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Optimal design of steel frames with semi-rigid connections and column bases using particle swarm optimization

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Abstract. The particle swarm optimization (PSO) method is utilized for two-dimensional steel frames featuring semi-rigid connections between beams and columns, as well as between columns and bases. The metaheuristic algorithm generates optimized steel frames by selecting suitable sections from a predetermined collection of standard steel sections. The standard steel sections, which include wide-flange profiles (W), are supplied by the American Institute of Steel Construction (AISC). The initial example is subject to displacement, inter-storey drift, and stress constraints, as outlined by the AISC Load and Resistance Factor Design (LRFD) regulations. In the last examples, the displacement, geometric, and stress constraints of the AISC Allowable Stress Design (ASD) standard are implemented. In the course of the optimization process, three distinct steel frames are examined, including configurations both with and without semi-rigid connections between the beams and columns, as well as between the columns and bases. All optimization techniques are implemented using the high-level programming language C# version 17.10.5, which is designed to integrate with version 22 of SAP2000's Open Application Programming Interface (OAPI). The results of the analysis indicate that the rigidity of the connections serves as a critical parameter in the optimization of weight for steel frames.

Keywords: Particle swarm optimization (PSO); Semi-rigid connection; Optimization of steel frames; SAP2000-OAPI; Column bases

1. Introduction

The process of optimizing steel frames structures begin with choosing steel profiles for the structural elements from a specific collection of available steel profile tables. This method reduces the steel frame's weight by considering various constraints, such as material properties, displacement, deflection, and dimensions (Prakash & Daloglu, 2023). The field of computer-aided optimization advanced significantly in the 1970s (Pincus, 1970; Talbi, 2009; Rao, 2019). Then, in the early 1990s, heuristic search techniques developed as a response to engineers and designers' requirements for cost-effective designs and improved optimization strategies (Jenkins, 1991; Tang & Yao, 2008; Yang, 2010).

In recent years, significant progress has been made in the development of innovative stochastic search methods within the field of structural optimization. These techniques, which are inspired by natural processes, effectively address the inconsistencies commonly associated with traditional mathematical programming-based optimization approaches. Typically, meta-heuristic algorithms aim to identify feasible solutions to optimization problems through a systematic 'trial and error' process within a reasonable computational timeframe (Arafa, Khalifa, & Alqedra, 2015). Numerous researchers have recently formulated and implemented diverse meta-heuristic strategies specifically designed for steel structures, accommodating both fully rigid and semi-rigid connections (Kameshki & Saka, 2003; Hayalioglu & Degertekin, 2004; Hayalioglu & Degertekin, 2005; Degertekin, Hayalioglu, & Gorgun, 2009; Rafiee, Talatahari, & Hadidi, 2013; Hadidi & Rafiee, 2014; Artar & Daloglu, 2015).

A thorough examination of the existing literature reveals a significant deficiency in studies utilizing metaheuristic algorithms for the optimal design of steel frames, specifically those incorporating semi-rigid connections and column bases. Among metaheuristic algorithms, the Particle Swarm Optimization (PSO) algorithm is well-regarded for its simplicity, computational efficiency, and ease of implementation. Unlike other methods that rely heavily on user-defined parameters, the PSO algorithm features a straightforward numerical structure governed by only a few control parameters: inertia weight (w), cognitive acceleration coefficient (c_1), and social acceleration coefficient (c_2). These parameters define the exploration-exploitation behavior of the swarm and can often be standardized for structural optimization problems (Clerc, 2010; Sengupta, Basak, & Peters, 2018). Moreover, PSO leverages these general parameters and does not require algorithm-specific operations such

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as crossover or pheromone update schemes, reducing implementation complexity (Papazoglou & Biskas, 2023; Adeyiga, Sotonwa, & Adenibuyan, 2022). In comparison, Genetic Algorithms (GA) require the specification of crossover probability, mutation rate, and selection method. Ant Colony Optimization (ACO) involves configuring four parameters and an extra parameter for search-space reduction in multiphase applications. Harmony Search (HS) necessitates the harmony memory consideration and pitch adjusting rate (Kennedy & Eberhart, 1995). The reduced parametric complexity of Particle Swarm Optimization (PSO) lessens its vulnerability to subjective tuning biases, enabling more robust convergence in complex design spaces (Clerc & Kennedy, 2002; Perez & Behdinan, 2007).

Consequently, the PSO technique can be utilized to enhance the design of steel frames featuring semi-rigid connections and column bases. By employing the PSO method, the optimal design for steel frames has been evaluated and compared to prior studies. Three particular case studies are examined for this analysis. The first case study originates from Değertekin (2008), while the other two are based on the work of Artar and Daloğlu (2015). The findings indicate that the PSO method performs effectively and is viable for addressing structural challenges.

2. Particle swarm optimization (PSO)

The PSO algorithm was initially conceived by Kennedy and Eberhart in 1995. Subsequent advancements to this algorithm were introduced by Pedersen and Chipperfield in 2010. PSO represents a stochastic search methodology that employs a population-based strategy for effectively exploring intricate solution spaces. The active foraging behavior of animal species, as they seek out food sources, serves as a basic concept for understanding this technique. This food-seeking behavior encompasses the formation of groups shaped by social influences that rely on the collective memory of each member and the data collected by the swarm.

