Steel and Composite Structures, *Vol. 27, No. 5 (2018) 613-622*
DOI: https://doi.org/10.12989/scs.2018.27.5.613

Performance based design optimum of CBFs using bee colony algorithm

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(*Received August 18, 2017, Revised January 4, 2018, Accepted March 30, 2018*)

Abstract. The requirement to safe and economical buildings caused to the exploitation of nonlinear capacity structures and optimization of them. This requirement leads to forming seismic design method based on performance. In this study, concentrically braced frames (CBFs) have been optimized at the immediate occupancy (IO) and collapse prevention (CP) levels. Minimizing structural weight is taken as objective function subjected to performance constraints on inter-story drift ratios at various performance levels. In order to evaluate the seismic capacity of the CBFs, pushover analysis is conducted, and the process of optimization has been done by using Bee Algorithm. Results indicate that performance based design caused to have minimum structural weight and due to increase capacity of CBFs.

Keywords: performance-based design; CBF; optimization; bee colony algorithm; pushover

1. Introduction

Recent incidents of earthquakes show that not only is the sustainability against earthquakes important, but also the social and economic consequences of earthquakes are significant. After Loma Prieta (1989) and Northridge (1994) earthquakes, this issue was noticed that considering life safety merely is not sufficient. Therefore, this requirements have led to the development of a performance-based seismic design methodology (Cornell *et al*. 2000, HAZUS 1997, Jalayer and Cornell 2003). The performance-based design (PBD) is a new framework to evaluate seismic hazard of structures and one of the last significant developments in earthquake engineering that is caused more accordance of design procedure on the real behavior of structure. The purpose of this method is increasing the safety level in structures (FEMA356 2000).

In majority of seismic design codes, firstly the design purposes and expected performances of structures are mentioned, then the criteria are presented. If the purposes of seismic design be express more clearly, and suggested criteria for supplying them be defined more appropriately, certainly, it can be said more confidently that designed structures provide considered purposes and expected performances (ATC40 1996, HAZUS 1997, SEAOC 1999).

In terms of seismic design, performance objectives can be satisfied via several possible structural designs. Best purpose design, often on the basis of economy. Being optimum of structure design is one of the purpose of performance. One of the significant parameters is the usage of materials in structures design. Designers always try to reduce the weight, cost and structures volume. In recent years, many researches have been done in design optimization field of steel structures.

Ganzerli *et al*. (2000) were amongst the first researchers to incorporate pushover analysis and performance-based design to optimum design of reinforced concrete structures. Their purpose is minimizing the materials weight with respect to the plastic hinge rotations. Also, the shear and moment resistance controlled after generating optimum design. Seismic reliability and Performance-Based Design (PBD) were investigated by Zhang and Foschi (2004). They used neural network to calculate the probability of structural damage. Liu *et al*. (2005) used GA for Performance-Based Seismic Design (PBSD) of steel moment-resisting frames (SMRFs). The researchers were defined initial material costs, lifetime seismic damage costs, and the number of different steel section types as three merit functions. Maximum interstory drift was used for the performance assessment of the frames using static pushover analysis. Fragiadakis *et al*. (2006) developed an optimization framework of PBSD of steel structures using the Evolutionary Algorithms (EA). The objective function was to minimize the cost subject to interstory drift. They utilized inelastic static and inelastic dynamic procedures with 10 earthquake records subject to each hazard level.

Lagaros and Papadrakakis (2007) applied the EA to the optimal PBSD of 3-D RC structures. Linear and nonlinear static procedures were conducted based on the European

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ISSN: 1229-9367 (Print), 1598-6233 (Online)

code and PBSD, respectively. The initial construction cost was selected as the objective function and the maximum interstory drift as constraint. Cross-sectional dimensions and the longitudinal and transverse reinforcement were considered as design variables. The researchers recognized that there was considerable difference between the results obtained from the European code and PBSD, and design solutions based on the former were more vulnerable to future earthquakes. (Möller *et al*. 2009) evaluated seismic vulnerability of reinforced concrete (RC) structures based on the reliability and performance levels. They utilized neural network and response levels in order to calculate seismic reliability. In this theory, seismic vulnerability of structures is defined as conditional probability of performance levels in a required performance and hazard level of earthquake. (Tehranizadeh and Moshref 2011) investigated the optimum design of SMRFs in different performance levels. Minimizing structural cost and earthquake damage with respect to the maximum hysteretic energy capacity of the structure are the goal of this research. They employed pushover analysis to evaluate seismic demand.

In similar research, Gong *et al*. (2012) presented an energy-based approach to the performance-based seismic optimization using non-linear time history analysis. Minimizing structural weight and energy and maximizing hysteretic energy were considered as objective function. Also, plastic hinge rotations and interstory drift were applied as constraints to the problem. (Gholizadeh 2015) presented two computational strategies for performance-based optimum seismic design (PBOSD) of SMRFs. The one of them is Modified Firefly Algorithm MFA) to efficiently find PBOSD at the performance levels. The other strategy is new neural network model termed as wavelet cascade-forward back-propagation (WCFBP) to reduce the computational burden.

