment and shear the base of the floors in the building.

4. Conclusions

The research is based on the analysis of the central core system with outrigger and viscous damper with axial force effect base on the Fourier series semi-analytical method, which is defined by the differential operator in Hilbert space.

1) Increasing the arm restraint length, which reduces the stiffness ratio of the central core to the surrounding columns, is a good way to reduce and control the relative displacement of the floors.
2) Applying of axial force (due to the weight of the central core to the surrounding columns) is associated with an increase in the imaginary part of the system mode frequencies.

3) Applying of axial force in the relevant equation has caused a difference of 2.5% relative lateral displacement of the floors and 3 % in the base shear and 2% in the base moment under harmonic loading

5. References

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