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OPTIMIZATION OF STEEL MOMENT FRAMES WITH PANEL-ZONE DESIGN USING AN ADAPTIVE DIFFERENTIAL EVOLUTION

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Abstract

Optimization of steel moment frames has been widely studied in the literature without considering shear deformation of panel-zones which is well-known to decrease the load-carrying capacity and increase the drift of structures. In this paper, a robust method for optimizing steel moment frames is developed in which the panel-zone design is considered by using doubler plates. The objective function is the total cost of beams, columns, and panel-zone reinforcement. The strength and serviceability constraints are evaluated by using a direct design method to capture the nonlinear inelastic behaviors of the structure. An adaptive differential evolution algorithm is developed for this optimization problem. The new algorithm is featured by a self-adaptive mutation strategy based on the p-best method to enhance the balance between global and local searches. A five-bay five-story steel moment frame subjected to several load combinations is studied to demonstrate the efficiency of the proposed method. The numerical results also show that panel-zone design should be included in the optimization process to yield more reasonable optimum designs.

Keywords: direct design; differential evolution; optimization; panel-zone; steel frame.

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1. Introduction

Moment frame or moment-resisting frame is a frame with rigid beam-to-column connections. This structure has been widely used for a long time since it is suitable for multi-story buildings and superior earthquake resistance. Cost optimization of a moment frame is often to minimize the total structural cost or weight by selecting the sections of beams and columns in a discrete pre-defined list while all strength, serviceability and constructability constraints are guaranteed. This implies that cost optimization of moment frames is highly nonlinear and finding optimal solutions is impossible in almost case studies. Normally, meta-heuristic algorithms that can find sufficiently good but not optimal solutions are employed. The efficiency of meta-heuristic algorithms for structural design has been proved by the results of many studies in the literature, for example, Refs. [1–8]. Besides that, lots of meta-heuristic algorithms have been proposed such as big bang–big crunch (BB–BC) [9],

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differential evolution (DE) [10], enhanced colliding bodies optimization (ECBO) [11], and harmony search (HS) [12].

In the optimization process, strength and serviceability constraints are evaluated by using structural analyses that can be categorized into 2 groups such as linear and nonlinear analyses. Using nonlinear analyses not only captures the nonlinear inelastic behaviors of structures but also yields lighter and more realistic optimum designs [13]. Among several methods for structural nonlinear analysis, the direct design has been favored recently. In the direct design approach, the ultimate load-carrying capacity of the whole system and nonlinear relationship between structural responses and applied loading are captured instead of the individual member check in the member-based design method. Some researches in the literature about direct design and using direct design for structural optimization are Refs. [14–18], among others. However, structural analysis using direct design methods requires much more time-computing compared to linear analysis methods, hence structural optimization using direct design often has an excessive computational effort.

In this study, a robust method for optimization of steel moment frames using a direct design method is introduced. A major advantage of the proposed method is that the time-computing is much more reduced, so the optimization of nonlinear steel frames can be performed with a very large number of objective function evaluations in an acceptable computational time. To do this, a direct design method using beam-column elements is used that saves significant time-computing. Furthermore, an improved DE method is developed using a self-adaptive mutation strategy based on the p-best method, named as EapDE, to enhance the balance between global and local searches. The panel-zone shear deformation is prevented by reinforcement of panel-zones using doubler plates. A five-bay five-story steel moment frame subjected to several load combinations is studied to demonstrate the efficiency of the proposed method.

2. Panel-zone reinforcement method

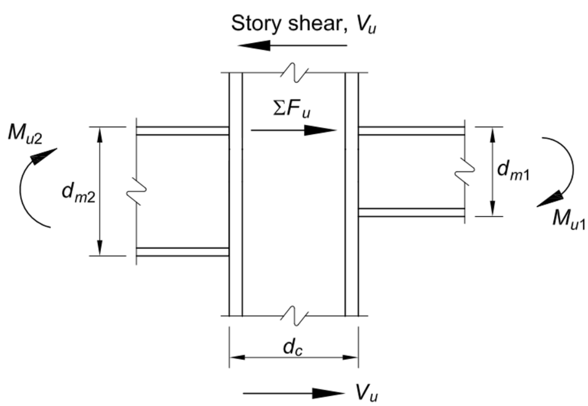


Figure 1. Typical panel-zone area [19]

Considering a typical panel-zone area as presented in Fig. 1. The shear force at the panel-zone is calculated as [20]:

$$\sum F_u = \frac{M_{u1}}{0.95 d_{m1}} + \frac{M_{u2}}{0.95 d_{m2}} - V_u \tag{1}$$

where M_{u1} and M_{u2} are the factored moments on the left and right beams, respectively; V_u is the factored shear force on column; d_{m1} and d_{m2} are the heights of the right and left beams, respectively.