Kaveh *et al*. (2015) developed an efficient framework to solve the performance based multi-objective optimal design problem with considering the initial cost and the seismic damage cost of SMRFs using nonlinear dynamic procedure. Artar (2016a) employed the harmony search and GA to find the optimum weight designs of steel trusses. Also, the same author (Artar 2016b) applied a teaching-learning based optimization method for allowable stress design of braced steel frames. (Hajirasouliha *et al*. 2016) investigated the influences of uncertainties on the seismic performance of optimum and conventionally designed frames. A new optimization method, namely, moth-flame optimization algorithm used by (Gholizadeh *et al*. 2017) to implement the optimize procedure of 2D and 3D steel structures. The gravity search and particle swarm optimization (PSO) algorithms were employed by (Mirzai *et al*. 2017) to find the optimum parameters of steel frames equipped with tuned mass dampers. (Qiao *et al*. 2017) applied the topology optimization method to high-rise steel braced frames. As a consequence, using brace improve the structural lateral stiffness and maximum roof displacement.

The purpose of this study is optimization of Concentrically Braced Frames (CBFs) at various performance levels. To arrive this purpose, a computer program was developed by coding in MATLAB for Artificial Bee Colony (ABC) algorithm. Structural analysis is done by the combination of MATLAB and OpenSees. Pushover analysis is employed to evaluate structural response at various seismic performance levels according to FEMA-356.

2. Performance-based design procedure

Performance-Based Design (PBD) methodology is one of the approaches that its objective is increasing the safety of structures subjected to predefined seismic hazard levels. According to FEMA-356, building performance levels are divided into three levels of structural and nonstructural components and combination of both them that immediate occupancy (IO), life safety (LS) and collapse prevention (CP) are used to define the structural performance levels (FEMA356 2000). Pushover analysis technique is a useful tool to evaluate seismic demands at various performance levels (Hasan *et al*. 2002). The essential feature of pushover analysis is increasing lateral load with specified distribution until a target displacement reaches.

2.1 Target displacement

According to the FEMA-356, the displacement coefficient is one of the methods to determine target displacement that is defined as follows

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\Pi^2} g \tag{1}$$

Where:

C_0 = Modification factor to relate spectral displacement and likely building roof displacement.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response.

C_3 = Modification factor to represent increased displacements due to dynamic $P - \Delta$ effects. For buildings with positive post-yield stiffness,

S_a = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g.

T_0 = Effective fundamental period of the building in the direction under consideration, sec.

2.2 Lateral load distribution

The Lateral load distribution P_S can be expressed as Eq. (2) over the height of the building in pushover analysis according to the FEMA-356.

$$P_S = V_b \left(\frac{G_S H_S^k}{\sum_{m=1}^{ns} G_m H_m^k} \right) \tag{2}$$

Where:

P_S = Lateral load applied at story s

V_b = Base shear

H_s, H_m = Height from the base of the building to stories s and m, respectively

G_s, G_m = Seismic weight for story level s and m, respectively

ns = Story number

k = Constant number determined by the period

The base shear obtained from FEMA-356 is calculated in any performance level as follows

$$V_b^i = W\left(\frac{S_a^i}{g}\right) \qquad i = CP, IO, LS \tag{3}$$

Where W and g are the seismic weight of the structure and gravitational acceleration, respectively

S_a^i is spectral acceleration being defined for each performance level as follows

$$S_a^i = \begin{cases} F_a S_s^i (0.4 + 3\frac{T}{T_0^i}) & 0 < T < 0.2T_0^i \\ F_a S_s^i & 0.2T_0^i < T < T_0^i \\ \frac{F_v S_1^i}{T} & T > T_0^i \end{cases} \tag{4}$$

$$T_0^i = \frac{F_v S_1^i}{F_a S_s^i} \tag{5}$$

Where:

T is the elastic fundamental period of the structure, which is computed here from the structural modal analysis.

S_s^i and S_l^i are the short-period and the first sec.-period response acceleration parameters, respectively.

F_a and F_v are the site coefficient determined respectively from FEMA-356.

Factor k in Eq. (6) which relates the spectral acceleration at T to the PGA is equal to

$$\mathrm{k} = \begin{cases} 1 & T < T_C \\ \frac{T_C}{T} & T_C < T < T_D \\ \frac{T_C - T_D}{T^2} & T > T_D \end{cases} \tag{6}$$

T_C and T_D = The fundamental parameters of the spectrum are the corner periods

2.3 Analysis and design of frames

The structure should be checked for gravity loads. In this study, the load combination of AISC-LRFD provisions is considered (AISC 2010)

$$Q = 1.2Q_{DL} + 1.6Q_{LL} \tag{7}$$

Where Q_{DL} and Q_{LL} are dead and live loads, respectively.

