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RESEARCH ARTICLE

Teaching-learning-based optimization of unbraced steel frames with semi-rigid connections and column bases

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Abstract

The teaching-learning optimization (TLBO) technique is applied to unbraced steel frames with semi-rigid beam-to-column connections and semi-rigid column-to-base connections. The algorithm for design produces optimized steel frames by choosing appropriate sections from a set of standard steel sections. These standard steel sections, including wide-flange profiles (W), are provided by the American Institute of Steel Construction (AISC). The first example is subjected to the displacement, inter-story drift, and stress constraints of AISC-Load and Resistance Factor Design (LRFD) regulations. In addition, the displacement and stress limitations of the AISC-Allowable Stress Design (ASD) standard and the geometric (size) requirements are implemented in the last two examples. In finite element analysis, optimal designs of three alternative unbraced frames are performed with and without semi-rigid beam-to-column and column-to-base plate connections. All optimization techniques are encoded in the high-level programming language C# version 17.4.4 to integrate with SAP2000 version 22's Open Application Programming Interface (OAPI). The analysis results indicate that the rigidity of connections is determining parameter in the weight optimization of unbraced steel frames.

1. Introduction

Generally, structural optimization of steel frames starts with selecting steel sections for the columns and beams from a discrete set of accessible steel section tables. This approach minimizes the steel frame's weight while considering restrictions, including material, strength, displacement, deflection, size, lateral torsional buckling, and beam-to-column connection behavior [1]. Since the 1970s, computer-aided optimization has often obtained more cost-effective designs [2-4]. In the first half of the 1990s, Heuristic search methods evolved in response to the need for engineers and designers to realize more cost-efficient designs and to seek more effective optimization strategies [5-7].

In the past few years, unique and creative stochastic search approaches have emerged in structural optimization. These stochastic search approaches use natural concepts and are not subject to the inconsistencies of mathematical programming-based optimal design methods. Typically, meta-heuristic algorithms aim to discover an acceptable solution to any optimization issue by 'trial and error' in a fair amount

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of computational time [8]. Numerous academics have devised and implemented several meta-heuristic methods for steel structures with completely rigid or semi-rigid connections in recent years.

One of the meta-heuristic methods introduced by Rao et al. [9] is teaching-learning-based optimization. This algorithm is based on the teaching-learning process in a classroom between the teacher and the student. Rao and Patel [10] simulate this approach to efficiently solve multidimensional, linear, and non-linear problems. The fundamental TLBO algorithm is upgraded in this study to improve its exploration and exploitation capabilities by introducing the concepts of some teachers, adaptive teaching factors, tutorial training, and self-motivated learning. Toğan [11] recently used this novel method for the optimum design of three planar steel frames. The study investigated three-story, ten-story, and 24-story frame designs under the AISC-LRFD standard. Dede and Ayvaz [12] utilized this method to optimize a ten-bar truss system, a twenty-five-bar space truss structure, a seventy-two-bar truss structure, and a two-hundred-bar plane truss structure. Artar [13] investigated the optimal design of two braced steel frames using this technique.

Simoes [14] conducted one of the earliest investigations on optimizing frames with semi-rigid connections. His research showed a linear description of the spring for frames with semi-rigid connections. Soares Filho et al. [15] explored the behavior of frame material and connections represented by linear elastic moment-rotation relationships in stiffness form. Choi and Kim [16] employed non-linear inelastic analysis to optimize the design of semi-rigid steel frames. Wang and Li [17] studied the stability of semi-rigid composite frameworks. Genetic algorithms or harmony search techniques have been utilized to optimize non-linear steel frames with semi-rigid connections and column bases [18-20]. Artar and Daloğlu [21] concentrated on optimizing the design of composite steel frames with semi-rigid connections and column bases. In the study, five-story, ten-story, and twenty-story structures are optimized using a genetic algorithm under the AISC-ASD standard. Steel frames with semi-rigid connections and fixed bases have been analyzed by Shallan et al. [22], who present an optimization model developed using teaching-learning-based optimization (TLBO) and genetic algorithm (GA) methods.

As stated previously, metaheuristic algorithm-based studies for the optimal design of steel frames with semi-rigid connections and column bases are insufficient. In many metaheuristic algorithms, the TLBO algorithm has a reduced numerical structure and is independent of a number of parameters that define the method's performance. Moreover, TLBO does not require the tuning of any algorithm parameters. In comparison, GA requires the crossover probability, mutation rate, and selection method, whereas ACO requires four parameters and an additional parameter for search space reduction in multiphase applications, and HS requires the harmony memory consideration rate and pitch adjusting rate [11].

Therefore, TLBO can be applied to optimize the steel frame with semi-rigid connections and column bases. Using the TLBO method, the best design of steel frames with semi-rigid beam-to-column and semi-rigid column-to-base connections have been analyzed and compared with previous literature. Three specific examples are assessed for this purpose. The first example is from the Toğan [11], and the remaining two are derived from Artar and Daloğlu [21]. The results show that the TLBO method works well and can be used to solve structural problems.

2. Teaching-Learning-Based Optimization (TLBO)

The optimization method concentrates on learning and teaching methods between teachers and students. The teacher is regarded with the most significant amount of knowledge and conveys it to the students in the classroom to improve their performance. Rao et al. [9] created this novel optimization approach consisting of two fundamental phases: teaching and learning. These two procedures provide optimization solutions for structural problems. Numerous structural issues in the research literature have been solved using the TLBO approach [11-13,22]. No algorithm-specific control settings are necessary because the algorithm's main inputs are population size and generation number. All TLBO technique steps are described in the following bullet points:

- **Initialise a class.** The following Eqs. 1 represents numerous students in the class [11]:

$$\text{Class or Population} = \begin{bmatrix} x_1^1 & x_2^1 & \cdots & x_{nv-1}^1 & x_{nv}^1 \\ x_1^2 & x_2^2 & \cdots & x_{nv-1}^2 & x_{nv}^2 \\ \vdots & \vdots & \vdots & \vdots & \vdots \\ x_1^{ns-1} & x_2^{ns-1} & \cdots & x_{nv-1}^{ns-1} & x_{nv}^{ns-1} \\ x_1^{ns} & x_2^{ns} & \cdots & x_{nv-1}^{ns} & x_{nv}^{ns} \end{bmatrix} \begin{array}{c} f(x^1) \\ f(x^2) \\ \vdots \\ f(x^{ns-1}) \\ f(x^{ns}) \end{array} \tag{1}$$

where ns denotes the number of students (population size), nv indicates the number of design variables, and $f(x^{1,2,\ldots\ldots,ns-1,ns})$ represents the unconstrained objective function for every student in the class. The initial class is generated arbitrarily.

