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Optimum study on wind-induced vibration control of high-rise buildings with viscous dampers

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Abstract. In this paper, optimum methods of wind-induced vibration control of high-rise buildings are mainly studied. Two optimum methods, genetic algorithms (GA) method and Rayleigh damping method, are firstly employed and proposed to perform optimum study on wind-induced vibration control, six target functions are presented in GA method based on spectrum analysis. Structural optimum analysis programs are developed based on Matlab software to calculate wind-induced structural responses. A high-rise steel building with 20-storey is adopted and 22 kinds of control plans are employed to perform comparison analysis to validate the feasibility and validity of the optimum methods considered. The results show that the distributions of damping coefficients along structural height for mass proportional damping (MPD) systems and stiffness proportional damping (SPD) systems are entirely opposite. Damping systems of MPD and GAMPD (genetic algorithms and mass proportional damping) have the best performance of reducing structural wind-induced vibration response and are superior to other damping systems. Standard deviations of structural responses are influenced greatly by different target functions and the influence is increasing slightly when higher modes are considered, as shown fully in section 5. Therefore, the influence of higher modes should be considered when strict requirement of wind-induced vibration comfort is needed for some special structures.

Keywords: high-rise buildings; wind-induced vibration; viscous damping optimum control; genetic algorithms; Rayleigh damping method.

1. Introduction

In recent years, many experts and scholars at home and abroad have performed a lot of theoretical and experimental studies on placement and parametric optimization of structures with viscous dampers and highly effective results were obtained, which established favorable foundation for engineering applications of dampers in all kinds of structures. Gürgöze and Müller (1992) presented a numerical method of finding the optimal placement and the optimal damping coefficient for a single viscous damper in a prescribed linear multi-degree-of-freedom system. Hahn and Sathiavageeswaran (1992) performed several parametric studies on the effects of damper distribution on earthquake response of shear buildings, and showed that, for a building with uniform story stiffness, dampers should be added to the lower half floors of the building. Zhang and Soong (1992) used a sequential

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optimization procedure to determine the optimal location of viscoelastic dampers in multi-storey building structures. Gluck, *et al*. (1996) utilized the optimal solution for the linear quadratic regulator problem to obtain the optimal damping matrix, which was then used to determine the damper coefficients in different storeys. Takewaki (1997) and Takewaki, *et al*. (1997) used a gradient-based approach to search for the optimal solution that would minimize a desired system transfer function. Recently, Singh and Moreschi (2001) also used a gradient-based approach to obtain the optimal distribution of classical viscous dampers for a continuous description of the performance function. Tsuji and Nakamura (1996) proposed an algorithm to find both the optimal story stiffness distribution and the optimal damper distribution for a shear building model subjected to a set of spectrum-compatible earthquakes. De Silva (1981) presented a gradient algorithm for the optimal design of discrete passive dampers in the vibration control of a class of flexible systems. The algorithm are aimed at making the modal damping and natural frequencies of the system reach the preassigned values. Constantinou and Tadjbakhsh (1983) derived the optimum damping coefficient for a damper placed on the first storey of a shear building subjected to horizontal random earthquake motions. Trombetti and Silvestri (2002, 2004, 2007) studied analytical formulation and efficiency of MPD system in shear-type structures with viscous dampers. Inaudi and Kelly (1993) proposed a procedure for finding the optimal isolation damping for minimum acceleration response of base-isolated structures subjected to stationary random excitation. Ou (1998) studied parameter influence of passive energy dissipators, viscous, viscoelastic, metallic and frictional dampers in series with bracing members on suppressive effectiveness of structural vibration under earthquake and suggested the rational range of parameters for the design of passive energy dissipation systems. Zhou and Xu (1998) proposed five different optimum design methods for installation of dampers in structure and presented an example about optimum control analysis of a ten-floor reinforced concrete. Xu and Zhou (1999) proposed the optimum design regarding the relative displacement as the control function and performed the seismic response analysis of the viscoelastic structure using the time history analysis method and the mode superposition method.

Most of the researches above aimed at optimization analysis of structure with dampers under earthquake action but not under wind-induced vibration action. However, with the rapid developments of new techniques, new materials and new structural systems, more and more high-rise buildings and super skyscrapers are built recently. The higher the structure is, the more flexible it becomes, and then wind load may become the controlled load in structural design of high-rise buildings. Therefore, it's much important and necessary for engineers and researchers to perform studies on structural wind-induced vibration, especially on damping control, because it is one of the most safely and effective methods to reduce structural dynamic response induced by wind and researches on structural wind-induced vibration optimum using both GA theory and Rayleigh damping theory are very few at present. The authors of this paper (2008) have studied wind-induced vibration control of high-rise building based on GA method and the results showed that GA method was feasible to optimize structural wind-induced responses. But the above studies are not limited and further studies on control optimization of structural wind-induced vibration are urgent needed.

In the present paper some key problems about wind-induced vibration for high-rise buildings are studied, such as optimal method and vibration comfort. Firstly, two kinds of control optimal technologies, genetic algorithms method and Rayleigh damping method, are introduced to optimize installation of dampers to obtain optimized damping effect. Secondly, based on spectrum analysis method of structural wind-induced vibration, six target functions are proposed to control the feedback condition of GA. And then calculation procedures are designed by Matlab software to obtain results corresponding to different analysis cases and to perform comparison analysis among cases. Finally, a twenty-storey high-rise steel building is taken as an example to validate rationality of the presented methods. Results of this paper show that GA method is able to distribute damping coefficient along structural height rationally based on structural dynamic characteristics. The proposed methods are very helpful of decreasing wind-induced vibration response and improving structural comfort. Systems of MPD and GAMPD have remarkable effect on reducing structural vibration induced by wind, which almost reaches about 71.9% and 76.1% respectively.

2. Wind-induced vibration effect analysis of structures

The governing equation of motion for the building with multi-degree-of-freedom subjected to wind loads is expressed as:

$$M\ddot{u} + C\dot{u} + Ku = F(t) \tag{1}$$

Where M, C and K are $n \times n$ mass, damping and stiffness matrix of the structure; $\ddot{u}$, $\dot{u}$ and u are $n \times 1$ acceleration vector, velocity vector and displacement vector respectively; $F(t)$ is $n \times 1$ force vector of wind loads.

