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Optimum design of composite steel frames with semi-rigid connections and column bases via genetic algorithm

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Abstract. A genetic algorithm-based minimum weight design method is presented for steel frames containing composite beams, semi-rigid connections and column bases. Genetic Algorithms carry out optimum steel frames by selecting suitable profile sections from a specified list including 128 W sections taken from American Institute of Steel Construction (AISC). The displacement and stress constraints obeying AISC Allowable Stress Design (ASD) specification and geometric (size) constraints are incorporated in the optimization process. Optimum designs of three different plane frames with semi-rigid beam-to-column and column-to-base plate connections are carried out first without considering concrete slab effects on floor beams in finite element analyses. The same optimization procedures are then repeated for the case of frames with composite beams. A program is coded in MATLAB for all optimization procedures. Results obtained from the examples show the applicability and robustness of the method. Moreover, it is proved that consideration of the contribution of concrete on the behavior of the floor beams enables a lighter and more economical design for steel frames with semi-rigid connections and column bases.

Keywords: AISC-ASD; genetic algorithm; weight optimization; composite beams; semi-rigid connection

1. Introduction

Minimum weight design of steel structures by using algorithm methods is one of major research areas in structural engineering. Various algorithms such as Genetic Algorithm (GA), Harmony Search Algorithm (HS), Ant Colony Algorithm (ACA), Particle Swarm Optimizer (PSO), Artificial Bee Colony Algorithm (ABC), Tabu Search Algorithm (TS), Simulated Annealing (SA) Algorithm, Teaching-Learning-Based Optimization Algorithm (TLBO) methods have been developed and performed for steel structures with fully rigid or semi-rigid connections by many researchers in the recent years. Although the first studies in the literature are mostly on the fully rigid frames or truss systems, subsequent studies are usually on the frames with semi-rigid connections.

Genetic Algorithm (GA) which is one of the above methods, was developed by Goldberg

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(1989). Rajeev and Krishnamoorthy (1992) used genetic algorithms for discrete optimization of structures. Although they investigated some simple structural examples including truss systems, their study explaining genetic algorithm procedures in details is an important reference for several researchers. Daloglu and Armutcu (1998) used genetic algorithm for optimum design of plane steel frames according to TS 648 (Turkish Building Code for Steel Structures). Their work includes some examples of planar frames and illustrates the application of the method. Erbatur *et al*. (2000) optimized planar and of space structures such as a 25-bar space truss, a 72-bar transmission tower, a 112-bar steel dome, a 22-bar plane truss, a cantilever beam and an industrial building by using genetic algorithms. In the later years, many studies on optimum design of fully rigid frames have been carried out for multi-storey steel frames using different methods. One of them is the study of Esen and Ülker (2008). They studied optimization of multi storey space steel frames by taking into account both materially and geometrically properties non-linear behaviors via the ANSYS program. Togan (2012) focused on optimum design of planar steel frames using Teaching–Learning Based Optimization which is a nature-inspired search method developed recently. In the study, three-story frame design, ten-story frame design and 24-story frame design were researched according to the AISC-LRFD specification. Aydogdu and Saka (2012) used Ant Colony Optimization for optimum design of irregular steel space frames including element warping effect. In the study, the design constraints were presented in details and different examples such as five-storey, 10-storey and 20-storey space frames were successfully optimized. Kaveh and Talatahari (2012) investigated a hybrid CSS and PSO algorithm for optimal design of structures to solve different examples such as a 942-bar spatial truss, 10-story spatial frame and a 60-elements grillage system. Dede and Ayvaz (2013) optimized a 10-bar truss system, a 25-bar space truss structure, a 72-bar truss structure and a 200-bar plane truss structure by using Teaching-Learning-Based Optimization Algorithm.

