Steel and Composite Structures, *Vol. 19, No. 2 (2015) 349-368*
DOI: http://dx.doi.org/10.12989/scs.2015.19.2.349

Optimal placement of elastic steel diagonal braces using artificial bee colony algorithm

E. Aydin [1], M. Sonmez [*2] and T. Karabork [2]

[1] *Department of Civil Engineering, Nigde University, 51240 Nigde, Turkey*
[2] *Department of Civil Engineering, Aksaray University, 68100 Aksaray, Turkey*

(*Received February 13, 2014, Revised November 21, 2014, Accepted January 13, 2015*)

Abstract. This paper presents a new algorithm to find the optimal distribution of steel diagonal braces (*SDB*) using artificial bee colony optimization technique. The four different objective functions are employed based on the transfer function amplitude of; the top displacement, the top absolute acceleration, the base shear and the base moment. The stiffness parameter of *SDB* at each floor level is taken into account as design variables and the sum of the stiffness parameter of the *SDB* is accepted as an active constraint. An optimization algorithm based on the Artificial Bee Colony (ABC) algorithm is proposed to minimize the objective functions. The proposed ABC algorithm is applied to determine the optimal *SDB* distribution for planar buildings in order to rehabilitate existing planar steel buildings or to design new steel buildings. Three planar building models are chosen as numerical examples to demonstrate the validity of the proposed method. The optimal *SDB* designs are compared with a uniform *SDB* design that uniformly distributes the total stiffness across the structure. The results of the analysis clearly show that each optimal *SDB* placement, which is determined based on different performance objectives, performs well for its own design aim.

Keywords: steel diagonal brace; artificial bee colony algorithm; structural optimization; transfer functions

1. Introduction

Many buildings have not performed well in recent earthquakes due to insufficient earthquake resistance. Many techniques have been developed to increase the earthquake resistance of these structures including; steel diagonal braces, shear walls and various structural control systems. Considering the ease of construction and the relatively low cost (Colunga and Vergara 1997) steel bracing appears to be an attractive alternative to the other shear resisting elements. Braced frames have a higher lateral stiffness when compared with moment resisting frames. Braced frames are of two types; eccentric and concentric bracing configurations. In eccentrically braced frames, the energy is dissipated by the eccentric brace elements, whereas the other structural elements respond elastically during a severe earthquake. In concentrically braced frames, the structural members with centrelines intersecting at a joint form a vertical truss system. The members of a concentrically braced frame carry only axial loads in the elastic range. The diagonal bracing

*Corresponding author, Associate Professor, E-mail: mustafasonmez@aksaray.edu.tr

http://www.techno-press.org/?journal=scs&subpage=8 ISSN: 1229-9367 (Print), 1598-6233 (Online)

members also respond elastically during a destructive earthquake.

In some cases, steel elements were commonly used to upgrade the seismic capacity of structures. These steel braces may be added either to new constructions or to existing building structures for rehabilitation. There are four types of application of steel systems to upgrade the seismic response of old structures: *SDB*s are added into the frame opening (Mitchell and Dandurand 1988), steel frames and trusses are placed on the building exterior (Turker and Bayraktar 2011), steel plate shear walls are inserted into the structure (Yamamato and Aoyama 1987) and pre-stressed steel cables are placed in the frame holes (Miranda 1991). Several construction details have been investigated to add *SDB*s to an existing reinforced concrete structure (Kawamata and Masaki 1980, Gorgulu *et al*. 2012). The use of an X steel braced system drastically increased the strength and stiffness of the reinforced concrete portal frame (Gorgulu *et al*. 2012). Bartera and Giacchetti (2003) presented an experimental investigation of the dynamic response of existing single storey reinforced concrete frame using the various types of steel bracing system. Analytical research has been conducted by several researchers (Downs *et al.* 1991, Valle *et al.* 1988, Valle 1980, Maheri and Sahebi 1997, Sarnoa and Elnashaib 2009, Ghobarah and Elfath 2001) in order to improve the dynamic response of structures. Maheri and Sahebi (1997) determined the effects of different *SDB* placements to increase the plane shear strength of the concrete frame. The advantage of eccentric steel braces in an existing building was presented by Ghobarah and Elfath (2001). A comparative study of an existing retrofit for a mid-rise steel building in downtown Mexico City using additional stiff steel braced-frames against an alternate retrofit using Adding Damping And Stiffness (ADAS) passive energy dissipation devices was presented by Colunga and Vergara (1997). Kim and Choi (2004) investigated a design procedure to increase the energy dissipation capacity of steel structures with buckling restrained *SDB*.

Hysteretic dampers such as buckling-restrained braces were widely used to protect structures against detrimental effects of earthquakes. For conventional design procedures the properties of hysteretic dampers have been determined by trial and error approaches (Choi and Kim 2005, Usami *et al.* 2005, Kanaji *et al*. 2003). The location and properties of the braces should be found in order to design the upgrading structures. The optimal placement of the added *SDB*s indicates the stiffness optimization. Some performance functions, such as the top displacement (Takewaki 2000), the base shear (Aydin *et al*. 2007), the top absolute acceleration (Takewaki 1999, Cimellaro 2007) and the base moment (Wang 2006) can be selected in order to determine the optimal structural design. The optimization techniques used in *SDB* placement problems can be classified as gradient-based techniques such as the steepest direction search algorithm (Aydin and Boduroglu 2008) and nongradient-based (direct search) techniques such as genetic algorithm (Farhad *et al*. 2009).

In the three last decades, several direct search techniques based on the models of social interaction among insects (e.g., bees, termites, and wasps) have been used because they have the capability to produce useful and very powerful search mechanisms to solve optimization problems. These insects, even with the very limited individual capability, can cooperatively perform many complex tasks that are necessary for their survival. Each member of the colony performs their tasks by interacting or communicating in a direct or indirect manner in their local environment (Frisch 1967). This intelligent behaviour that inspires scientists to develop new optimization algorithms mimicking behaviour of insects is generally called swarm intelligence. Swarm intelligence based algorithms, including particle swarm optimization (Lee and Geem 2004), ant colony optimization (ACO) algorithms (Kennedy *et al*. 2001) and bee based algorithms (Karaboga and Akay 2009, Karaboga and Basturk 2008, Karaboga 2005, Pham *et al*. 2006, Teodorovic 2009).

There are several bee-based optimization algorithms including bee colony optimization algorithm, virtual bee algorithm, bee algorithm and artificial bee colony algorithm. Recent studies presented by Karaboga and Akay (2009), Bansal *et al*. (2013) and Karaboga *et al*. (2014) show the ABC is the most widely used optimization algorithm among them so far. The three main advantages of the ABC optimization algorithm are: easy to implement, less control parameters and robustness. Although other direct search techniques have some fine-tune parameters (cross over fraction, elite count, etc. as in GA) and the performance of the algorithms (number of function evaluations) are heavily depends upon these control parameters. On the other hand, the ABC has no need such control parameters. Sonmez (2011a, b) used the ABC optimization algorithm to solve continuous and discrete structural problems. He demonstrated that the ABC provides results that are as good as or better than other optimization algorithms such as ant colony, genetic algorithm and particle swarm optimization. In addition, Sonmez stated that the ABC shows a remarkably robust performance with a 100% success rate. Although the ABC algorithm does not show any significant improvement in the speed of convergence in terms of the number of function evaluations performed to obtain the best designs when it compared to genetic algorithm, particle swarm optimization and harmony search. In addition, the ABC optimization algorithm has already been used for solving several different civil engineering optimization problems (Fiouz *et al.* 2012, Sonmez *et al*. 2013, Topal and Ozturk 2014).