The nominal strength at the panel-zone is calculated as [20]:

$$V_n = 0.60 F_y d_c t_w \qquad \text{if } P_r \le 0.40 P_y \tag{2a}$$

$$V_n = 0.60 F_y d_c t_w \left(1.4 - \frac{P_r}{P_y}\right) \qquad \text{if } P_r > 0.40 P_y \tag{2b}$$

where F_y is the yield strength of steel for the column; d_c and t_w are the height and the thickness of the column web, respectively; P_r and P_y are the axial force and axial yield strength of the column, respectively. P_y is determined as: $P_y = F_y A_g$ where A_g is the cross-sectional area of the column. If $\sum F_u$ is greater than ϕV_n, the panel-zone area will be yielded and the reinforcement design of the panel-zone area is necessary. ϕ is the resistance factor which is equal to 0.9 in this study.

Panel-zones can be designed by using [19]: (i) reinforcing the column web to guarantee the static behaviors for the panel-zone area and so the panel-zone shear deformation is ignored; and, (ii) allowing panel-zone yielded and then the panel-zone shear deformation has to be considered in structural design. In both approaches, the panel-zone reinforcement by using doubler plates or stiffeners requires. However, the first approach is simpler in the analysis but requires thicker doubler plates than the second approach. In this paper, the design of panel-zones using the first approach is used. The total thickness of the required doubler plate(s), t_{plate}, is calculated as follows:

$$t_{plate} = \sum F_u \Big/ \left(\phi 0.60 F_y d_c\right) - t_w \qquad \text{if } P_r \le 0.40 P_y \tag{3a}$$

$$t_{plate} = \sum F_u \Big/ \left(\phi 0.60 F_y d_c \left(1.4 - \frac{P_r}{P_y}\right)\right) - t_w \qquad \text{if } P_r > 0.40 P_y \tag{3b}$$

3. Formulation of the optimization problem

3.1. Objective function

Cost optimization of steel moment frames is defined as the minimization of the total cost of the structure including the cost of beams, columns, and panel-zone reinforcement. The cost of beams and columns is easily predicted by using the unit price of steel and the total weight of these members. However, the cost of panel-zone reinforcement including the material cost of doubler plates and welding cost is highly dependent on the labor cost that is based on the characteristics and location of each structure. For simplicity, Ha et al. [17] proposed an equation to transfer the panel-zone reinforcement cost to structural steel cost based on the current material and labor costs in the USA. The cost of a panel-zone reinforcement can be estimated as [17]

$$T^{panel} = c_{structuralsteel} \times \left(25000 \times t_{plate} \times (h + b) + 7850 \times t_{plate} \times h \times b\right) \text{ (kg)} \tag{4}$$

where h and b are the height and width of the doubler plate(s) with their unit of meter, respectively; $c_{structuralsteel}$ is the steel material price per weight. Assuming that the height of the doubler plate at a panel-zone is equal to the greater value of the heights of the left and right beams. And, the width of the doubler plated is equal to 95% the height of the column web. The cost objective function of the structure is therefore simplified as the following weight function by neglecting the steel price per weight (or assuming $c_{structuralsteel} = 1$):

$$\begin{aligned}\min T(\mathbf{X}) &= W(\mathbf{X}) + W_{panel}(\mathbf{X}) \\ &= \rho \sum_{i=1}^{nm} \left(A(x_i) \sum_{q=1}^{n_i} L_q \right) + \sum_{j=1}^{np} \left(25000 \times t_{plate,j} \times \left(h_j + b_j \right) + 7850 \times t_{plate,j} \times h_j \times b_j \right) \\ \mathbf{X} &= (x_1, x_2, \ldots, x_{nm}), \quad x_i \in [1, UB_i]\end{aligned} \tag{5}$$