One of the first studies on optimization of frames with semi-rigid connections was carried out by Simoes (1996). His study showed that a linear representation of the spring for frames with semi-rigid connections is quite adequate for the simple models. Filho *et al*. (2004) investigated the behavior of the frame material and connections described by linear elastic moment-rotation relationships, which are presented in the stiffness form. In their study, the moment-rotation relation of the connection is considered as linear elastic and a 20-storey steel frame was studied by linear static analysis. Choi and Kim (2006) focused on optimal design of semi-rigid steel frames using practical nonlinear inelastic analysis. Wang and Li (2007) researched stability analysis of semi-rigid composite frames. However, genetic algorithms or harmony search algorithms have been used for optimum design of nonlinear steel frames with semi-rigid connections (Kameshki and Saka 2001, Hayalioglu and Degertekin 2004a, b, Degertekin *et al*. 2009) and column bases (Hayalioglu and Degertekin 2005, 2010). Gorgun and Yılmaz (2012) studied geometrically nonlinear analysis of plane frames with semi-rigid connections accounting for shear deformations. In the study, they applied the nonlinear analysis method to three different planar steel structures. Hadidi and Rafiee (2014) used a harmony search based, improved Particle Swarm Optimization method for minimum cost design of semi-rigid steel frames. They optimized a nine-storey, single-bay frame, a ten-storey, four-bay frame and a twenty four-storey, three-bay frame systems to show the applicability of the method.

Many studies in the literature, as mentioned above, have been carried out for the optimum design of steel frames with semi-rigid connections and fewer studies on the steel frames with semi-rigid column bases. Moreover, concrete slab effects on the behavior of beams are not considered in analyses of these literature studies. So, in the present study, in order to compare results, optimum design of steel frames with semi-rigid steel beam to column connections and column bases are researched for the cases with and without considering concrete slab effects in FEM analyses. For this purpose, three different examples are examined. Two of them are taken from literature. Results obtained from the optimum designs of the semi-rigid frames with composite beams proved that the consideration of the concrete slab contribution on the behavior of beams provides lighter frames with semi-rigid connections and column bases.

2. Genetic algorithm

Genetic Algorithm (GA) conducts natural biological steps such as reproduction, crossover, mutation, etc. GA analyses start with random initial population comprised of individuals which are coded as binary digits. The binary codes of each individual in population are decoded and corresponding profiles are selected from available section lists. According to selected profiles, frames corresponding each individual are analyzed with finite element method (FEM). Then, objective, penalized objective and fitness functions are determined by FEM analyses results obtained. According to these results of individuals in the population, the individuals are arranged and reproduction, double-point crossover and mutation operators are applied. Thus, the initial population is replaced by a new population. An iteration step consists of these procedures and iterations are repeated until the convergence is obtained. More detailed information about GA steps can be found in the literature (Daloglu and Armutcu 1998, Kameshki and Saka 2001, Hayalioglu and Degertekin 2004a, 2005).

3. Analysis and connection details of planar frames with semi-rigid connections

In fact, the behaviors of the connections are nonlinear along all moment-rotation curves (Filho *et al*. 2004). However, suitability of a linear representation of spring in the analyses of semi-rigid frames for simple models was indicated by Simoes (1996). Moreover, it is assumed by Filho *et al*. (2004) that a linear approach is generally enough for the analysis of frames and they studied a 20-storey steel frame by using linear static analysis.

According to first-order analysis, the local stiffness matrix of a planar member with semi-rigid end connections is defined by Eq. (1) (Simoes 1996, Filho *et al*. 2004).

$$k_l = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3}\frac{(\alpha_1+\alpha_2+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} & \frac{6EI}{L^2}\frac{(2\alpha_1+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} & 0 & -\frac{12EI}{L^3}\frac{(\alpha_1+\alpha_2+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} & \frac{6EI}{L^2}\frac{(2\alpha_1+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} \\ & & \frac{4EI}{L}\frac{(3\alpha_1)}{(4-\alpha_1\alpha_2)} & 0 & -\frac{6EI}{L^2}\frac{(2\alpha_1+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} & \frac{2EI}{L}\frac{(3\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} \\ & & & \frac{EA}{L} & 0 & 0 \\ & & & & \frac{12EI}{L^3}\frac{(\alpha_1+\alpha_2+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} & -\frac{6EI}{L^2}\frac{(2\alpha_2+\alpha_1\alpha_2)}{(4-\alpha_1\alpha_2)} \\ & & & & & \frac{4EI}{L}\frac{(3\alpha_2)}{(4-\alpha_1\alpha_2)} \end{bmatrix} \tag{1}$$