The main purpose of this study is to find the optimal size and location of *SDB* in planar steel frames by using different transfer function vectors and the ABC optimization algorithm. During the optimization process, the structural response is defined via the transfer functions of displacements, internal forces and the absolute accelerations as the fundamental natural frequency of the structure. The amplitude of these transfer functions is chosen to be objective functions in the problem of the optimal placement of *SDB*. The stiffness parameter of added *SDB*s is defined as a design variable dependent on the cross section area, Young's modulus and the length of the *SDB*. The sum of the stiffness parameter is taken as an equality constraint specified in the proposed optimization algorithm. In order to find the optimal design of the added *SDB*, ABC algorithm is improved for planar steel buildings. Three different steel planar building models have been considered to determine the optimal distribution of the *SDB*s. In order to test the response of the bracing frames for a 20-storey steel building under the conditions of the El-Centro earthquake, time history analyses were also performed.

The remainder of this paper is arranged as follows: Section 2 briefly presents the formulation of transfer function vectors of the nodal displacements, elastic base shear, and absolute acceleration for planer steel frames with diagonal steel braces. In Section 3, the objected functions are presented, then the optimization process and the modelling of foraging behaviour of artificial bees are described. Numerical examples are given in Section 4. Finally, Section 5 presents the conclusions.

2. Structural model with *SDB*

Consider the *n*-storey planar steel building frame fixed base with *SDB* as shown in Fig. 1. The nodal mass and mass moments of inertia are assumed in every node. Let the *SDB*s be placed only in the mid-span of the planar building model. In this paper, the optimal placement (size and location optimization) of *SDB*s is investigated rather than the uniform distribution of *SDB*s. In the global coordinate system, the stiffness matrix including the axial force carrying *SDB*s can be written as

$$K_{SDB} = \sum_{i=1}^{n} T_i^T k_i T_i \tag{1}$$

where T_i is a square transformation matrix. k_i denotes stiffness parameter in the i-th storey as design variable and it can be written in terms of; the length L_i, Young's modulus E_i and the cross section area A_i of the i-th *SDB* as follows

$$k_i = \frac{E_i A_i}{L_i} \qquad i = 1,..., n \tag{2}$$

Matrix equation of motion of a frame without braces, subjected to the horizontal ground motion, may be given as

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = -Mr\ddot{u}_g(t) \tag{3}$$

where $u(t), \dot{u}(t), \ddot{u}(t)$ are, respectively, the displacement, velocity and acceleration vectors of the structural model; M, C and K denote the mass, structural damping and the stiffness matrices of the frame. r denotes the influence vector in terms of the direction of the base acceleration, $\ddot{u}_g(t)$ is the horizontal acceleration of the ground motion. Let $U(\omega)$ and $\ddot{U}_g(\omega)$ denote the Fourier Transform of $u(t)$ and $\ddot{u}_g(t)$, respectively. The Fourier Transform of Eq. (3) according to Clough and Penzien (1993) is presented as follows

$$(K + i\omega C - \omega^2 M)U(\omega) = -Mr\ddot{U}_g(\omega) \tag{4}$$

Let ω denote the circular frequency of excitation, i is $\sqrt{-1}$. If the *SDB*s are added to model structure, the Fourier Transform of the equation of motion can be rewritten as

$$\{(K + K_{SDB}) + i\omega C - \omega^2 M\}U_{SDB}(\omega) = -Mr\ddot{U}_g(\omega) \tag{5}$$

where K_{SDB} is the added stiffness matrix given in Eq. (1) and $U_{SDB}(\omega)$ represents the Fourier Transform of the displacement vector with the added *SDB*. The new quantity defined by Takewaki (2000) to find optimal damper placement can be written as

$$\hat{U}(\omega) = \frac{U_{SDB}(\omega)}{\ddot{U}_g(\omega)} \tag{6}$$

If the first natural frequency of the structure ω_1 is applied then Eqs. (6) and (5) can be rewritten as

$$A\hat{U}(\omega_1) = -Mr \tag{7}$$

$\hat{U}(\omega_1)$ presents the transfer function vector of the displacements and A includes the design variables (k_1, k_2,…, k_N), is as follow

$$A = K + K_{SDB} + i\omega_1 C - \omega_1^2 M \tag{8}$$

Eq. (7) can be solved for the transfer function vector of displacement $\hat{U}(\omega_1)$ and written as

$$\hat{U}(\omega_1) = -A^{-1}Mr \tag{9}$$

The transfer function of the displacements derived by Takewaki (2000) is shown in Eq. (9) however, Aydin and Boduroglu (2008) have proposed a new transfer function vector that is used to calculate the transfer function vector of elastic force as follows

$$F(\omega_1) = -KA^{-1}Mr \tag{10}$$

The transfer function vector of the absolute acceleration at the fundamental natural frequency of the structure derived with K_{SDB} by Cimellaro (2007) to find the placement of the optimal elastic braces is as follows

$$\hat{\ddot{U}}(\omega_1) = -M^{-1}(K + K_{SDB} + i\omega_1 C)\hat{U}(\omega_1) \tag{11}$$

The quantities of $\hat{U}_1$, F_i and $\hat{\ddot{U}}_1$, given in Eqs. (9), (10) and (11), can be written as

$$\hat{U}_1 = \mathrm{Re}[\hat{U}_1] + \mathrm{Im}[\hat{U}_1] \tag{12}$$

$$F_i = \mathrm{Re}[F_i] + \mathrm{Im}[F_i] \tag{13}$$

$$\hat{\ddot{U}}_1 = \mathrm{Re}[\hat{\ddot{U}}_1] + \mathrm{Im}[\hat{\ddot{U}}_1] \tag{14}$$

where $\hat{U}_1$, F_i and $\hat{\ddot{U}}_1$ are the transfer function values of the displacement, elastic shear forces and absolute acceleration of *i-th* storey, respectively. All these equations are in complex form.

The absolute value of $\hat{U}_1$, F_i and $\hat{\ddot{U}}_1$ can be written as

$$\left|\hat{U}_i\right| = \sqrt{(\mathrm{Re}[\hat{U}_i])^2 + (\mathrm{Im}[\hat{U}_i])^2} \tag{15}$$

$$\left|F_i\right| = \sqrt{(\mathrm{Re}[F_i])^2 + (\mathrm{Im}[F_i])^2} \tag{16}$$

$$\left|\hat{\ddot{U}}_i\right| = \sqrt{(\mathrm{Re}[\hat{\ddot{U}}_i])^2 + (\mathrm{Im}[\hat{\ddot{U}}_i])^2} \tag{17}$$

3. Optimization problem of *SDB*

A general structural optimization problem is defined to minimize an objective function while satisfying predefined constraints related to the design variables and structural response. The general mathematical statement of the *SDB* design optimization in the context of this study is as follows

$$\text{Minimize} \quad f(K_{SDB}) = f(k_1, k_2, k_3, \ldots, k_N) \tag{18}$$

This is subject to the inequality constraints on the upper and lower bounds of stiffness coefficients of each added *SDB* being as follows

