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Research paper

Optimization of nonlinear inelastic steel frames considering panel zones

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A R T I C L E I N F O

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A B S T R A C T

In this article, an efficient methodology is developed to optimize nonlinear steel frames under several load combinations. For that purpose, inelastic advanced analyses of steel frames are performed using plastic hinge beam–column elements to reduce computational efforts. An improved differential evolution (DE) algorithm is utilized as a global optimizer to refine the solution accuracy and enhance the convergence speed. Compared to the conventional DE algorithm, this newly developed method provides four major improvements such as: (1) a new mutation strategy based on the p-best method; (2) the multi-comparison technique (MCT) to decrease the number of unnecessary objective function evaluations; (3) a promising individual method (PIM) to choose trial individuals; and (4) a trial matrix containing all evaluated individuals to avoid objective function evaluations of duplicate individuals. Furthermore, panel zones are taken account of optimum design for the first time. Doubler plates are designed to prevent panel-zone shear deformations. Three mid- to large-size steel frames considering several load combinations required by AISC-LRFD are considered. Five new and efficient meta-heuristic algorithms are employed for comparison.

1. Introduction

Optimal solutions have been prioritized in design of steel frames since they save resources, money, materials, and time, while the structural performance is still guaranteed. Normally, design optimization of a steel frame is to minimize the total cost or weight of the structure subjected to various complex constraints including constructability, serviceability, and strength conditions and discrete design variables of the beam and column cross-sections (e.g., the W-shaped section list in AISC-LRFD [1]). Owing to these issues, design optimization of steel frames is highly nonlinear and multimodal. Finding optimal or even sub-optimal solutions is hence difficult. Normally, sufficiently good solutions, that are close to being optimal but not the “real” optimums, are acceptable. In light of this, meta-heuristic algorithms, which are also known as non-gradient-based ones, are preferable to gradient-based ones [2]. Indeed, these approaches use stochastic searching techniques to randomly choose potential solutions in a given search space, hence they are completely free from sensitivity analyses regarding derivatives of the objective function and constraints with respect to each of all design variable. Moreover, they require less mathematical knowledge. As a result, these methods are easy to implement and effective in finding global optimal solutions for optimization problems with highly nonlinear and non-convex properties. Nonetheless, since an optimum solution must be searched over the whole design domain without any directional information as that of derivative algorithms, their computational cost is often time-consuming and relatively expensive. Many improvements have been therefore proposed to deal with these shortcomings. The results of recent studies have confirmed that meta-heuristic algorithms work well for various structural optimization problems, including the sizing and topology optimization of truss structures [3–6], optimization of rigid and semi-rigid steel frames [7–11], reliability-based design optimization of structures [12–15], and optimization of steel frames under seismic loading [16–23]. In addition, many meta-heuristic algorithms have been developed, such as the harmony search (HS) [24], firefly algorithm (FA) [25], enhanced colliding bodies optimization (ECBO) [26], differential evolution (DE) [27], and big bang–big crunch (BB–BC) [28]. The reviews of several metaheuristic algorithms can be found in Refs. [29, 30].

Based on the structural analysis method, published steel frame optimization studies in the literature can be divided into two categories: (1) using linear analyses [9,15,26], and (2) using nonlinear analyses [31–36]. Compared to linear analyses, nonlinear analyses can estimate the nonlinear inelastic behaviors of the structure, resulting in optimum designs that are more realistic and lighter [13]. However, optimum design of a steel frame using nonlinear analysis is a problem involving

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high computational cost because optimization approaches using meta-heuristic algorithms requires lots of structural analyses while the computing time for a nonlinear analysis is much greater than for a linear analysis. This fact is more notable in the case of the optimization of real steel frames where several load combinations are considered since the number of structural analysis is much greater than when only one load combination is considered. Due to the highly computational efforts, in most published studies of steel frame optimization using nonlinear analysis, the authors have tried to develop optimization methods which have fast convergence and then verify these methods using small case studies and a small number (often < 10,000) of objective function evaluations [7, 8,37–39].

Panel zone is the column web area located at a beam-to-column connection. Many works concerning behaviors of panel zones in the literature have proved that high shear forces can develop in the panel zone, which can cause columns to yield before the initial yielding point of the structure [40–42]. This phenomenon is more likely to happen in an optimum steel frame design since cross-sections of beams and columns are minimized to save material. Therefore, the design of panel zones should be considered in the steel frame optimization. Unfortunately, to the best knowledge of authors, design of panel zones has not been considered in steel frame optimization problems.

A panel zone can be designed according two following methods provided by AISC-LRFD [1]: (1) limiting panel-zone behavior to the elastic range or the effect of panel zone deformation on frame stability is not considered, and (2) allowing panel zone yielding or panel-zone shear yielding effects are considered. In both cases, the reinforcement of panel zones by adding doubler plates or stiffeners is often used. However, compared to the second method the first method is simpler in analysis but often requires thicker doubler plates or stiffeners which increases material and welding costs. The above mentions imply that total cost of the panel zones needs to be added into the objective functions of the optimization. Total cost of a panel zone is dependent on the column web height and beam depth which are variables of the optimization. Adding panel zone design in the optimization will therefore generate another optimization problem that seems more complicated.

This study proposes an efficient method that can perform the optimization of nonlinear steel frames considering several load combinations. A nonlinear inelastic analysis using beam-column elements is employed for the structural analysis, which can significantly reduce the computational costs compared to that utilized in existing commercial software such as ABAQUS, ANSYS, etc. For optimizer, a modified version of the efficient p-best differential evolution (EpDE) method [43] is developed which is named as mEpDE. In mEpDE some improvements are made to improve the performance of EpDE such as: (1) a simple but efficient method for choosing trial individuals is proposed and named as Promising Individual Method (PIM), and (2) a trial matrix is used to contain all individuals that have been already evaluated to avoid repetitive objective function evaluations of the same individual. Three mid- to large-scale steel frames are optimized to demonstrate the robustness of the proposed method. Some new and efficient meta-heuristic algorithms are used for comparison, such as enhanced colliding body optimization (ECBO) [26], improved differential evolution (IDE) [15], adaptive harmony search (AHS) [44], and micro-genetic algorithm (micro-GA) [8]. Another contribution of this study is the consideration of the design of panel zones in the optimization through the use of doubler plates in the column webs to prevent panel-zone shear deformations.

2. Optimization problem formulation

In this section, the formulation of optimizing nonlinear steel frames considering several load combinations is presented. The design variables are the cross-sections of the beams and columns, which are selected from a W-shaped section list provided by AISC-LRFD [1]. The objective function is to minimize the total cost of structural members, including beams, columns, and panel zones. The constraints include constructability, serviceability, and strength conditions. The beam-to-column connections are assumed to be fully rigid. The details of the objective function and constraints for such an optimization problem are presented below.

2.1. Objective function

The objective function of the optimization problem is the total cost of beams, columns, and panel zones that can be formulated as

$$\begin{aligned} Min \quad & T(\mathrm{X}) = T_{beam\&column}(\mathrm{X}) + T_{panel-zone}(\mathrm{X}), \\ & \mathrm{X} = (x_1, x_2, \ldots, x_{nm}),\ x_i \in [1, UB_i], \end{aligned} \tag{1}$$

where $T_{beam\&column}$ is the total cost of beams and columns; $T_{panel\ -\ zone}$ is the total cost of panel zones; nm is the number of design variables that are the cross-sections of the beams and columns; x_i is an integer value in $[1, UB_i]$ representing the sequence number of the ith design variable in the variable space; UB_i is the number of W-shaped sections available for the ith design variable. $T_{beam\&column}$ is determined as

$$\begin{aligned} T_{beam\&column}(X) &= c_{structuralsteel} W(X) \\ &= c_{structuralsteel} \rho \sum_{i=1}^{nm} \left(A(x_i) \sum_{q=1}^{n_i} L_q \right), \end{aligned} \tag{2}$$

where $c_{structuralsteel}$ is the price per weight of steel material; W(X) is the total weight of the beams and columns; ρ is the specific weight of steel; n_i is the number of frame members in the ith group; $A(x_i)$ is the cross-section of the ith design variable; and, L_q is the length of member q in the ith group.

The total cost of panel zones $T_{panel\ -\ zone}$ is determined as

$$T_{panel-zone}(X) = \sum_{j=1}^{np} T_j^{panel}(X), \tag{3}$$

where np is the number of panel zones needed to design the doubler plates; and, T_j^{panel} is the cost of doubler plate(s) at the jth panel zone. It should be noted that T_j^{panel} including the cost of doubler plates, welding cost including materials and labor can only be calculated exactly based on the real construction costs. Welding cost is especially dependent on the welding thickness because a greater welding thickness requires not only larger welding material volume but also longer time to complete. In fact, the relationship of welding leg and welding cost is not linear. However, for simplicity, in this study we assume this relationship is linear. Furthermore, $T_j^{panel}$is estimated by using the current prices of steel material and welding labor in the United States as following. The cost of structural steel, $c_{structuralsteel}$, is about 0.8 (USD/kg) and the welding cost including material and labor is 40 (USD) per 1 (m) of the welding length with the welding leg about 4 (mm). This means that the total cost of welding having the leg of 4 (mm) can be represented by (50 × $c_{structuralsteel}$ × welding length) kilogram of structural steel. In addition, welding leg is equal to the thickness of the doubler plate. Therefore, T_j^{panel} is calculated as

$$\begin{aligned} T_j^{panel}(X) = &\ \text{cost of welding+cost of doubler plates material} \\ = &\ c_{\text{structuralsteel}} \times (25000 \times t_j \times (h_j + b_j) + 7850 \times t_j \times h_j \times b_j)\ (kg), \end{aligned} \tag{4}$$

where: t_j, h_j, and b_j are the thickness, height, and width of the designed doubler plate at the $j$thpanel zone, respectively. Note that the unit of t_j, h_j, and b_j for using Eq. (4) is meter. In this study, the height and width of the doubler plate are assumed to equal to the web heights of the beam and column, respectively.

The optimization presented in Eq. (1) can be rewritten as

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

2.2. Constraints

2.2.1. Constructability constraints

Owing to practical requirements, at each column-to-column connection, the section depth of the lower column must be greater than that of the upper column. Also, at each beam-to-column connection, the flange width of the beam must be smaller than that of the column if the beam is connected to the column flange. If the beam is connected to the column web, the flange width of the beam must be smaller than the web height of the column. These constructability constraints are formulated as follows:

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

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

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

in which n_{c-c} is the number of column-to-column connections; n_{b-c1} is the number of beam-to-column connections where the beam is connected to the column flange; n_{b-c2} is the number of beam-to-column connections where the beam is connected to the column web; $D_c^{uppercolumn}$ and $D_c^{lowercolumn}$ are the depths of upper- and lower- columns at a column-to-column connection; b_{cf} and b_{bf} are the flange widths of the column and beam, respectively, at a beam-to-column connection where the beam is connected to the column flange; b_{bf2} is the flange width of the beam and T_c is the web height of the column at a beam-to-column connection where the beam is connected to the column web. In addition, Fig. 1 illustrates the beam to column constructability.