High-rise buildings are recognized as flexible systems with low damping ratio. The off-diagonal parts of the structural damping matrix are usually assumed to be neglected and then the structural damping matrix is considered to be satisfying orthogonality condition. So, N independent equations with single-degree-of-freedom can be obtained from Eq. (1):

$$\ddot{q}_i + 2\xi_i\lambda_i\dot{q}_i + \lambda_i^2\dot{q}_i = F_i^*(t) \ (i = 1, 2, \cdots, N) \tag{2}$$

Where $F_i^*(t)$ is generalized wind-force of i-th mode; λ_i is base frequency of i-th mode; ξ_i is damping ratio of i-th mode.

The analysis result of recording wind time-series show that if the initial non-stationary zone is ignored, fluctuating wind is very approximate with stationary random process, the probability distribution of each sample is also approximately equal and its probability density can be expressed by Gaussian distribution, and then Fluctuating wind is always considered by Gauss process (Zhao and Xie 2006, Qu 1991). So, in this paper, fluctuating wind force is assumed to be zero-mean Gaussian stationary stochastic process in this paper. So, generalized fluctuating wind force also has the same characteristic with fluctuating wind force. Then, according to the knowledge of stochastic vibration theory, cross correlation function of generalized fluctuating wind forces can be expressed as:

$$R_{F_i^*F_j^*}(\tau) = E[F_i^*(t)F_j^*(t+\tau)] = \frac{1}{M_i^*M_j^*}\{\varphi\}_i^T[R_{\{p\}}(\tau)]\{\varphi\}_j \tag{3}$$

Where $[R_{\{p\}}(\tau)] = E[F(t)F(t+\tau)^T]$ is cross correlation function matrix of fluctuating wind force F.

According to the relationships between cross power-spectrum function and cross correlation function, power-spectrum density function of fluctuating wind force vector and cross correlation function matrix, the power-spectrum density function of fluctuating wind-load can be obtained:

$$S_{F_i^*F_j^*}(\omega) = \frac{1}{M_i^*M_j^*}\{\varphi\}_i^T[S_F]\{\varphi\}_j S_f(\omega) \tag{4}$$

Where $[S_F]$ and $S_f(\omega)$ are coefficient matrix of power-spectrum density function for fluctuating wind-load vector and normal fluctuating wind-speed power-spectrum separately, which can be defined as (Davenport power spectrum is selected as fluctuating wind-speed spectrum in this paper):

$$\begin{cases} (S_F) = \gamma_{ij} P_i P_j \quad (i, j = 1, 2, \cdots, N) \\ S_f(\omega) = \dfrac{4\pi x^2}{3\omega(1+x^2)^{4/3}} \quad \left(x = \dfrac{1200n}{\bar{v}_{10}}\right) \end{cases} \tag{5}$$

Where P_i is wind force of i-th point; ω is frequency of fluctuating wind-load and $\omega = 2\pi n$; $\bar{v}_{10}$ is the mean wind speed at 10m height; γ_{ij} is cross correlation function between i-th point and j-th point.

From Eqs. (2) equation (4), self power-spectrum density function of general coordinate $q_i(t)$ can be obtained:

$$S_{q_i}(\omega) = |H_i(\omega)|^2 S_{F_i^*}(\omega) \tag{6}$$

Where $S_{F_i^*}(\omega)$ is self power-spectrum density function of general fluctuating wind-load, which can be obtained easily by Eq. (3) when $i = j$; $H(\omega)$ is transform function.

So, the self power-spectrum density function of structural storey-displacement response can then be obtained:

$$S_{u_i}(\omega) = \sum_{j=1}^{N} (\varphi_{ij})^2 S_{q_j}(\omega) = \sum_{j=1}^{N} (\varphi_{ij})^2 |H_j(\omega)|^2 S_{F_i^*}(\omega) = \sum_{j=1}^{N} (\varphi_{ij})^2 \Lambda_j |H_j(\omega)|^2 S_f(\omega) \tag{7}$$

Where φ_{ij} is i-th element of j-th mode; $\Lambda_j = \dfrac{1}{(M_j^*)^2}\{\varphi\}_j^T [S_F]\{\varphi\}_j$.

The self power-spectrum density function of story-acceleration response and story-velocity response can be deduced from Eq. (7):

$$\begin{cases} S_{\dot{u}_i}(\omega) = \omega^2 S_{u_i}(\omega) \\ S_{\ddot{u}_i}(\omega) = \omega^4 S_{u_i}(\omega) \end{cases} \tag{8}$$

Finally, structural response variances of displacement, velocity and acceleration can be achieved according to its relationship with self power-spectrum density function:

$$\begin{aligned} \sigma_{u_i}^2 &= \int_{-\infty}^{\infty} S_{u_i}(\omega) d\omega = \sum_{j=1}^{N} (\varphi_{ij})^2 \Lambda_j \int_{-\infty}^{\infty} |H_j(\omega)|^2 S_f(\omega) d\omega \\ \sigma_{\dot{u}_i}^2 &= \int_{-\infty}^{\infty} \omega^2 S_{u_i}(\omega) d\omega = \sum_{j=1}^{N} (\varphi_{ij})^2 \Lambda_j \int_{-\infty}^{\infty} \omega^2 |H_j(\omega)|^2 S_f(\omega) d\omega \\ \sigma_{\ddot{u}_i}^2 &= \int_{-\infty}^{\infty} \omega^4 S_{u_i}(\omega) d\omega = \sum_{j=1}^{N} (\varphi_{ij})^2 \Lambda_j \int_{-\infty}^{\infty} \omega^4 |H_j(\omega)|^2 S_f(\omega) d\omega \end{aligned} \tag{9}$$

3. Work principle and computing mode of viscous damper

Oil dampers are mainly made up of viscous fluid, piston, cylinder tube, as shown in Fig. 2. Piston will be forced to generate reciprocating motion in cylinder tube under earthquake action or strong wind action. Then viscous fluid in viscous dampers under pressure force will be flowing with a high speed through damping holes or gaps from one side to the other side. In this process, two kinds of forces are caused: one is damping force (the most important force for dampers), which is generated by viscosity of viscous fluid and has capacity of dissipating energy greatly induced by earthquake or wind. The other is spring force (very little and be ignored usually for viscous dampers relative to damping force), which is generated by compressibility of viscous fluid and embodies stiffness characteristic of oil dampers in the process of working.