- **Teaching phase.** During this stage, a teacher uses the following Eqs. 2 to try to improve the class's average performance (i.e., population fitness) in the subject (i.e., variable) taught by them [22].

$$x^{new,i} = x^i + r(x_{teacher} - T_F x_{mean}) \tag{2}$$

where $x^{new,i}$ represents the new solution and x^i represents the current solution, r is a random number between 0 and 1, and T_F is a teaching factor (either 1 or 2). x_{mean} is the class mean as determined by Eqs. 3 [13].

$$x_{mean} = \left(mean(x_1) \ldots mean(x_{ns})\right) \tag{3}$$

In this phase, If the new solution $x^{new,i}$ comes up with a better solution $f(x^{new,i})$ than the current solution $(f(x^i))$, the current solution replaces the new solution.

- **Learner phase.** This phase's processes are like those stated in the previous stage. In this stage, learners engage in random interactions with other learners i and j to enhance their knowledge. Using the modification formula Eqs. 4, a better solution is obtained from the learning between students [9].

$$\begin{array}{ll} if\ f(x^i) < f(x^j) & \Rightarrow x^{new,i} = x^i + r(x^i - x^j) \\ \text{Otherwise} & \Rightarrow x^{new,i} = x^i + r(x^j - x^i) \end{array} \tag{4}$$

As previously stated during the teaching phase, if the new solution, $x^{new,i}$, is better $(f(x^{new,i}))$ than the current solution $(f(x^{,i}))$, the new solution is replaced with the current solution.

3. Analysis and modeling of steel frame with semi-rigid connections

The connection is assumed as completely pinned or rigid in the analysis of the frame. However, the results of the experiments demonstrate that most connections exhibit the moment-rotation behavior between these two connections (pinned and rigid); therefore, it can be categorized as a semi-rigid connection. For simple models, Simoes [14] indicated a linear representation of spring in the analyses of semi-rigid frames. Moreover, Filho et al. [15] adopted a linear approach and studied a 20-story steel frame using linear static analysis. In reality, the behavior of the connections along all moment-rotation curves is nonlinear [21]. Therefore, the moment-rotation relationship of the connection is regarded as nonlinear in this paper.

In many published studies, beam-to-column or column-to-base connections are assumed to be completely rigid [11,13]. However, semi-rigid connections vary from completely pinned to fully rigid, depending on the type of connection. When examining the bending moment (M) at the connections, it is vital to account for the semi-rigid connections, as this can lead to rotational movement θ_r [21]. Fig. 1 and Fig. 2 illustrate moment-rotation curves and connection types, respectively [18].

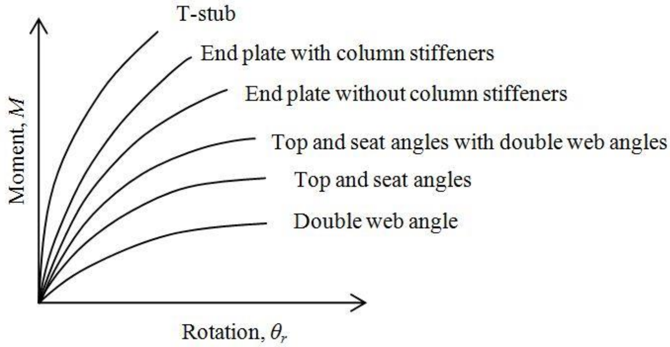

Fig. 1. Moment-rotation curves of semi-rigid connections [18]

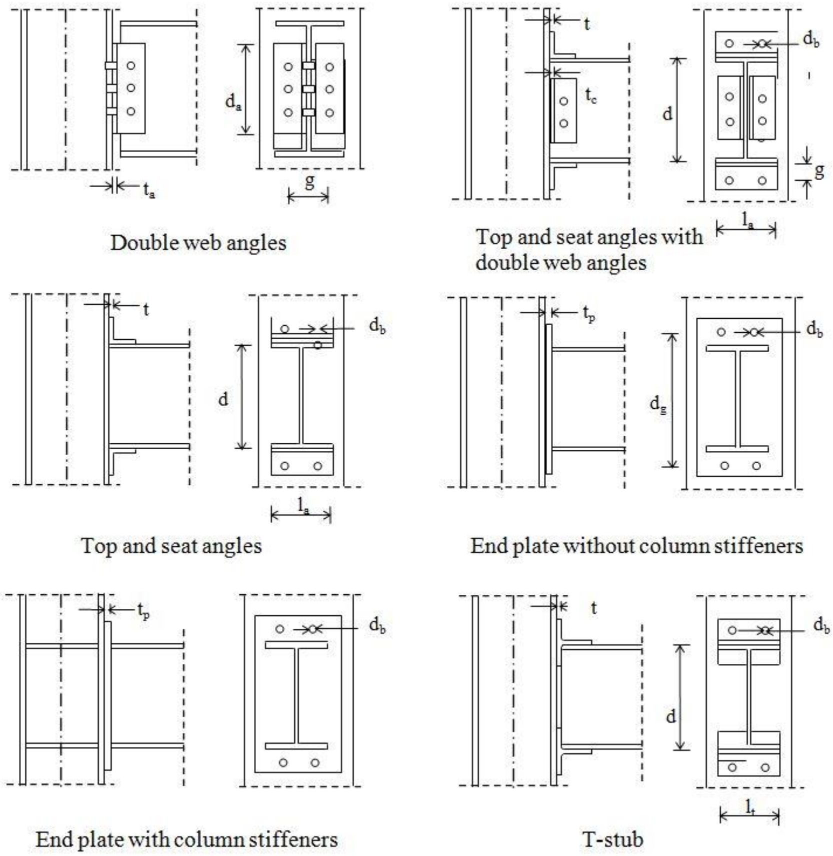

Fig. 2. Semi-rigid beam-to-column connection types [18]