The approach begins with random solutions and then progressively modifies the velocity and location of particles through iteration. Every particle is characterized by its position x_i^m and velocity v_i^m. The initial position x_i^0 is randomly scattered and taken from the given solution. The initial velocity v_i^0 is presumed to be zero. The initial parameters can be obtained from Eqs. 1-2.

$$x_i^0 = x_{min} + r(x_{max} - x_{min}) \tag{1}$$

$$v_i^0 = 0 \tag{2}$$

In the provided equations, r denotes a random integer that can take a value between 0 and 1. At the same time, x_{min} and x_{max} signify the lower and top limit of the design parameters, respectively.

This particle velocity reflects the extent of change, while particle positions denote possible solutions for optimal design challenges. Each particle is updated based on two optimal values during every iteration of its movement. The first optimal value is called $p_{best,i}^m$, which signifies the best position attained by the i^{th} particle. The second optimal value, p_{gbest}^m, indicates the best position among all particles at time m. The position and velocity of the i^{th} particle for the m iteration are adjusted according to Eqs. 3-4.

$$v_i^{m+1} = w^m v_i^m + c_1 r_1 \left(p_{best,i}^m - x_i^m\right) + c_2 r_2 (p_{gbest}^m - x_i^m) \tag{3}$$

$$x_i^{m+1} = x_i^m + v_i^{m+1} \tag{4}$$

In the formulas mentioned above, r_1 and r_2 serve as arbitrary values ranging from 0 to 1. The particle's inertia, denoted as w^m,influences the algorithm's exploration characteristics. The trust parameters c_1 and c_2 represent the coefficients for individual and collective learning, respectively. For further information on the PSO algorithm, one can refer to various sources, including Kennedy and Eberhart (1995), Saka and Geem (2013), and Gad (2022).

3. Two-dimensional steel frame modeling with semi-rigid connections

The analysis of the frame is based on the assumption that the connection between the column and beam is entirely rigid, facilitating straightforward calculations. However, experimental findings indicate that most connections exhibit moment-rotation behavior that lies between pinned and rigid connections. As a result, these connections can be classified as semi-rigid. In the context of simple models, Simoes (1996) presented a linear model for the analysis of semi-rigid frameworks. Furthermore, Soares Filho et al. (2004) employed a linear technique in examining a 20-storey steel frame employing linear static analysis. Nevertheless, it is important to note that the actual behavior of the connections across all moment-rotation curves is nonlinear (Artar & Daloglu, 2015). As a result, this paper acknowledges the moment-rotation relationship of the connection as being nonlinear.

In numerous published studies, it is often assumed that connections between beams and columns or between columns and their bases are entirely rigid (Toğan, 2012; Artar, 2016). Nonetheless, semi-rigid connections can range from being fully pinned to completely rigid, depending on their specific type. When analyzing the bending moment (M) at these connections, it is crucial to consider semi-rigid connections, as they can cause rotational movement (θ_r) (Artar & Daloglu, 2015). Figs. 1 and 2 depict moment-rotation curves and the types of connections, respectively (Hayalioglu & Degertekin, 2004).

In the present study, a polynomial model proposed by Frye and Morris (1975) is utilized. The subsequent equations, denoted as Eq. 5, pertaining to odd-power polynomials, elucidate this model:

$$\theta_r = C_1(\kappa M)^1 + C_2(\kappa M)^3 + C_3(\kappa M)^5 \tag{5}$$

In this context, κ is defined as the standardization constant, contingent upon the geometrical variables and the characteristics of the connections. The constants C_1, C_2, and C_3 serve as the coefficients employed for the purpose of curve fitting.

In the analysis of the steel frames, as mentioned in Fig. 2, two semi-rigid connections are applied in this study. Table 1 provides these two connections' curve fitting and standardization constants (Faella, Piluso, & Rizzano, 1999). The values for rotational stiffness are 2.26×10^8 kNmm/rad for end plates lacking column stiffeners and 3.39×10^8 kNmm\rad for end plates including column stiffeners (Hayalioglu & Degertekin, 2004). Additionally, the semi-rigid connection for the column base utilized by Hayaloglu and Degertekin (2004) is incorporated, as depicted in Fig. 3.