Because of the spring force of viscous dampers are very little and can be ignored, the damping force of viscous dampers can be defined as:

$$F = Cv^{\alpha} \tag{10}$$

Where F, C, α, v are damping force, damping efficient, velocity factor and motion velocity of viscous dampers separately.

The relationship curve of force-velocity of viscous dampers is shown in Fig. 2. Apparently, force-velocity curve of dampers is linear and dampers have characteristic of linear relation between damping force and motion velocity when $\alpha = 1$. Force-velocity curve is nonlinear and dampers have characteristic that damping force appears very bigger relative to less motion velocity but arise little with increase of the motion velocity when $\alpha < 1$. Force-velocity curve is also nonlinear but dampers have opposite characteristic with case $\alpha < 1$, which have less damping force relative to less motion velocity but arise promptly with increase of the motion velocity when $\alpha < 1$.

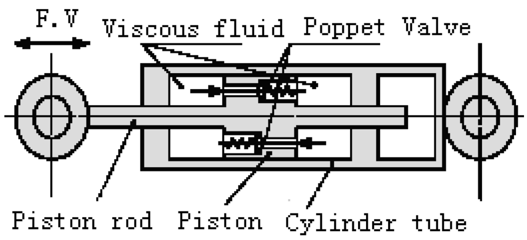

Fig. 2 Structure figure of oil damper

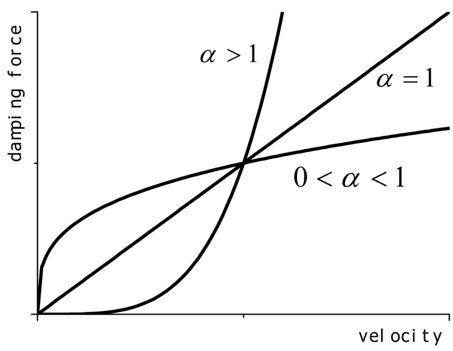

Fig. 2 Force-velocity curve of viscous damper

4. Base theory of Genetic Algorithms (GA)

4.1. Basic principle

Genetic algorithms (GA) presume that the potential solution of any problem is an individual and can be represented by a set of parameters. These parameters are regarded as the genes of a chromosome and can be structured by a string of values in binary form. A positive value, generally known as a fitness value, is used to reflect the degree of “goodness” of the chromosome for the problem that would be highly related to its objective value. Genetic algorithms are heuristic random search techniques based on the concept of natural selection and natural genetics of population, and so they are of “population-based” method of searching large combinatorial design spaces to find the optimum combination of design variables. Detailed discussion on the mechanisms of GA can be found in Holland (1975) and Goldberg (1992). GA has been proven to be a versatile and effective approach of solving optimization problems and have been used in many research fields. The successful application of genetic algorithms to both combinatorial and discrete optimization problems (Koumousis and Georgiou 1994) motivated the employment of GA to solve mixed-discrete non-linear optimization problems (Jenkins 1997, Lin, *et al*. 1995). Furuya (1998) applied the genetic algorithm to obtain the proper placement of the passive dampers in each storey of a building.

The basic steps of Genetic algorithm can be expressed as follows and also be shown in Fig. 3.

1. Some cells with problem information will be created by stochastic manner, and each cell is made up of one kinds of code (such as binary encoder, Gray encoder and so on);
2. The sufficiency of target function for each cell in grope will be appraised. Higher sufficiency cells can be reserved and washout lower cells;
3. New daughter lines can be created by there kinds of methods, namely ① father cells creates daughter cells by cross method, ② father cells creates daughter cells by mutation method, ③ excellent father cells are copied and form daughter cells;
4. The new created gropes perform next genetic optimum operation, and then cycle operation for step 2 and step 3 until optimization result arrives predefined target.

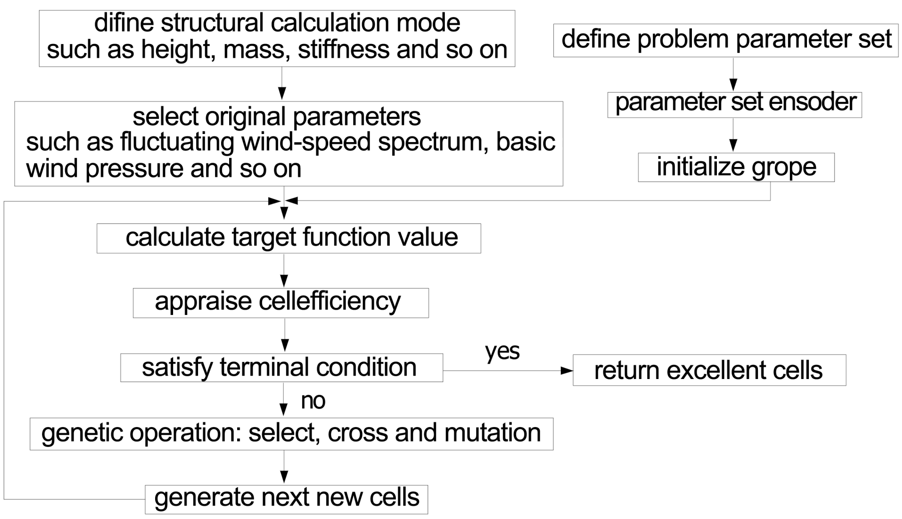

Fig. 3 Flow chart of GA

4.2. Target functions

Target functions are employed by Genetic algorithms to decide selection, cross and mutation characteristic of creating new daughter cells. Apparently, the better or inferior results obtained by GA optimum are influenced by target functions greatly. So, the rational selection of target functions is very important for Genetic algorithms to obtain the best excellent cells.