In this work, a polynomial model suggested by Frye and Morris [23] is utilized due to its ease to use. The following Eqs. 5 for odd-power polynomials express this model:

$$\theta_r = c_1(\kappa M)^1 + c_2(\kappa M)^3 + c_3(\kappa M)^5 \tag{5}$$

where κ represents the standardization constant that depends on the geometrical parameters and type of connections, the constants c_1, c_2, and c_3 are the curve-fitting constants.

Within the context of an unbraced steel frame analysis, two of six possible semi-rigid connection types are implemented here. The value of curve fitting and standardization constants for two connections is provided in Table 1 [24]. Their accepted values for rotational stiffness are 2.26×10^8 kNmm/rad for end plates without column stiffeners and 3.39×10^8 kNmm/rad for end plates with column stiffeners [18]. These values for semi-rigid connections utilized in the analysis fall under permanent connection size parameters. These size parameters $t_p = 1.74$ cm and $d_b = 2.54$ cm are considered according to Hayaloglu and Degertekin [18]. Moreover, the same semi-rigid column-to-base characteristics utilized by Hayaloglu and Degertekin [18] are used in this investigation, as shown in Fig. 3.

4. The formulation of the optimal design

The minimum weight of unbraced steel frames is considered the unconstrained objective function in discrete problems. The objective and penalized functions are given below in Eqs. 6-9 [25].

$$\min w = \sum_{s=1}^{ng} A_s \sum_{i=1}^{ns} \rho_i L_i \tag{6}$$

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

$$\mathrm{g_i}(x) \leq 0 \rightarrow c_i = 0 \tag{8}$$

$$\varphi(x) = w(x)\left(1 + P\sum_{i=1}^{m} c_i\right) \tag{9}$$

where w denotes the weight of the steel frame, A_s indicates group (s) 's cross-section area, ρ_i and L_i denotes the member's density and length, ng symbolizes the total number of groups, ns represents the total number of members in group s. $\varphi(x)$ represents the penalized objective function, P represents the penalty constant, c_i represents constraint violation and g_i represents the constraints.

In the present study, several constraints are considered according to Toğan [11] for the first example and Hayalioglu and Degertekin [18] for the other two models, which are formulated in the following.

The stress constraints from AISC-LRFD [1] are given in Eqs. 10.

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

Table 1. The material features for the RC beam

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	$c_1 = 1.79 \times 10^{-3}$ $c_2 = 1.76 \times 10^{-4}$ $c_3 = 2.04 \times 10^{-4}$	$\kappa = d_g^{-2.4} t_p^{-0.6}$

where P_u and P_n represents the required axial and the nominal strength, respectively, M_{ux} and M_{uy} represent the required flexural strength about a major axis and minor axis, respectively (for a 2-dimensional frame, $M_{uy} = 0$), M_{nx} and M_{ny} represent the nominal flexural strength about a major axis and minor axis, respectively, ϕ represents the resistance factor indicated as ϕ_c for compression (0.85), ϕ_t for tension (0.90) and ϕ_b for flexural (0.90).

The stress constraints from AISC-ASD [26] are given in Eqs. 11-12.

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

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

If $\frac{f_a}{F_a} \leq 0.15$, Eqs. 13 is calculated rather than Eqs. 11 and Eqs. 12.

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

where f_a represents computed axial stress, F_a represents permissible axial stress under axial compression force alone, and it is based on the elastic or inelastic buckling of the member following the slenderness ratio, which is based on the effective length factor (K), F_{bx} and f_{bx} represent the permissible compressive bending stresses and the computed bending stresses about its major axis, respectively, F'_{ex} represents the Euler stresses, F_y represents the yield stress of steel, nc represents the total number of members, C_{mx} represents a factor that is assumed to be 0.85 for unbraced frame members. The effective length factor K is represented by Dumonteil [27] as follows in Eqs. 14.

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

where G_A and G_B represent the relative stiffness factors at A^{th} and B^{th} ends of columns which can be calculated by following Eqs. 15.

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

$$\alpha_{uf} = \frac{1}{\left(1 + \frac{6 E I_g}{L_g K^*}\right)} \tag{16}$$

where I_c represents the moment of inertia of the column section that corresponds to the buckling plane, L_c represents the length of the column, I_b represents the moment of inertia of the beam section that corresponds to the bending plane, L_c represents the length of the beam, K^* represents the rotational spring stiffness of the member end which expresses as M/θ, and α_{uf} represents a coefficient that indicates the state of the connection. It is equal to 1 for rigid connections. For semi-rigid connections, it is calculated by Eqs. 16 [28].

The displacement constraints are given in Eqs. 17.

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

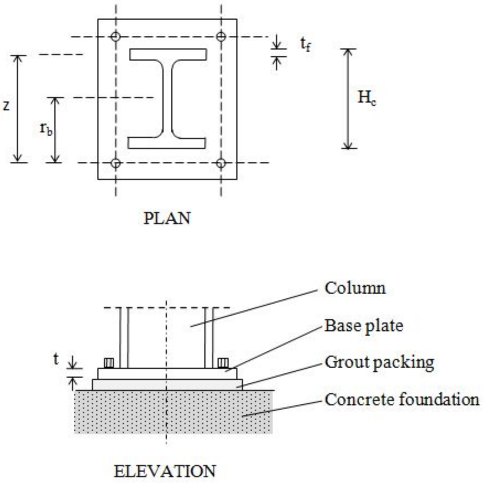

Fig. 3. Semi-rigid column-to-base connection types [18]

where δ_{jl} represents displacement of j^{th} degree of freedom under the load case l, δ_{ju} represents the upper bound, m represents the number of constrained displacements, nl represents the total number of loading cases.