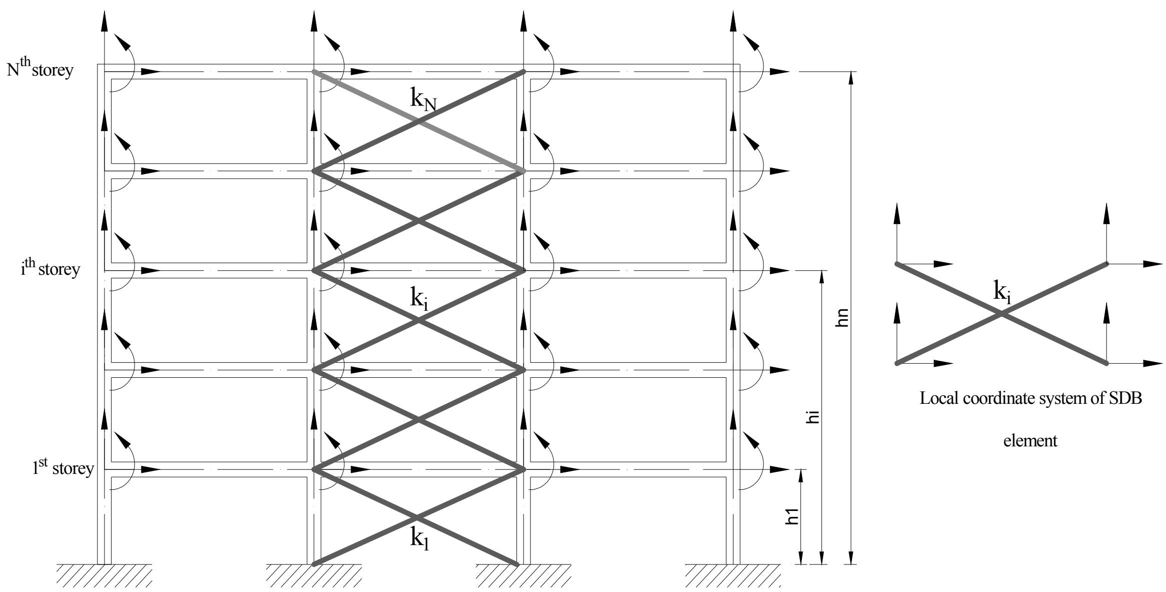


Fig. 1 An *n*-storey planar steel building with X brace system

$$0 \le k_i \le \bar{k} \quad (i = 1, 2, \ldots, N) \tag{19}$$

where $\bar{k}_i$ is the upper bound of the stiffness coefficient of the *SDB* in *i-th* storey and an equality constraint on the sum of stiffness coefficients is written as

$$\sum_{i=1}^{N} k_i = \bar{K} \tag{20}$$

where $\bar{K}$ is the sum of stiffness coefficients of the added *SDB*s.

3.1 Proposed objective functions

To find the optimal *SDB* design, objective functions are generally chosen as the top displacement or the base shear (Takewaki 2000, Aydin *et al.* 2007, Takewaki 1999, Cimellaro 2007, Wang 2006, Aydin and Boduroglu 2008). In the current study, the top absolute acceleration and base moment are also taken as new objective functions to determine the optimal *SDB* design.

When the amplitude of the top displacement transfer function is minimized, the first optimization problem has taken the following form

$$\min f_1 = \left|\hat{U}_t(k_i)\right| \quad (i = 1, 2, \ldots, n) \tag{21}$$

where $\left|\hat{U}_t(k_i)\right|$ corresponds to the transfer function amplitude of the top story displacement evaluated at the first natural frequency, which is in displacement vector $\hat{U}(\omega_1)$. The top storey displacement is taken as the sum of each of the inter-storey drifts.

The second optimization problem based on the minimization of the amplitude of the elastic base shear transfer function can be defined as

$$\min f_2 = |V_b(k_i)| \quad (i = 1, 2, ..., n) \tag{22}$$

where

$$|V_b(k)| = |F_1(k)| + |F_2(k)| + \cdots + |F_n(k)| \tag{23}$$

in which $|F_i|$ is the transfer function amplitude of the *i-th* storey elastic shear force evaluated at the first natural frequency, in vector $F(\omega_1)$. The transfer function amplitude of the base shear $|V_b(k_i)|$ is calculated as the sum of the elastic shear force of each storey. This performance function was proposed to find the optimal damper by Aydin *et al*. (2007) and to find the optimal *SDB* design by Aydin and Boduroglu (2008).

The third optimization problem based on the minimization of the amplitude of the transfer function of the top absolute acceleration at the fundamental natural frequency of a structure can be described as follows

$$\min f_3 = \left|\hat{\ddot{U}}_t(k_i)\right| \quad (i = 1, 2, ..., n) \tag{24}$$

where $|\hat{\ddot{U}}_t(k_i)|$ corresponds to the transfer function amplitude of the top absolute acceleration evaluated at the undamped fundamental natural frequency of the structure in vector $|\ddot{U}(\omega_i)|$. This objective function was proposed to calculate the optimal distribution of the visco-elastic dampers in shear building structures (Cimellaro 2007). This performance function has been adapted to find an optimal *SDB* design in the current study.

For the last objective function, the amplitude of the transfer function of the elastic base moment was selected as follows

$$\min f_4 = |M_B(k_i)| \quad (i = 1, 2, ..., n) \tag{25}$$

and the absolute value of the base moment is calculated as

$$|M_B(k_i)| = |F_1(k)|h_1 + |F_2(k)|h_2 + \cdots + |F_i(k)|h_i + \cdots + |F_N(k)|h_n \tag{26}$$

where $|F_i|$ is the transfer function amplitude of elastic shear force evaluated at the undamped fundamental natural frequency in vector $F(\omega_1)$, and h_i is the height of the *i*-th storey.

3.2 Concept of Artificial Bee Colony Algorithm (ABCA)

Swarm intelligence is a relatively new interdisciplinary field of research, which has gained wide popularity in recent years. Researchers got inspiration from the collective intelligence emerging from the behaviour of a group of social insects (e.g., bees, termites, and wasps). These insects, even with the very limited individual capability, can cooperatively perform many complex tasks necessary for their survival. Swarm intelligence systems are typically made up of a population of simple agents interacting locally with one another and their environment (Bonabeau *et al*. 1999, Kennedy *et al*. 2001).

Like other social insects, honeybees live as members of a community known as colony. Each bee of a colony has own tasks to perform such as cleaning, waxing and foraging in their hive. Those bees, designated for foraging when they reach three weeks old, can fly long distances up to 14 km in several directions to discover abundant food sources. This foraging process starts with the scout bees flying randomly from one area to another to discover fertile flower patches and gather nectar from those flowers. When returning to the hive, scout bees that have found a patch that is above a certain quality threshold deposit their nectar and go to the “dance area” to perform the Waggle Dance. This dance shows that the discovered food source is in the neighbourhood of the hive. The direction and duration of the dance are closely correlated to the distance and location of the patch of flowers. The longer the duration of the dance, the better the quality of the food source; therefore, a bee indicating a high quality food source being on the dance floor for longer has a greater probability of being selected by other bees (Karaboga and Basturk 2008, Karaboga 2005). In principle, flower patches with a plentiful amount of nectar that can be reached with less effort should attract more bees. In Fig. 2, bees transmit information about the location of the found source. Each of the forager bees can find various different quality food sources during their journeys. Hence, after unloading the nectar they have found, each bee can follow one of three options.

(a) Abandon the current food source and search for another promising flower patch if they cannot find any nectar.
(b) Continue to forage at the food source without recruiting nest mates if the nectar is does not exceed the desired quality.
(c) If they find a plentiful nectar source before returning the food source, they can perform a bee dances to recruit nest mates.

The option selected is based on the food level of the nectar source the individual bee has found. If a bee has found a nectar source that is above a certain limit, it follows the option (c). If the nectar

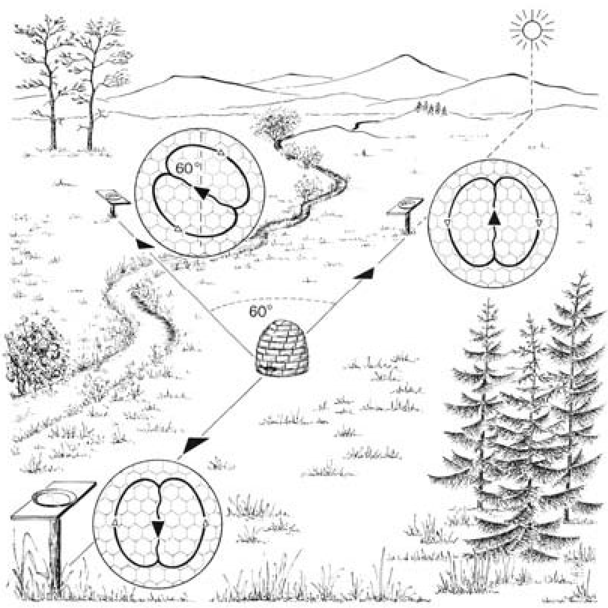


Fig. 2 Waggle dances for different food sources (Lemmens 2007)

source is average, the bee goes to forage at the food source without recruiting nest mates (option (b)). Otherwise, the bee continues to search for other promising nectar sources (the option (a)). The main goal of the bees is to find the most abundant nectar sourced.