The strength constraints for the non-seismic load combinations taken from LRFD-AISC (2010) are presented as follows

$$\left(\frac{P_u}{\phi_c P_n}\right) < 0.2 \rightarrow g_{\sigma,l}(X) = \left[\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right)\right] - 1 \le 0, \quad l = 1, \ldots, ne \tag{8}$$

$$\left(\frac{P_u}{\phi_c P_n}\right) \ge 0.2 \rightarrow g_{\sigma,l}(X) = \left[\frac{P_u}{2\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right)\right] - 1 \le 0, \quad l = 1, \ldots, ne \tag{9}$$

That:

P P_u = Required strength (tension or compression)

P_n = Nominal axial strength (tension or compression)

∅ ϕ_c = Resistance factor (0.9 for tension, 0.85 for comparison)

∅ b = Flexural resistance reduction factor (Mux, M_{ux}, M_{uy} = Required flexural strengths in the x and y directions;

M_{nx}, M_{ny} = Nominal flexural strengths in the x and y directions

X = Vector of design variables

In order to calculate the maximum inter-story drift, pushover analysis is done at various performance levels that the load combination has been considered to evaluate seismic demands as follows

$$Q = 1.1(Q_{DL} + Q_{LL}) \tag{10}$$

The other constraints used in this optimum design of the X- braced frames in accordance with FEMA-356 as defined:

- Yield rotation

$$\text{Beams:} \quad \theta_y = \frac{ZF_{ye} l_b}{6EI_b}$$
$$\text{Column:} \quad \theta_y = \frac{ZF_{ye} l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \tag{11}$$

Z = Plastic section modulus

F_{ye} = Expected yield strength of the material

l_b = Beam length

l_c = Column length

E = Modulus of elasticity

P = Axial force in the member

P_{ye} = Expected axial yield force of the member = $A_g F_{ye}$

- plastic rotation deformation

$$\begin{aligned} &\mathrm{a:} \quad \frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}} \ \text{or} \ \frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}} \ \text{for} \ LS = 6\theta_y,\ CP = 8\theta_y \\ &\mathrm{b:} \quad \frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}} \ \text{or} \ \frac{h}{t_w} \le \frac{640}{\sqrt{F_{ye}}} \ \text{for} \ LS = 2\theta_y,\ CP = 3\theta_y \end{aligned} \tag{12}$$

Other: Linear interpolation between the values on lines *a* and *b*.

- Maximum Allowable length alteration for square-shaped Compression braces

$$
\begin{aligned}
&\text{a}: \quad \frac{d}{t} \le \frac{1500}{F_y} \text{ for } LS = 4\Delta_C,\ CP = 6\Delta_C \\
&\text{b}: \quad \frac{d}{t} \ge \frac{6000}{F_y} \text{ for } LS = 4\Delta_C,\ CP = 2\Delta_C \\
&\text{c}: \quad \frac{1500}{F_y} \le \frac{d}{t} \le \frac{6400}{F_y} \text{ for Linear interpolation shall be used}
\end{aligned}
\tag{13}
$$

Where Δ_C is the axial deformation at expected buckling load.

- Inter-story drift

$$
\begin{aligned}
g^{IO}_{\theta,k}(X) &= \frac{\theta^{IO}_k}{\theta^{IO}_{all}} - 1 \le 0 \qquad k = 1, 2, ..., ns \\
g^{IO}_{\theta,k}(X) &= \frac{\theta^{IO}_k}{\theta^{IO}_{all}} - 1 \le 0 \qquad k = 1, 2, ..., ns
\end{aligned}
\tag{14}
$$

Where θ_k^{IO}, θ_k^{LS} and θ_k^{CP} are the k-th story of the braced steel frame in IO, LS and CP performance level which its allowable values are 0.5%, 1.5% and 2.0%, respectively.

3. Problem formulation for optimization

In the structural optimization problem, usually the cost of material is chosen as objective function, because it is one of the important parameters of structures design. In this study, the minimum cost has been selected as the objective function as follows

$$\text{Find:} \quad A^T = \left[A_1, A_2, A_3, ..., A_{ng}\right] \tag{15}$$

$$\text{Minimize:} \quad W(A_1, A_2, A_3, ..., A_{ng}) = \sum_{i=1}^{ng} \sum_{j=1}^{nm} \rho_i A_i l_j \tag{16}$$

$$\text{Subject to:} \quad g_1(x) \le 0, g_2(x) \le 0, ..., g_{nc}(x) \le 0 \tag{17}$$

n_g = Number of design groups
nm = Number of members
ρ_i = Material density of the i-th group section
A_i = Cross-sectional area of the *i*-th group section
l_i = Length of the j-th element in the *i*-th group
nc = Number of constraints

The general form of constrained optimization problem should be converted to an unconstrained problem with a modified objective function. In this study, the adaptive penalty function method has been used for constraint optimization problem. The modified objective function φ is defined as Krishnamoorthy *et al*. (2002)

$$\phi(X) = F(X) + R_P \sum_{j=1} \max(0, g_j(X))^2 \tag{18}$$

In the present function, $\varphi(X)$, R_P and m are the new objective function (pseudo objective function), penalty coefficient and the number of problem's constraints, respectively. It is noted that the more detail can be found in (Krishnamoorthy *et al*. 2002).

4. Artificial bee colony optimization algorithm

Artificial Bee Colony (ABC) algorithm is one of the foraging behavior-based algorithms that first proposed by (Karaboga 2005).This algorithm inspires from the behavior of honey bees. In ABC algorithm, each food source represents a possible solution to the optimization problem and the fitness of the food source corresponds to the value of the objective function to be optimized. ABC algorithm consists of three groups of bees: employed bees, onlookers and scouts. The employed bees search food in the neighborhood of a food source. Onlooker bees watch the dances of the employed bees, and they tend to select good food sources from those found by the employed bees. Scouts bees explore the search space randomly. The employed bees share the information about the location and quality of food sources to the onlooker bees (Zhu and Kwong 2010).