$$\alpha_1 = \frac{1}{1+3EI/S_1L}; \quad \alpha_2 = \frac{1}{1+3EI/S_2L} \tag{2}$$

$$K = T^t k T \tag{3}$$

where L, A and I is length, area of cross-section and moment of inertia of the member, respectively. E is elastic modulus, α_1 and α_2 are fixity factors defined by Eq. (2). The values of fixity factor change between 1 and 0 indicating fully rigid and pinned joints, respectively. S_1 and S_2 in Eq. (2) are rotational spring stiffness values of semi-rigid connections. After local matrix of each member in the frame is defined, global stiffness matrix expression is evaluated by Eq. (3). Finally, displacements and the nodes and stresses of each member in the planar frame are easily calculated by using finite elements methodology.

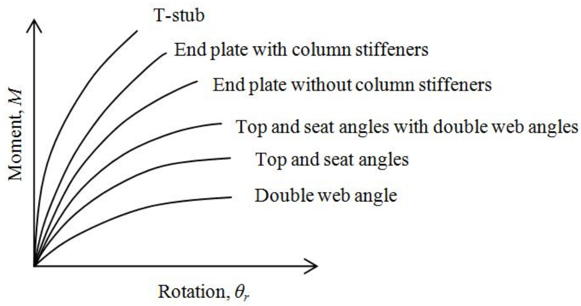


Fig. 1 Moment-rotation curves of semi-rigid connections (Hayalioglu and Degertekin 2004a)

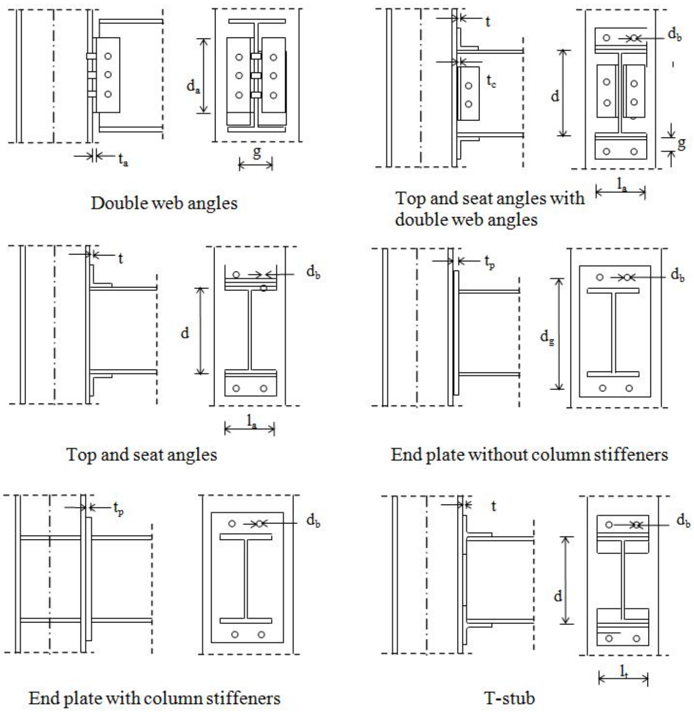


Fig. 2 Semi-rigid beam-to-column steel connection types (Hayalioglu and Degertekin 2004a)

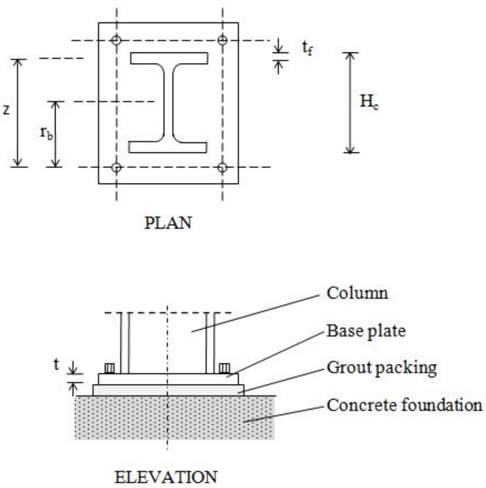


Fig. 3 Semi-rigid column-to-base details (Hayalioglu and Degertekin 2004a)

Beam-to-column or column-to-base connections are usually assumed as fully rigid in some of the studies in literature (Daloglu and Armutcu 1998, Esen and Ulker 2008, Togan 2012, Aydogdu and Saka 2012). However, semi-rigid connections change between totally pinned and fully rigid according to connection types. Types of semi-rigid connections are very significant for bending moment (M) at the connections since that leads to some rotation. Moment-rotation curves and connection types are shown in Figs. 1 and 2 (Hayalioglu and Degertekin 2004a).