Inter-story drift constraints are expressed in Eqs. 18.

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

where Δ_{jil} represents the inter-story drift of i^{th} column in the j^{th} the story under load case l, Δ_{ju} represents the limit value, ns represents the number of stories, nsc represents the number of columns in a story.

Column-to-column geometric constraints are shown in Eqs. 19.

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

where D_{un} and D_{ln} represent the depth of the upper and lower floors, respectively.

Beam-to-column geometric constraints are given in Eqs. 20.

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

where $b_{fbk,i}$ and $b_{fck,i}$ represent the flange width of the beam and column element, respectively.

5. Numerical examples

Three different unbraced steel frames are carried out in the present study to investigate the effect of semi-rigid beam-to-column connections and semi-rigid column-to-base connections using the TLBO optimization algorithm. The first example is studied by many researchers [11,29,30,31], and the other last two samples are examined by Hayalioglu and Degertekin [18] and Artar and Daloglu [21]. In the current study, the allowable lateral displacement is restricted to $H/250$, where H is the total height of the steel frame in cm [18], and the maximum inter-story drift is limited to $h_r/300$, where h_r is the story height in inches [21]. Rotational stiffness for column-to-base semi-rigid connection in all frames is adopted as 1.13×10^8 kNmm/

rad [20]. The frame members were presumed to be constructed from a steel material with the young modulus $E_s = 200$ GPa, yield stress $f_y = 248.2$ MPa, material density $\rho = 7.85$ ton/m^3 [21].

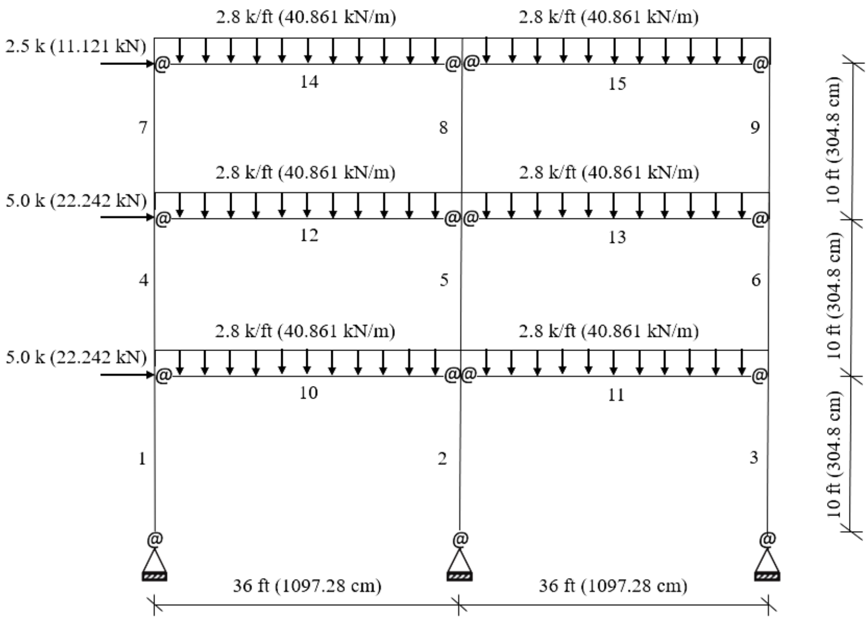

Fig. 4. A schematic diagram of a two-bay, three-story steel frame

Table 2. Optimum design of two-bay, three-bay story frame

Element Group	Member	Rigid frame					Semi-rigid frame	
							Present study	
		Pezeshk et al. [29]	Camp et al. [30]	Değertekin [31]	Toğan [11]	Present study	End plate without column stiffeners	End plate with column stiffeners
		GA	ACO	HS	TLBO	TLBO	TLBO	TLBO
1 (Beam)	10-15	24 × 62	24 × 62	21 × 62	24 × 62	21 × 55	21 × 62	18 × 55
2 (Column)	1-9	10 × 60	10 × 60	10 × 54	10 × 49	10 × 60	10 × 60	10 × 68
Minimum weight (kg)		8524	8524	8297	8069	7838	8524	8165
Maximum displacement (cm)		-	-	-	-	1.5048	1.5392	1.6425
Maximum inter-story drift (cm)		-	-	-	-	0.9445	0.9729	0.9659