In this paper, in order to perform comparison analysis, select preferable target functions and then obtain some valuable conclusions to structural optimum design of wind-induced vibration control, six kinds of target functions are present according to the deduction of section 1.

4.2.1. Target function T1

Taking structural top-acceleration response variance of the first mode as target function $T1$, which can be expressed as:

$$T1 = \sigma_{\ddot{u}_N} = (\varphi_{N1})\sqrt{\Lambda_1 \int_{-\infty}^{\infty} \omega^4 |H_1(\omega)|^2 S_f(\omega) d\omega} \tag{11}$$

4.2.2. Target function T2

Taking the mean value of inter-story displacement angle variance of the first mode as target function $T2$:

$$T2 = \frac{1}{N}\sum_{i=1}^{N} \frac{\sigma_{u_i}}{h_i} = \frac{1}{N}\sum_{i=1}^{N} \frac{1}{h_i}(\varphi_{i1})\sqrt{\Lambda_1 \int_{-\infty}^{\infty} |H_{\Delta 1}(\omega)|^2 S_f(\omega) d\omega} \tag{12}$$

$$\text{Where} \begin{cases} H_\Delta(\omega) = T \cdot H(\omega) \\ T = \begin{bmatrix} 1 & 0 & & & & 0 \\ -1 & 1 & 0 & & & \\ 0 & -1 & 1 & 0 & \cdots & \\ & & & \ddots & & \\ & & & & & 0 \\ 0 & & & 0 & -1 & 1 \end{bmatrix} \end{cases}$$

4.2.3. Target function T3

Taking structural top-acceleration response variance of the first ten modes as target function $T3$:

$$T3(1) = \sigma_{\ddot{u}_N} = \sqrt{\sum_{j=1}^{10} (\varphi_{Nj})^2 \Lambda_j \int_{-\infty}^{\infty} \omega^4 |H_j(\omega)|^2 S_f(\omega) d\omega} \tag{13}$$

4.2.4. Target function T4

Taking mean value of inter-story displacement angle variance of the first ten modes as target function $T4$:

$$T4 = \frac{1}{N}\sum_{i=1}^{N}\frac{\sigma_{u_i}}{h_i} = \frac{1}{N}\sum_{i=1}^{N}\frac{1}{h_i}\sqrt{\sum_{j=1}^{10}(\varphi_{Nj})^2\Lambda_j\int_{-\infty}^{\infty}|H_{\Delta j}(\omega)|^2 S_f(\omega)d\omega} \tag{14}$$

4.2.5. Target function T5

Taking both structural top-acceleration response variance and mean value of inter-story displacement angle variance of the first ten mode as Target function $T5$:

$$T5 = T1 \times T2 \tag{15}$$

4.2.6. Target function T6

Taking base-shear variance of the first mode as target function $T6$:

$$T6 = k_1\sigma_{u,1} = k_1\varphi_{11}\sqrt{\Lambda_1\int_{-\infty}^{\infty}|H_1(\omega)|^2 S_f(\omega)(d\omega)} \tag{16}$$

5. Rayleigh damping optimum theory

In the dynamic systems considered in this paper internal damping is neglected, so that the damping matrix derives from the effects of added viscous dampers only. Considering a system added viscous dampers which leads, for the generic N-storey linear elastic structure (show as Fig. 4), to a Rayleigh damping matrix. The Rayleigh damping matrix has the following expression:

$$C^R = \alpha M + \beta K \tag{17}$$

Where M and K are structural mass- matrix and stiffness-matrix; α and β are constant and with units of S^{-1} and S separately.

Two following damping matrices can be defined according to Eq. (17) (Trombetti and Silvestri 2004, 2007, Trombetti, *et al*. 2002):

1. If the damping matrix of dampers is proportional to the storey mass matrix, this system is called "MPD system" (show as Fig. 5a) and its damping matrix can be defined:

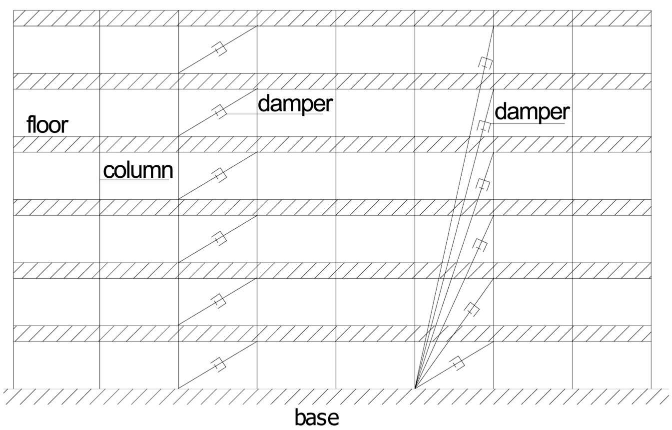

Fig. 4 Six-storey shear-type structure equipped with Rayleigh damping system

$$C^{MPD} = \alpha M \tag{18}$$

(2) If the damping matrix of dampers is proportional to the lateral stiffness matrix, this system is called “SPD system” (show as Fig. 5b) and its damping matrix can be defined:

$$C^{SPD} = \beta M \tag{19}$$

The installation ways of dampers are different for MPD and SPD systems, which can be easily seen from Fig. 5. Dampers are installed between one story and a fixed point for MPD system and dampers are installed between two adjacent stories for SPD system. So, the two damping systems can be defined as follows:

MPD system: dampers are placed in such a way as to connect each storey to a fixed point and the damping coefficient matrix is proportional to the corresponding storey mass matrix;

SPD system: dampers are placed in such a way as to connect two adjacent storeys and the damping coefficient matrix is proportional to the corresponding lateral stiffness matrix.