In the present work, two of six different semi-rigid beam-to-column connection types are applied in frame analysis with or without considering concrete slab effects on behavior of the floor beams. These two types and their adopted rotational stiffness values are 2.26×10^8 kNmm/rad for end plate without column stiffeners, and 3.39×10^8 kNmm/rad for end plate with column stiffeners, (Hayalioglu and Degertekin 2004a). These rotational stiffness values used in the analyses depend on the fixed connection size parameters. The parameters $t_p = 1.746$ cm and $d_b = 2.54$ cm are considered in the present study as in the study of Hayalioglu and Degertekin (2004a). The rotational stiffness value 2.26×10^8 kNmm/rad can be also used for Top and Seat Angles connection type when the fixed connection size parameters are assumed as $t = 2.54$ cm, $d_b = 2.858$ cm (Hayalioglu and Degertekin 2005). Moreover, the same semi-rigid column-to-base details used by Hayalioglu and Degertekin (2004a) are applied in this study, Fig. 3.

4. Formulation of optimum design

Minimum weight of planar frames is considered as objective function in discrete optimum design problem. The objective, penalized objective and fitness functions are shown as below (Daloglu and Armutcu 1998)

$$\min W = \sum_{k=1}^{ng} A_k \sum_{i=1}^{nk} \rho_i L_i \tag{4}$$

$$g_i(x) > 0 \rightarrow c_i = g_i(x) \tag{5}$$

$$g_i(x) \le 0 \rightarrow c_i = 0 \tag{6}$$

$$\varphi(x) = W(x)\left(1 + P\sum_{i=1}^{m} c_i\right) \tag{7}$$

$$F_i = \left(\varphi(x)_{\max} + \varphi(x)_{\min}\right) - \varphi(x)_i \tag{8}$$

where W is the weight of the frame, A_k is cross-sectional area of group k, ρ_i and L_i are density and length of member i, ng is total numbers of groups, nk is the total numbers of members in group k. g_i is the constraints, c_i is constraint violations, P is a penalty constant, $\varphi(x)$ is penalized objective function, F_i is fitness function.

In this study, maximum lateral displacement constraints and stress constraints of AISC Allowable Stress Design (ASD) and geometric constraints for column-to-column and beam-to-column are considered as follows, Hayalioglu and Degertekin (2004a),

The stress constraints taken from AISC–ASD (1989) are shown in Eqs. (9) and (10).

$$g_i(x) = \left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 + \dfrac{f_a}{F'_{ex}} F_{bx}\right)}\right] - 1.0 \le 0 \qquad i = 1,\ldots,nc \tag{9}$$

$$g_i(x) = \left[\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}}\right] - 1.0 \le 0 \qquad i = 1,\ldots,nc \tag{10}$$

When $\dfrac{f_a}{F_a} \le 0.15$, Eq.(11) is calculated instead of Eqs.(9) and (10).

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

where nc is total number of members subjected to both axial compression and bending stresses, f_a is computed axial stress, F_a is allowable axial stress under axial compression force alone, and is calculated depending on elastic or inelastic bucking of the member according to slenderness ratio depending on effective length factor (K), f_{bx} is computed bending stresses due to bending of the member about its major (x), F_{bx} is allowable compressive bending stresses about major, C_{mx} is a factor which taken as 0.85 for unbraced frame members, F'_{ex} is Euler stresses, F_y is yield stress of steel. The effective length factor K for unbraced frames is determined as follows (Dumonteil 1992)

$$K = \sqrt{\frac{1.6G_A G_B + 4.0(G_A + G_B) + 7.50}{G_A + G_B + 7.50}} \tag{12}$$

where G_A and G_B are the relative stiffness factors at *A*th and *B*th ends of columns.