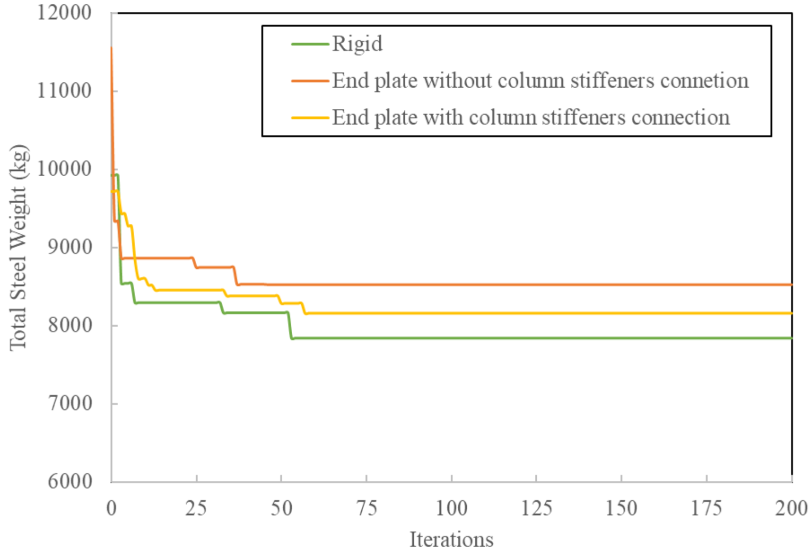

Fig. 5. Design histories of a two-bay, three-story steel frame

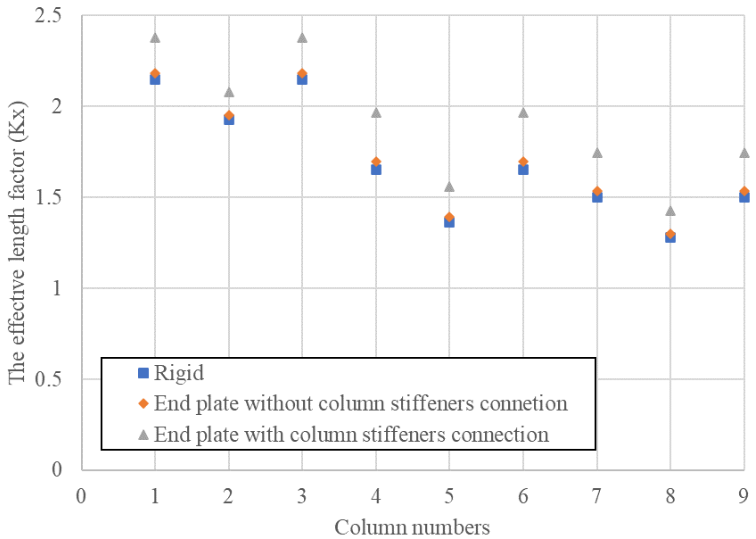

Fig. 6. Effective length factor (K_x) of column elements in the two-bay, three-story steel frame

5.1. Two-bay, three-story frame design

A two-bay, three-story unbraced frame is illustrated in Fig. 4, which is consisted of 15 members. Several authors evaluated the steel frame devoid of semi-rigid connections [11,29-31]. In the present work, the steel frame with and without semi-rigid connections is optimized by teaching-learning-based optimization in FEM software. The stress constraints of AISC-LRFD [1], displacement constraints, and inter-story constraints used by Doğan [11] are employed on the fully rigid and semi-rigid frame. The maximum lateral displacement and inter-story drift are limited to 3.658 cm and 1.016 cm, respectively.

The steel frame elements are divided into two groups: one group consists of beam elements, and the second group consists of column elements. In the design, beam elements are selected from the list of 276-W profiles, and column elements are selected from the list of 38-W profiles. Dumonteil's [27] approximation equation is utilized to get the effective length factor about the major axis K_x. The effective length factor about the minor axis K_y is considered 1.0 for all column elements and $1/6^{th}$ of the span length for each beam element.

The optimum results achieved for the rigid frame are stated in Table 2 to compare with the solutions available in the literature. The optimum cross-sections of the semi-rigid structure completed for the end plate without column stiffeners and end plate with column stiffeners semi-rigid connections are also illustrated in Table 2. Fig. 5 describes the total steel weight design histories with iteration for the rigid and semi-rigid frames. The values of the effective length factor along the major axis of rigid and semi-rigid frame column elements are depicted in Fig. 6.

Based on the TLBO solution in Table 2, which is based on the rigid frame, it can be said that the weight of the unbraced steel frame meets the minimum best design from the past literature. The minimum weight, 7838 kg, achieved by the current study is approximately 8%, 8%, 5.5%, and 2.86% less than the minimum weight gained by Pezeshk et al. [29], Camp et al. [30], Değertekin [31] and Toğan [11], respectively. It is also observed that this study has got a minimum weight than Toğan [11] because of random initialization, stochastic search, and convergence criteria [33]. The maximum top and inter-story drifts are 1.5048 cm and 0.9445 cm, respectively. The top and inter-story drifts are considerably less than the permissible values. It is also observed from Table 2 that the minimum weights of the semi-rigid frame with end plate without column stiffeners and end plate with column stiffeners are 8.75 and 4.17% heavier than the minimum weight of the rigid structure, respectively.

The optimal steel frame design is developed by the TLBO algorithm after 200 iterations with a population size of 20, leading to an analysis of 8000 frames. In addition, Fig. 6 reveals that the effective length factor about the major axis (K_x) for each column depends on the fixity factor, which is based on the rotational spring stiffness of the semi-rigid frame. Consequently, a reduction in rotational stiffness leads to increased rotation at the joints, a significant increase in lateral displacements, and an increase in the effective length factor (K_x). Therefore, columns with more extensive cross-section profiles are preferred in semi-rigid frame designs. Consequently, a reduction in the rotational stiffness value of the connection between the beam and the column causes a substantial increase in the minimum weight. Nonetheless, the results of this study are more cost-effective than those of previous research.

5.2. Two-bay, five-story frame

A two-bay, five-story unbraced steel frame is illustrated in Fig. 7, which is consisted of 25 elements. Hayaloglu and Degertekin [18] and Artar and Daloglu [21] utilized a genetic algorithm to analyze the steel frame with rigid and semi-rigid connections. The work optimizes the steel frame with and without semi-rigid connections using teaching-learning-based optimization in FEM software. The current work applies AISC-ASD stress constraints, lateral displacement restrictions, and column-to-column and beam-to-column size constraints to the completely rigid and semi-rigid frame. The maximum lateral displacement and inter-story drift are limited to 7.2 cm and 2.96 cm, respectively.

In the design, the steel frame elements' cross-sections are chosen from a list of 174-W sections from the American Institute of Steel Construction (AISC). Table 3 summarises the optimal cross-sections, minimal weight, maximum displacement, and maximum inter-story drift found for the completely rigid and semi-rigid frames with the solution available in the literature. Fig. 8 describes the total steel weight design histories with iteration for the rigid and semi-rigid frames. Fig. 9 illustrates the values of the effective length factor about the major axis of the column elements of the rigid and semi-rigid structure.