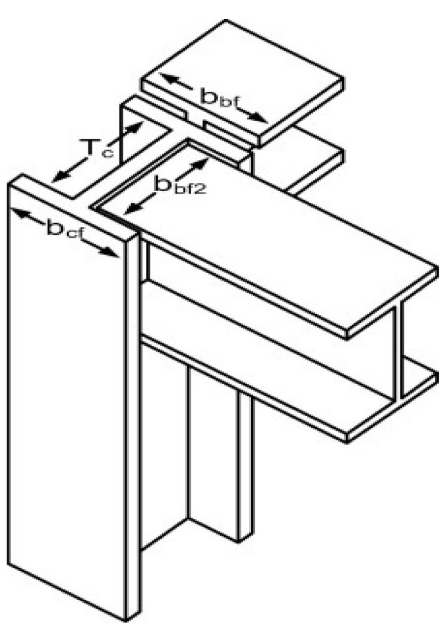


Fig. 1. Constructability of beam-to-column connections.

2.2.2. Strength constraints

The strength constraints of the optimization are used to guaranty the safety of the frame under different strength load combinations provided in design codes such as ASCE/SEI 7–05 [45]. Generally, the strength constraint for the frame under the jth strength load combination can be written as follows:

$$C_j^{str}(\mathrm{X}) = 1 - \frac{R_j}{S_j} \leq 0 \tag{7}$$

where R_j and S_j are the structural load-carrying capacity and the factored loads corresponding to the j^{th} strength load combination, respectively; and, n_{str} is the number of the considered strength load combinations. In Eq. (6), the ratio $\frac{R_j}{S_j}$ is also well-known as the ultimate load factor of the frame subjected to the jth strength load combination that can be directly estimated by using a nonlinear inelastic analysis.

2.2.3. Serviceability constraints

According to AISC-LRFD [1], serviceability requirements include: (1) deflection, drift, and vibration; (2) connection slip; (3) corrosion; (4) camber; and (5) thermal expansion and contraction. In this study, the serviceability constraints limit the drift because deflection commonly affects the serviceability performance of steel frames. The drift constraints consist of lateral drift for the top story sway and inter-story drifts for each floor for the frame subjected to the kth serviceability load combination are defined as follows:

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

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

where D_k and D_k^u are the lateral drift of the top story and its allowable value corresponding to the kth serviceability load combination, 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. The deflections of the frame subjected to a serviceability load combination can be calculated using a nonlinear elastic analysis where all geometric nonlinearities are included in the analysis while the material limits in the elastic region. This condition can be formulated as the load factor of the structure subjected to the kth serviceability load combination at the initial yielding point of the structure, lf_k^{iyp}, greater than 1.0:

$$C_k^{iyp}(\mathrm{X}) = \frac{1}{lf_k^{iyp}} - 1 < 0. \tag{9}$$

2.3. Penalty function method for constraint handling

To solve the afore-exhibited constrained optimization problem, the penalty method is used to transform a constrained optimization problem into an equivalent unconstrained optimization problem as follows:

$$T_{uncons}(\mathrm{X}) = W(\mathrm{X}) \times (1 + \alpha_{con}\beta_1 + \alpha_{str}\beta_2 + \alpha_{iyp}\beta_3 + \alpha_{ins}\beta_4) + W_{panel}(\mathrm{X}), \tag{10.a}$$

where:

$$\begin{aligned} \beta_1 &= \sum_{j=1}^{n_{con}} (\max(C_{i,1}^{con}, 0) + \max(C_{i,2}^{con}, 0) + \max(C_{i,3}^{con}, 0)) \\ \beta_2 &= \sum_{j=1}^{n_{str}} (\max(C_j^{str}, 0)) \\ \beta_3 &= \sum_{k=1}^{n_{ser}} (C_k^{iyp}) \\ \beta_4 &= \sum_{k=1}^{n_{ser}} (\max(C_k^{drift}, 0) + \sum_{l=1}^{n_{story}} \max(C_k^{int,l}, 0)) \end{aligned}, \tag{10.b}$$

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

As presented in Eq. (10), the constraint violations are measured by adding a penalty weight to the structural total weight. The penalty weight is determined using the penalty parameters chosen to be non-zero when constraints are violated and zero when constraints are not violated. By this way, both structural weight and constraint violations are minimized to find the smaller unconstrained objective functions in the optimization process. When no constraints are violated, the value of unconstrained objective function is equal to the value of the constrained objective function. The penalty parameters are usually sufficiently large for making a great penalty weight in order to ensure that infeasible individuals are removed in the optimization process.

2.4. Solution methodology

In the optimization, the computational time for constructability constraints estimation, which are calculated without using structural analyses, is small. Most of computational effort of the optimization process is used for performing time-consuming structural nonlinear analyses to evaluate the strength and serviceability constraints. Normally, the number of the required structural nonlinear analyses, N_{sa}, in an optimization process is calculated as follows:

$$N_{sa} = \text{total\_iteration} \times n_{\text{individual}} \times n_{\text{LoadCombination}}, \tag{11}$$

where *total_iteration* is the iteration number of the optimization; $n_{individual}$ is the number of individuals in the population; and, $n_{LoadCombination}$ is the number of load combinations considered. From Eq. (11), if an optimization problem has 4 considered load combinations, 25 individuals in the population and the total iteration of the optimization process of 10,000, the total required structural analyses are equal to 400,000 which require an excessive computational effort. This implies that optimization of nonlinear steel frames subjected to several load combinations is very computationally expensive. To solve this problem, we use plastic hinge beam–column elements for modelling structures to reduce the time-computing of a structural analysis and develop an improved DE algorithm for optimization that can efficiently decrease the total required structural analyses.

3. Nonlinear inelastic analysis of steel frames

3.1. Modelling of steel frames

In this study, plastic hinge beam–column elements are used for modeling columns and beams since this model has an acceptable accuracy but requires much shorter computational time compared to plastic zone methods [46, 47]. To account for geometric nonlinearity, the stability functions proposed by Chen and Liew [48] are used to capture the second-order effect ($P - \delta$ effect) related to the local deformation associated with the element chord between end nodes. The inelastic effects due to residual stresses of the member subjected to an axial force are captured using the Column Research Council (CRC) tangent modulus concept proposed by Chen and Lui [49]. In the case of both axial force and bending moment, the gradual stiffness degradation model for plastic hinges proposed by Kim and Choi [50] is used to describe the partial plasticization effects. The yield surface proposed by Orbison et al. [51] is employed in the stiffness degradation model as it requires the least number of elements. The effects of transverse shear deformation are also considered through the shear deformation stiffness matrix proposed by Chen et al. [52]. Lateral–torsional buckling effects are directly accounted for in the derivation of the stiffness matrices by using a linearized form of the incremental virtual work equation [53]. Furthermore, the global system imperfections are considered by modelling an initial story out-of-plumbness. Note that, local imperfections are not considered in this study because the cross-sectional areas of the structural members which are chosen from the W-shaped section list in AISC-LRFD [1] are compact. In addition, warping deformation associated with lateral torsional buckling is also not taken into account.

3.2. Nonlinear solution method

To solve the nonlinear equations, the generalized displacement control (GDC) method [54] is employed since it efficiently solves nonlinear problems with multiple critical points. For more detailed information, interested readers are suggested to refer to Ref. [13]. Note that, for the analysis of a structure under strength load combinations, the structural analysis program is set to stop when a predefined number of loading steps is reached. For the serviceability load combinations, the program will stop upon reaching the full-service loads.

3.3. Panel-zone design

Since the nonlinear inelastic analysis method presented above cannot directly account for panel zone deformation, the design of the panel zones in this study utilizes doubler plates extending the depth of the column web to ensure that the panel zones are elastic. With this approach, the effects of panel zone deformation can be acceptably approximated by using the member centerline-to-centerline distance in the structural modeling [55].

From Fig. 2, the shear force acting in the panel zone can be calculated as follows [55]:

$$F_{panel} = \frac{M_{u1}}{0.95d_{b1}} + \frac{M_{u2}}{0.95d_{b2}} - V_u, \tag{12}$$

where M_{u1} and M_{u2} are the factored beam moments, d_{b1} and d_{b2} are the beam depths, and V_u is the factored column shear force.

The nominal strength, R_n, at the panel zone is calculated as follows:

$$R_n = 0.60F_y d_c t_w \ \ for\ \ P_r \le 0.40P_y, \tag{13.a}$$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_y}\right) \text{for}\ P_r > 0.40P_y, \tag{13.b}$$

where F_y is the specified minimum yield stress of the column web; d_c and t_w are the column depth and total thickness of the column web including the thickness of the doubler plates, respectively; P_r and P_y are the design axial force and axial yield strength of the column, respectively. Here, $P_y = F_y A_g$, where A_g is the gross cross-section of the column.

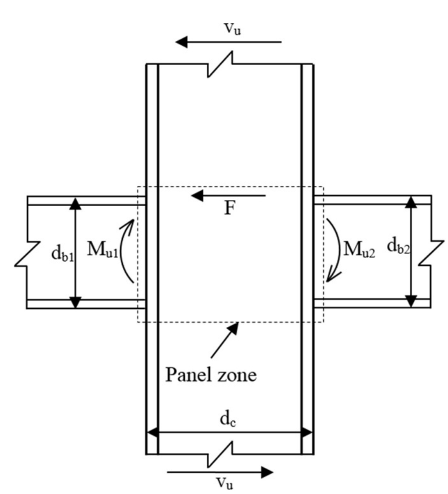


Fig. 2. Panel zone location.

The panel zone design procedure is summarized as follows:

Panel zone design procedure
01: Define all n_{str} considered strength load combinations and n_{ser} considered serviceability load combinations
02: **Do** $i = 1, (n_{str} + n_{ser})$
03: Perform nonlinear inelastic analysis with the ith load combination
04: Get forces at all panel zones of the structure
05: Calculate the shear force F_{panel} acting in panel zones using Eq. (12)
06: Calculate the strength of panel zones R_n using Eq. (13)
07: **If** $(F_{panel} > R_n)$ **then** design doubler plate(s) for the panel zone
08: **End Do**
09: Choose the thickness of the doubler plate(s) at panel zones so that all load combinations considered are satisfied
10: **End**

Note that, it is more economical in practice to have the plate on one side of column web if the required thickness of the plate is less than or equal to one inch (25.4 mm).

4. Improved DE algorithm

To solve highly computational cost problems such optimization of nonlinear steel frames subjected to several load combinations, a modification of the efficient p-best Differential Evolution (EpDE) method proposed by Truong and Kim [43] is developed in this study. The new method is named as mEpDE. Compared to the conventional DE algorithm [27], the EpDE method provides two major improvements: (1) it proposes a new mutation strategy based on the p-best method; and (2) it develops the multi-comparison technique (MCT), which can decrease the amount of unnecessary objective function evaluations. These improvements are summarized in Sections 4.1 and 4.2. In mEpDE some improvements are made to further improve the performance of EpDE such as: (1) a simple but efficient method for choosing trial individuals is proposed and named as Promising Individual Method (PIM), and (2) a trial matrix is used to contain all individuals that have been already evaluated to avoid repetitive objective function evaluations of the same individual. These improvements are presented in Section 4.3.

4.1. New mutation strategy based on the p-best method

Two common methods used for mutation in the conventional DE algorithm are ‘DE/rand/1′ and ‘DE/best/1′. In ‘DE/rand/1′, the trial individual is generated based on a random individual in the current population. Therefore, this method is good at maintaining the global search but poor at local exploitation, and its convergence speed is slow. In contrast, in ‘DE/best/1′, the trial individual is created based on the best individual of the current population. Hence, this method converges faster than ‘DE/rand/1′ and is good at local exploitation, but it can be more easily trapped in local optima. To take advantage of these two methods, the mutation strategy ‘DE/*p*best/1′ is used in EpDE. In ‘DE/*p*best/1′, the trial individual U is created as follows:

$$U = X_{pbest} + F \times (X_1 - X_2), \tag{14}$$

where X_{pbest} is a random individual in the top 100p% ($p \in (0, 1]$) of the current population, F is a scale factor, and X_1 and X_2 are two other random individuals in the current population. Obviously, ‘DE/rand/1′ and ‘DE/best/1′ are simply ‘DE/*p*best/1′ with p values of 1.0 and $1/NP$, respectively, where NP is the number of individuals in the population. Furthermore, if the value of p is decreased, the convergence speed and local exploitation are improved, but the global exploration may be reduced, and the optimization is easily trapped at local optima.