$$G = \left(\frac{\sum I_c / L_c}{\sum \alpha_{uf} (I_g / L_g)} \right); \tag{13}$$

$$\alpha_{uf} = \frac{1}{\left(1 + \dfrac{6EI}{Lk} \right)} \tag{14}$$

where I_c is moment of inertia of column section corresponding to plane of buckling, L_c is unbraced length of column, I_g is moment of inertia of beam corresponding to plane of bending, L_g is unbraced length of beam, *S* is rotational spring stiffness of corresponding end, α_{uf} is a coefficient which shows the connection condition and it is equal to 1 for rigid connections. It is calculated by Eq. (14) (Dhillon and O'Malley 1999, Hayalioglu and Degertekin 2004a), when the beams are not rigidly connected to columns. *k* in the related equation is corresponding spring stiffness, and expressed as M/θ_r. However, in this study, adopted rotational stiffness of connection, *S*, is used instead of *k* in Eq. (14).

The displacement constraints are shown in Eq. (15)

$$g_{jl}(x) = \frac{\delta_{jl}}{\delta_{ju}} - 1 \le 0 \qquad \begin{array}{l} j = 1,...,m \\ l = 1,...,nl \end{array} \tag{15}$$

where δ_{jl} is displacement of *j*th degree of freedom under load case *l*, δ_{ju} is upper bound, *m* is number of restricted displacements, *nl* is total number of loading cases.

Column-to-column geometric constraints (size constraints) are expressed in Eq. (16)

$$g_n(x) = \frac{D_{un}}{D_{ln}} - 1 \le 0 \qquad n = 2,...,ns \tag{16}$$

where D_{un} is depth of upper floor column, D_{ln} is depth of lower floor column.

Beam-to-column geometric constraints are shown in Eq. (17)

$$g_{bb,i}(x) = \frac{b_{kbk,i}}{b_{fck,i}} - 1 \le 0 \qquad i = 1,...,n_{bf} \tag{17}$$

where n_{bf} is number of joints where beams are connected to the flange of column, $b_{fbk,i}$ and $b_{fck,i}$ are flange widths of beam and column, respectively.

5. Composite beams

Concrete slabs on steel beams are taken into account in the analysis. Effective width of concrete slab as shown in Fig. 4 is determined as follows (Salmon and Johnson 1980). As seen in Fig. 4, while slab extending is only on one side for exterior beam, it is on both sides for interior

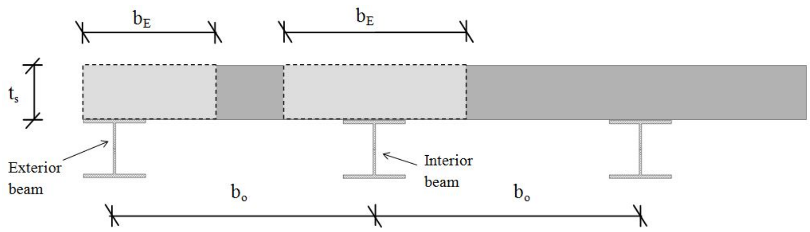


Fig. 4 Effective width of composite beam

beam (Salmon and Johnson 1980). The contribution of the concrete on interior beams is greater than the contribution on exterior beams. In space frames, the contribution of concrete should be considered for exterior or interior beams separately. However, in planar frame models, the contributions of concrete on the all beams can be assumed as these on exterior beams or interior beams. In this study, the contributions can be considered as these on exterior beams.

$$\text{for an interior beam,} \quad \begin{aligned} & b_E \le \frac{L}{4} \\ & b_E \le b_0 \\ & b_E \le b_f + 16t_s \end{aligned} \tag{18}$$

$$\text{for an exterior beam,} \quad \begin{aligned} & b_E \le \frac{L}{12} + b_f \\ & b_E \le \frac{1}{2}\left(b_0 + b_f\right) \\ & b_E \le b_f + 6t_s \end{aligned} \tag{19}$$

where b_E is effective width of concrete slab, L is span length of steel beam, b_f is flange width of steel beam, b_o is interval between two beams, t_s is thickness of concrete slab. The effective width of concrete slab is transformed by Eq. (20)

$$b_{E(transformed)} = b_E \frac{E_c}{E_s} \tag{20}$$

where E_c is elastic modulus of concrete and E_s is elastic modulus of steel. Composite beam section properties such as center of gravity of the cross section, moment of inertia about major and minor axes…etc., are determined for the analyses of whole structure.