The ABC generates a randomly distributed initial population of SN solutions, where SN is equal to the number of employed bees or onlooker bees.

Each solution X_{ij} is a *D*-dimensional vector which *D* is the number of parameters of the function to be optimized. This operation can be defined as

$$
\begin{aligned}
X_{ij} &= X_{\min j} + rand[0,1](X_{\max j} - X_{\min j}) \\
i &= 1,2,..., N \text{ and } j = 1, 2,...D
\end{aligned}
\tag{19}
$$

Where $X_{\max j}$ and $X_{\min j}$ are the maximum and minimum value of solution for *j* dimension, respectively.

Each employed bee X_i generates a new candidate solution v_i in the neighborhood of its present position as follows

$$v_{ij} = x_{ij} + \phi_{ij}(x_{ij} - x_{kj}) \qquad k \ne j \tag{20}$$

Where $k \in \{1, 2,..., SN\}$ is randomly chosen index and ϕ_{ij} is a random number within [-1, 1].

After the generation of the new source position, v_{ij} is produced and evaluated by the artificial bee. Then its quality (fitness) is compared with old food source. If the new food has equal or better nectar than the old source, it is replaced with the old one in the memory. Otherwise, the old one is retained in the memory.

After sharing the information by employed bees of food sources, the onlooker bees choose a food source based on the probability value of fitness of each food source. This value is described as follows (Parsopoulos and Vrahatis 2002)

$$P_i = \frac{fit_i}{\sum_{j=1}^{SN} fit_j} \tag{21}$$

Where fit_i is the fitness value of the ith solution in the colony and SN is the number of food sources. The solution i is the higher the probability of the ith food source selected.

If a position of food source cannot be improved, the food source is released, and employed bee related on food source is converted to scouts. Then, the scout bee creates a randomly food source and new food source is replaced with X_i as follows

$$X_{ij} = X_{\min j} + rand[0,1](X_{\max j} - X_{\min j}) \quad \text{for} \quad j = 1,2,..., D \tag{22}$$

Where rand (0, 1) is a random number within [0, 1] based on a normal distribution.

This process is repeated until number of iterations reach to the maximum value. Finally, ABC algorithm is stop.

5. Design examples

In this study, braced steel frame of 5 and 9 stories has been simulated. 2D modeling of frames have been implemented using finite element software OpenSees, and NonLinear Beam-Column element with Steel 02 materials has been employed to model beam, column and braces. The braced frames are assumed to have fixed supports, connections and structure foundation as rigid. According to the FEMA-356, soil type is D-type soil and the interaction between soil and structure have been ignored, also P – Δ effect has been considered.

Supposed design variations have been chosen from AISC database for structural elements (AISC 2010).

The dead load of QD = 6 kN/m^2 and live load of QL = 2 kN/m^2 are applied to the all beams. The soil type D, based on FEMA-356 is considered and the value of the modulus of elasticity is 210 GPa and the yield stress is f_y = 235 MPa.

Artificial Bee Colony Algorithm has been used to achieve optimum weight of frames. Optimization process has been coded in MATLAB software. In order to utilize optimization algorithm the OpenSees and MATLAB software links have been used.

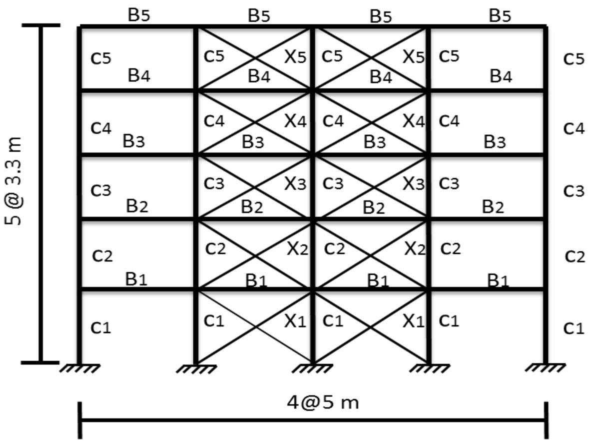


Fig. 1 Example 1: 5-story, 4-bay braced frame with grouping details

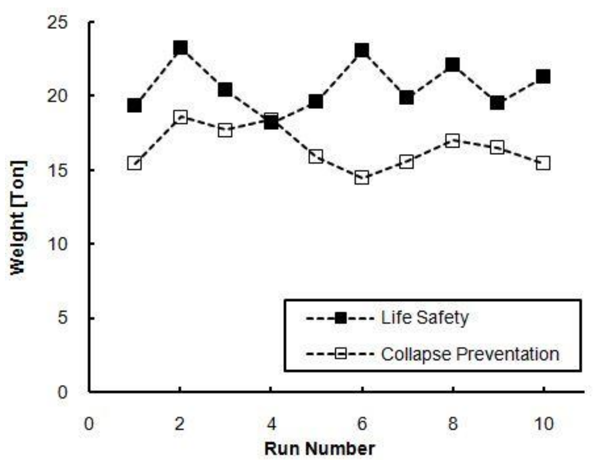


Fig. 2 Example 1: Optimum weight for 5-story braced frame

5.1 Example 1: 5-story braced steel frame

In this example, a braced steel frame with 4-bay and 5-story is assumed as Fig. 1. Height of each story is 3.3 m and each span is 5 m. The framework consists of 65 members, 5 continuous design variables, C1–C5 for columns and 5 continuous design variables, B1–B5 for beams and 5 continuous design variables, X1-X5 for braces.