It should be noted that in the early stage of the optimization process, the individuals are often highly dispersed, so maintenance of the diversity of individuals is preferred. In the later optimization stages when the convergence of the individuals increases significantly, a fast convergence speed is prioritized. Thus, the value of p for the k^{th} iteration of the optimization is calculated as

Table 1
Parameters for optimization methods.

	One bay-ten story steel frame	Five bay-five story steel frame	Three bay-ten story steel frame
Termination	Presented in Section 5	Presented in Section 5	Presented in Section 5
Proposed method (mEpDE) and EpDE	Population = 25 Iteration = 4000 A = 1.0; B = 1.0 F = 0.7; CR = Rand(0,1)	Population = 25 Iteration = 4000 A = 1.0; B = 1.0 F = 0.7; CR = Rand(0,1)	Population = 25 Iteration = 4000 A = 1.0; B = 1.0 F = 0.7; CR = Rand(0,1)
ECBO	Population = 20 Iteration = 5000 Colliding Memory = 2 Pro-parameter = 0.3	Population = 20 Iteration = 5000 Colliding Memory = 2 Pro-parameter = 0.3	Population = 30 Iteration = 3333 Colliding Memory = 2 Pro-parameter = 0.3
IDE	Population = 25 Iteration = 4000 F = 0.8 CR = 0.9	Population = 25 Iteration = 4000 F = 0.8 CR = 0.9	Population = 30 Iteration = 3333 F = 0.8 CR = 0.9
AHS	HMS = 50 $HMCR$ = 0.8 PAR = 0.4 $MaxItr$ = 100,000	HMS = 50 $HMCR$ = 0.8 PAR = 0.4 $MaxItr$ = 100,000	HMS = 100 $HMCR$ = 0.8 PAR = 0.4 $MaxItr$ = 100,000
Micro-GA	Population = 10 Iteration = 10,000 Mutation: No Crossover: Uniform with crossover rate = 0.5 Elitism scheme: Yes	Population = 10 Iteration = 10,000 Mutation: No Crossover: Uniform with crossover rate = 0.5 Elitism scheme: Yes	Population = 10 Iteration = 10,000 Mutation: No Crossover: Uniform with crossover rate = 0.5 Elitism scheme: Yes

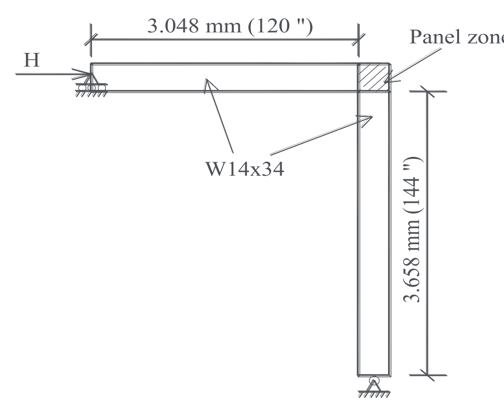


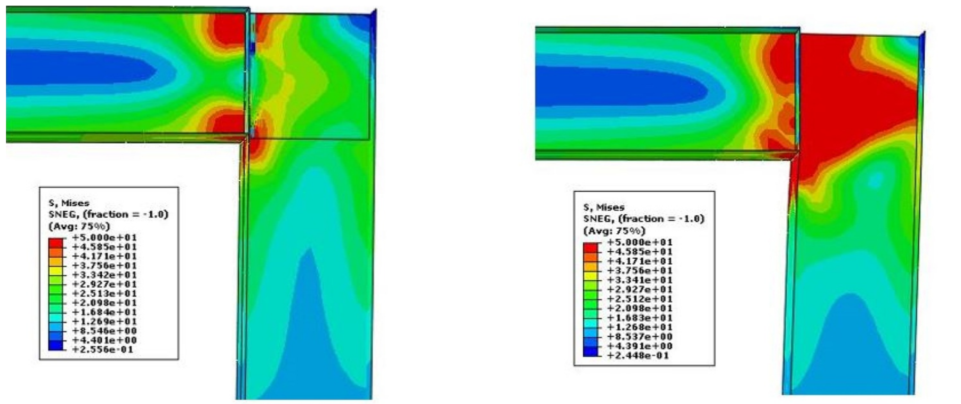
Fig. 3. Two-bar steel frame.

$$p(k) = A \times nm^{\left(-B \times \frac{k-1}{total\_iteration-1}\right)}, \tag{15}$$

in which A and B are predefined parameters. Obviously, $p(1)$ is equal to A, so parameter A plays the role of controlling the number of the best individuals used at the beginning of the optimization process. In addition, if B increases the decline of p increases. Therefore, parameter B plays the role of controlling the speed of decline in the number of the best individuals in the optimization process. It should be noted that the conventional ‘DE/pbest/1′ method corresponds to a case in which $B = 0$. In addition, if A and B are equal 1.0, ‘DE/rand/1′ and ‘DE/best/1′ are used at the beginning and at the end of the optimization, respectively.

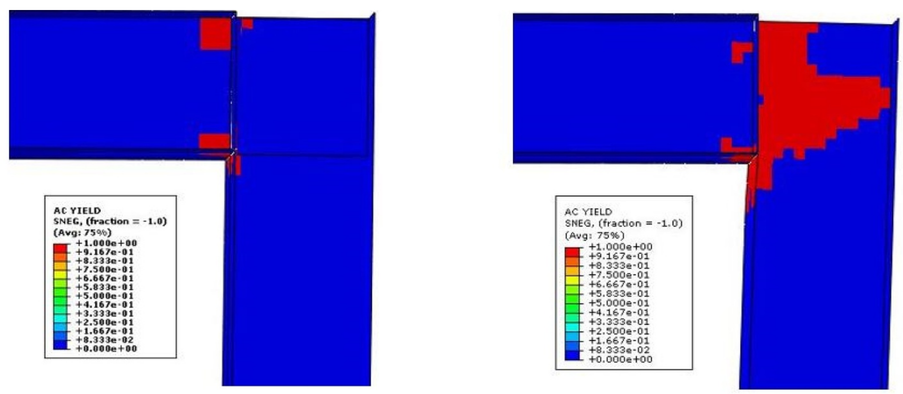
4.2. Multi-comparison technique (MCT)

As mentioned above, to calculate the objective function of an individual, several nonlinear inelastic analyses are required to evaluate the strength and serviceability constraints of the optimization. For this reason, optimization methods using conventional DE algorithms often incur excessive computational costs, as the objective function of every



a) Case 1: Model using doubler plate b) Case 2: Model without using doubler plate

Fig. 4. Stress distribution of the two-bar steel frame
a) Case 1: Model using doubler plate (b) Case 2: Model without using doubler plate.



a) Case 1: Model using doubler plate b) Case 2: Model without using doubler plate

Fig. 5. Plastic zone of the two-bar steel frame
(a) Case 1: Model using doubler plate (b) Case 2: Model without using doubler plate.

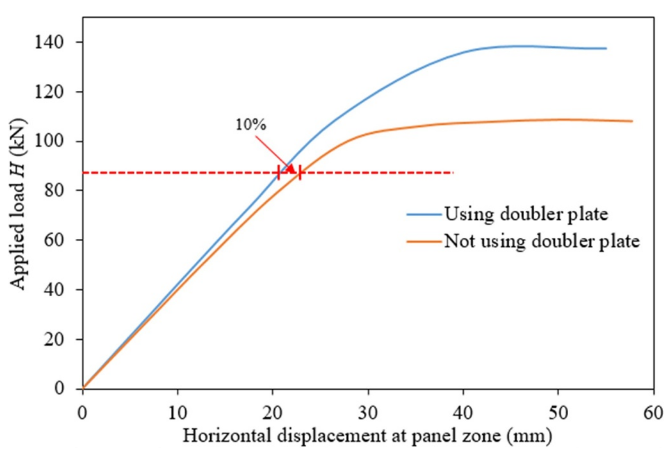


Fig. 6. Load-deflection curves of the two-bar frame for two cases.

trial individual is evaluated. It should be noted that only the trial individuals, which are better than the target, are chosen in the optimization process. Therefore, a trial individual can be immediately neglected without calculation of its objective function if it can be assumed that its objective function is greater than that of the target individual. To this end, the proposed MCT method will check the constraints that have been rearranged according to their computational cost and ordered from the shortest to the longest computation time. For a constraint, the objective function of the trial individual including the calculated constraints is evaluated. If it is greater than the objective function of the target individual, the trial individual is neglected immediately. To explain the MCT method in more detail, an unconstrained objective function of the trial individual Y is determined using Eq. (10.a) as follows:

$$\begin{aligned} T_{uncons}(\mathrm{Y}) &= W(\mathrm{Y}) + (\alpha_{con}\beta_1) \times W(\mathrm{Y}) + (\alpha_{str}\beta_2 + \alpha_{iyp}\beta_3) \times W(\mathrm{Y}) + (\alpha_{ins}\beta_4) \times W(\mathrm{Y}) + W_{panel}(\mathrm{Y}) \\ &= W(\mathrm{Y}) + C_{con} \times W(\mathrm{Y}) + C_{str} \times W(\mathrm{Y}) + C_{ins} \times W(\mathrm{Y}) + W_{panel}(\mathrm{Y}), \end{aligned} \tag{16}$$

where C_{con}, C_{str}, and C_{ins} are parameters relating to the geometric constructability, strength, and serviceability constraints of trial individual Y, respectively The comparison between Y and the target individual X is performed as follows:

Step 1: The total weight of beams and columns, W(Y), is calculated. If $W(\mathrm{Y}) > T_{uncons}(\mathrm{X})$, Y is neglected and the selection operator corresponding to X is terminated. Otherwise, proceed to Step 2.
Step 2: The constructability constraints are evaluated, which do not

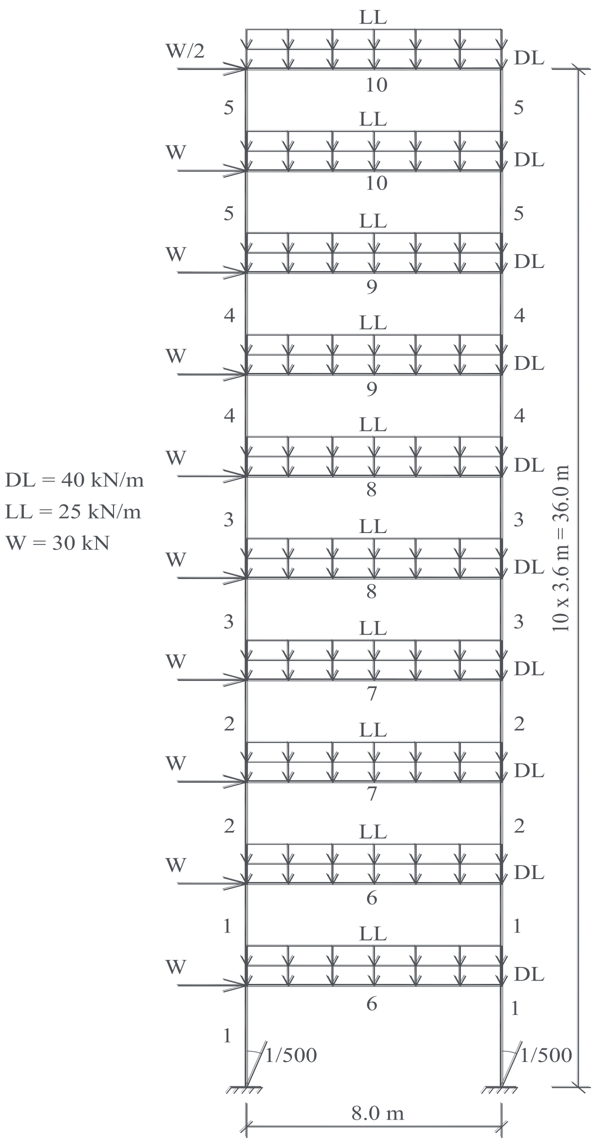


Fig. 7. One bay-ten story steel frame.

require nonlinear inelastic analysis. If (W(Y) + C_{con} × W(Y)) > T_{uncons}(X), Y is neglected and the selection operator corresponding to X is terminated. Otherwise, proceed to Step 3.