where $W(\mathbf{X})$ and $W_{panel}(\mathbf{X})$ are the total weight of the beams and columns and the reinforcement cost of panel-zones, respectively; $\mathbf{X}$ is the vector of design variables which are the integer values representing the sequence numbers of the cross-section types used for the beams and column in the variable space; UB_i is the number of W-shaped sections available for the i^{th} group of beams and columns; ρ is the specific weight of steel; n_i is the number of frame members in the i^{th} group; $A(x_i)$ is the cross-section of the i^{th} design variable; and, L_q is the fabricated length of member q in the i^{th} group; np is the number of reinforced panel-zones. Note that, the length of a beam is the distance between two column nodes but not include the column height.

3.2. Constraints

In this study, constructability constraints include the provisions at column-to-column connections so that the height of the upper column segment must not be larger than the lower column segment. Besides, at the beam-to-column connections, the width of the beam flanges should not be greater than the width of the column flange. If the beam is connected to the column web, the width of the beam flange should not be greater than the height of the column web. These conditions are formulated as follows:

$$C_{i,1}^{con}(\mathbf{X}) = \left(\frac{D_c^{uppercolumn}}{D_c^{lowercolumn}} \right)_i - 1 \le 0, \quad i = 1, \ldots, n_{c-c} \tag{6a}$$

$$C_{i,2}^{con}(\mathbf{X}) = \left(\frac{b_{bf}}{b_{cf}} \right)_i - 1 \le 0, \quad i = 1, \ldots, n_{b-c1} \tag{6b}$$

$$C_{i,3}^{con}(\mathbf{X}) = \left(\frac{b_{bf2}}{T_c} \right)_i - 1 \le 0, \quad i = 1, \ldots, n_{b-c2} \tag{6c}$$

in which n_{c-c}, n_{b-c1} and n_{b-c2} are the connection numbers of column-to-column, beam-to-column flange, and beam-to-column web, respectively; $D_c^{uppercolumn}$ and $D_c^{lowercolumn}$ are the upper- and lower-column segment depths at a column-to-column joint, respectively; b_{cf} and b_{bf} are the flange widths of the column and beam at a beam-to-column flange joint, respectively; b_{bf2} and T_c are the beam flange width column web height at a beam-to-column web joint.

In this paper, the strength constraint of the frame subjected to the j^{th} strength load combination is evaluated by using direct design as presented as follows:

$$C_j^{str}(\mathbf{X}) = 1 - \frac{R_j}{S_j} \le 0, \quad j = 1, \ldots, n_{str} \tag{7}$$

where R_j and S_j are the structural load-carrying capacity and the factored loads. The ratio R_j/S_j is called the structural ultimate load factor.

The serviceability constraints include the lateral drift for the top story sway and inter-story drifts for each floor that are formulated as

$$C_k^{drift}(\mathbf{X}) = \left|\frac{D_k}{D_k^u}\right| - 1 \leq 0, \quad j = 1, \ldots, n_{str}, \quad k = 1, \ldots, n_{ser} \tag{8a}$$

$$C_k^{int,l}(\mathbf{X}) = \left|\frac{d_k^l}{d_k^{u,l}}\right| - 1 \leq 0, \quad l = 1, \ldots, n_{story}, \quad k = 1, \ldots, n_{ser} \tag{8b}$$

where D_k and D_k^u are the lateral drift of the top story and its allowable value, respectively; d_k^l and $d_k^{u,l}$ are the inter-story drift of the l^{th}story and its allowable value, respectively; n_{story} is the number of structural stories; and, n_{ser} is the number of the considered serviceability load combinations.

3.3. Constraint handling using the penalty function method

The above-constrained optimization problem can be transformed into an unconstrained optimization problem by using the penalty function method as follows:

$$T_{uncons}(\mathbf{X}) = W(\mathbf{X}) \times (1 + \alpha_{con}\beta_1 + \alpha_{str}\beta_2 + \alpha_{ins}\beta_3) + W_{panel}(\mathbf{X}) \tag{9a}$$

where

$$\begin{aligned} \beta_1 &= \sum_{j=1}^{n_{con}} \left(\max\left(C_{i,1}^{con}, 0\right) + \max\left(C_{i,2}^{con}, 0\right) + \max\left(C_{i,3}^{con}, 0\right)\right) \\ \beta_2 &= \sum_{j=1}^{n_{str}} \left(\max\left(C_j^{str}, 0\right)\right) \\ \beta_3 &= \sum_{k=1}^{n_{ser}} \left(\max\left(C_k^{drift}, 0\right) + \sum_{l=1}^{n_{story}} \max\left(C_k^{int,l}, 0\right)\right) \end{aligned} \tag{9b}$$

in which α_{con}, α_{str}, and α_{ins} are the penalty parameters of the geometric constructability, strength, and inter-story drift constraints, respectively.