In Fig. 2, the graph of attained optimum weight in 10 design cycles has been shown for 5-story building in LS and CP performance levels. The optimization results of 5-story braced frame have been presented in Table 1 for CP and LS performance levels.

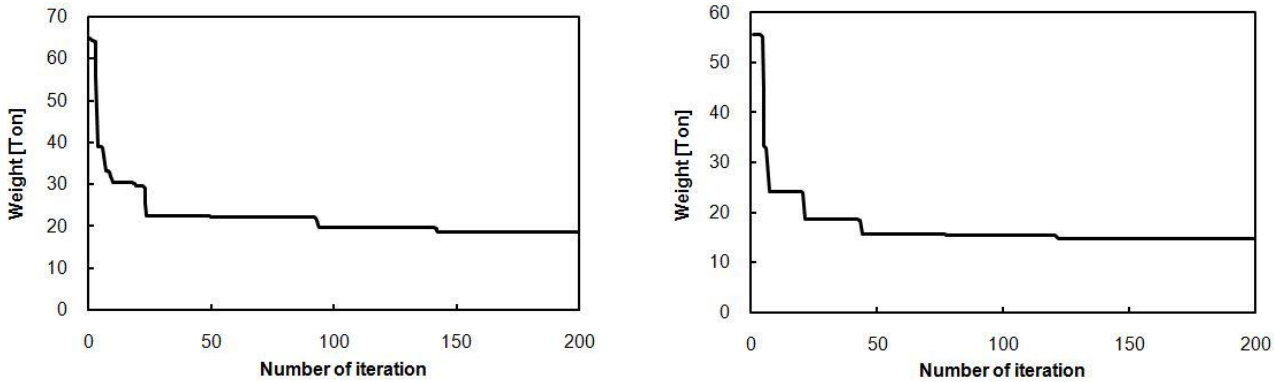
In Fig. 3, the convergence graph of 5-story structure has been presented for LS and CP performance levels. The minimum weight of 5-story frame for CP and LS performance levels has been achieved 18.21 and 14.69 tons, respectively.

Fig. 4 shows the capacity curve graph of 5-story structure for LS and CP performance levels.

In Fig. 5, relative displacement stories of 5-story

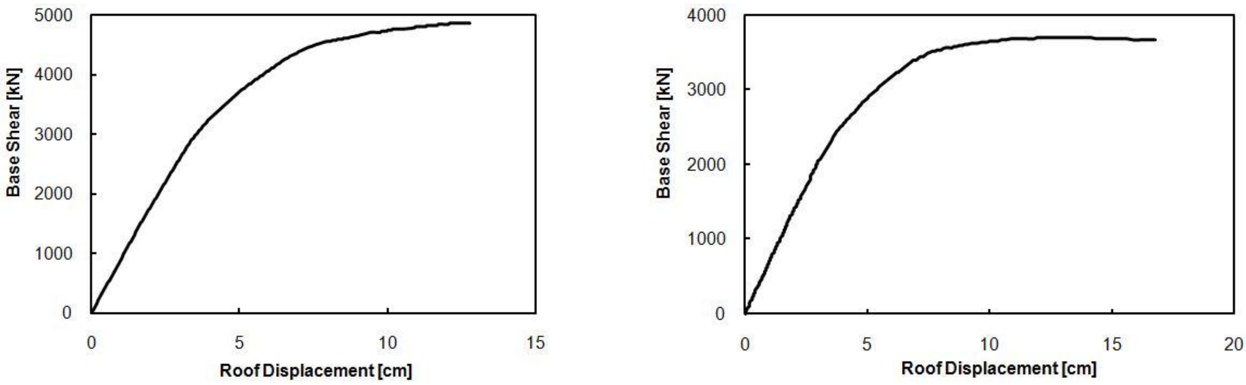
Table 1 Example 1: Design history results for 5-story

	Weight (Ton)	
NO. cycle design	Life Safety (LS)	Collapse Prevention (CP)
1	19.36	15.49
2	23.26	18.61
3	2.43	17.72
4	18.21	18.46
5	19.61	15.93
6	23.08	14.69
7	19.92	15.60
8	22.15	17.03
9	19.50	16.54
10	21.29	15.46
Average	20.68	16.53
Minimum	18.21	14.49