Table 3 shows that the optimal design results of this study are highly comparable to the solutions provided by previous literature. Based on the optimum design results of a fully rigid frame, the current study's minimum weight, 8765 kg, is approximately 1.38% and 5.29% less than the minimum weight gained by

Hayaloglu and Degertekin [18] and Artar and Daloglu [21], respectively. It also stands true for optimal designs of semi-rigid frames (End Plate without Column Stiffeners and End Plate with Column Stiffeners connections, as illustrated in Table 3). Additionally, the weight of 8765 kg is approximately 6.5% and 3.9% less than the minimum weights of 9375 kg and 9121 kg derived from optimal designs of the two semi-rigid frames, respectively. The top and inter-story drifts are considerably less than the permissible values. Consequently, stress and size constraints play a significant role in the best designs of both fully and semi-rigid frames.

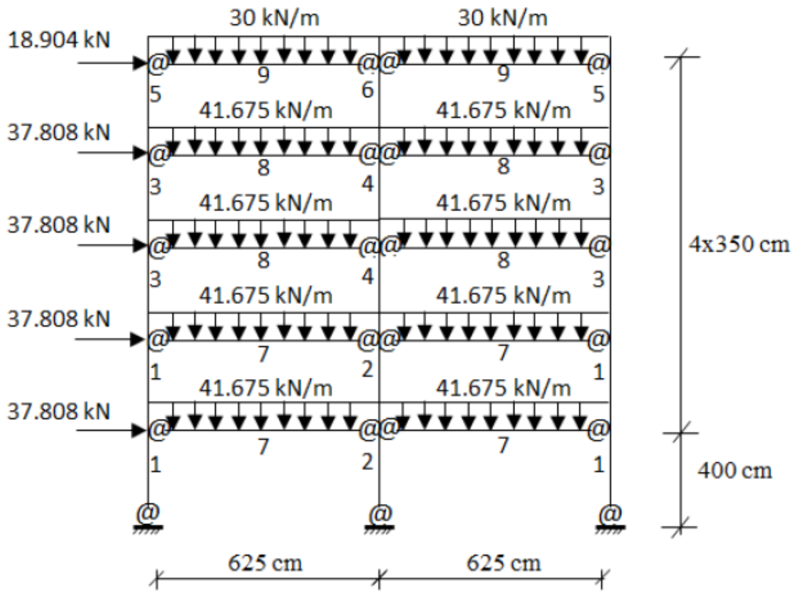

Fig. 7. A schematic diagram of a two-bay, five-story frame

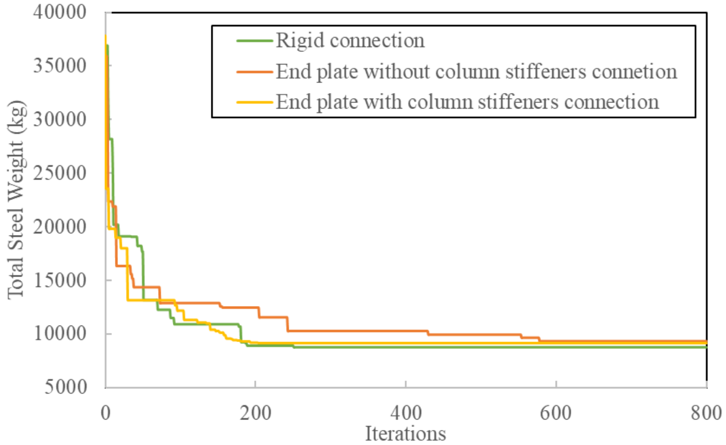

Fig. 8. Design histories of the two-bay, five-story steel frame

Table 3. Optimum design of a two-bay, five-story steel frame

Element group	Hayalioglu and Degertekin [18]			Artar and Daloğlu [21]			Present study		
		Semirigid			Semirigid			Semirigid	
	Fully rigid	End plate without column stiffeners	End plate with column stiffeners	Fully rigid	End plate without column stiffeners	End plate with column stiffeners	Fully rigid	End plate without column stiffeners	End plate with column stiffeners
	GA	GA	GA	GA	GA	GA	TLBO	TLBO	TLBO
1	18×55	21×62	21×62	21×68	21×68	16×67	21×55	18×50	24×68
2	18×65	21×62	21×68	16×100	24×94	21×101	24×68	21×62	24×68
3	14×61	16×67	21×62	16×45	12×45	14×48	21×55	14×68	21×48
4	14×43	21×62	14×61	$1 \times x50$	16×57	16×50	21×62	18×76	21×48
5	14×43	12×53	14×61	12×26	8×28	8×24	18×50	14×43	14×38
6	8×31	14×61	10×54	8×24	8×31	14×30	14×61	14×82	18×50
7	18×50	18×60	21×62	24×55	21×62	24×68	21×55	21×57	18×55
8	14×43	12×50	21×62	24×55	16×57	21×50	21×48	21×55	21×48
9	21×50	8×40	14×53	18×35	18×35	18×35	10×17	10×17	18×46
Minimum weight (kg)	8888	9831	10432	9255	9646	9616	8765	9375	9121
Maximum displacement (cm)	3.68	4.48	3.30	2.17	3.28	2.75	2.49	3.48	3.15
Maximum inter-story drift (cm)	-	-	-	-	-	-	0.631	1.079	0.914

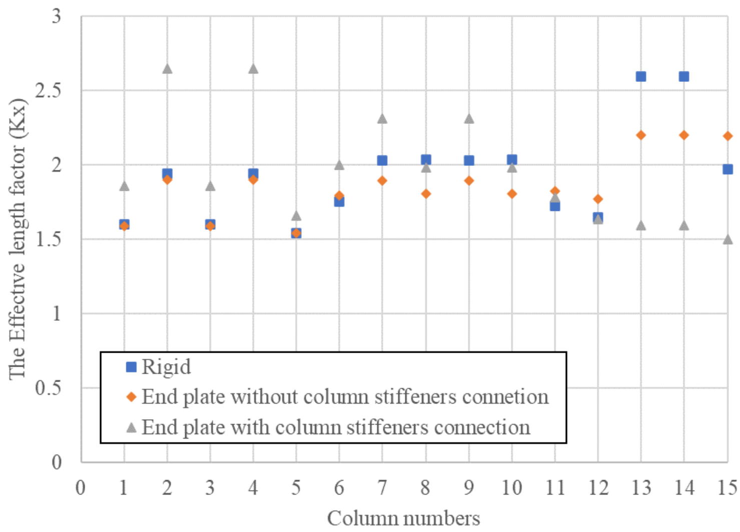

Fig. 9. Effective length factor (K_x) of column elements in a two-bay, five-story steel frame