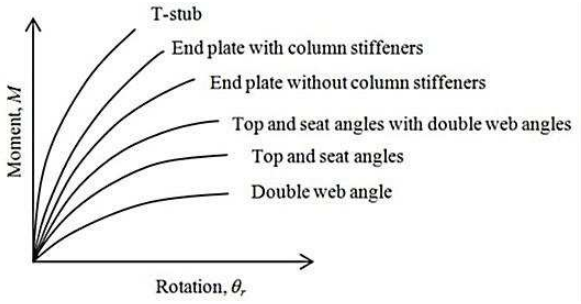


Fig. 1. The moment-rotation curves for semi-rigid connections (Hayalioglu & Degertekin, 2004)

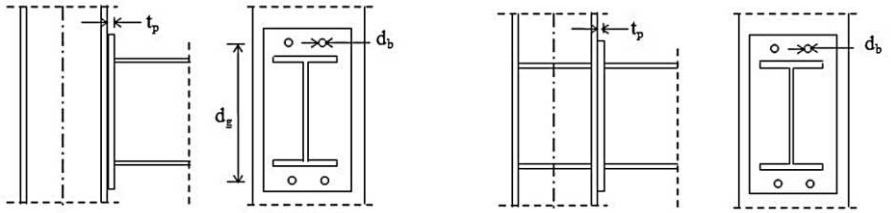


Fig. 2. Types of semi-rigid connections used in the study

Table 1. The details of semi-rigid connections used in the study

Connection Types	Curve fitting constants	Standardization constants
End plate without column stiffeners	$C_1 = 1.83 \times 10^{-3}$ $C_2 = 1.04 \times 10^{-4}$ $C_3 = 6.38 \times 10^{-6}$	$\kappa = d_g^{-2.4} t_p^{-0.4} d_b^{-1.5}$
End plate with column stiffeners	$C_1 = 1.79 \times 10^{-3}$ $C_2 = 1.76 \times 10^{-4}$ $C_3 = 2.04 \times 10^{-4}$	$\kappa = d_g^{-2.4} t_p^{-0.6}$

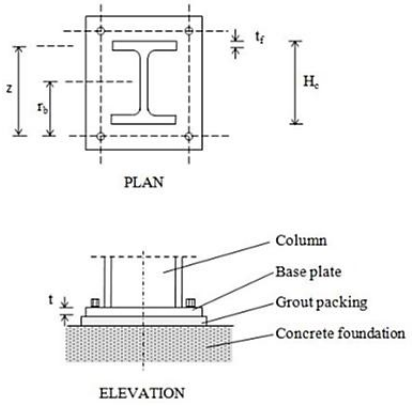


Fig. 3. Type of semi-rigid connection for column bases (Hayalioglu & Degertekin, 2004)

4. Mathematical formulation of optimal structural design

The lowest weight of unbraced steel structures is treated as the unconstrained objective function in discrete optimization problems. The formulations for the objective function and the penalized functions are provided in Eqs. 6-9 (Daloglu & Armutçu, 1998).

$$Min\ W_{sf} = \sum_{f=1}^{ng} A_f \sum_{i=1}^{nf} \rho_i L_i \tag{6}$$

$$\mathrm{g_i}(x) > 0 \rightarrow c_{vi} = \mathrm{g_i}(x) \tag{7}$$

$$\mathrm{g_i}(x) < 0 \rightarrow c_{vi} = 0 \tag{8}$$

$$\varphi_{sf}(x) = W_{sf}(x)\left(1 + P\sum_{i=1}^{m} c_{vi}\right) \tag{9}$$

In this context, the variable W_{sf} represents the total weight of the unbraced steel frame, while A_f denotes the cross-sectional area of element group f. The symbols ρ_i and L_i refer to the density and length of the steel members. The variable ng indicates the total number of groups, and nf signifies the total number of members within group f. The function $\varphi_{sf}(x)$ is referred to as the penalized objective function, and P representing the penalty constant. The variable c_{vi} corresponds to constraint violations, and $\mathrm{g_i}(x)$ designates the constraints applicable to the design.

This study considers various limitations based on Prakash and Daloglu (2023), which are expressed as follows:

The stress limits according to AISC-LRFD (2010) are presented in Eq. 10.

$$\mathrm{g_i}(x) = \begin{cases} \dfrac{P_u}{\phi P_n} + \dfrac{8}{9}\left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) - 1.0 \le 0 & if\ \dfrac{P_u}{\phi P_n} \ge 0.2 \\ \dfrac{P_u}{2\phi P_n} + \left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) - 1.0 \le 0 & if\ \dfrac{P_u}{\phi P_n} \le 0.2 \end{cases} \tag{10}$$

In this context, the symbol ϕ signifies the resistance factor, which is defined as ϕ_c for compression (0.85), ϕ_t for tension (0.90), and ϕ_b for flexural (0.90). P_n and P_u denote the nominal strength and required axial strength, respectively. M_{nx} and M_{ny} indicate the nominal flexural strength about the major and minor axes, respectively. M_{ux} and M_{uy} represent the necessary flexural strength around the major and minor axes, respectively, with the understanding that for a two-dimensional frame, M_{uy} is equal to zero.