In order to make meaningful comparisons of the dissipative performances offered by different

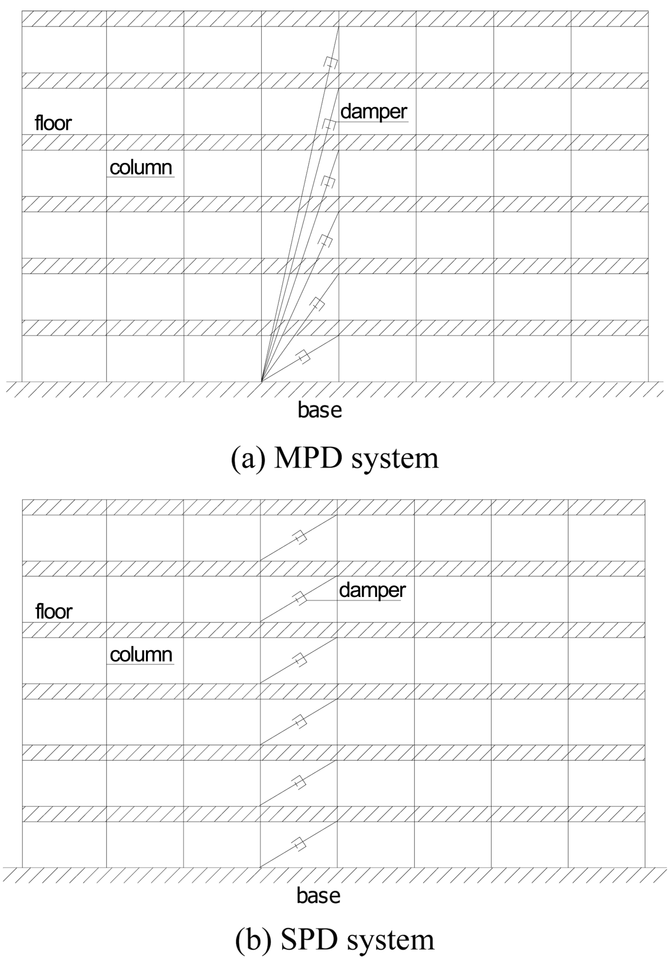

Fig. 5 Sketch figures of MPD and SPD systems

damper systems, it is necessary to introduce a constraint upon their total size. The total size, $\hat{c}$, of a generic damper system made up of N added viscous dampers is defined herein as the sum of the damping coefficients of all N viscous dampers, as expressed by:

$$\hat{c} = \alpha\sum_{j=1}^{N} m_j + \beta\sum_{j=1}^{N} k_j \tag{20}$$

When various damper systems are compared, the equal "total size" constraint implies that all systems must have the same value $\hat{c}$.

Substituting Eq. (20) into Eq. (17) gives Rayleigh damping matrix:

$$C = \bar{\alpha}(1-\gamma)M + \bar{\beta}\gamma K \tag{21}$$

Where $\bar{\alpha} = \hat{c}/\sum_{j=1}^{N} m_j$, $\bar{\beta} = \hat{c}/\sum_{j=1}^{N} k_j$, γ is constant without units and has span of [0,1]. Note that $\gamma = 0$ identifies the MPD system, whilst $\gamma = 1$ identifies the SPD system.

6. Example analysis

A 20-storey high-rise steel building is taken as an example in this section. The basic parameters of the building are given in table 1 and the first ten periods and frequencies of the building are shown in Table 2.

Some first-phase preparations are here presented in order to perform further optimum analysis. Firstly, Linear viscous dampers are installed in the example building and the whole damping coefficients $\hat{c}$ of all viscous dampers in different cases are defined to be equal, $\hat{c} = 1.02\times10^{9} N\cdot s/m$, so as to analysis structural dynamic performance comparatively. Secondly, some original parameters,

Table 1 Basic parameters of the building

Height of the building (m)	72
Area along wind (m^2)	2356
Story-mass (kg)	$m_1 = m_2 = \cdots m_{20} = 3.2\times10^4$
Story-stiffness (N/m)	$k_1 = \cdots k_4 = 3.5\times10^7$ $k_5 = \cdots k_{10} = 3.0\times10^7$ $k_{11} = \cdots k_{16} = 2.5\times10^7$ $k_{17} = \cdots k_{20} = 2.2\times10^7$
Field type	D
Basic wind pressure (kN/m^2)	1.0
Basic wind speed (m/s)	40.0

Table 2 The first ten periods and frequencies of the building

	Mode									
	1	2	3	4	5	6	7	8	9	10
T_i/s	2.664	0.935	0.564	0.409	0.319	0.263	0.226	0.198	0.178	0.162
λ_i/rad/s	2.358	6.718	11.137	15.345	19.686	23.868	27.785	31.734	35.265	38.782

Table 3 Original parameters of GA

Parameters	Cell number	Genetic generations	Variable dimension	Variable binary number	Cross probability	Generation gap
Original value	40	200	20	20	0.7	0.9

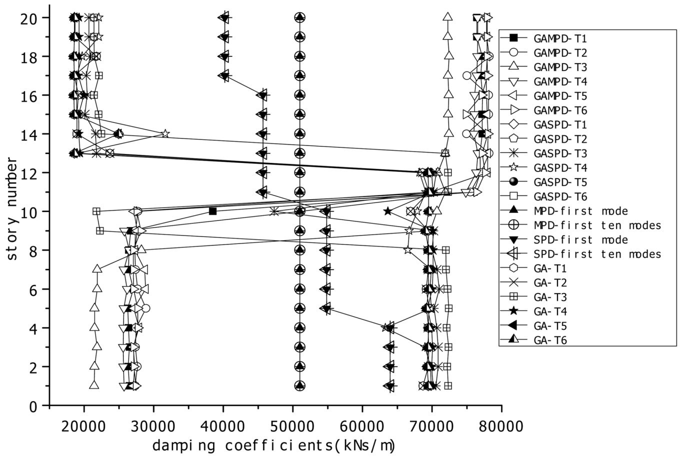

Fig. 6 Distributions of damping coefficients along height for different damping systems

which are very important and necessary for the optimum analysis of Genetic Algorithm, are presented in Table 3. Finally, according to the Genetic Algorithms method and Rayleigh damping method deduced in section 3 and section 4, Structural optimum analysis programs are developed based on Matlab software to calculate structural wind-induced responses.