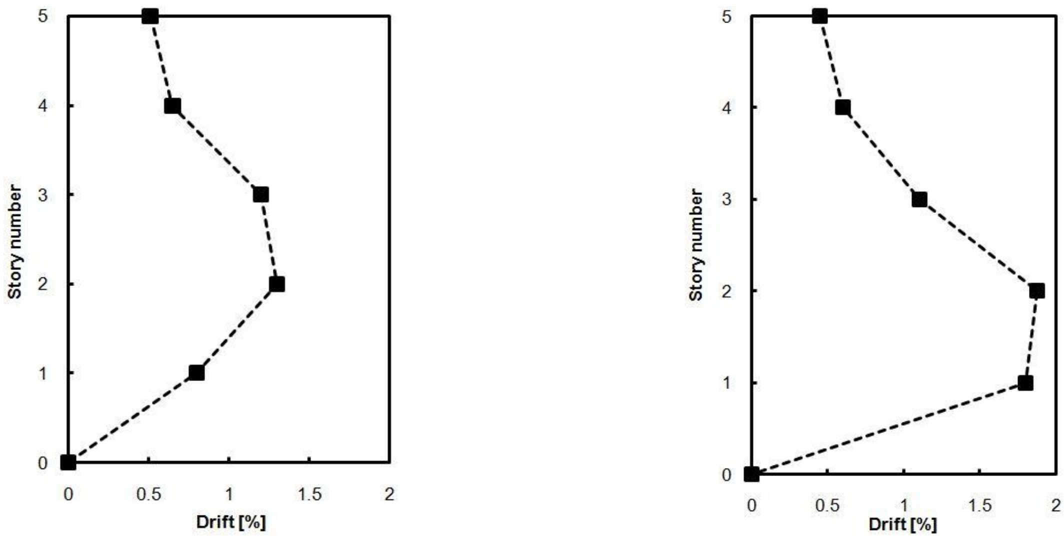
(a) LS level (b) CP level

Fig. 3 Example 1: Convergence histories for 5-story braced frame



(a) LS level (b) CP level

Fig. 4 Example 1: Capacity curve for 5-story braced frame



(a) LS level (b) CP level

Fig. 5 Example 1: Story drifts for 5-story braced frame

structure have been presented, permissible drift is 1.5 and 2 percent for LS and CP performance levels, respectively. As it is seen, structure has allowable drift.

The characteristics of obtained optimum sections for 5-story frame have been illustrated in Tables 2 and 3 for LS and CP performance levels.

5.2 Example 2: *9-story braced steel frame*

In this example, the structure considered is 9-story, 4-

Table 2 Example 1: Optimum designs Results for 5-Story – LS level

STORY	Column	Beam	Brace
1	W14X82	W18X40	HSS7X7X1/2
2	W14X74	W18X40	HSS6X6X1/2
3	W14X68	W16X31	HSS6X6X1/2
4	W14X53	W14X26	HSS5X5X5/16
5	W14X48	W12X26	HSS4X4X3/8

Table 3 Example 1: Optimum designs Results for 5-Story – CP level

STORY	Column	Beam	Brace
1	W14X53	W14X26	HSS10X10X1/2
2	W14X53	W14X22	HSS10X10X1/2
3	W14X68	W12X19	HSS8X8X1/2
4	W14X26	W12X19	HSS6X6X3/8
5	W14X26	W12X19	HSS4X4X3/8

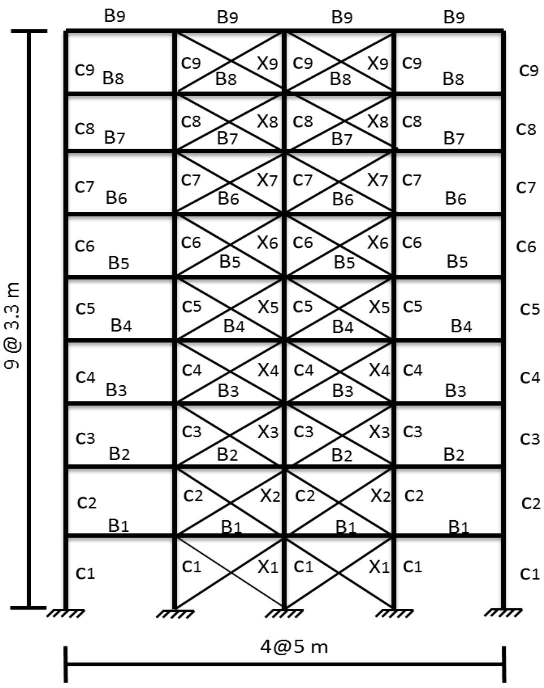


Fig. 6 Example 2: 9-story, 4-bay braced frame with grouping details

bay braced frame as shown in Fig. 6. The length of each bay and the height of each story are 5 and 3.3 m, respectively. The geometric characteristics and the group members are also shown in Fig. 6.

The final results have been presented in Table 4 obtaining from optimization of 9-story building for LS and CP performance level.

In Fig. 7, obtained optimum weight in 10 design cycles has been shown for 5-story building in two performance levels of LS and CP.

In Fig. 8, convergence history has been shown for 9-story frame in LS and CP performance levels. The results of convergence show that the minimum optimum weight of 9-story frame is 47.83 tons for LS performance level and CP level is 37.5 tons.