4. Improved DE algorithm

The DE, a population-based metaheuristics algorithm, was proposed by Storn and Price [10] in 1997. Up to now, many modified versions of DE have been developed in the literature and prove this algorithm as one of the most efficient methods and is suitable for solving various optimization problems. Regarding the optimization of steel frames, the authors and the colleague introduced a new and efficient DE-based method in 2020, named as mEpDE [17]. Compared to the conventional DE method, mEpDE has several improvements such as (i) using a new mutation strategy based on the p-best method to balance the local and global searches; (ii) Developing the multi-comparison technique (MCT) to efficiently reduce the number of structural analysis calls for evaluating the strength and serviceability constraints; (ii) Developing the Promising Individual Method (PIM) that effectively chooses trial individuals; (iv) Avoiding repetitive same individual evaluations by using a matrix to contain all evaluated individuals. Numerical results provided in Ref. [17] showed the robustness of mEpDE compared to several new and efficient metaheuristic algorithms for steel frame optimization. However, in this study, we will use a self-adaptive mutation strategy based on the p-best method that can improve the performance of mEpDE for optimization of steel moment frames. Other techniques remain the same as implemented in mEpDE. The new optimization method is named as EapDE.

In the conventional DE method, 'DE/rand/1' and 'DE/best/1' are two common mutation strategies that have opposite effects in the balance of global and local searches of the optimization. Specifically, the trial individual is generated based on a random individual and the best individual corresponding to using 'DE/rand/1' and 'DE/best/1'. Therefore, 'DE/rand/1' is better at global exploration but converges more slowly compared to 'DE/best/1'. To take advantage of these methods, the 'DE/pbest/1' strategy is used in the mEpDE method where p for the k^{th} iteration of the optimization process is calculated as

$$p\,(k) = A \times nm^{\left(-B \times \frac{k-1}{total\_iteration-1}\right)} \tag{10}$$

where A and B are predefined parameters; $total\_iteration$ is the predefined value for the maximum number of iterations. In the 'DE/pbest/1' strategy, the trial individual is generated based on a random individual in the top $100p\%$ ($p \in (0, 1]$) of the current population. Furthermore, from Eq. (10) we have $p\,(1) = A$ so A is the parameter to control the number of the best individuals used at the beginning of the optimization process. And, if B increases the decline of p increases. Hence, B is used to control the decline speed of the number of the best individuals used. Besides that, if A and B are equal to 1.0, 'DE/rand/1' and 'DE/best/1' are used at the beginning and the end of the optimization, respectively. Eq. (10) is an approach where the value p is predefined without considering the population characteristics and their changes in the optimization process. It should be noted that the diversity and convergence of the population can be predicted based on the change values of the individuals in the population. Many indicators representing the diversity of the population are developed in the literature, for example [21]:

$$DI_{(t)} = \frac{1}{NP}\sum_{k=1}^{NP}\sqrt{\sum_{i=1}^{D}\left(\frac{x_{k,i} - x_{C,i}}{x_i^{UB} - x_i^{LB}}\right)^2}, \quad x_{C,i} = \frac{1}{NP}\sum_{k=1}^{NP} x_{k,i} \tag{11}$$

where NP is the number of individuals in the population; D is the number of design variables; $x_{k,i}$ is the value of the design variable i^{th} of the individual k^{th}; x_i^{UB} and x_i^{LB} are the upper- and lower-bounds of the design variable i^{th}; and, $DI_{(t)}$ is defined as the diversity index of the population at the k^{th} iteration. DI_t represents the individual distribution around the center of the current population. If $DI_{(t)}$ is great, we can guess that the individuals are still highly dispersed, so maintenance of the diversity of individuals is preferred or large p value should be used and vice versa. In light of this, the following equation is used to calculate p [21]:

$$p = \frac{1}{NP} + \left(1 - \frac{1}{NP}\right) \times \frac{DI_{(t)}}{DI_{(0)}} \tag{12}$$