The stress limits according to AISC-ASD (1989) are presented in Eqs. 11-12.

$$\mathrm{g_i}(x) = \left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}} F_{bx}\right)}\right] - 1.0 \le 0 \qquad i = 1, 2, \ldots, nc \tag{11}$$

$$\mathrm{g_i}(x) = \left[\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}}\right] - 1.0 \le 0 \qquad i = 1, 2, \ldots, nc \tag{12}$$

If $\frac{f_a}{F_a}$ is less than or equal to 0.15, then Eq. 13 is used for calculations instead of Eqs. 11 and 12.

$$\mathrm{g_i}(x) = \left[\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}}\right] - 1.0 \le 0 \qquad i = 1, 2, \ldots, nc \tag{13}$$

In this context, The variable nc represents the total number of members, and C_{mx} is a factor typically assumed to be 0.85 for unbraced frame members. The terms F_{bx} and f_{bx} correspond to the allowable compressive bending stresses and the computed bending stresses about the major axis, respectively. Furthermore, F'_{ex} indicates the Euler stresses, and F_y refers to the yield stress of steel. f_a denotes the computed axial stress, while F_a signifies the allowable axial stress under axial compression, which is determined by the elastic or inelastic buckling characteristics of the member depending on its slenderness ratio, influenced by the effective length factor (K).

According to Dumonteil (1992), the effective length factor K is defined in Eq. 14.

$$K = \sqrt{\frac{1.6G_A G_B + 4.0(G_A + G_B) + 7.50}{G_A + G_B + 7.50}} \tag{14}$$

In this context, G_A and G_B denote the stiffness factors corresponding to the A^{th} and B^{th} ends of the columns, which can be determined using the equations provided in Eq. 15.

$$G = \left(\frac{\sum I_c/L_c}{\sum \alpha_{uf}(I_g/L_g)}\right) \tag{15}$$

In this context, I_c denotes the moment of inertia related to the column section that aligns with the buckling plane, while L_c signifies the column length. I_g refers to the moment of inertia associated with the beam section pertinent to the bending plane, and L_g represents the length of the beam. The coefficient α_{uf} reflects the connection state, equal to 1 for rigid connections. Semi-rigid connections are determined using Eq. 16 (Doğan, 2010).

$$\alpha_{uf} = \frac{1}{\left(1 + \frac{6EI_g}{L_g K'}\right)} \tag{16}$$

Where, K' indicates the rotational spring stiffness at the member's end, expressed as M/θ_r.

The displacement constraints are delineated in Eq. 17.

$$g_{jl}(x) = \frac{\delta_{jl}}{\delta_{ju}} - 1.0 \leq 0 \qquad \begin{matrix} j = 1, \dots, cd \\ l = 1, \dots, nl \end{matrix} \tag{17}$$

In this context, δ_{jl} represents the displacement associated with the j^{th} degree of freedom for the specified load case l. Additionally, δ_{ju} indicates the established upper limit, while cd signifies the total count of constrained displacements. Lastly, nl refers to the overall number of loading scenarios considered.

Inter-storey drift constraints are delineated in Eq. 18.

$$g_{jil}(x) = \frac{\Delta_{jil}}{\Delta_{ju}} - 1.0 \leq 0, \quad i = 1, \dots, nsc, j = 1, \dots, ns, l = 1, \dots, nl \tag{18}$$

In this context, Δ_{jil} denotes the inter-storey drift of the i^{th} column in the j^{th} storey during load case l. The symbol Δ_{ju} indicates the permissible limit value, ns refers to the total number of stories, and nsc signifies the count of columns within a storey.

Column-to-column geometric constraints are delineated in Eq. 19.

$$g_n(x) = \frac{D_{un}}{D_{ln}} - 1.0 \leq 0 \qquad n = 2, \dots, ns. \tag{19}$$

In this context, D_{un} and D_{ln} signify the depths of the upper and lower floors, respectively.

Beam-to-column geometric constraints are delineated in Eq. 20.

$$g_{bb,i}(x) = \frac{b_{fbk,i}}{b_{fck,i}} - 1.0 \leq 0 \qquad i = 1, \dots, n_{bf}. \tag{20}$$

In this context, $b_{fbk,i}$ and $b_{fck,i}$ signify the flange widths of the beam and column elements, respectively.

5. Implementation and results

Three particular unbraced steel frames are examined in this study to explore the impact of semi-rigid connections between beams and columns, as well as between columns and bases, utilizing the PSO optimization algorithm. The first example has been analyzed by several researchers (Pezeshk, Camp, & Chen, 2000; Camp, Bichon, & Stovall, 2005; Degertekin S. O., 2008), while the last two examples were investigated by Hayalioglu and Degertekin (2004) and Artar and Daloglu (2015). In this study, the permissible lateral displacement is limited to $H/250$, with H representing the overall height of the steel frame in centimeters, and the maximum inter-storey drift is constrained to $h_r/\,300$, where h_r denotes the height of the storey in centimeters. The rotational stiffness for the semi-rigid column-to-base connection in all frames is set at 1.13×10^8 kNmm/rad. The frame members are assumed to be made of steel with a Young's modulus of $E_s = 200$ GPa, a yield stress of $f_y = 248.2$ MPa, and a material density of $\rho_i = 7.85$ ton/m^3.