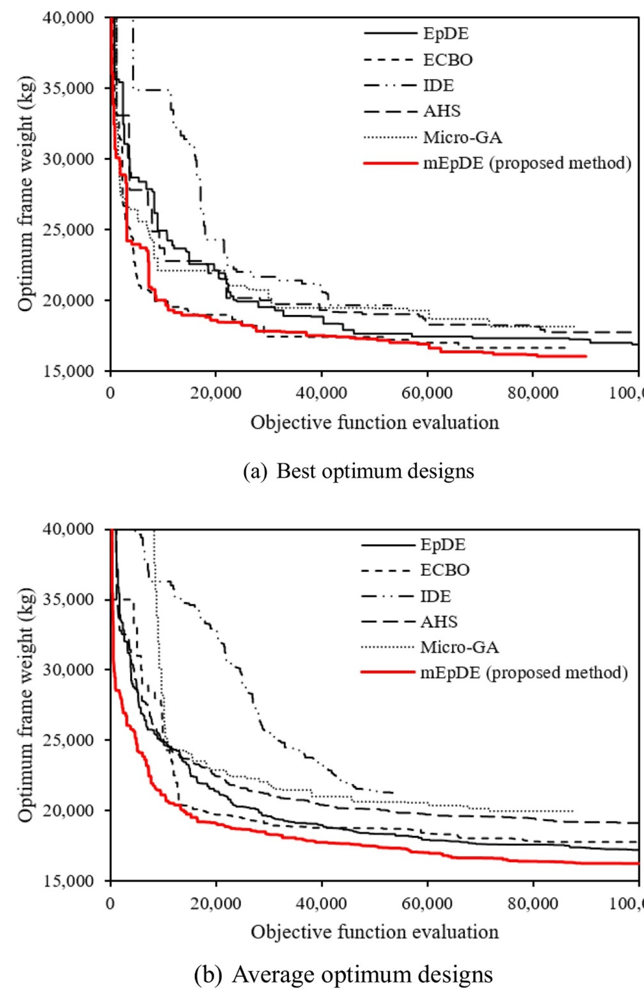


Fig. 8. Convergence histories for the one bay-ten story steel frame (a) Best optimum designs (b) Average optimum designs.

Step 3: The strength constraints are evaluated using nonlinear inelastic analysis. If (W(Y) + C_{con} × W(Y) + C_{str} × W(Y)) > T_{uncons}(X), Y is neglected. Otherwise, proceed to Step 4.

Step 4: The serviceability constraints are evaluated using nonlinear inelastic analysis. If T_{uncons}(Y) > T_{uncons}(X), Y is neglected. Otherwise, Y is chosen to replace X in the population.

Table 2
Optimization results of one bay-ten story steel frame.

Element group of best design	mEpDE	EpDE	ECBO	IDE	AHS	Micro-GA
1	W27 × 94	W27 × 114	W24 × 104	W27 × 102	W27 × 129	W27 × 161
2	W27 × 94	W24 × 104	W21 × 101	W24 × 117	W24 × 131	W24 × 117
3	W24 × 76	W24 × 76	W18 × 76	W24 × 94	W24 × 94	W24 × 68
4	W21 × 55	W18 × 60	W18 × 60	W14 × 120	W21 × 62	W21 × 68
5	W21 × 44	W18 × 35	W18 × 35	W14 × 43	W21 × 44	W21 × 44
6	W27 × 84	W27 × 84	W30 × 90	W27 × 102	W24 × 76	W24 × 68
7	W27 × 94	W30 × 99	W27 × 84	W30 × 90	W30 × 90	W30 × 99
8	W24 × 76	W24 × 68	W27 × 84	W21 × 73	W24 × 76	W24 × 68
9	W24 × 55	W24 × 62	W21 × 57	W24 × 62	W21 × 48	W18 × 65
10	W18 × 35	W18 × 35	W18 × 35	W16 × 57	W16 × 40	W18 × 46
Best weight (kg)	16,028	16,873	16,628	19,640	17,736	18,109
Panel zone cost of the best design (kg)	0	217.02	217.02	275.31	0	0
Worst weight (kg)	16,580	18,007	19,077	23,798	20,246	21,761
Average weight (kg)	16,253	17,222	17,782	21,280	19,127	19,949
Std (kg)	218	558	843	1511	820	1480
Avg. number of structural analysis	15,158	13,827	154,836	121,350	228,254	243,468
Avg. computational time (hour)	4.7	4.4	49	39	73	77

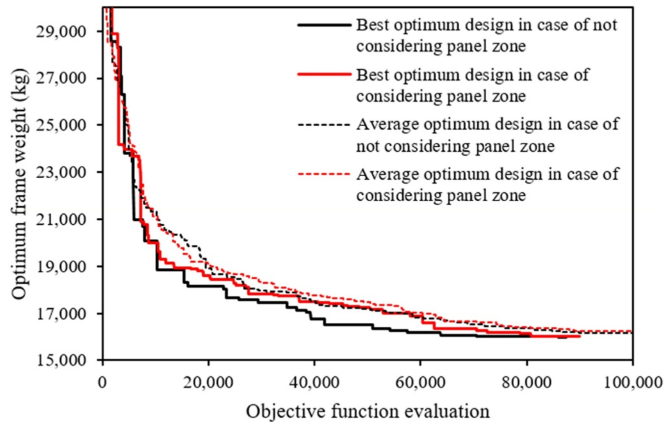


Fig. 9. Convergence histories for the one bay-ten story steel frame.

4.3. Improvements of EPDE

The MCT method procedure presented in Section 4.2 shows that a trial individual, Y, that does not satisfy $W(\mathrm{Y}) < T_{uncons}(\mathrm{X})$ and $(W(\mathrm{Y}) + C_{con} \times W(\mathrm{Y})) < T_{uncons}(\mathrm{X})$ is neglected immediately and the selection process is terminated. From the fact that the computation time to evaluate $W(\mathrm{Y})$ and $(W(\mathrm{Y}) + C_{con} \times W(\mathrm{Y}))$ is negligible, the optimization performance can be improved by selecting trial individuals satisfying the condition $(W(\mathrm{Y}) + C_{con} \times W(\mathrm{Y})) > T_{uncons}(\mathrm{X})$. Based on this, a simple but highly efficient method for choosing trial individuals is proposed: The Promising Individual Method (PIM). The details of PIM are as follows:

PIM algorithm for the selection phase

01: Input: target vector X with objective function $T_{uncons}(\mathrm{X})$
02: Perform mutation and crossover operators to generate a trial individual, Y
03: Calculate the weight of beams and columns, $W(\mathrm{Y})$
04: **If** $(W(\mathrm{Y}) > T_{uncons}(\mathrm{X}))$ **then** Neglect Y; Go back to line 02
05: Evaluate the constructability constraints to calculate C_{con}
06: **If** $((W(\mathrm{Y}) + C_{con} \times W(\mathrm{Y})) > T_{uncons}(\mathrm{X}))$ **then** Neglect Y; Go back to line 02
07: Continue

Another problem is that individuals of the population during the last stage of the optimization process are converged. At that point, new trial individuals usually overlap with previously created individuals, particularly for a discrete optimization problem like steel frame optimization. Therefore, there are many unnecessary objective function evaluations in this stage. To prevent this, another improvement made to EpDE in this study is that a single matrix, *DEstor*, is used to store all the individuals and their objective function values. When a new trial individual, Y, is generated, it is first checked against the members of *DEstor*. If Y is identical to an existing member of *DEstor*, it is discarded and a new trial individual is created; if it is unique, the selection operator is performed. Y and its objective function value are then stored in *DEstor*.

5. Proposed optimization procedure of steel frames using nonlinear inelastic analysis

The overall computational procedure of the proposed method for optimization of steel frames is presented as follows:

Proposed numerical optimization procedure

01: Begin
02: Input optimization parameters: *A, B* for Eq. (12), scale factor $F = 0.7$, *total_iteration*, number of individuals in population *NP*
03: Randomly generate *NP* individuals $\mathrm{X}_i(i = 1, .., NP)$ and save them into matrix *DEm*
04: **For** *i* **from** 1 **to** *NP*
05: Calculate penalty parameters for checking constraints of the individual X_i by using Eq. (7b)
06: Design panel zones for X_i and calculate total cost of the designed panel zones using Eq. (7)
07: Calculate total cost $W(\mathrm{X}_i)$ using Eq. (8)
08: Determine unconstrained objective functions $T_{uncons}(\mathrm{X}_i)$ using Eq.-(10.a)
09: **End**
10: Save all *DEm* members and their unconstrained objective functions into matrix *DEstor*
11: **While** *the terminal conditions are not satisfied*
12: **For** *j* **from** 1 **to** *NP*
13: Determine *p* using Eq. (15)
14: Generate randomly an integer value k_{rand} in the range [1, *nm*]
15: **For** *k* **from** 1 **to** *nm*
16: Generate randomly *CR* in the range [0, 1]
17: Generate randomly *Rand* in the range [0, 1]
18: **If** $(k = k_{rand})$ or $(Rand \le CR)$ **then**
19: Choose X_{pbest}, X_{r1}, and X_{r2} so that $\mathrm{X}_{r1} \neq \mathrm{X}_{r2} \neq \mathrm{X}_{pbest} \neq \mathrm{X}_j$
20: $u_k =$ the nearest integer of $(x_{pbest,k} + F \times (x_{r1,k} - x_{r2,k}))$
21: **Else**
22: $u_k = x_{j,k}$
23: **End if**
24: **End**
25: **If** $(\mathrm{U}(u_1,.., u_{NP})$ was stored in *DEstor*) **then**
26: Use the unconstrained objective functions stored in *DEstor* for U
27: If U is better than X_i, U will replace X_i in *DEm*
28: **Else:**
29: Apply MCT and PIM methods for selection U
30: Save U and its unconstrained objective function into *DEstor*
31: **End if**
32: **End**
33: **End**
34: **Stop**

At line 29 of the proposed numerical optimization procedure, when the MCT method is used, the exact unconstrained objective function of U maybe not evaluated if we can predict that U is not better than the target individual X_i. For example, U violates an strength constraint and its current unconstrained objective function is greater than the constrained objective function of X_i. In this case, the current unconstrained objective function of U is used to save into *DEstor*. This is acceptable because U is not a feasible candidate and the recorded value for U is enough to ensure that U is neglected in the optimization process.