Fig. 6 shows the distribution manners of damping coefficients along structural height for different cases ("GAMPD" indicates that analysis result given is optimized by both Genetic Algorithms method and Rayleigh damping method, "$T1$" indicates that target $T1$ is selected, "the first mode" indicates that optimum analysis only considers the first mode. Therefore, it's easy to know the means of other cases). This figure shows that the change tendency of damping coefficients for MPD system and SPD system without optimized is proportional to storey mass and story-stiffness respectively: The former keeps invariable along height and the latter presents stepladder-like change shape along structural height. However, the distribution manners of damping coefficients for all of the optimized cases change greatly and obviously. For example, the distribution of MPD system shows that damping coefficients at the lower part of the structure are less than that at the upper part of building. However, the distribution of SPD system shows that damping coefficients at the lower part of the structure are bigger than that at the upper part of structure.

Table 4 gives standard deviation of roof-acceleration response of the structure equipped with 22 damping systems considered. Fig. 7 shows standard deviation of storey acceleration for the structure equipped with 22 damping systems considered. Fig. 8 shows standard deviation of storey displacement for the structure equipped with 11 damping systems considered. Fig. 9 shows standard deviation of interstorey drift angle for the structure equipped with 11 damping systems considered. Following results can be obtained:

① Damping systems of MPD and GAMPD have the best performance of reducing structural wind-induced vibration responses and are superior to systems of SPD, GASPD and GA. Take the

Table 4 Standard deviation of roof responses of different damping systems(units: m/s^2)

Plans	GAMPD						GASPD					
Cases	*T*1	*T*2	*T*3	*T*4	*T*5	*T*6	*T*1	*T*2	*T*3	*T*4	*T*5	*T*6
$\sigma_{\ddot{u}}$	0.255	0.256	0.315	0.316	0.281	0.246	1.038	1.036	1.121	1.122	1.091	1.028
Average	0.278						1.073					
σ_u	77.92	77.98	94.03	93.8	93.5	77.94	189.6	189.3	199.4	199.6	199	189.5
Average	85.86						194.4					

Plans	GA						MPD		SPD	
Cases	*T*1	*T*2	*T*3	*T*4	*T*5	*T*6	First	Ten	First	Ten
$\sigma_{\ddot{u}}$	0.856	0.856	0.900	0.901	0.862	0.847	0.315	0.338	1.151	1.175
Average	0.87						0.327		1.163	
σ_u	165.6	165.6	165.9	165.8	166	165.6	115.2	115.5	210	210.1
Average	165.75						115.35		210.05	

Notes: "First" represents that the first mode is considered in this plan, "Ten" represents that the first ten modes are considered in this case.

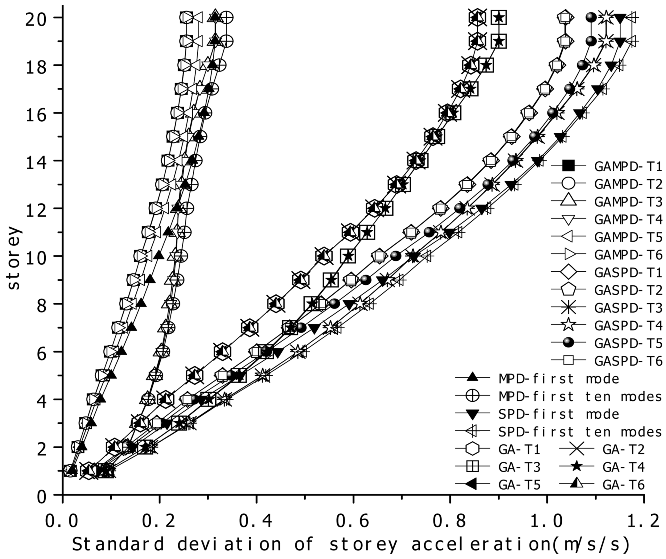

Fig. 7 Standard deviation of storey acceleration for the structure equipped with 22 damping systems considered

standard deviation of roof responses as an example, the average values of roof acceleration for damping systems of GAMPD, MPD, GA, GASPD and SPD are 0.278 m/s^2, 0.327 m/s^2, 0.87 m/s^2, 1.073 m/s^2, 1.163 m/s^2 respectively. Relative to SPD system, the damping ratios of GAMPD, MPD, GA and GASPD are 76.1%, 71.9%, 25.2% and 7.74%. The reason of that is that additional damping ratio of the first mode for MPD system is very greater than SPD system (Trombetti and Silvestri 2006), which is very helpful for reducing structural wind-induced responses.

② Energy dissipation effect of the structure with dampers optimized only by GA is better than systems of GASPD and SPD but worse than plans of GAMPD and MPD. GA theory has the

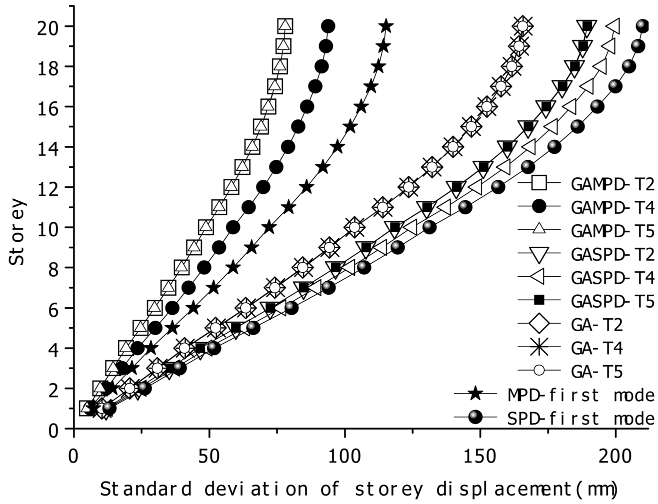

Fig. 8 Standard deviation of storey displacement for the structure equipped with 11 damping systems considered