5.1. Case study-1: 2-bay, 3-storey frame design

A 2-bay, 3-story, 2-dimensional frame is depicted in Fig. 4, comprised of 15 members. Various researchers have assessed the steel frame, such as Pezeshk et al. (2000), Camp et al. (2005), Degertekin (2008). In this study, the steel frame is optimized with and without semi-rigid connections using particle swarm optimization in a hybrid SAP2000 and C# programming language approach. The stress limitations of AISC-LRFD, displacement limits, and inter-story constraints are applied to both the fully rigid and semi-rigid frames. The maximum lateral deflection and inter-story drift are constrained to 3.658 cm and 1.016 cm, respectively.

The frame components are categorized into two groups: the first group includes beam components, while the second group comprises column components. For the design, beam components are chosen from a selection of 276-W profiles, and column components are chosen from 38-W profiles. Dumonteil (1992) approximation equation is applied to determine the effective length factor regarding the major axis (K_x). The effective length factor concerning the minor axis K_y is assumed to be 1.0 for all column components and is considered to be 1/6th of the span length for each beam component.

The optimal outcomes for the rigid frame are presented in Table 2 for comparison with existing solutions in the literature. The ideal cross-sections for the semi-rigid structure, featuring both end plates without column stiffeners and those with stiffeners at the connections, are also shown in Table 2. Fig. 5 illustrates the design history of total steel weight through iterations for both the rigid and semi-rigid frames.

According to the PSO solution presented in Table 2, which pertains to the rigid frame, the weight of the 2-dimensional frame aligns with the minimum optimal design established in previous studies. The minimum weight of 8165 kg obtained in this study is roughly 4.4%, 4.4%, and 1.6% lighter than the lowest weights reported by Pezeshk et al. (2000), Camp et al. (2005), and Değertekin (2008), respectively.

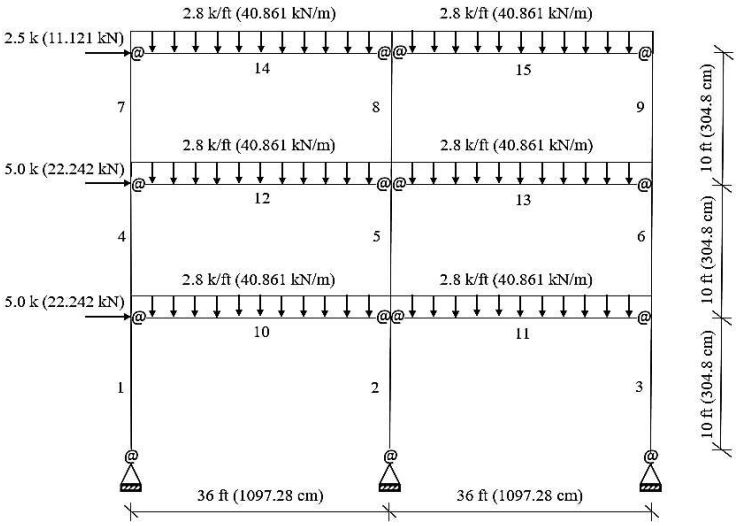


Fig. 4. A 2-bay, 3-storey steel frame

Table 2. Optimum design of 2-bay, 3-storey frame

Element Group	Member	Rigid frame				Semi-rigid frame	
						Present study	
		Pezeshk et al. (2000)	Camp et al. (2005)	Değertekin (2008)	Present study	End plate without column stiffeners	End plate with column stiffeners
		GA	ACO	HS	PSO	PSO	PSO
1 (Beam)	10-15	24 × 62	24 × 62	21 × 62	21 × 55	18 × 60	18 × 55
2 (Column)	1-9	10 × 60	10 × 60	10 × 54	10 × 68	10 × 68	10 × 77
Minimum weight (kg)		8524	8524	8297	8165	8655	8532
Maximum displacement (cm)		-	-	-	1.3947	1.6071	1.5433
Maximum inter-storey drift (cm)		-	-	-	0.8509	0.9571	0.8814