5. Case study

In this section, a five-bay five-story steel moment frame with the geometry presented in Fig. 2 is studied to demonstrate the efficiency of the proposed method. The initial story out-of-plumbness is $1/500$. The initial imperfection of beams is not considered. The steel material used for the whole structure is ASTM A992 with the elastic modulus of $E = 200$ GPa, the yield stress of $F_y = 344.7$ MPa and the weight per unit volume of 7,850 kg/m^3. Doubler plates are reinforced using 4 thicknesses such as 3/16 inches (4.7625 mm), 3/8 inches (9.525 mm), 5/8 inches (15.875 mm), and 1 inches (25.4 mm).

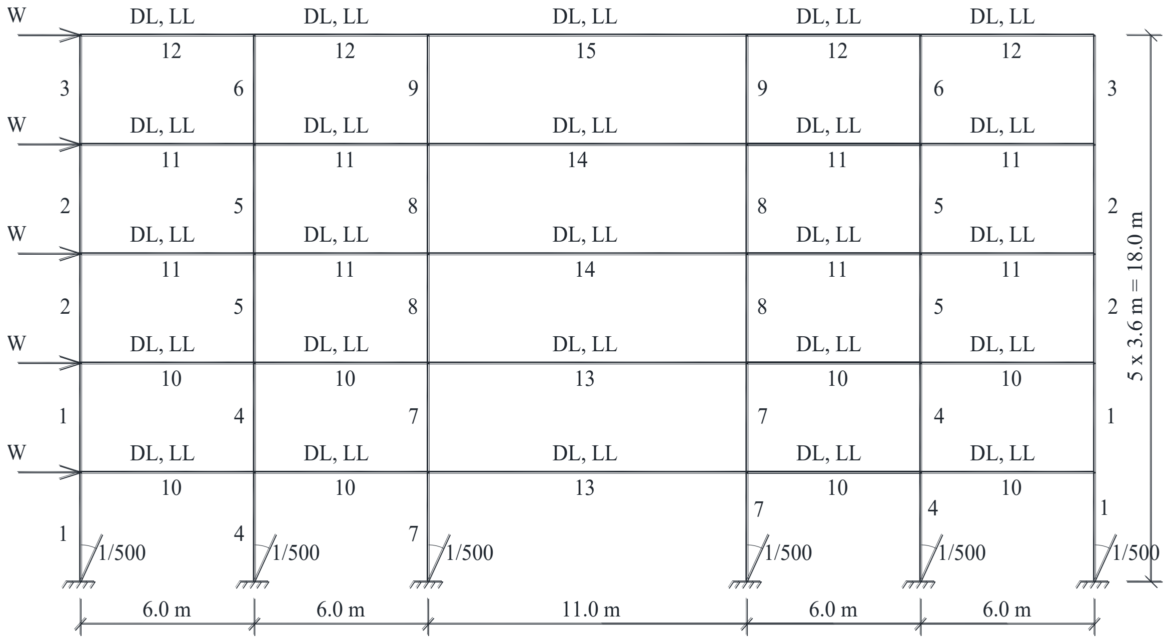


Figure 2. Five bay-five story steel frame [17]

The dead load (DL), live load (LL) and wind load (W) as presented in Fig. 2 are equal to 35 kN/m, 25 kN/m and 28 kN, respectively.

The columns and beams are grouped into 15 cross-sections where 267 sections from W10–W44 of AISC-LRFD are used for the beam members and 158 sections from W12, W14, W18, W21, W24, and W27 are used for the column members. Two strength load combinations: ($1.2DL + 1.6LL$) and ($1.2DL + 1.6W + 0.5LL$) and one serviceability load combination ($1.0DL + 0.7W + 0.5LL$) are considered. The allowable inter-story drift is $h/400$, where h is the frame story height. There are a total of 21 constraints considered including 18 constructability constraints, 2 strength constraints, and 1 serviceability constraint.