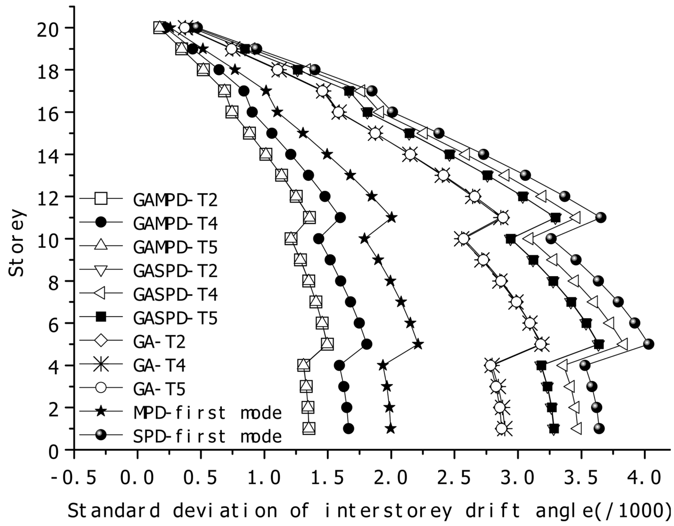

Fig. 9 Standard deviation of interstorey drift angle for the structure equipped with 11 damping systems considered

property of distributing damping coefficient according to the given structural dynamic response, and make the best use of energy dissipation performance of viscous dampers (Wang and Zhou 2008). The average values of displacement standard deviation for system GA, GAMPD and GASPD are 165.75 mm, 85.86 mm, 194.04 mm, which are 78.91%, 40.88% and 92.55% relative to the average value of SPD system, 210.05 mm.

③ Standard deviations of structural responses show different change tendency if varied target functions are selected, which can be seen easily from system GASPD, GASPD and GA. According to the vibration-absorbing effect, the best plan of a optimum system is the damping system equipped with target *T*6, and then target *T*1, target *T*2 and target *T*5 sequentially.

④ Standard deviations of structural responses become increase slightly when considering influence of higher modes (such as target $T3$ and target $T4$). The standard deviations of roof acceleration and displacement are 1.038 m/s^2 and 189.6 mm for damping system GASPD with target function $T1$, and 1.121 m/s^2 and 199.4 mm for the same system with target function $T3$. Of course, similar patterns can be seen in others systems.

Fig. 10 shows distribution of damping force for the structure equipped with five damping systems considered. Fig. 11 shows distribution of total damping force for the structure equipped with 22 damping systems considered. Results can be obtained from the two figures. Firstly, the distribution of damping forces throughout the structural height is the opposite for the MPD and SPD systems: the MPD system transmits the largest dissipative force at the top of the structure, whilst the SPD system transmits the largest dissipative force at the bottom of the structure. Secondly, the total damping forces of all damping systems are almost the same: it is 6932kN for the "GAMPD-$T1$" system, 7530kN for the "GASPD-$T6$" system, 7971kN for the "SPD-first mode" system, 6790kN for the "GA-$T4$" system and et. al. Finally, dissipative effect of damping systems equipped with MPD (such as the MPD system or GAMPD system) is better than that equipped with SPD (such as the SPD system or GASPD system), the reason of that is not due to the formation of larger damping force but due to the natural characteristics of the damping systems.

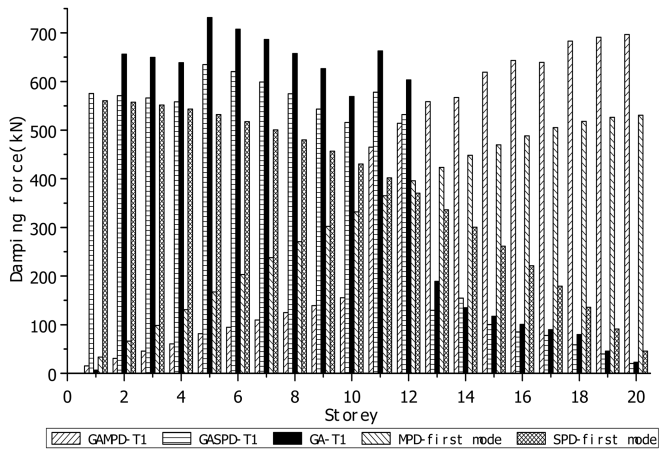

Fig. 10 Distribution of damping force for the structure equipped with five damping systems considered

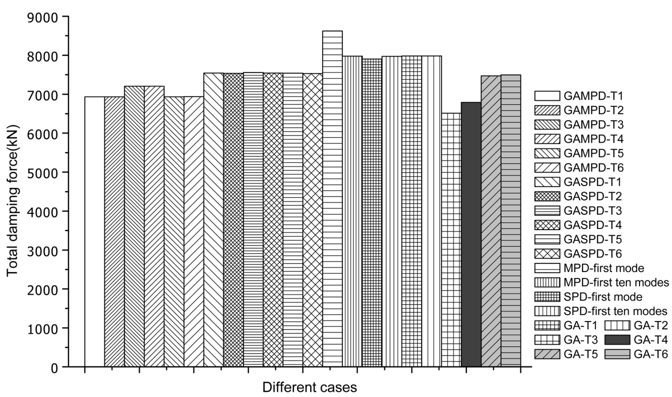

Fig. 11 Distribution of total damping force for the structure equipped with 22 damping systems considered

7. Conclusions

In this paper, optimum methods of wind-induced vibration control of high-rise buildings are mainly studied. Two optimum methods, genetic algorithms method and Rayleigh damping method, are firstly employed and proposed to perform optimum study on wind-induced vibration control with the help of structural frequency-domain analysis. Six target functions are presented in genetic algorithms and structural optimum analysis procedures are relevantly designed. A high-rise building with 20-storey is adopted and 22 kinds of control plans are employed to perform comparison analysis to validate the feasibility and validity of the optimum methods proposed. Some conclusions are summarized as follows:

1. The distributions of damping coefficients for MPD systems and SPD systems are entirely opposite: the damping coefficients of MPD systems at the upper part of the structure are bigger than that of SPD systems, whilst the damping coefficients of SPD systems at the lower part of the structure are bigger than that of MPD systems.
2. Damping systems of MPD and GAMPD have the best performance of reducing structural wind-induced vibration response and are superior to other damping systems.
3. GA optimum method can distribute damping coefficients along structural height rationally according to the specific dynamic properties. The energy dissipation effect of the structure optimized only by GA is better than plans of GASPD and SPD but worse than plans of GAMPD and MPD.
4. Standard deviations of structural responses show different change tendency if varied target functions are selected. The best plan of a optimum system is the damping system equipped with target $T6$, and then target $T1$, target $T2$ and target $T5$ sequentially.
5. Standard deviations of structural responses become increasing slightly when considering the influence of higher modes. The influence of higher modes should be considered when strict requirement of wind-induced vibration comfort is needed for some special structures.