The optimal steel frame design is developed by the TLBO algorithm after 800 iterations with a population size of 20, leading to an analysis of 32000 frames. In addition, Fig. 9 reveals that the effective length factor about the major axis (K_x) for each column depends on the fixity factor, which is based on the rotational spring stiffness of the semi-rigid frame. Consequently, a reduction in rotational stiffness leads to increased rotation at the joints, a significant increase in lateral displacements, and an increase in the effective length factor (K_x). Therefore, columns with more extensive cross-section profiles are preferred in semi-rigid frame designs. Consequently, a reduction in the rotational stiffness value of the connection between the beam and the column causes a substantial increase in the minimum weight. Nonetheless, the results of this study are more cost-effective than those of previous research.

5.3. Single-bay, ten-story frame

A Single-bay, ten-story unbraced steel frame is illustrated in Fig. 10, which is consisted of 30 elements. Hayaloglu and Degertekin [18] and Artar and Daloglu [21] utilized a genetic algorithm to analyze the steel frame with rigid and semi-rigid connections. The work optimizes the steel frame with and without semi-rigid connections using teaching-learning-based optimization in FEM software. The present work imposes the same constraints as the preceding frame. The maximum lateral displacement and inter-story drift are limited to 12 cm and 2.54 cm, respectively.

The steel frame elements' cross-sections in the design are chosen from 276-W sections from the American Institute of Steel Construction (AISC). Table 4 summarises the optimal cross-sections, minimal weight, maximum displacement, and maximum inter-story drift found for the completely rigid and semi-rigid frames with the solution available in the literature. Fig. 11 describes the total steel weight design histories with iteration for the rigid and semi-rigid frames. Fig. 12 illustrates the values of the effective length factor about the major axis of the column elements of the rigid and semi-rigid structure.

Table 4 shows that the optimal design results of this study are highly comparable to the solutions provided by previous literature. Based on the optimal design findings of a fully rigid frame, the minimum weight, 12785 kg, is around 5.68% greater than the minimum weight gained by Hayaloglu and Degertekin [18] and 8.38% less than the minimum weight achieved by Artar and Daloglu [21]. It also stands true for optimal designs of semi-rigid frames (End Plate without Column Stiffeners and End Plate with Column Stiffeners connections, as illustrated in Table 4). Additionally, the weight of 12785 kg is approximately 5.71% and

2.26% less than the minimum weights of 13560 kg and 13081 kg derived from optimal designs of the two semi-rigid frames, respectively. The top and inter-story drift are considerably less than the permissible values. Consequently, stress and size constraints play a significant role in the best designs of both fully and semi-rigid frames.

The optimal steel frame design is developed by the TLBO algorithm after 400 iterations with a population size of 40, leading to an analysis of 32000 frames. In addition, Fig. 12 reveals that the effective length factor about the major axis (K_x) for each column depends on the fixity factor, which is based on the rotational spring stiffness of the semi-rigid frame. Consequently, a reduction in rotational stiffness leads to increased rotation at the joints, a significant increase in lateral displacements, and an increase in the effective length factor (K_x). Therefore, columns with more extensive cross-section profiles are preferred in semi-rigid frame designs. Consequently, a reduction in the rotational stiffness value of the connection between the beam and the column causes a substantial increase in the minimum weight. Nonetheless, the results of this study are more cost-effective than those of previous research.