In addition, the terminal conditions of the optimization process presented at line 11 are (a) the number of iteration reaches the defined value or (b) the difference between the worst and the best members is smaller 0.01% or (c) the best individual is not improved in 1000 consecutive iterations.

Table 3
Effect of panel zone design on optimum design of one bay-ten story steel frame.

	Panel zone design in optimization process			Panel zone design after optimization process		
	Columns and beams cost	Panel cost	Total cost	Columns and beams cost	Panel cost	Total cost
Best weight (kg)	16,028	0	16,028	15,974	326	16,300
Worst weight (kg)	16,580	0	16,580	16,363	293	16,656
Average weight (kg)	16,253	0	16,253	16,072	436	16,508

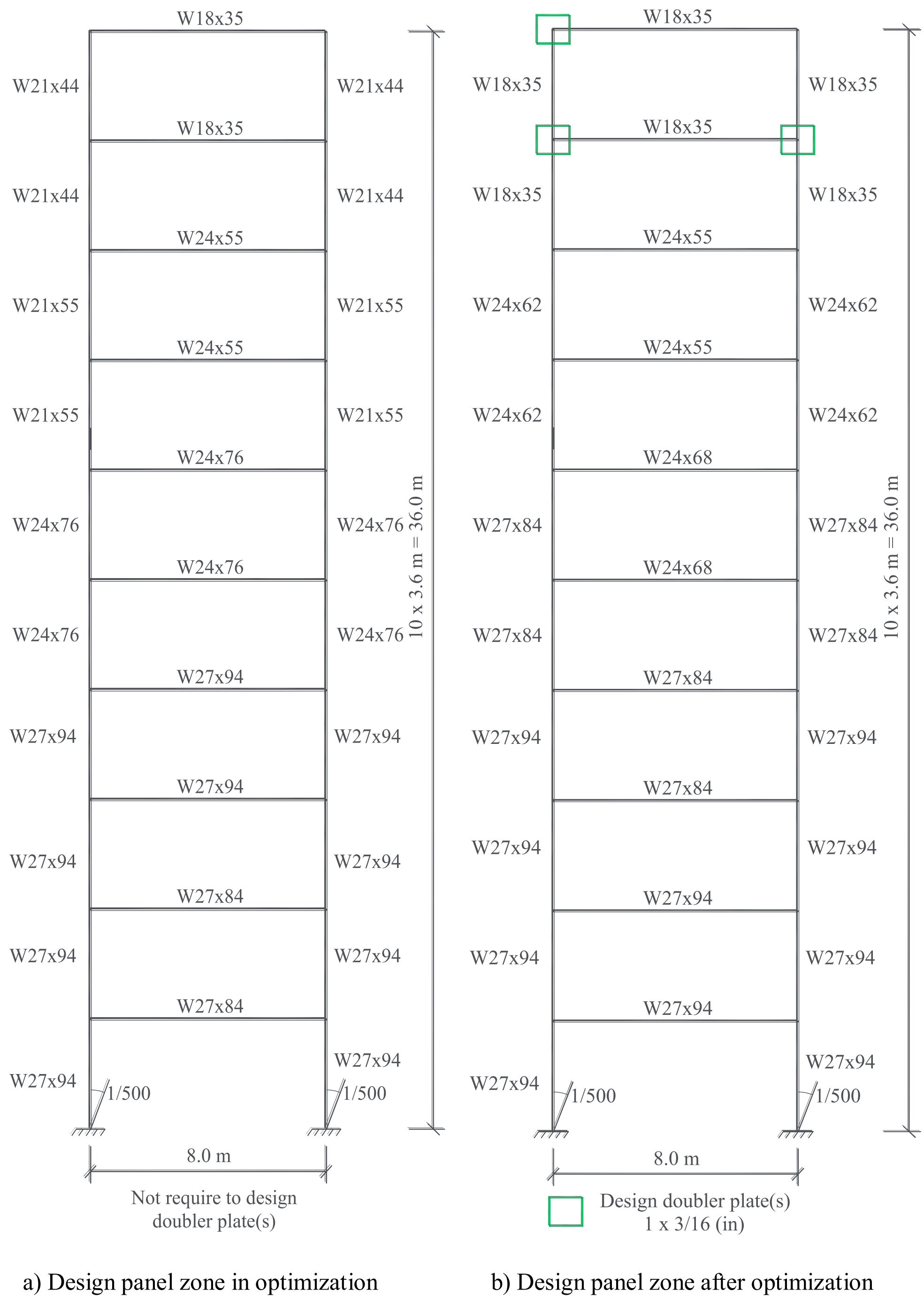


Fig. 10. Best optimum design of one bay-ten story steel frame
(a) Design panel zone in optimization (b) Design panel zone after optimization.

6. Numerical examples

In this section, a two-bar frame is studied first to perform the effects of panel zone on structural behaviors. Three steel frames (one bay-ten story, five bay-five story, and three bay-ten story steel frames) are then analyzed to demonstrate the efficiency of the proposed method. The following features are considered in all case studies.

The objective function is the total weight of the structure presented in Eq. (5). AISC W-shaped sections are used for the design of the beams and columns where 267 sections from W10–W44 for the beam members and 158 sections from W12, W14, W18, W21, W24, and W27 for the column members are considered. The properties of these sections are used to develop two different design pools, one for beam members and one for column members. In each pool, the sections, which are sorted with respect to their area, are represented by their sequence number that indicates their positions in the pool. The design variable of each structural member group is defined as the position of the section used for this member group in the corresponding pool. This means that a design variable is an integer value between 1 and the number of sections in the pool. Regarding the strength constraints, two load combinations are considered: ($1.2DL + 1.6LL$), and ($1.2DL + 1.6W + 0.5LL$), where DL, LL, and W are the dead, live, and wind loads, respectively. Load combination ($1.0DL + 0.7W + 0.5LL$) is used for the serviceability constraint with an allowable inter-story drift of $h/400$, where h is the height of the frame story. This means that 3 structural analyses are required to calculate the strength and serviceability constraints corresponding to the

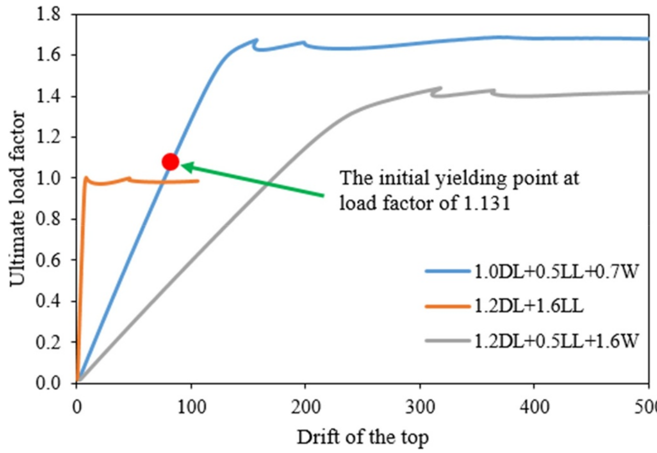


Fig. 11. Load-deflection curves of the best optimum design of one bay-ten story steel frame obtained using the proposed method.

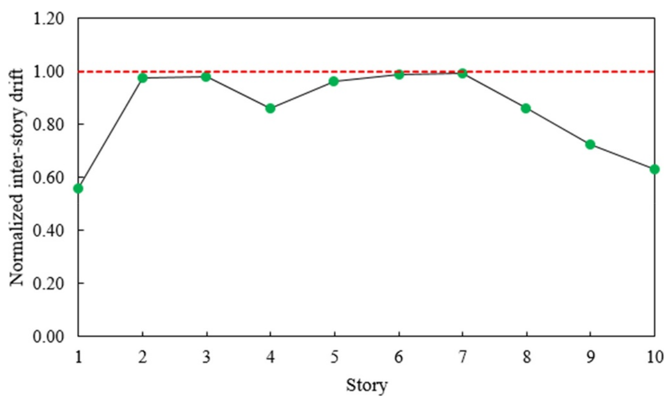


Fig. 12. Convergence histories for the one bay-ten story steel frame.

objective function evaluation of one individual. The optimization is terminated when the total number of objective function evaluations equals 100,000. ASTM A992 steel with the yield stress of $F_y = 344.7$ (*MPa*) and the elastic modulus of $E = 200$ (*GPa*) is used for all beams and columns. The weight per unit volume of the material is 7850 (kg/m^3). Regarding the thickness of doubler plates, the use of many thickness types for doubler plates is not encouraged from the viewpoint of construction. Therefore, in this study only four types of doubler plates' thickness are used 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).

Five recently developed optimization methods for steel frames are employed for comparison: EpDE [43], ECBO [26], IDE [15], AHS [44], and micro-GA [8]. The parameters of the optimization methods are summarized in Table 1 that are selected similar to the values used in the aforementioned references. An Intel® Core™ i7-8700 K CPU@3.20 Hz computer configuration is used for the analyses.

6.1. A two-bar steel frame

Fig. 3 presents the frame with the beam and column sections of W14 × 34. The beam-to-column connection is assumed to be rigid. The frame has only one panel zone at the column web area located at beam-to-column connection. Two cases are now studied to consider the effect of panel zone deformation on the structural behaviors. In the first case, a doubler plate with the thickness of 10 (*mm*) is used to extend the depth of the column web at the panel zone area. In the second case, the panel zone is not designed. Two cases are loaded by a horizontal force H using nonlinear inelastic analysis until they are failed.

Figs. 4 and 5 present the stress distribution and plastic zone, respectively, of the two-bar steel frame for the two cases at the ultimate state of the Case 2. It is observed from Fig. 4b that the stress of the beam and column at the panel zone reaches to the yield strength, 345 (MPa). This means that the frame is failed due to yielding of the panel zone in the Case 2. In addition, it can be seen from Fig. 5b that the failure of the frame is due to the formation of a plastic hinge at both panel zone and junction of the beam and column for Case 2. However, the failure only occurs by the yielding of the steel at the junction of the beam and column for the Case 1 as can be observed from Figs. 4a and 5a. This is because the panel zone in the Case 1 is reinforced by the doubler plate which increases the stiffness of the connection.

On the other hand, Fig. 6 shows the load-deflection curves of two-bar frame for two cases. It is clear that the strength of the connection significantly increases when the frame is reinforced the doubler plate at the panel zone. Additionally, it can be observed at working load, 10% drift of the frame is due to panel zone deformation. This indicates that panel zone deformation should be taken into consideration in design of steel frames.