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References

Constantinou, M.C. and Tadjbakhsh, I.G. (1983), "Optimum design of a first story damping system", *Comput. Struct*. **17**, 305-310.

De Silva, C.W. (1981), "An algorithm for the optimal design of passive vibration controllers for flexible systems", *J. Sound Vib.*, **75**(4), 495-502.

Furuya, O., Hamazaki, H. and Fujita, S. (1998), "Proper placement of energy absorbing devices for reduction of wind-induce vibration caused in high-rise buildings", *J. Wind Eng. Ind. Aerodyn*., **74-76**, 931-992.

Gluck, N., Reinhorn, A.M., Gluck, J. and Levy, R. (1996), "Design of supplemental dampers for control for structures", *J. Struct. Eng.*, **122**(12), 1394-1399.

Gurgoze, M. and Muller, P.C. (1992), "Optimal positioning of dampers in multi-body systems", *J. Sound Vib.*, **158**(3), 517-530.

Hahn, G.D. and Sathiavageeswaran, K.R. (1992), "Effects of added-damper distribution on the seismic response of buildings", *Comput. Struct.*, **43**(5), 941-950.

Holland, J.H. (1975), *Adaptation in Natural and Artificial Systems,* University of Michigan Press, Ann Arbor, MI.

Holland, J.H. (1992), *Genetic Algorithms*, Scientific American.

Inaudi, J.A. and Kelly, J.M. (1993), "Optimum damping in linear isolation systems", *Earthq. Eng. Struct. Dyn.*, **22**, 583-598.

Jenkins, W.M. (1997), "On the application of natural algorithms to structural design optimization", *Eng. Struct*. **19**(4), 302-308.

Koumousis, V.K. and Georgiou, P.G. (1994), "Genetic algorithms in discrete optimization of steel truss roofs", *J. Comput. Civ. Eng*., ASCE, **8**(3), 309-325.

Lin, S.S, Zhan, g.C. and Wang, H.P. (1995), "On mixed-discrete nonlinear optimization problems: a comparative study", *Eng. Optim*. **23**, 287-300.

OU, J.P., Wu, B. and Long, X. (1998), "Parameter analysis of passive energy dissipation systems", *Earthq. Eng. Eng. Vib.*, **18**(1), 60-70.

Qu W. L. (1991), *Wind-Induced Vibration Control Design of High-Rise and Super High-Rise Buildings*, Pressed by Survey Science and Technology University of Wu Han.

Singh, M.P. and Moreschi, L.M. (2001), "Optimal seismic response control with dampers", *Earthq. Eng. Struct. Dyn.*, **30**, 553-572.

Takewaki, I. (1997), "Optimal damper placement for minimum transfer functions", *Earthq. Eng. Struct. Dyn.*, **26**, 1113-1124.

Takewaki, I. (1998), "Optimal damper positioning in beams for minimum dynamic compliance", *Comput. Methods Appl. Mech. Eng.*, **156**, 363-373.

Trombetti T. and Silvestri S. (2007), "Novel schemes for inserting seismic dampers in shear-typesystems based upon the mass proportional component of the Rayleigh damping matrix", *J. Sound Vib.*, **302**(3), 486-526.

Trombetti, T. and Silvestri, S. (2004), "Added viscous dampers in shear-type structures: the effectiveness of mass proportional damping", *J. Earthq. Eng.*, **8**(2), 275-313.

Trombetti, T. and Silvestri, S. (2006), "On the model damping ratios of shear-type structures equipped with Rayleigh damping systems", *J. Sound Vib.*, **292**, 21-58.

Trombetti, T., Silvestri, S. and Ceccoli, C. (2002), "Inserting viscous dampers in shear-type structures: analytical formulation and efficiency of MPD System", *Proceedings of the Second International Conference on Advances in Structural Engineering and Mechanics (ASEM'0 2),* Pusan, Korea, 21-23 August 2002.

Trombetti, T., Silvestri, S. and Ceccoli, C. (2002), "On the first modal damping ratios of MPD and SPD systems", Technical Report: Nota Tecnica nº 64, 2002, Department DISTART, University of Bologna, Italy.

Tsuji, M. and Nakamura, T. (1996), "Optimum viscous dampers for stiffness design of shear buildings", *The Structural Design of Tall Buildings*, **5**, 217-234.

Wang, D.Y. and Zhou, Y. (2008), "Wind-induced vibration control research of high-rise buildings based on GA optimum theory", *China Civil Engineering J.*, **41**(1), 42-47.

Xu, Z.D., Zhou, Y. and Zhao, H.T. (1999), "Optimum design method of the viscoelastic structure", *J. Xi'an University of Architecture and Technology*, **31**(3), 246-252.

Zhang, R.H. and Soong, T.T. (1992), "Seismic design of viscoelastic dampers for structural applications", *J. Struct. Eng. ASCE*, **118**(5), 1375-1392.

Zhao Jianfei, and Xie Buying. (2006), "The simulation of stochastic wind loading on long-span bridges and development of the program", *Structural Engineers*, **22**(2), 42-49.

Zhou, Y., Xu, Z.D. and Deng, X.S. (1998), "Optimum installation of the dampers in the viscoelastic structures", *World Information on Earthquake Engineering*, **14**(3), 15-20.

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