Table 4 Example 2: Design history results for 9-story

	Weight (Ton)	
NO. cycle design	Life Safety (LS)	Collapse Prevention (CP)
1	50.62	38.92
2	48.07	41.33
3	58.79	37.50
4	61.95	38.88
5	47.83	45.23
6	68.38	39.10
7	58.50	37.72
8	48.47	41.12
9	51.05	40.64
10	49.19	39.59
Average	54.28	40.00
Minimum	47.83	37.50

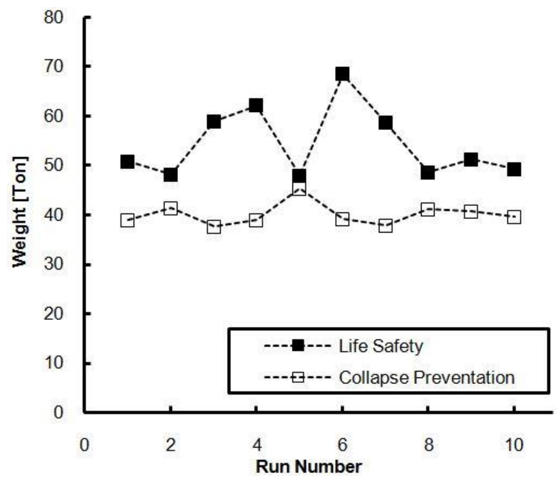


Fig. 7 Example 2: optimum weight for 9-story braced frame

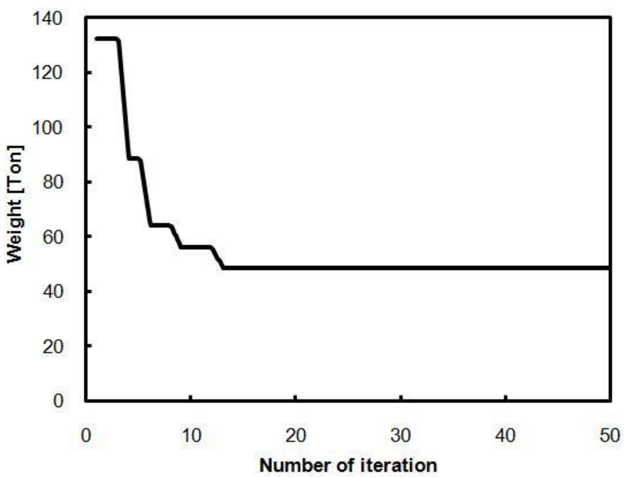
Fig. 9 describes the capacity curve of 9-story structure for LS and CP performance levels. As it is observed, the optimized structure for LS performance level has higher capacity than CP performance level.

In Fig. 10, relative displacement of 9-story building has been demonstrated. As it is observed, drift values of stories don't exceed permissible values.

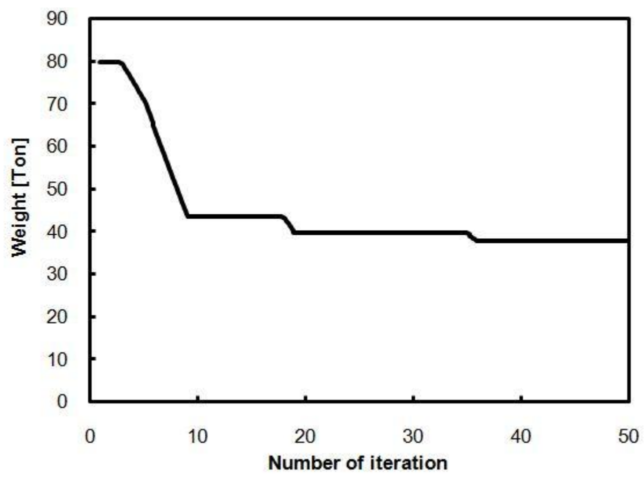
The characteristics of obtained optimum sections are in Tables 5 and 6 for 9-story frame.

6. Conclusions

In this research, it has been dealt with the optimum design of 5- and 9-story braced steel frames by using Artificial Bee Colony Algorithm. Modeling and analyzing the frames has been done in OpenSees software.