3.3 ABCA for SDB design

In the ABC optimization algorithm, the nectar qualities characterize possible solutions to a given optimization problem and the position of the food source corresponds to the design variables ($k_1, k_2,\ldots, k_i,\ldots, k_n$). The general structure of the ABC optimization algorithm is given as follows:

(1) Initialize
(2) REPEAT
  (a) Place the employed bees on the food sources
  (b) Place the onlooker bees on the food sources
  (c) Send the scouts to the search new food sources
  (d) Memorize the best solution achieved so far
(3) UNTIL (requirements are met).

In general, the ABC algorithm uses three control parameters: (a) the number of worker bees (*BN*); (b) a predefined iteration number (LIMIT) if there is no improvement in the amount of nectar from a food source; and (c) the maximum iteration number for searching food (MCN).

All these parameters must be initialized at the first step of the ABC algorithm. Then all the foraging bees (*BN*) go to explore for promising flower areas. The position of a food source, *s*, can be calculated as

$$ {}_s k_i^{new} = k_i^{low} + \gamma (k_i^{up} - k_i^{low}) \tag{27} $$

$$ k_i^{low} \le k_i \le k_i^{up} \quad (i = 1, 2, \ldots, n) \tag{28} $$

where γ is a random number between 0 and 1. k_i^{low} and k_i^{up} are the lower and upper bounds of the *i-th* design variable, respectively.

After the bees come back to the hive with a certain amount of nectar (f_j) determined using one of Eqs. (21), (22), (24) or (25), the first half of the scout bees (*SN*) which found the best food sources become the "employed bees" (step (2(a))). The remainder of the bees watches the dancing bees to decide which one of the employed bee they will follow. The bees that watch the dance are called "onlooker bees" and they select a food source according to a probability proportional to the amount of nectar to be found at that food source. The probability P_s for that source, *s*, is computed in the following way

$$ P_s = \left(\frac{1}{{}_s f_j(k)} \right) \Bigg/ \left(\sum_{s=1}^{SN} \frac{1}{{}_s f_j(k)} \right) \tag{29} $$

where *j* denotes the objective function number (*j* = 1, 2, 3, 4). Each food source has only one employed bee; that is, the number of food sources is equal to the number of employed bees. The number of onlooker bees which will go to a food source depends upon the amount of nectar at the source (step (2(b))). The onlooker bees select a food source according to the quality of the nectar. More unemployed bees will choose to visit an abundant nectar source while fewer or no onlooker bees will choose the food source having less nectar than others.

After an employed bee has done the waggle dance whether it has recruited any bees, it will leave the hive to find a better food source (called candidate food sources) in the neighbourhood of the previous food sources it or the other employed bees have discovered. This means that the ABCA uses the location of the previous food source (${}_{s}k^{old}$) to search for a candidate food source (${}_{s}k^{new}$). Numerically, the location of a candidate food source, s, is determined as

$$ {}_{s}k_{i}^{new} = {}_{s}k_{i}^{old} + \phi({}_{s}k_{i}^{new} - {}_{m}k_{i}^{old}) \tag{30}$$

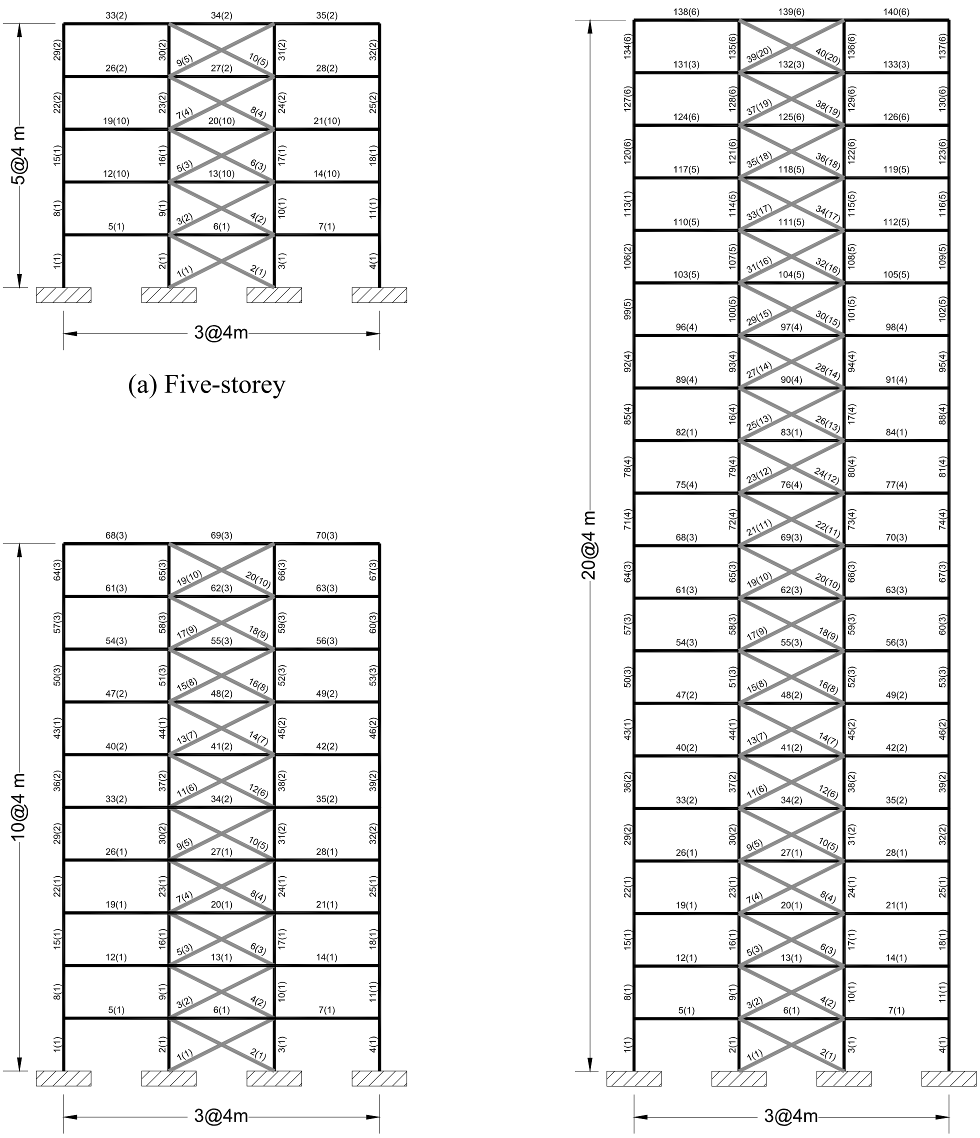
where ϕ is a random number between -1 and 1. ${}_{s}k_{i}^{new}$ is an updated design variable. The left hand subscripts correspond to the solution number (food source, s = 1, 2,…, SN) while the right hand script denote the design variable number (m = 1, 2,…, SN). m is a randomly chosen integer number but cannot be equal to s. ${}_{s}k_{i}^{old}$ plays an important role in the convergence behaviour of the ABC algorithm since it is used to control the exploration abilities of the bees. It directly influences the location of the new food source, which is based on the previous position of other food sources. If the level of food in the new position is better than old one, the new position becomes the food source; otherwise, the old location is retained as the best food source (step (2(d))).