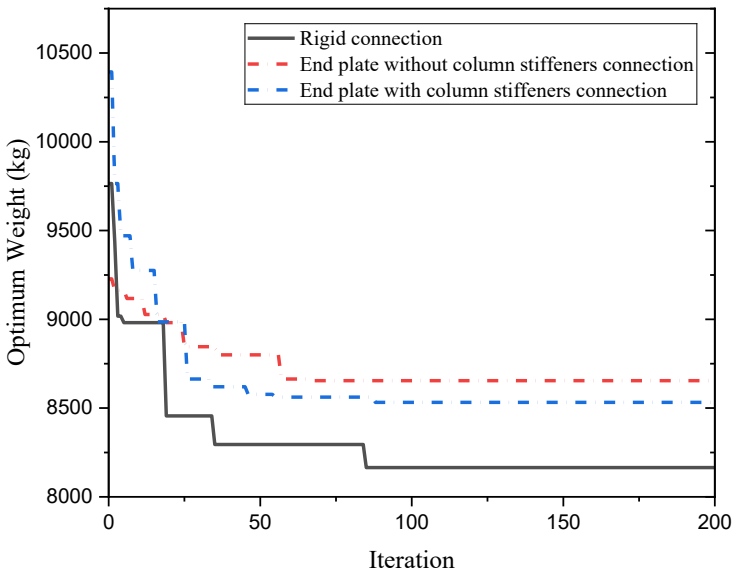


Fig. 5. Design histories of a 2-bay, 3-storey steel frame

Additionally, this research demonstrates a lower minimum weight than previous literature due to random initialization, stochastic search, and convergence criteria. The maximum top drift and inter-story drift are recorded at 1.3947 cm and 0.8509 cm, respectively. These values for top and inter-story drifts are significantly below the acceptable limits. Furthermore, as indicated in Table 2, the minimum weights for the semi-rigid frame with the end plate without column stiffeners and the end plate with column stiffeners are 6.0% and 4.5% heavier than the minimum weight of the rigid frame structure, respectively.

The optimal design of the steel frame is generated through the PSO algorithm after 200 iterations with a population of 20, resulting in an evaluation of 4000 frames. Additionally, a decrease in the rotational stiffness of the connection between the beam and the column leads to a considerable increase in the minimum weight. However, this research's findings are more cost-efficient than those of earlier studies.

5.2. Case study-2: 2-bay, 5-storey frame design

A 2-bay, 5-storey, 2-dimensional frame is depicted in Fig. 6, consisting of 25 elements. Hayaloglu and Degertekin (2004) and Artar and Daloglu (2015) employed a genetic algorithm to analyze the steel frame. This research optimizes the steel frame with and without semi-rigid connections using particle swarm optimization.

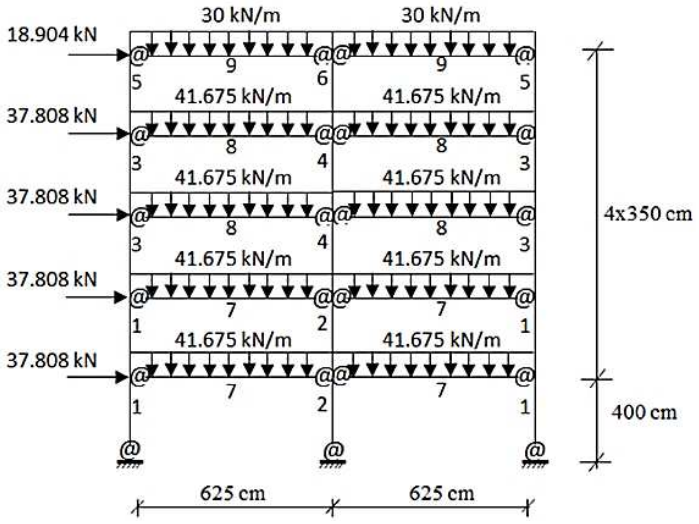


Fig. 6. A 2-bay, 5-storey steel frame

Table 3. Optimum design of 2-bay, 5-storey frame

Element Group	Rigid frame			Semi-rigid frame	
				Present study	
	Hayalioglu and Degertekin (2004)	Artar and Daloğlu (2015)	Present study	End plate without column stiffeners	End plate with column stiffeners
	GA	GA	PSO	PSO	PSO
1	18 × 55	21 × 68	16 × 36	12 × 120	21 × 55
2	18 × 65	16 × 100	24 × 146	21 × 73	21 × 68
3	14 × 61	16 × 45	12 × 53	10 × 60	18 × 71
4	14 × 43	1 × x50	21 × 101	16 × 67	14 × 193
5	14 × 43	12 × 26	8 × 10	8 × 10	8 × 10
6	8 × 31	8 × 24	16 × 89	8 × 10	8 × 18
7	18 × 50	24 × 55	18 × 46	14 × 43	16 × 40
8	14 × 43	24 × 55	8 × 48	12 × 53	21 × 50
9	21 × 50	18 × 35	8 × 10	8 × 10	8 × 10
Minimum weight (kg)	8888	9255	8840	9355	9208
Maximum displacement (cm)	3.68	2.17	5.10	6.74	4.97
Maximum inter-storey drift (cm)	-	-	1.254	1.822	1.395