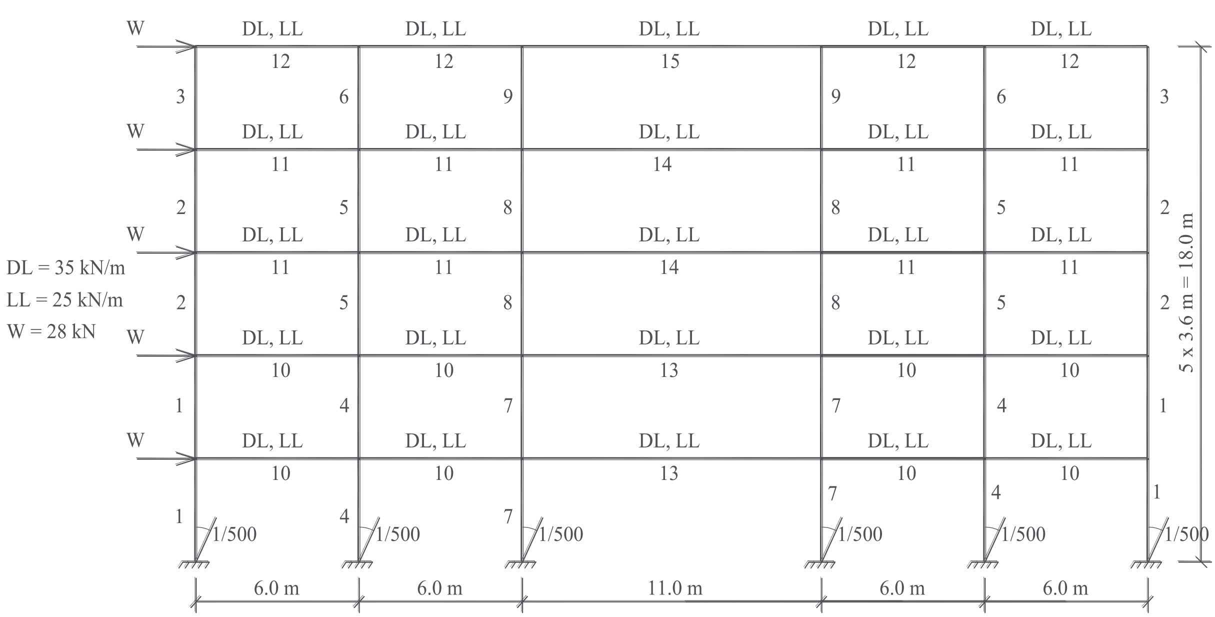


Fig. 13. Five bay-five story steel frame.

Table 4
Optimization results of five bay-five story steel frame.

Element group of best design	mEpDE	EpDE	ECBO	IDE	AHS	Micro-GA
1	W18 × 35	W18 × 40	W18 × 35	W24 × 62	W21 × 57	W21 × 44
2	W18 × 35	W18 × 35	W18 × 35	W21 × 50	W18 × 40	W21 × 48
3	W18 × 35	W18 × 35	W18 × 35	W12 × 35	W18 × 35	W12 × 26
4	W24 × 62	W24 × 62	W24 × 68	W24 × 94	W21 × 68	W21 × 57
5	W24 × 55	W24 × 55	W24 × 55	W24 × 62	W21 × 57	W18 × 40
6	W21 × 50	W21 × 48	W21 × 73	W12 × 96	W14 × 53	W14 × 74
7	W27 × 94	W27 × 114	W27 × 114	W24 × 84	W21 × 111	W27 × 84
8	W27 × 94	W24 × 68	W24 × 55	W24 × 84	W21 × 83	W24 × 55
9	W27 × 84	W21 × 93	W14 × 43	W14 × 48	W21 × 62	W21 × 101
10	W14 × 22	W14 × 22	W14 × 22	W18 × 35	W14 × 26	W14 × 22
11	W14 × 22	W14 × 22	W14 × 22	W14 × 22	W14 × 22	W16 × 26
12	W14 × 22	W16 × 26	W16 × 31	W18 × 40	W16 × 31	W16 × 31
13	W21 × 48	W21 × 50	W21 × 48	W24 × 55	W21 × 62	W21 × 57
14	W24 × 55	W21 × 50	W24 × 55	W21 × 50	W21 × 50	W21 × 50
15	W24 × 55	W24 × 55	W24 × 62	W24 × 55	W24 × 55	W18 × 55
Best weight (kg)	19,045	19,706	19,854	23,240	21,527	21,252
Panel cost of the best design (kg)	922	1180	1773	2077	1862	3085
Worst weight (kg)	19,419	20,962	21,016	27,582	24,274	25,091
Average weight (kg)	19,209	20,243	20,485	24,949	22,922	23,064
Std (kg)	157	448	518	1764	809	1374
Avg. number of structural analysis	30,870	27,936	229,251	148,326	286,174	264,960
Avg. computational time (hour)	6.7	6.2	51	33	64	59

6.2. One bay-ten story frame design

Fig. 7 shows the geometry and design loading conditions of a one-bay, ten-story frame consisting of 20 panel zones and 30 frame members. The initial story out-of-plumbness is equal to 1/500. All beam members have a uniformly distributed dead load equal to 40 (kN/m) and a uniformly distributed live load equal to 25 (kN/m). Furthermore, wind load equal to 30 (kN) acts at each floor level. From the 30 frame members, the columns are divided into 5 groups and the beams are divided into 5 groups. The numbering of member groups is presented in Fig. 7. This implies that the design variable space includes more than 1.33E+23 permutations. The constraints of the optimization problem are comprised of 9 constructability constraints, 2 strength constraints, and 1 serviceability constraint. In order to obtain statistically significant results and avoid the effect of the initial solution on the result of the optimization procedure, 20 optimization runs are performed for each case. In this way, the effect of the stochastic nature associated with the metaheuristic algorithms is isolated and meaningful results are obtained.

Table 2 summarizes the optimization results of the considered algorithms. The comparison between the proposed method (mEpDE) and EpDE is considered first to evaluate the efficiency of the improvement proposed in this study. As can be observed from Table 2, mEpDE yields the best optimum design of the frame with the total cost of 16,028 (*kg*) which is lower than the ones of EpDE by 5.27%. Similarly, Table 2 reveals that the average and worst weights of the optimum structural designs obtained with mEpDE are smaller than those of EpDE. These results prove that mEpDE is better than EpDE in searching the optimal solutions.

Table 2 also indicates that the best, the average, and the worst values of the optimum structural design obtained with mEpDE are smaller than those of the EpDE, ECBO, IDE, AHS, and micro-GA methods. The worst optimum weight of 16,580 (kg) obtained with mEpDE is smaller than even the lowest weights of 16,873, 16,628, 19,640, 17,736, and 18,109 (kg) found with the EpDE, ECBO, IDE, AHS, and micro-GA methods, respectively. This means that the proposed method can yield better optimum structural designs than the EpDE, ECBO, IDE, AHS, and micro-GA methods.

Furthermore, the standard deviation in the weight of the optimum structural design found using the proposed method is 218 (kg), which is equal to 39.05%, 25.84%, 14.41%, 26.55%, and 14.71% that of the EpDE, ECBO, IDE, AHS, and micro-GA methods, respectively. From these results, it can be concluded that the proposed method is more stable for finding the optimum structural design than the other considered optimization algorithms.

Regarding the computational cost, Table 2 indicates that mEpDE requires an average of 15,158 structural analyses for one optimization process, which is much smaller than 154,836, 121,350, 228,254, 243,468 structural analyses required when using the ECBO, IDE, AHS, and micro-GA methods, respectively. As a consequence, the average computation time of the proposed method is only 4.7 (hours) while ECBO, IDE, AHS, and micro-GA require more than 30 (hours) of computation time. The proposed method is hence more robust than the other considered optimization algorithms.

The convergence curves of the best and average optimum designs obtained with the considered optimization methods are shown in Fig. 8a and b, respectively. These figures show that mEpDE and ECBO are converged much better than EpDE, IDE, AHS, and micro-GA. Besides that, the optimum structural weights obtained at the 10,000th objective function evaluation are much larger than those at the 100,000th objective function evaluation. Therefore, using a small number ($<$10,000) of objective function evaluations will not yield satisfactory optimum structural designs in this case study.

In optimization of steel frames considering panel zone design as presented in Eq. (5), the total cost of panel zones that is dependent upon the depth of the beams and columns is included. It will therefore increase the number of dependent variables in the optimization problem. In order to investigate the influence of considering panel zone design upon convergence speed of the optimization process, mEpDE is used to perform the comparison of two optimization problems such as (1) considering panel zone design in optimization process and (2) regardless panel zone design in optimization. The results are presented in Fig. 9. Obviously, the convergence speed of the second case is better than of the first case. From this result, a question arises here whether the optimization method, where the panel zone design is only considered for the optimum design obtained, is better than the proposed optimization problem or not? To clarify this issue, a comparison between the optimum designs of two aforementioned optimization problems is conducted and the results are presented in Table 3. As can be seen in this table, the optimum designs yielded in Case 1 have smaller total cost than in Case 2. Therefore, considering panel zone design in the optimization process is necessary. In addition, Fig. 10 shows the panel zone design for best optimum designs obtained from Cases 1 and 2. In Fig. 10a, when panel zone cost includes in the objective function, greater columns are preferred to use in order to reduce the reinforcement by doubler plates. Meanwhile, three panel zone positions need to be added doubler plates in the Case 2 where panel zone design is

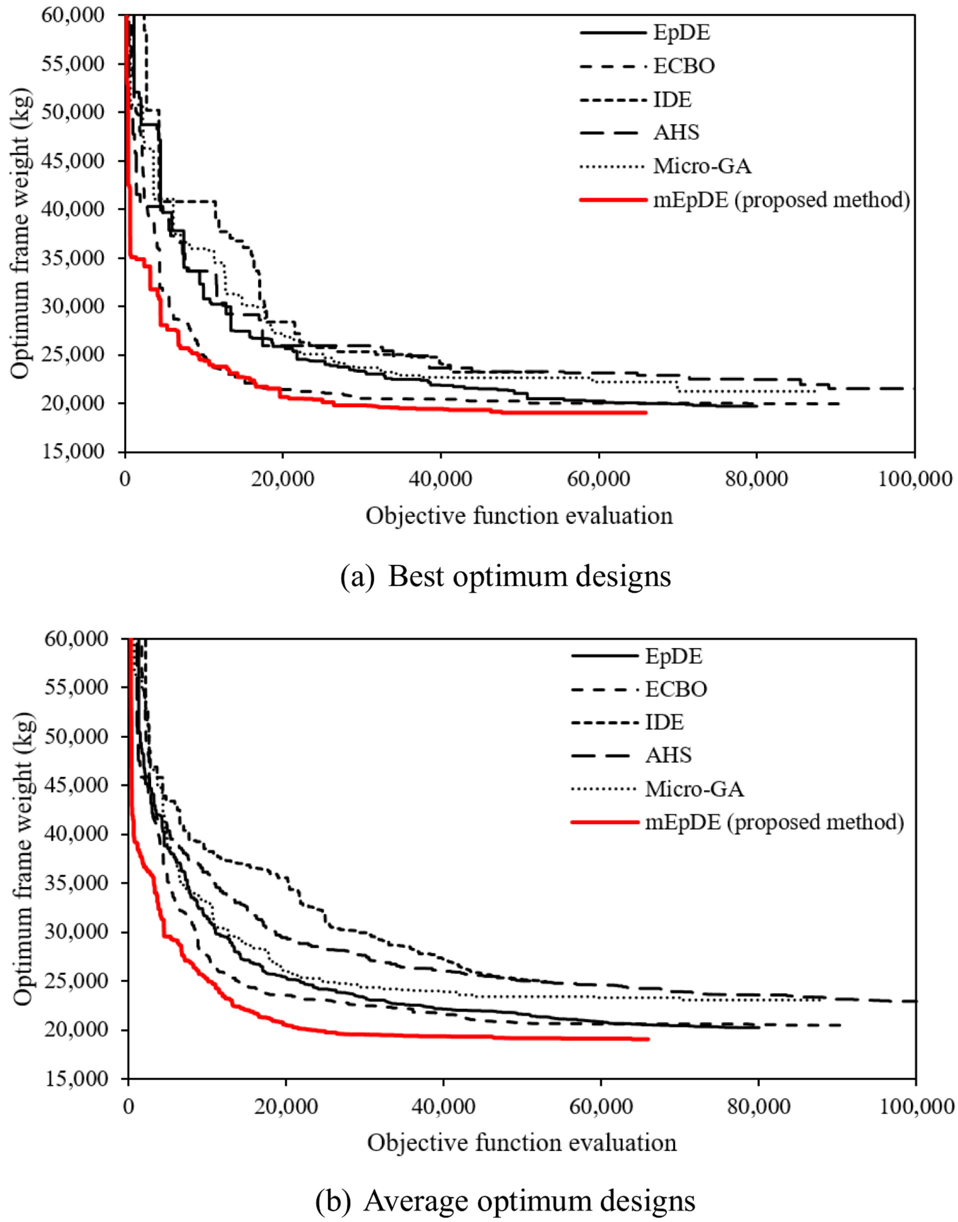


Fig. 14. Convergence histories for five bay-five story steel frame.