If there is no improvement in the amount of nectar from a food source after a predetermined number of cycles, (LIMIT), this food source should be abandoned by its employed bee and this employed bee becomes a “scout bee.” This type of bees is primarily concerned with finding any kind of nectar sources and they may accidentally discover rich, entirely unknown food sources (step (c)). If a scout bee finds a better food source than other employed bees’ food source, it becomes an employed bee. Like all direct optimization algorithms, the ABC algorithm is iterative, too. The ABC optimization process is performed until the number of cycles is reached to the predefined maximum number of cycles (step (4)).

4. Numerical examples

Three different examples were considered to determine the effect of *SDB*s to the behaviour of selected frames. Each frame had 5, 10 and 20 storeys as shown in Fig. 3. All frames had 3 bays with a span of length 8 m and a storey height of 4 m. The properties of the frame elements are given in Tables 1, 2 and 3 for the 5, 10 and 20 storey frames, respectively. The members as well as the material numbers, given in parenthesis, can be seen in these figures. Young’s Modulus was 2.06×10^5 MPa for the columns and beams. The shear deformation of the structural elements was not taken into account. Only the axial deformation in the added *SDB*s and both bending and axial deformations in the columns and beams was considered. The undamped fundamental natural frequencies of 5, 10, 20 storey model structures were computed as 8.5534 rad/s, 47458 rad/s and 2.7358 rad/s, respectively. The critical damping ratio of the model steel structure was taken to be ξ = 0.02 in the undamped first natural frequency. The structural member properties are given in Tables 1, 2 and 3 for the 5, 10 and 20 storey frames, respectively. The sum of the stiffness coefficients of the *SDB*s was assumed to be $\overline{K} = 3.42 \times 10^9$ N/m of 5-storey building and $\overline{K} = 6.85 \times 10^9$ N/m for 10 and 20 storey buildings.

Optimal *SDB* designs were determined using the ABC algorithm for top displacement, top absolute acceleration, base shear and base moment as objective functions. In order to find the location and amount of diagonal bracing, the artificial bee colony algorithm was used for as the optimizer. The number of bees was considered to be 3 times the number of design variables and



(a) Five-storey

(b) Ten-storey

(c) Twenty-storey

Fig. 3 Three-span planar steel braced frames

that maximum number of cycles was 10 times the number of the design variables. Six independent optimization runs were performed for each optimization case, and then the best, worst and average values were presented in the following format.

The maximum numbers of cycles were set to 50, 100 and 200 for the 5, 10 and 20 storey models respectively that are defined as 10 times of the number of design variable. The ABC method is performed to optimally place the added *SDB*s, random numbers are used, for this reason; the ABC algorithm is run at least six times. Then the worst, the best and the average performance values obtained from the ABC algorithm are selected to represent in Table 4-6.

Table 1 The properties of the frame elements of a 5-storey building frame

Element No.	Storey level	Beams		Columns		Lumped mass (kg)		Mass moment of inertia (kg.m^2)	
(Material No.)		*A* (m^2) (10^{-4})	*I* (m^4) (10^{-5})	*A* (m^2) (10^{-4})	*I* (m^4) (10^{-5})	Interior node (10^3)	Exterior node (10^3)	Interior node (10^5)	Exterior node (10^5)
1-18 (1)	1-3	683	353	683	353	51.2	25.6	5.46	1.71
19-35(2)	4-5	365	205	365	205	51.2	25.6	5.46	1.71

Table 2 The properties of the frame elements of a 10-storey building frame

Element No.	Storey level	Beams		Columns		Lumped mass (kg)		Mass moment of inertia (kg.m^2)	
(Material No.)		*A* (m^2) (10^{-4})	*I* (m^4) (10^{-5})	*A* (m^2) (10^{-4})	*I* (m^4) (10^{-5})	Interior node (10^3)	Exterior node (10^3)	Interior node (10^5)	Exterior node (10^5)
1-28 (1)	1-4	756	383	756	383	51.2	25.6	5.46	1.71
29-49 (2)	5-7	683	353	683	353	51.2	25.6	5.46	1.71
50-70 (3)	8-10	365	205	365	205	51.2	25.6	5.46	1.71

Table 3 The properties of the frame elements of a 20-storey building frame

Element No.	Storey level	Beams		Columns		Lumped mass (kg)		Mass moment of inertia (kg.m^2)	
(Material No.)		*A* (m^2) (10^{-4})	*I* (m^4) (10^{-5})	*A* (m^2) (10^{-4})	*I* (m^4) (10^{-5})	Interior node (10^3)	Exterior node (10^3)	Interior node (10^5)	Exterior node (10^5)
01-28 (1)	1-4	1512	689.4	1512	689.4	51.2	25.6	5.46	1.71
29-49 (2)	5-7	1210	574.5	1210	574.5	51.2	25.6	5.46	1.71
50-70 (3)	8-10	983	459.6	983	459.6	51.2	25.6	5.46	1.71
71-98 (4)	11-14	756	383.0	756	383	51.2	25.6	5.46	1.71
99-119 (5)	15-17	683	353.0	683	353	51.2	25.6	5.46	1.71
120-140 (6)	18-20	365	205.0	365	205	51.2	25.6	5.46	1.71

Different optimal distributions of the *SDB*s for the 5, 10 and 20 storey buildings were found for four performance functions as shown in Figs. 4-6. The optimal *SDB* design for the top displacement shown in Fig. 4 requires placing the *SDB* on all the storeys in decreasing quantities from the base to the top of the building frame. The other optimal *SDB* design methods give similar *SDB* distributions that are concentrated on the first-three storey for the five storey building model. The uniform *SDB* placement, which is commonly used in practice, is also plotted in Figs. 4-6.

Fig. 5 presents the final value of the stiffness coefficients of the optimal *SDB*s for the ten-storey building model. Five and ten-storey building models show almost same the optimal *SDB* distribution pattern for top displacement minimization; however, a slight increase according to the lower storey can be noted in the 8th storey. The other optimal *SDB* design methods; the top absolute acceleration minimization, the base shear minimization and the base moment

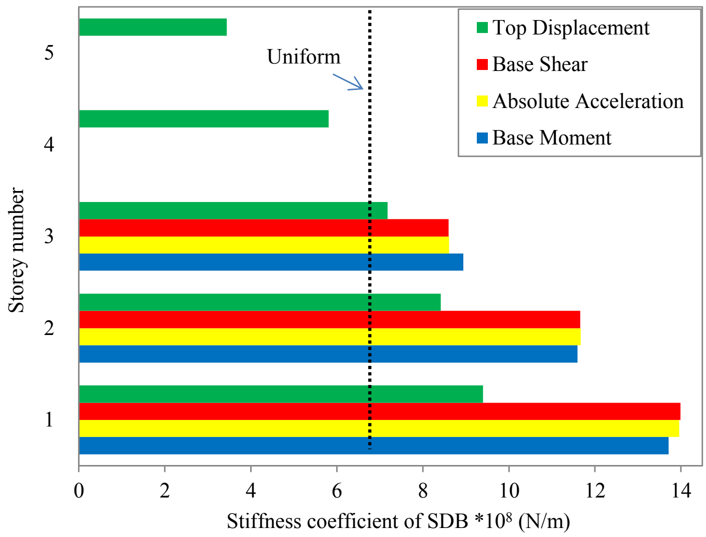


Fig. 4 Optimal distribution of the stiffness coefficients of *SDB*s for a 5-storey frame