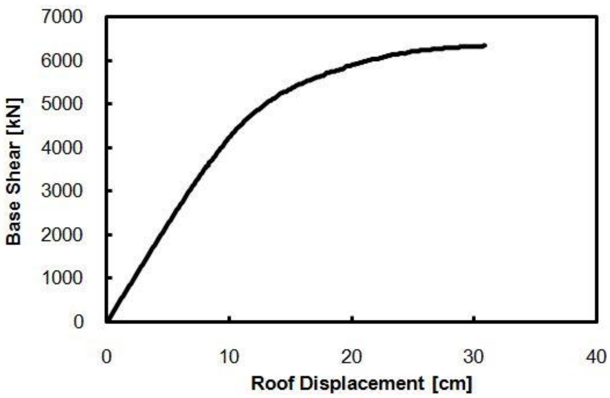


(a) LS level

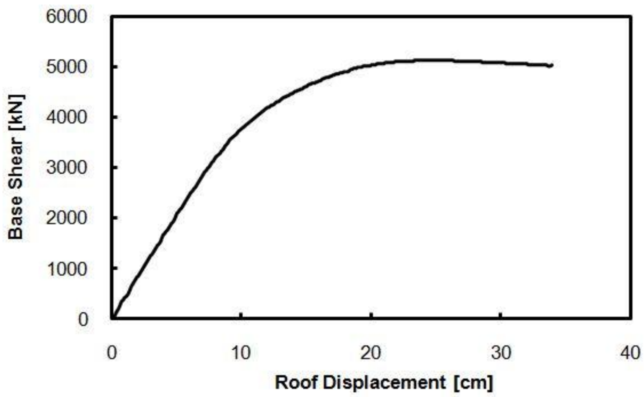


(b) CP level

Fig. 8 Example 2: Convergence histories for 9-story braced frame

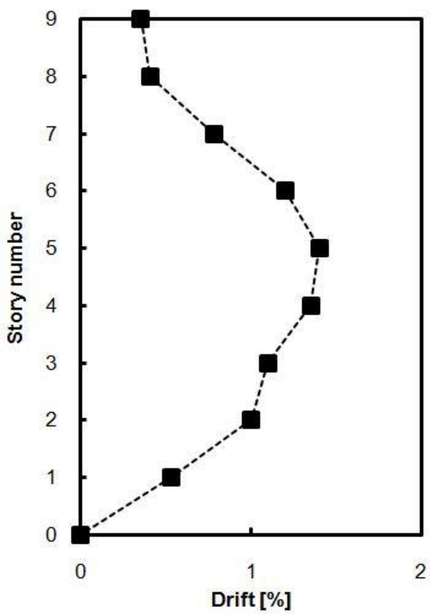


(a) LS level

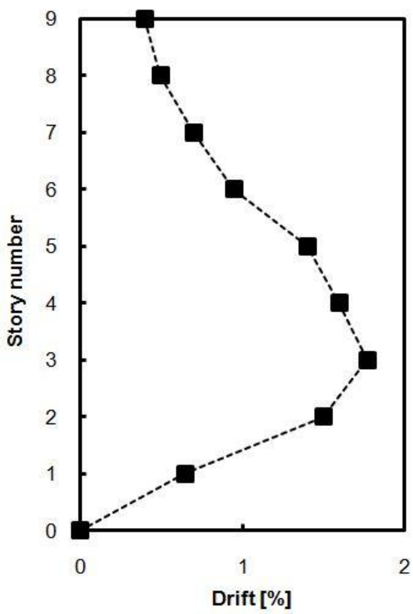


(b) CP level

Fig. 9 Example 2: Capacity curve for 9-story braced frame



(a) LS level



(b) CP level

Fig. 10 Example 2: Story drifts for 9-story braced frame

Table 5 Example 2: Optimum designs Results for 9-Story – LS level

STORY	Column	Beam	Brace
1	W14×99	W18X62	HSS12X12X5/8
2	W14×90	W18X62	HSS12X12X5/8
3	W14X74	W18X50	HSS10X10X5/8
4	W14×68	W18X40	HSS10X10X5/8
5	W14×68	W16X31	HSS10X10X1/2
6	W14X53	W14×26	HSS10X10X5/8
7	W14X53	W12×26	HSS8X8X1/2
8	W14×48	W12×26	HSS10X10X5/8
9	W14×48	W12×19	HSS8X8X1/2

Table 6 Example 2: Optimum designs results for 9-story – CP level

STORY	Column	Beam	Brace
1	W14×74	W16X31	HSS14X14X5/8
2	W14×68	W16×31	HSS12×12×5/8
3	W14×61	W14×26	HSS12×12×5/8
4	W14×53	W12X26	HSS10×10×5/8
5	W14×48	W12×22	HSS12×12×5/8
6	W14×48	W12×22	HSS6×6×5/8
7	W14×43	W12×19	HSS6×6×5/8
8	W14×43	W12×19	HSS4X4X.375
9	W14×43	W12×19	HSS4X4X.375

Optimization procedure has been coded in MATLAB software and then has been linked to OpenSees software.

Achieved results of this study are as follows:

Optimizing for studied frames has been fulfilled in LS and CP performance levels. Optimization procedure has been repeated 10 times and ultimately the least weight has been selected as the optimum weight. Achieved results show that Artificial Bee Optimization Algorithm has the suitable ability and velocity to gain the optimum design, as though, optimum results average of the structure has little difference with the best result of optimization, and also Convergence history shows that bee algorithm approaches to the optimum response in first steps.

Utilizing design approach based on the performance, simultaneous with structure optimization not only has got to minimize the structure weight, but also to apply the maximum capacity of structure in order to reach the required performance level. In the other words, structure segments are able to use their nonlinear capability in allowable range

The capacity curve of optimum structures shows that these structures have had suitable stiffness before reaching the yielding boundary and their nonlinear behavior indicates that optimized structure has upper capacity in energy dissipation. And required ductility which is appropriate to performance level has been provided.

Optimized frames for LS performance level have more weight and base shear in comparison with CP performance level in the time of reaching the purpose displacement, meaning that these frames are more resistance and safer.

Optimized frames for CP performance level are able to tolerate more rotation and deformation in comparison with LS performance level. For this reason, they have more nonlinear behavior. Whilst, the capacity of optimized frames is because of their linear behavior for LS performance level to a great extent.

The optimum weight of 9-story frame for CP performance level has got 22% less than performance level of LS. Likewise, the optimum weight of 5-story frame for CP performance level has got 25% less than performance level of LS.

Base shear, including the purpose displacement in 9-story optimum frame for LS performance level, is 19% more than performance level of CP and for 5-story optimum frame, it is 22% more than performance level of CP.

Acknowledgments

This research was supported by a grant (17CTAP-C129811-01) from Land Transport Technology Promotion Research Project Program funded by Ministry of Land, Infrastructure and Transport of Korean government.

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