Table 5
Effect of panel zone design on optimum design of five bay-five story steel frame.

	Panel zone design in optimization process			Panel zone design after optimization process		
	Columns and beams cost	Panel cost	Total cost	Columns and beams cost	Panel cost	Total cost
Weight of best optimum design (kg)	18,123	922	19,045	14,101	9074	23,175
Weight of worst optimum design (kg)	17,642	1777	19,419	14,208	8576	22,784
Avg. weight of optimum design (kg)	18,029	1180	19,209	14,137	9023	23,160

ignored in the optimization process.

Fig. 11 presents the load-deflection curves of the best optimum structural design obtained by using the proposed method corresponding to the strength load combinations. The ultimate load factors of the frame are equal to 1.001 and 1.436 corresponding to the load combinations ($1.2DL + 1.6LL$)and ($1.2DL + 1.6W + 0.5LL$), respectively. This means that all the strength constraints are satisfied. Fig. 9 also shows the initial yielding point of the load combination ($1.0DL + 0.7W + 0.5LL$) at the load factor of 1.131. This means that the structure is still in the elastic region under the load combination ($1.0DL + 0.7W + 0.5LL$). The inter-story drift constraints of this optimum design is also presented in Fig. 12. The normalized inter-story drift is the ratio of the inter-story drift and its allowable value. In this figure, the normalized inter-story drifts at all story are smaller than 1.0. This implies that the inter-story drift constraints are guaranteed.

6.3. Five bay-five story frame design

Fig. 13 shows the geometry and applied loads of a five-bay, five-story frame with an initial story out-of-plumbness of 1/500. The frame

Fig. 15. Best optimum design of five bay-five story steel frame
(a) Design panel zone in optimization (b) Design panel zone after optimization.

consists of 30 columns, 25 beams, and 25 panel zones. The uniformly distributed dead load and live load on all the beams are equal to 35 (kN/m) and 25 (kN/m), respectively. The design wind load acting at each floor level is 28 (kN). The column and beam elements of the frame are grouped into 9 column design variables and 6 beam design variables. Therefore, the design variable space of this case study includes

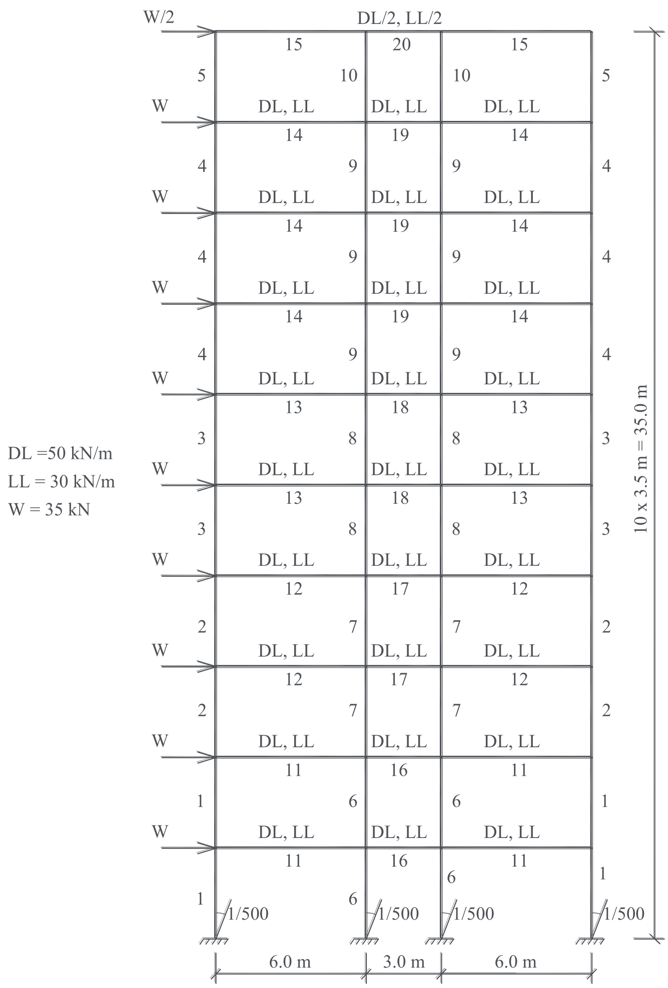


Fig. 16. Three bay-ten story steel frame.

more than 2.22E + 34 permutations. There are a total of 21 constraints in this case, including 18 constructability constraints, 2 strength constraints, and 1 serviceability constraint.

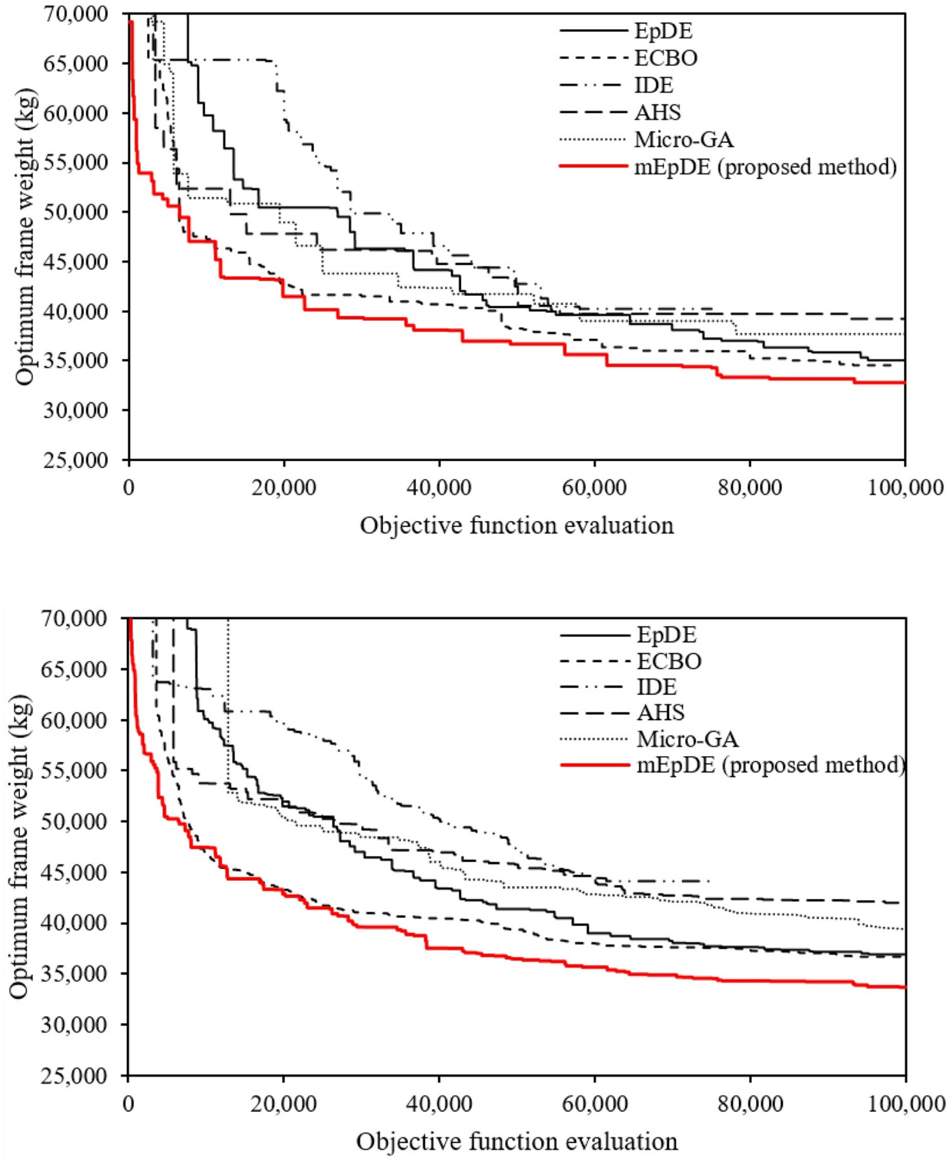
Table 4 summarizes the performance of the considered optimization algorithms, in which each algorithm is executed 10 times. Similar to the previous case study, the weight of 19,045 (kg) for the best optimum structural design found using the mEpDE method is much lower than the best optimum results of 19,706, 19,854, 23,240, 21,527, and 21,252 (kg) found using the EpDE, ECBO, IDE, AHS, and micro-GA methods, respectively. The average and worst weights of 19,209 and 19,419 (kg), respectively, of the optimum structural designs obtained using mEpDE are also smaller than those obtained using EpDE, ECBO, IDE, AHS, and micro-GA. Furthermore, the standard deviation of the weight of 157 (kg) for the optimum structural design found using mEpDE is much smaller than 448, 518, 1764, 809, and 1374 (kg) of EpDE, ECBO, IDE, AHS, and micro-GA methods, respectively. This indicates that the performance of the proposed method is better than the other considered optimization algorithms. Regarding the computational cost, mEpDE requires approximately 6.7 (hours) of computation time for an average of 30,870 structural analyses per optimization process. This is a little greater than 6.2 (hours) of EpDE but much shorter than 51, 33, 64, and 59 (hours) required for using ECBO, IDE, AHS, and micro-GA methods, respectively.

Fig. 14a and b show the convergence curves for the best and average optimum designs obtained with the considered optimization methods, respectively. These figures show that in this example, the convergence speed of the mEpDE method is better than the EpDE, ECBO, IDE, AHS, and micro-GA methods. In addition, the optimum structural weights obtained at the 10,000th objective function evaluation are much larger than those obtained at the 100,000th objective function evaluation. Therefore, using a small number (< 10,000) of objective function evaluations in this case study will not yield good optimum structural designs.

Table 5 presents the comparison results between two case studies: (1) considering and (2) regardless panel zone design in optimization process. Compared to Case 2, the optimum designs yielded in Case 1 has the bigger total weight of columns and beams but much smaller total weight since the total cost of panel zones is minimized in the

Table 6
Optimization results of three bay-ten story steel frame.