To demonstrate the efficiency of the proposed method, only the mEpDE method is employed for comparison since mEpDE is much better than several new and efficient optimization methods for the optimization of steel frames as provided in Ref. [17]. The parameters used for the proposed method and mEpDE are: $NP = 25$, $max\_iteration = 4000$; $A = 1.0$; $B = 1.0$; scale factor $F = 0.7$; crossover rate CR is randomly generated in the range (0,1). The termination of the optimization process is defined as the best objective function is not improved in 1,000 consecutive iterations or the number of iteration reaches *max_iteration*. The strength and serviceability constraints are evaluated by using the PAAP program, a robust direct design program for steel structures [22].

Table 1 presents the best optimum designs obtained by using the proposed method (EapDE) and mEpDE, where 20 optimization runs are performed for each case. As can be seen in this table, the EapDE yields the best optimum design with a total weight of the frame of 18,566 kg, which is smaller than one of mEpDE with 18,687 kg. The worst weight of the optimum design of 19,073 kg by using EapDE is also smaller than 19,149 kg of mEpDE. This means that EapDE can find a better optimum design of the frame than mEpDE. The required structural analyses of EapDE are only 22,733 that is smaller than 20,462 of mEpDE. The reason is that, in the EapDE method, the p value is changed ac-

Table 1. Optimization results of five bay-five story steel frame

Element group of best design	EapDE	mEpDE
1	W18 × 40	W18 × 35
2	W18 × 40	W14 × 30
3	W12 × 26	W12 × 26
4	W24 × 62	W24 × 68
5	W24 × 55	W24 × 55
6	W24 × 55	W24 × 55
7	W27 × 114	W27 × 102
8	W24 × 62	W24 × 62
9	W24 × 55	W24 × 62
10	W12 × 22	W14 × 22
11	W14 × 22	W16 × 26
12	W16 × 26	W16 × 26
13	W21 × 44	W18 × 46
14	W24 × 55	W24 × 55
15	W21 × 57	W24 × 55
Best weight (kg)	18,566	18,730
Beams weight (kg)	7,656	7,961
Columns weight (kg)	9,491	9,115
Panel cost of the best design (kg)	1,419	1,654
Normalized constraint evaluation of $(1.2DL + 1.6LL)$	1.0058	1.0042
Normalized constraint evaluation of $(1.2DL + 1.6W + 0.5LL)$	1.3909	1.4314
Normalized constraint evaluation of $(1.0DL + 0.7W + 0.5LL)$	0.6354	0.6238
Worst weight (kg)	19,073	19,149
Avg. weight (kg)	18,707	18,872
Avg. number of structural analysis	22,733	20,462
Avg. computational time (hour)	6.2	6.2