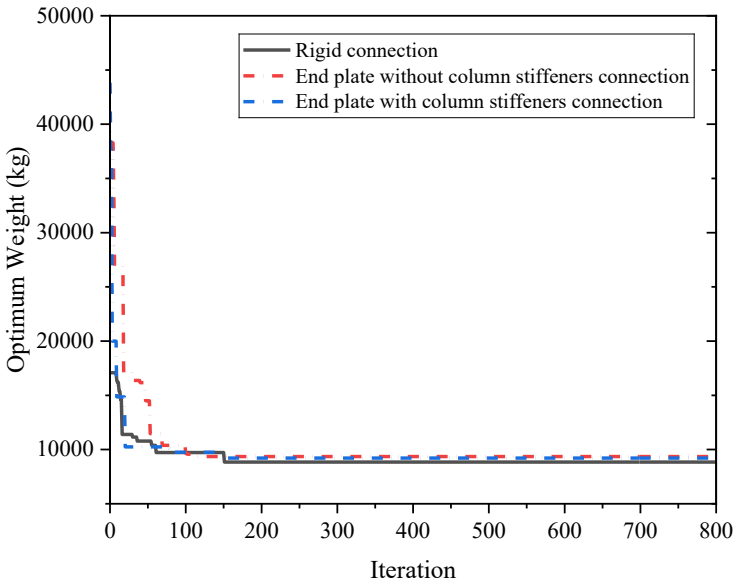


Fig. 7. Design histories of a 2-bay, 5-storey steel frame

The present study applies AISC-ASD stress limitations, restrictions on lateral displacement, and size constraints for column-to-column and beam-to-column connections to fully rigid and semi-rigid frames. The maximum allowable lateral displacement and inter-story drift are capped at 7.2 cm and 2.96 cm, respectively.

In the procedure, the cross-sections of the frame components are selected from a collection of 174-W sections provided by the American Institute of Steel Construction (AISC). Table 3 presents the optimal solutions identified for the completely rigid and semi-rigid frame based on solutions found in existing literature. Fig. 7 depicts the history of total steel weight design with iterations for both the rigid and semi-rigid frames.

Table 3 indicates that the optimal design outcomes from this research closely match the findings of earlier studies. According to the optimal design results for a fully rigid frame, the minimum weight achieved in this study is 8840 kg, which is about 0.5% and 4.5% lighter than the minimum weights reported by Hayaloglu and Degertekin (2004) and Artar and Daloglu (2015), respectively. This observation similarly applies to the optimal designs of semi-rigid frames (End Plate without Column Stiffeners and End Plate with Column Stiffeners connections, as detailed in Table 3). Furthermore, the weight of 8840 kg is roughly 5.5% and 4.0% below the minimum weights of 9355 kg and 9208 kg obtained from the optimal designs of the two semi-rigid frames, respectively. The maximum top and inter-story drifts are significantly below the allowable limits. As a result, stress and size constraints are crucial in achieving the best designs for fully and semi-rigid frames.

The PSO algorithm creates the optimal steel frame design after 800 iterations with a population of 20, resulting in the analysis of 16000 frames. Additionally, reducing the rotational stiffness value of the connection between the beam and column leads to a considerable increase in the minimum weight. Nonetheless, this study's findings are more cost-effective than earlier research's.

5.3. Case study-3: one-bay, 10-storey frame design

Fig. 8 depicts a one-bay, 10-storey 2-dimensional steel frame comprising 30 elements. Hayaloglu and Degertekin (2004) and Artar and Daloglu (2015) employed a genetic algorithm to examine the steel frame. The study optimizes the steel frame with and without semi-rigid connections using particle swarm optimization. This current study applies the same constraints as the earlier frame. The maximum lateral displacement and inter-story drift limits are set at 12 cm and 2.54 cm, respectively.

The cross-sectional dimensions of the frame components in the design are selected from 276-W sections as specified by the American Institute of Steel Construction (AISC). Table 4 summarizes the optimal solutions identified for the completely rigid and semi-rigid frames based on existing literature. Fig. 9 shows the total steel weight design histories with iterations for the rigid and semi-rigid frames.

Table 4 indicates that the optimal design results obtained from this study are very similar to the solutions found in existing literature. According to the optimal design for a fully rigid frame, the minimum weight of 12977 kg is approximately 7.3% heavier than the minimum weight reported by Hayaloglu and Degertekin (2004) and 7.0% lighter than the minimum weight noted by Artar and Daloglu (2015). This observation is also accurate for the optimal designs of semi-rigid frames (End Plate without Column Stiffeners and End Plate with Column Stiffeners connections, as shown in Table 4). Furthermore, the weight of 12977 kg is roughly 4.9% and 0.6% lighter than the

minimum weights of 13646 kg and 13054 kg obtained from the optimal designs of the two semi-rigid frames, respectively. The top and inter-story drift measurements are significantly below the allowable limits.