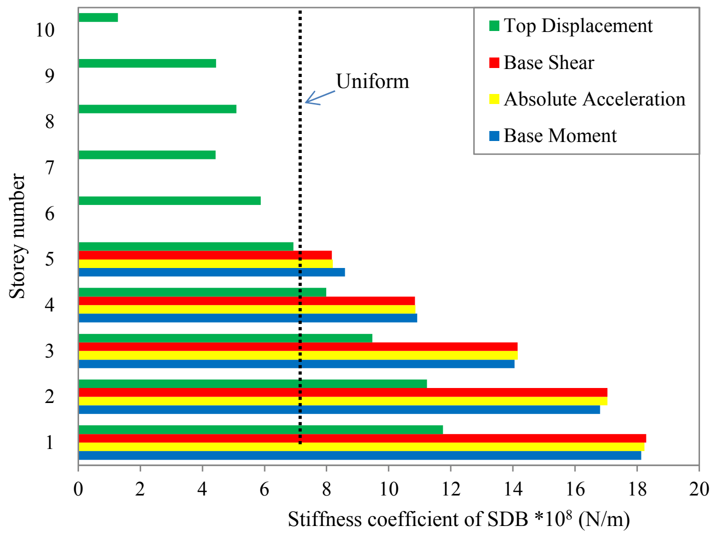


Fig. 5 Optimal distribution of the stiffness coefficients of *SDB*s for a 10-storey frame

minimization place *SDB*s into the first-five storey of ten storey building model.

In the twenty-storey building model, the optimal stiffness coefficients of *SDB*s were plotted for the four optimal design methods shown in Fig. 6. The optimal *SDB*s for top displacement minimization are generally distributed to all the storeys in decreasing quantities from the base to the top of the building frame. The force based optimal *SDB* design methods reveal an *SDB* distribution from the base to the middle storey of the building in decreasing quantities except for the second storey.

Tables 4 to 6 also show the final and first step values of the fundamental frequency of the 5, 10 and 20 storey building frames. The numerical results show that all the *SDB* designs are truly

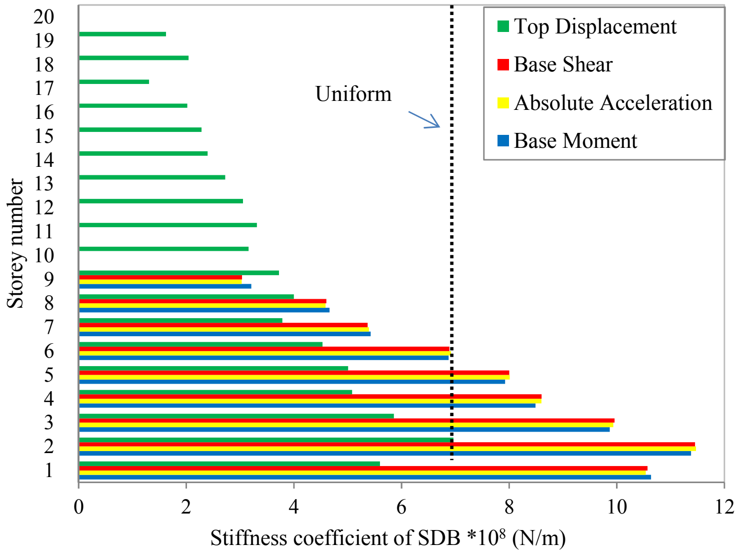


Fig. 6 Optimal distribution of the stiffness coefficients of *SDB*s for a 20-storey frame

effective in reducing the response of structures. The corresponding fundamental frequency for top displacement is higher than the value of the other optimal *SDB* designs. The top displacement design method minimizes the displacement. The fundamental frequency increased when the displacements are decreased as shown Tables 4 to 6. The final values of the objective functions were effectively decreased according to the value of the objective at the first step.

Table 4 The values of the objective functions and fundamental frequencies of a 5-storey building frame

Objective function		Transfer function amplitude		Fundamental frequency	
		First step	Final step	First step (rad/s)	Final step (rad/s)
Base shear	Best	6785039.2	4713640.9	19.1754	14.3355
	Worst	7190859.5	4713641.6	17.1321	14.3356
	Average	7319198.7	4713641.2	18.3003	14.3356
Top displacement	Best	0.044310	0.042378	18.7119	19.4891
	Worst	0.045325	0.042400	17.7731	19.4887
	Average	0.046423	0.042382	18.3624	19.4888
Base moment	Best	99377005.6	79000125.0	18.7956	14.3486
	Worst	99323517.4	79000150.5	15.7920	14.3486
	Average	102668720.0	79000131.7	17.6694	14.3486
Top absolute acceleration	Best	174.0531	122.9327	17.0510	14.3362
	Worst	207.8231	122.9336	15.3087	14.3362
	Average	186.6425	122.9330	16.3468	14.3362

Table 5 The values of the objective functions and fundamental frequencies of a 10-storey building frame

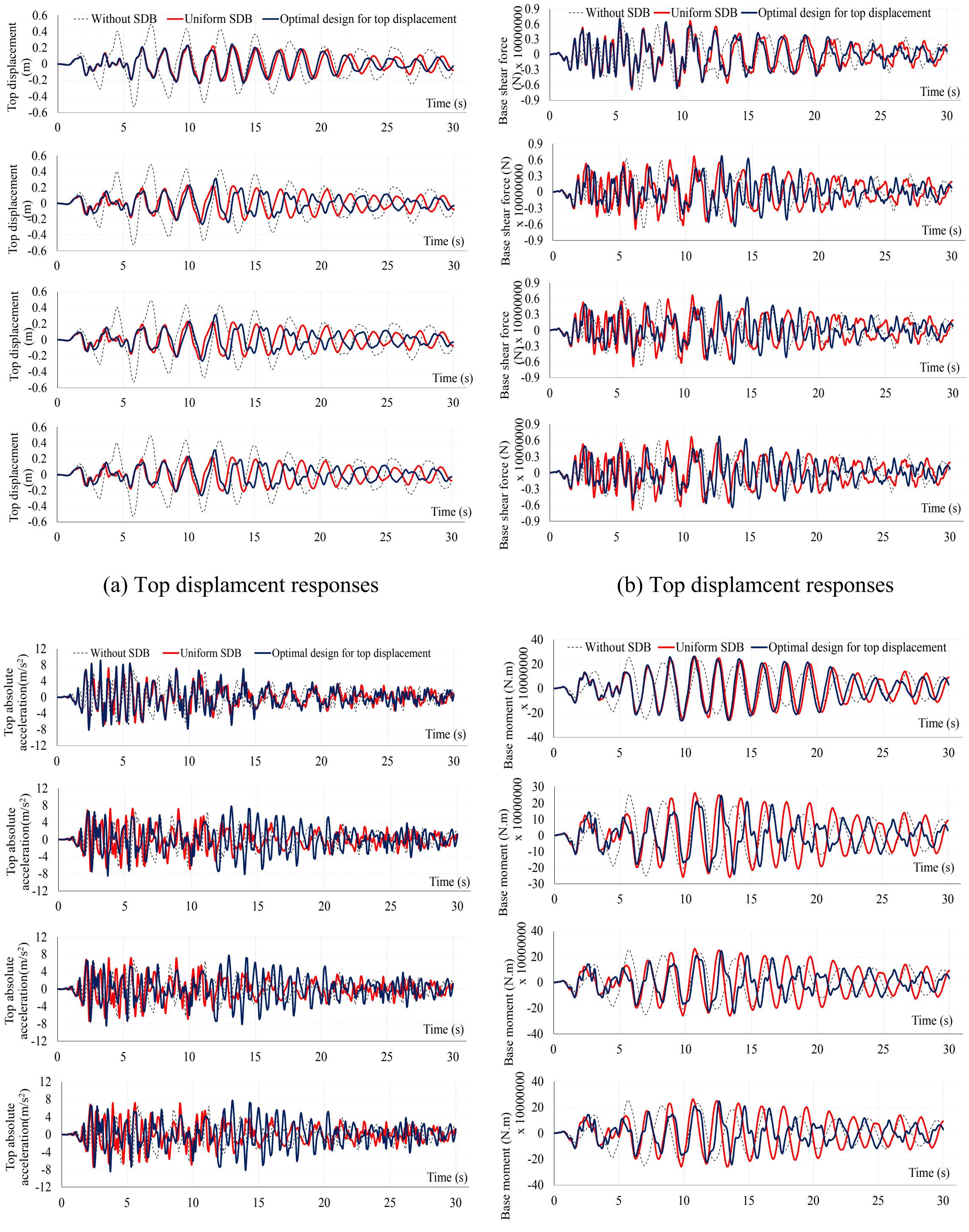
Objective function		Transfer function amplitude		Fundamental frequency	
		First step	Final step	First step (rad/s)	Final step (rad/s)
Base shear	Best	13352433.1	10665071.2	8.1262	7.1304
	Worst	13461966.0	10665077.4	7.8988	7.1309
	Average	13924294.0	10665073.8	8.0507	7.1307
Top displacement	Best	0.2622486	0.232211	8.2940	8.6836
	Worst	0.2464526	0.232211	7.8861	8.6834
	Average	0.2646201	0.232211	8.1071	8.6835
Base moment	Best	379499229.0	333920940	8.3215	7.1348
	Worst	411781543.0	333921013	8.1092	7.1348
	Average	395547523.0	333920964.5	8.1885	7.1347
Top absolute acceleration	Best	358.906707	278.10597	8.5333	7.1306
	Worst	370.622435	278.10613	8.0282	7.1312
	Average	366.428714	278.10604	8.1978	7.1310