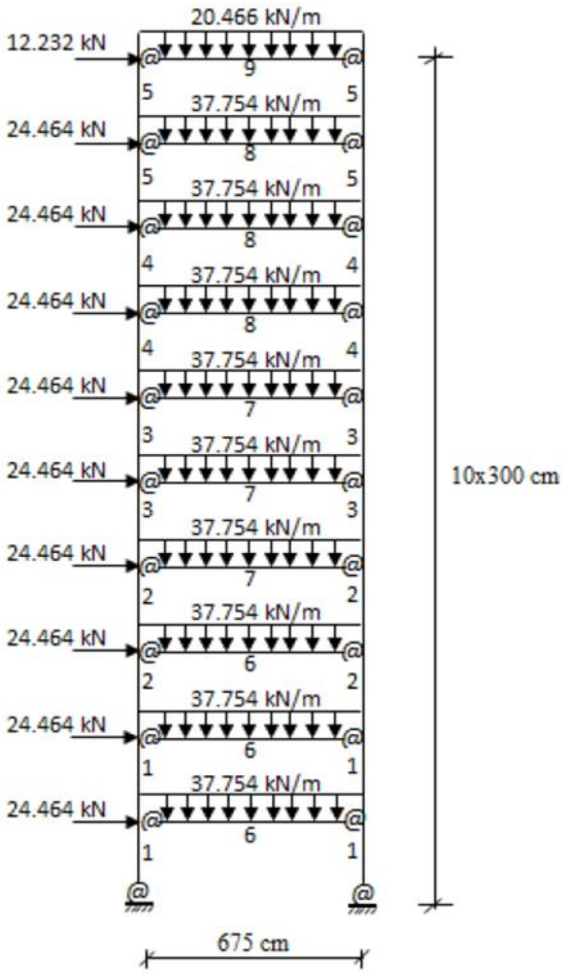

Fig. 10. A schematic diagram of a single-bay, ten-story frame

Table 4. Optimum design of a single-bay, ten-story steel frame

Element group	Hayalioglu and Degertekin [18]			Artar and Daloğlu [21]			Present study		
		Semirigid			Semirigid			Semirigid	
	Fully rigid	End plate without column stiffeners	End plate with column stiffeners	Fully rigid	End plate without column stiffeners	End plate with column stiffeners	Fully rigid	End plate without column stiffeners	End plate with column stiffeners
	GA	GA	GA	GA	GA	GA	TLBO	TLBO	TLBO
1	14×109	14×109	14×109	24×117	24×117	21×101	30×124	30×108	30×108
2	14×90	14×109	14×109	24×117	21×101	21×101	27×84	27×84	30×90
3	14×90	14×90	12×79	24×68	21×101	18×76	24×68	27×84	30×90
4	14×90	12×79	12×79	16×57	16×67	18×60	24×55	27×84	24×68
5	12×79	12×79	12×79	12×40	16×67	16×36	21×48	24×68	21×48
6	16×77	18×60	21×83	27×94	24×76	30×108	24×68	24×68	24×68
7	21×62	18×86	18×76	21×68	18×76	18×76	24×62	24×62	24×62
8	14×53	21×68	21×73	24×55	24×68	21×57	24×55	21×48	18×50
9	14×48	18×65	18×65	16×26	16×26	14×30	21×44	24×55	21×44
Minimum weight (kg)	12098	12691	15053	13951	15016	14245	12785	13560	13081
Maximum displacement (cm)	8.73	11.71	11.89	5.57	8.46	7.55	5.74	8.58	8.13
Maximum inter-story drift (cm)	-	-	-	-	-	-	0.781	1.153	1.029

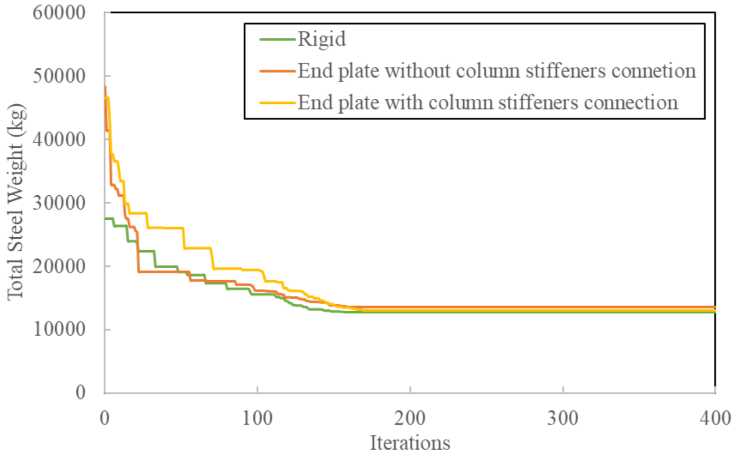

Fig. 11. Design histories of a single-bay, ten-storey frame

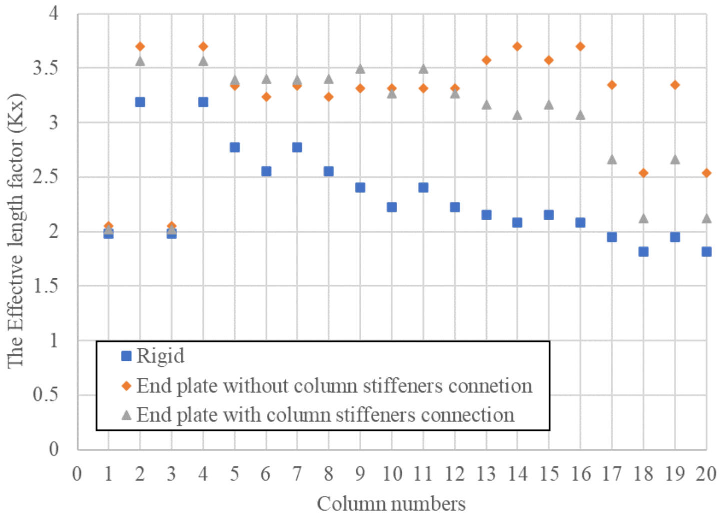

Fig. 12. Effective length factor (K_x) of column elements in a single-bay, ten-story steel frame

6. Conclusions

The primary purpose of this study is to apply teaching-learning-based optimization (TLBO) to the unbraced steel frames with semi-rigid connections and column bases to determine the optimal weight. The optimization technique is encoded in the high-level programming language C# to integrate with SAP2000 version 22 Open Application Programming Interface (OAPI). AISC-LRFD stress constraints, displacement constraints, and inter-story drift constraints are applied to the first example. In two other examples, the AISC-Allowable Stress Design (ASD) stress constraints, displacement constraints, inter-story drift constraints, and geometric (dimension) constraints are implemented. The analysis outcomes are shown in tabular and graphical formats. The most significant findings of the study are briefly presented below:

- The research findings from all three examples indicate that semi-rigid connections positively impact lateral displacements, resulting in the selection of more enormous profile columns. However, due to the rotational stiffness value, the semi-rigid structure's minimum weight is heavier than the minimum weight of an unbraced steel frame.

- Based on the design histories of the unbraced steel frames, the third example is completed with 400 iterations, whilst the first example and second example are constructed with 200 iterations and 800 iterations, respectively.
- When frames' rotational spring stiffness or fixity factor reduces, the values of the effective length factor (K_x) and the buckling lengths of columns increase simultaneously. In the first case, the highest (K_x) value for a fully rigid frame is approximately 2.15, which reaches 2.37 for a semi-rigid structure. It is also valid for other examples. As a result, optimal design weights decrease with the rotational spring stiffness increase.
- The comparisons demonstrate that the present work carries less weight than the preceding ones for the first two examples. But for the third example, the current work has less weight than Artar and Daloglu [21] and heavier weight than Hayalioglu and Degertekin [18].

Conflict of interests

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

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Author contributions

Jay Prakash: Conceptualization, Methodology, Data curation, Writing-Original Draft Preparation. Ayşe Daloglu: Supervision, Writing-Reviewing and Editing.

Data availability statement

The data presented in this study are available upon request from the corresponding author.

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