Element group of best design	mEpDE	EpDE	ECBO	IDE	AHS	Micro-GA
1	W21 × 48	W24 × 68	W24 × 68	W27 × 129	W27 × 102	W27 × 94
2	W18 × 119	W18 × 86	W18 × 65	W27 × 94	W21 × 62	W24 × 84
3	W18 × 55	W18 × 76	W18 × 60	W24 × 103	W14 × 109	W24 × 68
4	W18 × 35	W12 × 35	W12 × 30	W12 × 45	W14 × 43	W21 × 55
5	W12 × 26	W12 × 40	W12 × 58	W12 × 40	W14 × 43	W14 × 43
6	W27 × 258	W27 × 194	W27 × 235	W27 × 258	W27 × 281	W27 × 161
7	W27 × 114	W27 × 161	W21 × 111	W24 × 104	W24 × 207	W27 × 114
8	W27 × 84	W21 × 73	W21 × 101	W14 × 82	W21 × 111	W24 × 104
9	W21 × 68	W21 × 68	W18 × 76	W14 × 109	W21 × 68	W14 × 68
10	W14 × 34	W14 × 43	W12 × 152	W12 × 87	W14 × 38	W12 × 96
11	W18 × 46	W14 × 43	W24 × 55	W21 × 68	W16 × 36	W16 × 45
12	W21 × 83	W30 × 99	W24 × 62	W21 × 48	W21 × 57	W24 × 76
13	W21 × 44	W21 × 55	W21 × 44	W24 × 76	W21 × 55	W21 × 62
14	W18 × 35	W18 × 35	W18 × 40	W21 × 44	W18 × 46	W14 × 48
15	W14 × 22	W16 × 26	W12 × 26	W14 × 22	W14 × 26	W10 × 26
16	W16 × 36	W40 × 149	W14 × 48	W21 × 55	W21 × 136	W33 × 118
17	W21 × 62	W18 × 60	W16 × 67	W33 × 118	W24 × 104	W16 × 36
18	W18 × 40	W21 × 55	W27 × 129	W21 × 50	W12 × 120	W21 × 73
19	W18 × 35	W16 × 36	W18 × 35	W21 × 68	W21 × 57	W12 × 53
20	W14 × 22	W12 × 30	W10 × 26	W18 × 40	W10 × 26	W14 × 22
Best weight (kg)	32,785	35,028	34,535	40,239	39,226	37,695
Panel cost of the best design (kg)	1711	1707	1686	2141	1677	3704
Worst weight (kg)	34,460	38,832	37,932	46,571	44,368	40,997
Average weight (kg)	33,683	36,907	36,529	44,123	41,993	39,404
Std (kg)	708	1264	1177	2646	2064	1468
Avg. number of structural analysis	20,809	15,189	243,177	168,298	254,459	279,904
Avg. computational time (hour)	13.1	8.3	133	92	139	153



(b) Average optimum designs

Fig. 17. Convergence histories for three bay-ten story steel frame
(a) Best optimum designs (b) Average optimum designs.

Table 7
Effect of panel zone design on optimum design of five bay-five story steel frame.

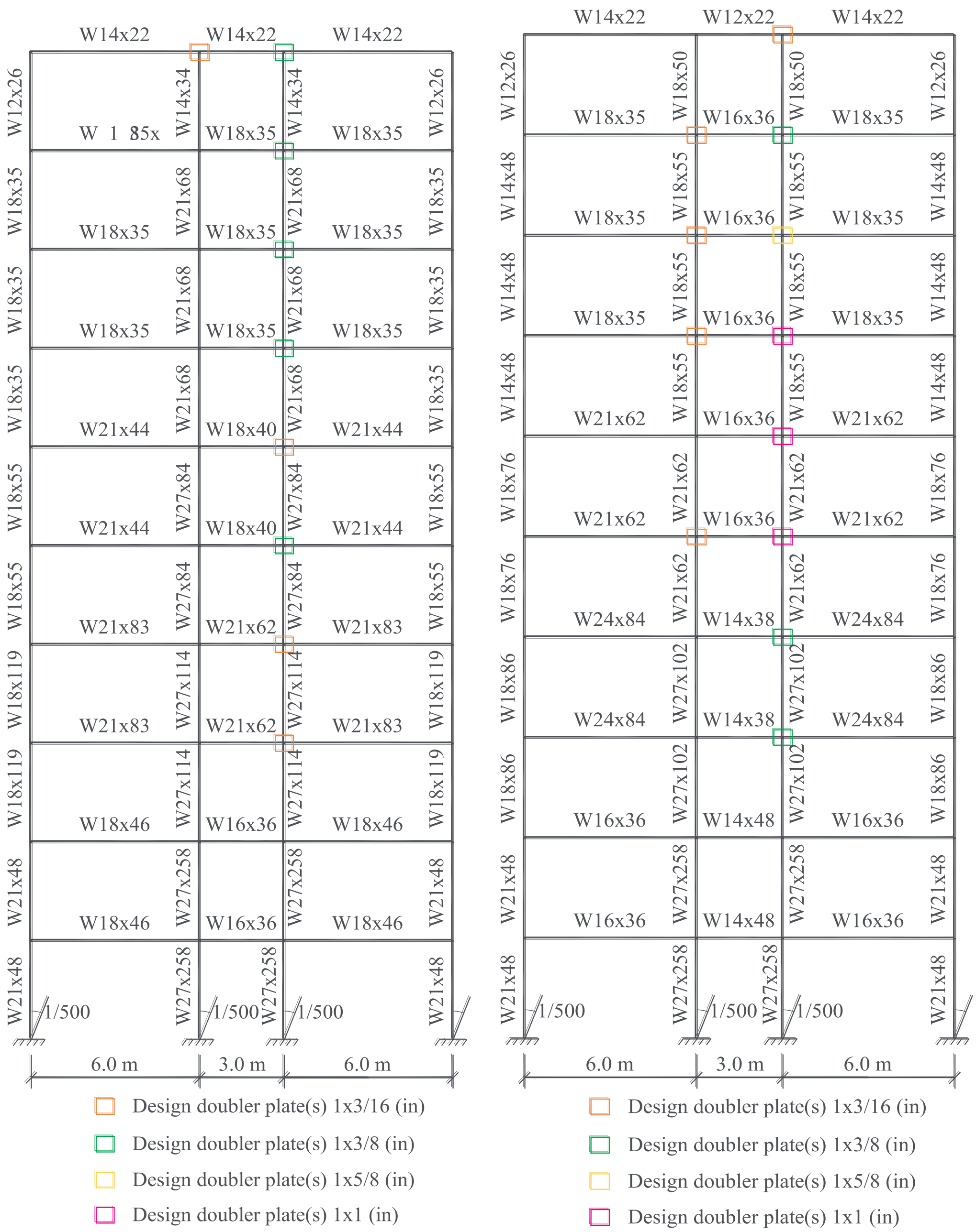
	Panel zone design in optimization process			Panel zone design after optimization process		
	Columns and beams cost	Panel cost	Total cost	Columns and beams cost	Panel cost	Total cost
Weight of best optimum design (kg)	31,074	1711	32,785	30,593	3758	34,351
Weight of worst optimum design (kg)	32,250	2210	34,460	33,894	3822	37,716
Avg. weight of optimum design (kg)	31,745	1938	33,683	31,260	4525	35,785

optimization process. Furthermore, Fig. 15 shows the panel zone design for best optimum designs obtained from Cases 1 and 2. Since panel zone cost is considered in the optimization process, the optimum design of Case 1 requires less and thinner doubler plates compared to Case 2.

6.4. Three bay-ten story frame design

The geometry and applied loads of a three-bay, ten-story steel frame consisting of 40 columns, 30 beams, and 40 panel zones are shown in Fig. 16. The initial story out-of-plumbness of the frame is equal to 1/500. All the beams are subjected to a uniformly distributed dead load of 50 (kN/m) and a uniformly distributed live load of 30 (kN/m). In addition, the wind load acting at each floor level is equal to 35 (*kN*). The beam and column elements of the frame are grouped into 10 column design variables and 10 beam design variables. The design variable space thus includes more than 1.78E+46 permutations. The total number of constraints is 26, including 23 constructability constraints, 2 strength constraints, and 1 serviceability constraint.

The optimization results are presented in Table 6, in which each optimization algorithm is executed 10 times. Once again, the weight of



a) Design panel zone in optimization b) Design panel zone after optimization

Fig. 18. Best optimum design of three bay-ten story steel frame
(a) Design panel zone in optimization (b) Design panel zone after optimization.

32,785 (kg) for the best optimum structural design obtained using the proposed method is smaller than the best optimum results of 35,028, 34,535, 40,239, 39,226, and 37,695 (kg) obtained with the EpDE, ECBO, IDE, AHS, and micro-GA methods, respectively. The average weight of 33,683 (kg) of the optimum structural designs obtained with the proposed method is lower than the average optimum results for the EpDE, ECBO, IDE, AHS, and micro-GA methods by approximately 9.57%, 8.45%, 30.99%, 24.67%, and 16.98%, respectively. Therefore, it can be concluded that the proposed method yields better optimum structural designs than EpDE, ECBO, IDE, AHS, and micro-GA in this example. In addition, the proposed method is more stable than the other considered algorithms, as the standard deviation in the weight of the optimum design found using the proposed method is 708 (kg), which is much smaller than the results of 1264, 1177, 2646, 2064, and 1468 (kg) for the EpDE, ECBO, IDE, AHS, and micro-GA methods, respectively. Furthermore, the proposed method requires an average of 20,809 structural analyses per optimization process which is greater than 15,189 structural analyses of the EpDE method but much smaller than the number of structural analyses required for the ECBO, IDE, AHS, and micro-GA methods. Therefore, while ECBO, IDE, AHS, and micro-GA require more than 90 (hours) of computation time, the proposed method requires only 13.1 (hours) for one optimization process. The proposed method is hence more efficient than other considered optimization algorithms.

Fig. 17a and b show the convergence curves for the best and average optimum designs obtained with the considered optimization methods, respectively. As these figures show, the convergence speed of the proposed method is much better than the EpDE, ECBO, IDE, AHS, and micro-GA methods. Once again, the convergence curves show that the optimum structural weights obtained at the 10,000th objective function evaluation are much larger than those found at the 100,000th objective function evaluation. Therefore, using a small number (< 10,000) of objective function evaluations does not produce good optimum structural designs.

The comparison results between two case studies: (1) considering and (2) regardless panel zone design in optimization process are presented in Table 7. Once again, the optimum designs yielded in Case 1 has the bigger total weight of columns and beams but much smaller total weight. In addition, Fig. 18a and b show the panel zone design for best optimum designs of the Case 1 and 2, respectively. The optimum design of Case 1 requires less and thinner doubler plates compared to Case 2.

7. Conclusion

Several conclusions have been drawn from this work as follows:

1 An efficient method for the optimum design of nonlinear steel frames considering panel zone design is developed using nonlinear inelastic analysis and an improved DE algorithm named as mEpDE.
2 Compared to the conventional DE algorithm, mEpDE contains four major improvements:(1) a new mutation strategy based on the pbest method; (2) the multi-comparison technique (MCT) for the decrease of the number of unnecessary objective function evaluations; (3) a promising individual method (PIM) for choosing trial individuals; and (4) a trial matrix containing all evaluated individuals to avoid multiple individual objective function evaluations of an individual.
3 The numerical results confirm that the proposed method yields better optimum structural designs, requires smaller number of structural analyses, and is more stable than ECBO, IDE, AHS, and micro-GA.
4 Considering panel zone design in optimization of steel frames yields more realistic and reasonable optimum structural designs since the total cost of panel zone, beams, and columns is minimized.
5 With a computational cost of approximately 20% of that of ECBO, IDE, AHS, and micro-GA, the proposed method is relatively robust and can be considered as an efficient tool supporting the practical design of steel frames.

CRediT authorship contribution statement

Manh-Hung Ha: Software, Writing - original draft, Formal analysis, Visualization, Resources. **Quang-Viet Vu:** Software, Writing - review & editing. **Viet-Hung Truong:** Conceptualization, Methodology, Software, Formal analysis, Investigation, Resources, Data curation, Writing - review & editing, Supervision, Project administration.

Declaration of Competing Interests

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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