Table 6 The values of the objective functions and fundamental frequencies of a 20-storey building frame

Objective function		Transfer function amplitude		Fundamental frequency	
		First Step	Final Step	First Step (rad/s)	Final Step (rad/s)
Base shear	Best	27739266.6	23259190.3	3.7974	3.4469
	Worst	28652663.0	23259206.1	3.7168	3.4473
	Average	27914712.0	23259195.32	3.7524	3.4445
Top displacement	Best	1.2452	1.184600	3.7595	3.9042
	Worst	1.2877	1.184610	3.6461	3.9047
	Average	1.2833569	1.184604	3.7057	3.9041
Base moment	Best	1534744180.0	1384530590.0	3.8105	3.4483
	Worst	1585062270.0	1384530660.0	3.6677	3.4440
	Average	1545375530.0	1384530600.0	3.7153	3.4462
Top absolute acceleration	Best	737.67678	606.69270	3.8033	3.4431
	Worst	729.998728	606.69289	3.6381	3.4511
	Average	732.125271	606.69279	3.7255	3.4449

4.1 Time history analyses

The optimal *SDB* designs obtained in the frequency domain and the ABC algorithm were tested by the time history analysis using the El-Centro (NS) earthquake acceleration data. Time history analyses were performed only for 20-storey building. In order to investigate the seismic response of the structure, the optimal *SDB* designs based on the base shear, the top displacement, the base moment and the top absolute acceleration are used. In addition to the optimal designs, the uniform



(a) Top displamcent responses (b) Top displamcent responses

(c) Top absolute acceleration responses (d) Base moment responses

Fig. 7 Calculated responses of a 20-storey building with different *SDB* designs design and the design without braces were taken into consideration for the comparison. The results in Fig. 7 showed that the optimal damper designs performed well under the given earthquake forces. As shown in Fig. 7(a), all the optimal designs reduce the top displacement response according to the design without braces. However, in terms of the peak value of displacement, the optimal design for top displacement shows a better performance than the other optimal designs. The response of the uniform *SDB* design is almost close to the optimal *SDB* design for the top displacement.

The optimal *SDB* designs for the top absolute acceleration, base shear and base moment can be called force based methods. As a natural consequence of this situation, these optimal *SDB* designs are mainly equivalent in terms of the *SDB* distributions and their compatibility with respect to structural responses. Fig. 7(b) shows the time history responses of the top absolute acceleration for the *SDB* designs in the 20-storey building model. In particular, both the optimal *SDB* design for top absolute acceleration and the *SDB* design for the base shear and base moment outperformed the displacement based design when there are large earthquake accelerations. It is common knowledge that when the *SDB* elements are added to the structure, the stiffness of the structure increases. And this can be due to the rising accelerations and base forces. The key point is to maintain the rise of the structural response (accelerations and internal forces) at the minimum level. Force based designs give a better response than the displacement based design in terms of the top absolute acceleration, base shear and base moment as shown Figs. 7(b) to (d). The results of analyses performed for the optimal *SDB* designs in case of the top absolute acceleration, the base shear and the base moment show that these three optimal design methods are mainly equivalent. The results of the dynamic analysis clearly indicate that each optimal *SDB* design, which is determined based on different performance objectives, also fulfils its own design purpose. The results of the time history analysis verify the results of the transfer function in the optimization stage.

5. Conclusions

This paper presents a new algorithm to determine the optimal distribution of steel diagonal braces (*SDB*) using bee colony optimization. A stiffness parameter of each *SDB* was defined as the design variable and an active constraint was given as the sum of the stiffness parameter of the *SDB*s. Four different objective functions, namely the transfer function amplitude of top displacement, the top absolute acceleration, the base shear and the base moment, were used in optimization process.

According to the four specified objective functions, the ABC algorithm was used as an optimizer for this problem. The ABC algorithm was applied to three planar building models and the following conclusions are drawn:

- Optimal *SDB* distribution, transfer function responses and fundamental frequency are very close to each other for three of the objective functions; however, the top displacement minimization presented different *SDB* design and response. For example, the fundamental frequency of the 5 storey model is approximately 14.35 rad/s for the absolute acceleration, base shear and base moment but this is 19.5 for top displacement optimizations.
- One of the pioneering points of this study is that the base moment and top absolute acceleration were used as an objective function in the optimization of *SDB*. This objective function can be used an alternative performance function for base shear minimization. All

three optimal *SDB* design procedures are derived from a force based design approach.

- The proposed ABC algorithm for the *SDB* design showed positive performance in three aspects. First is the initial point independency. This means that the algorithm does not need an initial guess for the design variables. The second aspect is that the algorithm does not require the evaluation of the gradients of the objective and constraint functions. The last aspect is that the algorithm showed a remarkably robust performance with a 100% success rate. The difference between the best and the worst results for all examples was less than 1%.
- Time history analysis performed for all the *SDB* designs using the El-Centro earthquake records. The results showed that each optimal *SDB* design, which was determined based on different performance objectives, also fulfilled its own design purpose. The analyses showed that the results of the time history analysis verified the results of the transfer function in the optimization stage.

For all the example, *SDB*s were distributed in decreasing quantity from bottom to top storey for the top displacement minimization; on the other hand, for other objective functions, *SDB*s were placed from the base to the middle of buildings.

References

Aydin, E. and Boduroglu, M.H. (2008), "Optimal placement of steel diagonal braces for upgrading the seismic capacity of the existing structures and its comparison with optimal dampers", *J. Constr. Steel Res.*, **64**(1), 72-86.

Aydin, E., Boduroglu, M.H. and Guney, D. (2007), "Optimal damper distribution for seismic rehabilitation of planar building structures", *Eng. Struct.*, **29**(2), 176-185.

Bansal, J.C., Sharma, H. and Jadon, S.S. (2013), "Artificial bee colony algorithm: A survey", *Int. J. Adv. Intell. Paradigms*, **5**(1-2), 123-159.

Bartera, F. and Giacchetti, R. (2003), "Steel dissipating braces for upgrading existing building frames", *J. Constr. Steel Res.*, **60**(3-5), 751-769.