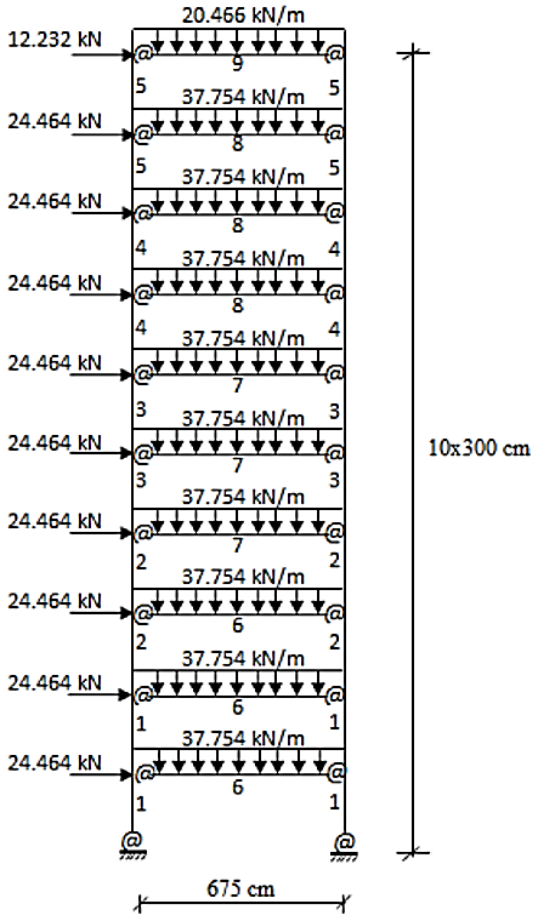


Fig. 8. A one-bay, 10-storey steel frame

Table 4. Optimum design of a one-bay, 10-storey frame

Element Group	Rigid frame			Semi-rigid frame	
				Present study	
	Hayalioglu and Degertekin (2004)	Artar and Daloğlu (2015)	Present study	End plate without column stiffeners	End plate with column stiffeners
	GA	GA	PSO	PSO	PSO
1	14 × 109	24 × 117	30 × 108	30 × 90	30 × 99
2	14 × 90	24 × 117	27 × 94	30 × 90	30 × 90
3	14 × 90	24 × 68	27 × 84	27 × 84	27 × 102
4	14 × 90	16 × 57	27 × 84	27 × 84	27 × 94
5	12 × 79	12 × 40	24 × 76	24 × 104	16 × 40
6	16 × 77	27 × 94	21 × 50	24 × 55	21 × 48
7	21 × 62	21 × 68	21 × 50	21 × 48	24 × 76
8	14 × 53	24 × 55	21 × 48	21 × 44	21 × 44
9	14 × 48	16 × 26	24 × 55	27 × 114	18 × 40
Minimum weight (kg)	12098	13951	12977	13646	13054
Maximum displacement (cm)	8.73	5.57	7.59	10.09	8.98
Maximum inter-storey drift (cm)	-	-	1.110	1.349	1.360

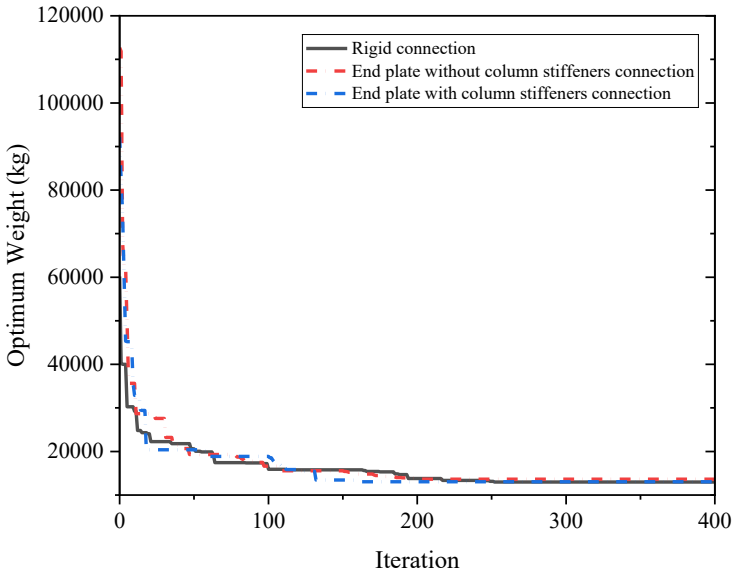


Fig. 9. Design histories of a one-bay, 10-storey steel frame

The PSO algorithm has yielded the most efficient steel frame design after conducting 400 iterations with a population of 40, resulting in an analysis of 16000 frames. In addition, reducing the rotational stiffness of the joint between the column and beam improves the minimum weight. Nevertheless, this research's findings are more economical than those from earlier studies.

6. Conclusion

The primary objective of this research is to utilize particle swarm optimization (PSO) for 2-dimensional frames featuring semi-rigid connections and column bases, aiming to determine the optimal weight of these steel frames. The high-level programming language C# implements the optimization technique to integrate with SAP2000 version 22 Open Application Programming Interface (OAPI). Stress constraints based on AISC-LRFD, along with displacement constraints and inter-story drift constraints, are applied in the first example. The subsequent two examples incorporate stress constraints based on AISC-ASD, displacement constraints, inter-story drift constraints, and geometric (dimension) constraints. The analysis results are presented in both tabular and graphical formats.

Key findings from the study are summarized below:

- The research outcomes across all three examples indicate that semi-rigid connections positively affect lateral displacements, leading to the selection of larger column profiles.
- Due to the influence of the rotational stiffness value, the minimum weight of the semi-rigid frames is greater than that of the fully rigidly connected frames.
- Analyzing the design histories of the 2-dimensional frames indicates that the third example was finalized after 400 iterations, whereas the first and second examples had 200 iterations and 800 iterations, respectively.
- Comparative analysis shows that the current study results are less weighted than previous works for the first two examples. However, for the third example, the current findings exhibit less weight than those of Artar and Daloglu (2015) yet greater weight than those of Hayalioglu and Degertekin (2004).

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