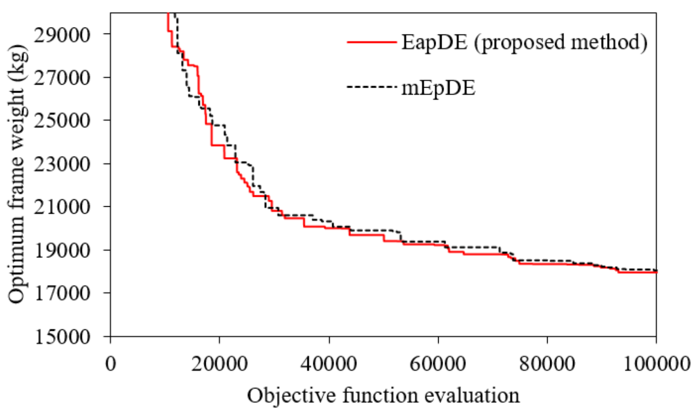
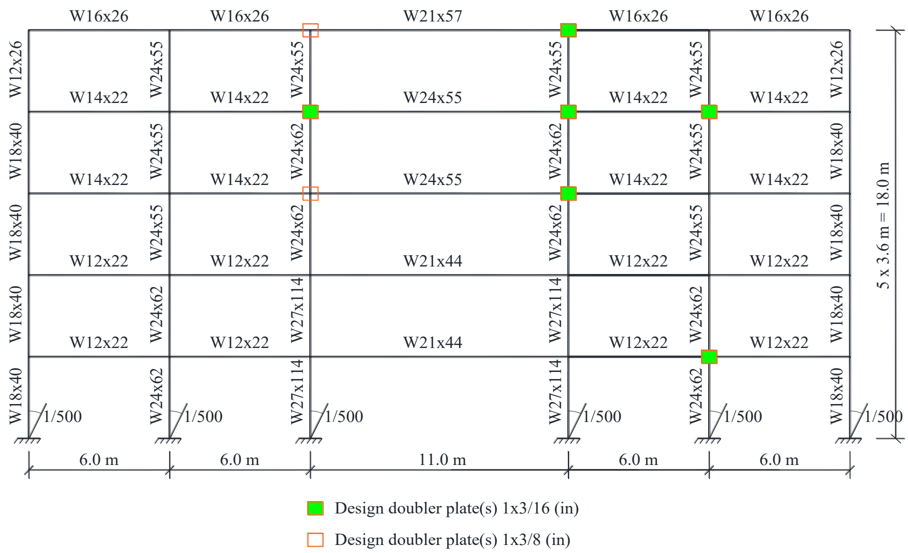
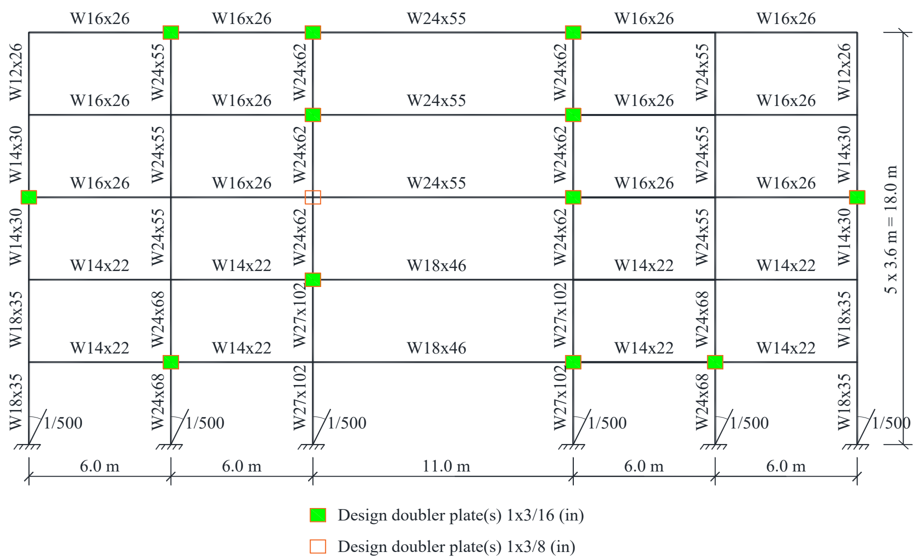


Figure 3. Convergence histories of best optimum designs

cording to the convergence speed of the population. Therefore, the diversity of the population remains better than ones of mEpDE where the p value is predefined as discussed in Section 4. It is also should be noted that with three load combinations considered, the total number of structural analyses for this optimization problem is 300,000. Therefore, the time-computing of both methods is only about 6.2 hours although the total objective function evaluations of 300,000 are very great. This means that both EapDE and mEpDE efficiently reduce the number of required structural analyses. Furthermore, Fig. 3 presents the convergence histories of the best optimum designs of EapDE and mEpDE. As can be seen in this figure, the convergence speeds of the two methods are almost the same. Besides that, Fig. 4 shows the panel-zone reinforcement of the best optimum design of two methods.



(a) Using the EapDE method



(b) Using the mEpDE method

Figure 4. Best optimum design of five bay-five story steel frame

6. Conclusions

An efficient method for optimizing steel moment frames with the panel-zone design using doubler plate(s) was successfully developed in this work. In the proposed method, a direct design method using beam-column elements is used to model the structure that significantly reduces the computational cost. An improved DE method is developed using a self-adaptive mutation strategy based on the p-best method to enhance the balance between global and local searches. Numerical results of the optimization of a five-bay five-story steel moment frame subjected to several load combinations prove that the proposed method not only can find better the optimum design of the structure but also efficiently saves the computational efforts.

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References

[1] Hayalioglu, M. S., Degertekin, S. O. (2005). Minimum cost design of steel frames with semi-rigid connections and column bases via genetic optimization. *Computers & Structures*, 83(21-22):1849–1863.