Bonabeau, E., Dorigo, M. and Theraulaz, G. (1999), *Swarm Intelligence: From Natural to Artificial Systems*, Oxford University Press, New York, NY, USA.

Choi, H. and Kim, J. (2005), "Energy-based seismic design of buckling-restrained braced frames using hysteretic energy spectrum", *Eng. Struct.*, **28**(2), 304-311.

Cimellaro, G.P. (2007), "Simultaneous stiffness-damping optimization of structures with respect to acceleration displacement and base shear", *Eng. Struct.*, **29**(11), 2853-2870.

Clough, R.W. and Penzien, J. (1993), *Dynamics of Structures,* (2nd Ed.), McGraw-Hill, NewYork, NY, USA.

Colunga, A.T. and Vergara, A.A. (1997), "Comparative study on the seismic retrofit of a mid-rise steel building: steel bracing vs. energy dissipation", *Earthq. Eng. Struct. D.*, **26**(6), 637-655.

Downs, R.E., Hjelmstat, K.D. and Foutch, D.A. (1991), "*Evaluation of two RC building retrofit with steel bracing*", Structural Research Series No: 563, Department of Civil Engineering, University of Illinois at Urbana-Champaign, IL, USA.

Farhad, F., Nakamura, S. and Takahashi, K. (2009), "Application of genetic algorithm to optimization of buckling restrained braces for seismic upgrading of existing structures", *Comput. Struct.*, **87**(1-2), 110-119.

Fiouz, A.R.M., Obeydi, M., Forouzani, H. and Keshavarz, A. (2012), "Discrete optimization of trusses using an artificial Bee Colony (ABC) algorithm and the fly-back mechanism", *Struct. Eng. Mech.*, Int. J., **44**(4), 501-519.

Frisch, V.K. (1967), *Dance Language and Orientation of Bees*, Harvard University Press, Cambridge, MA, USA.

Ghobarah, A. and Elfath, A.H. (2001), "Rehabilitation of a reinforced concrete frame using eccentric steel bracing", *Eng. Struct*., **23**(7), 745-755.

Gorgulu, T., Tama, Y.S., Yilmaz, S. and Kaplan, H. (2012), "Strengthening of reinforced concrete structures with external steel shear walls", *J. Constr. Steel Res*., **70**, 226-235.

Kanaji, H., Kitazawa, M. and Suzuki, N. (2003), "Seismic retrofit strategy using damage control design concept and the response reduction effect for a long-span truss bridge", *Proceedings of the 19th US–Japan Bridge Engineering Workshop*, Tsukuba, Japan, October.

Karaboga, D. (2005), "An idea based on honey bee swarm for numerical optimization", Technical Report-TR06; Computer Engineering Department: Computer Engineering Department, Erciyes University, (in Turkish).

Karaboga, D. and Basturk, B. (2008), "On the performance of artificial bee Colony (ABC)", *Appl. Soft. Comp*., **8**, 687-697.

Karaboga, D. and Akay, B. (2009), "Survey: Algorithms simulating bee swarm intelligence", *Artif. Intell Rev*., **31**, 61-85.

Karaboga, D., Gorkemli, B., Ozturk, C. and Karaboga, N. (2014), "A comprehensive survey: Artificial bee colony (ABC) algorithm and applications", *Artif. Intell. Rev*., **42**, 21-57.

Kawamata, S. and Masaki, Q. (1980), "Strengthening effect of eccentric steel diagonal braces to existing RC frames", *Proceedings of the 7th World Conference on Earthquake Engineering,* Istanbul, Turkey, August.

Kennedy, J., Eberhart, R.C. and Shi, Y. (2001), *Swarm Intelligence,* Morgan Kaufmann Publishers, San Francisco, CA, USA.

Kim, J. and Choi, H. (2004), "Behaviour and design of structures with buckling-restrained braces", *Eng. Struct*., **26**(6), 693-706.

Lee, K.S. and Geem, Z.W. (2004), "A new structural optimization method based on the harmony search algorithm", *Comput. Struct*., **82**(9-10), 781-798.

Lemmens, N., Jong, S., Tuyls, K. and Nowe, A. (2007), "Bee behaviour in multi-agent systems: A bee foraging algorithm", *Proceedings of the 7th ALAMAS Symposium*, The Hague, Netherlands, April.

Maheri, M.R. and Sahebi, A. (1997), "Use of steel bracing in reinforced concrete frames", *Eng. Struct*., **19**(12), 1018-1024.

Miranda, E. (1991), "Seismic evaluation and upgrading of existing structures", Ph.D. Thesis; University of California at Berkeley, CA, USA.

Mitchell, D. and Dandurand, A. (1988), "Repair and upgrading of concrete structures in Mexico City after the 1985 earthquake", *Can. J. Civil Eng*., **15**(6), 1052-1066.

Pham, D.T., Granbarzadeh, A., Koc, E., Otri, S., Rahim, S. and Zaidi, M.T. (2006), "The bee algorithm– A novel tool for complex optimization problems", *Proceedings of Intelligent Production Machines and Systems (IPROMS)*, Cardiff, UK, July.

Sarnoa, L.D. and Elnashaib, A.S. (2009), "Bracing systems for seismic retrofitting of steel frames", *J. Construct. Steel Res*., **65**(2), 452-465.

Sonmez, M. (2011a), "Artificial bee colony algorithm for optimization of truss structures", *Appl. Soft Comput. J*., **11**(2), 2406-2418.

Sonmez, M. (2011b), "Discrete optimum design of truss structures using artificial bee colony algorithm", *Struct. Multidisc. Optim*., **43**(1), 85-97.

Sonmez, M., Aydin, E. and Karabork, T. (2013), "Using an artificial bee colony algorithm for optimal placement of viscous damper in planar building frames", *Struct. Multidisc. Optim*., **48**(2), 395-409.

Takewaki, I. (1999), "Displacement-acceleration control via stiffness-damping collaboration", *Earthq. Eng. Struct. D*., **28**(12), 1567-1585.

Takewaki, I. (2000), "Optimum damper placement for planar building frames using transfer functions", *Struct. Mult.-Disp. Optim*., **20**(4), 280-287.

Teodorovic, D. (2009), "Bee Colony Optimization (BCO)", *Innov. Swarm. Intell*., **248**, 39-60.

Topal, U. and Ozturk, H.T. (2014), "Buckling load optimization of laminated plates via artificial bee colony algorithm", *Struct. Eng. Mech*., *Int*. *J*., **52**(4), 755-765.

Turker, T. and Bayraktar, A. (2011), "Experimental and numerical investigation of brace configuration effects on steel structures", *J. Construct. Steel Res*., **67**(5), 854-865.

Usami, T., Lu, Z.H. and Ge, H.B. (2005), "A seismic upgrading method for steel arch bridges using buckling-restrained braces", *Earthq. Eng. Struct. D*., **34**(4-5), 471-496.

Valle, C.E.D. (1980), "Some lessons from the March 14, 1979 earthquake in Mexico City", *Proceedings of 7th World Conference on Earthquake Engineering*, Istanbul, Turkey, September.

Valle, C.E.D., Foutch, D.A., Hjelmstad, K.D., Gutierrez, E.F. and Colunga, A.T. (1988), "Seismic retrofit of RC building: A case study", *Proceedings of 9th World Conference on Earthquake Engineering*, Kyoto, Tokyo, Japan, August.

Wang, D. (2006), "Optimal design of structural support positions for minimizing maximal bending moment", *Finite Elem. Anal. Des*., **43**, 95-102.

Yamamato, Y. and Aoyama, H. (1987), "Seismic behaviour of existing RC frame strengthened with retrofitting steel elements", *Proceedings of U.S.-Japan Seminar on Repair and Retrofit of Existing Structures, UJNR*, Tsukuba, Japan, May.

CC