[2] Maheri, M. R., Shokrian, H., Narimani, M. M. (2017). An enhanced honey bee mating optimization algorithm for design of side sway steel frames. *Advances in Engineering Software*, 109:62–72.

[3] Le, L. A., Bui-Vinh, T., Ho-Huu, V., Nguyen-Thoi, T. (2017). An efficient coupled numerical method for reliability-based design optimization of steel frames. *Journal of Constructional Steel Research*, 138: 389–400.

[4] Gholizadeh, S., Baghchevan, A. (2017). Multi-objective seismic design optimization of steel frames by a chaotic meta-heuristic algorithm. *Engineering with Computers*, 33(4):1045–1060.

[5] Lieu, Q. X., Do, D. T. T., Lee, J. (2018). An adaptive hybrid evolutionary firefly algorithm for shape and size optimization of truss structures with frequency constraints. *Computers & Structures*, 195:99–112.

[6] Truong, V.-H., Kim, S.-E. (2018). Reliability-based design optimization of nonlinear inelastic trusses using improved differential evolution algorithm. *Advances in Engineering Software*, 121:59–74.

[7] Ha, M.-H., Vu, Q.-A., Truong, V.-H. (2018). Optimum design of stay cables of steel cable-stayed bridges using nonlinear inelastic analysis and genetic algorithm. *Structures*, 16:288–302.

[8] Pham, H. A. (2016). Truss optimization with frequency constraints using enhanced differential evolution based on adaptive directional mutation and nearest neighbor comparison. *Advances in Engineering Software*, 102:142–154.

[9] Erol, O. K., Eksin, I. (2006). A new optimization method: big bang–big crunch. *Advances in Engineering Software*, 37(2):106–111.

[10] Storn, R., Price, K. (1997). Differential evolution–a simple and efficient heuristic for global optimization over continuous spaces. *Journal of Global Optimization*, 11(4):341–359.

[11] Kaveh, A., BolandGerami, A. (2017). Optimal design of large-scale space steel frames using cascade enhanced colliding body optimization. *Structural and Multidisciplinary Optimization*, 55(1):237–256.

[12] Lee, K. S., Geem, Z. W. (2004). A new structural optimization method based on the harmony search algorithm. *Computers & Structures*, 82(9-10):781–798.

[13] Yang, I.-T., Hsieh, Y.-H. (2013). Reliability-based design optimization with cooperation between support vector machine and particle swarm optimization. *Engineering with Computers*, 29(2):151–163.

[14] Truong, V.-H., Nguyen, P.-C., Kim, S.-E. (2017). An efficient method for optimizing space steel frames with semi-rigid joints using practical advanced analysis and the micro-genetic algorithm. *Journal of Constructional Steel Research*, 128:416–427.

[15] Kim, S.-E., Truong, V.-H. (2020). Reliability evaluation of semirigid steel frames using advanced analysis. *Journal of Structural Engineering*, 146(5):04020064.

[16] Truong, V.-H., Kim, S.-E. (2018). A robust method for optimization of semi-rigid steel frames subject to seismic loading. *Journal of Constructional Steel Research*, 145:184–195.

[17] Ha, M.-H., Vu, Q.-V., Truong, V.-H. (2020). Optimization of nonlinear inelastic steel frames considering panel zones. *Advances in Engineering Software*, 142:102771.

[18] Hung, T. V., Viet, V. Q., Thuat, D. V. (2019). A deep learning-based procedure for estimation of ultimate load carrying of steel trusses using advanced analysis. *Journal of Science and Technology in Civil Engineering (STCE)-NUCE*, 13(3):113–123.

[19] ANSI/AISC 360-10 (2010). *Specification for structural steel buildings*. Chicago (IL): American Institute of Steel Construction.

[20] Liew, J. R., Chen, W.-F. (1995). Analysis and design of steel frames considering panel joint deformations. *Journal of Structural Engineering*, 121(10):1531–1540.

[21] Pham, A. H. (2016). Discrete optimal sizing of truss using adaptive directional differential evolution. *Advances in Computational Design*, 1(3):275–296.

[22] Thai, H.-T., Kim, S.-E. (2009). Practical advanced analysis software for nonlinear inelastic analysis of space steel structures. *Advances in Engineering Software*, 40(9):786–797.