6. Design examples

Three different planar frames with semi-rigid beam to column steel connections and column bases are carried out for two cases with and without considering concrete slab effects in FEM analyses. First two examples were previously studied by Hayalioglu and Degertekin (2004a). In the present study, concrete slab effect on behavior of beams is also considered in optimum design of semi-rigid frames, and composite beams are placed as seen in Fig. 4. Thickness of concrete slab is taken to be 10 cm and the modulus of elasticity, E, is 30 GPa. Optimum cross sections for both cases are selected from a specified list including 128 W sections taken from American Institute of Steel Construction (AISC). The maximum lateral displacements (top storey sway) is limited to H/250, where H is total height of frame (Hayalioglu and Degertekin 2004a). Adopted rotational stiffness for all column bases in first two examples is 1.13×10^8 kNmm/rad and it is 2.26×10^8 kNmm/rad for third example (Hayalioglu and Degertekin 2005). Also, material properties of steel in all three examples are E_s = 200 GPa, yield stress f_y = 248.2 MPa, material density ρ = 7.85 ton/m^3.

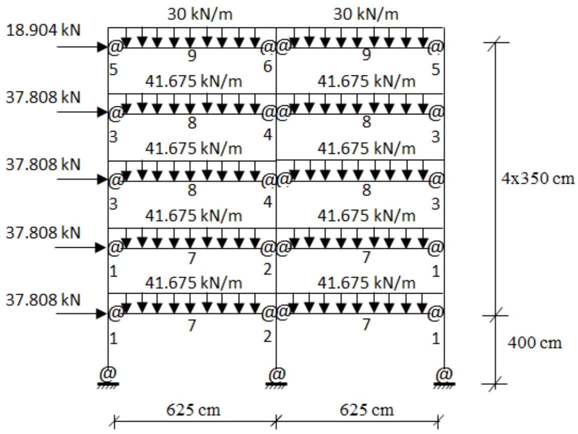


Fig. 5 Five-storey, two-bay frame

Table 1 Optimum cross sections of fully rigid frame

Group no	Hayalioglu and Degertekin (2004a) (without composite beams)		Present study	
			Fully rigid	
	Fully rigid		Without composite beams	With composite beams
1	14×68	18×55	21×68	21×68
2	16×67	18×65	16×100	24×76
3	8×31	14×61	16×45	14×48
4	14×61	14×43	1×x50	14×48
5	8×31	14×43	12×26	14×48
6	10×33	8×31	8×24	8×24
7	18×55	18×50	24×55	18×60
8	16×50	14×43	24×55	18×40
9	21×50	21×50	18×35	16×26
Total weight (kg)	8668	8888	9255	8704
Top storey sway (cm)	3.53	3.68	2.17	1.83

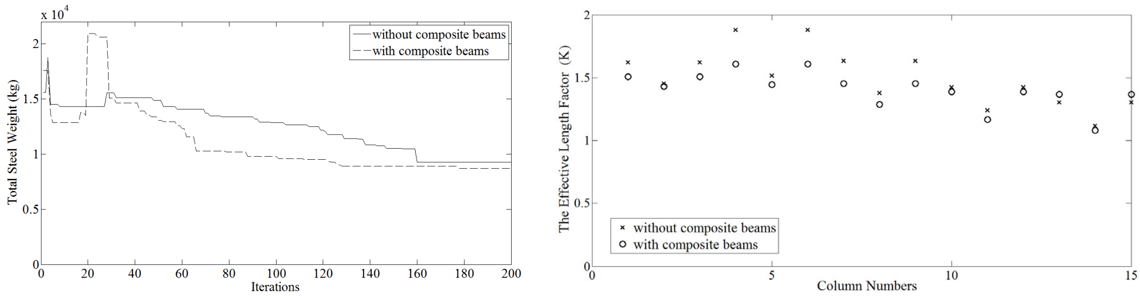
Table 2 Optimum cross sections of semi rigid frame

Group no	Hayalioglu and Degertekin (2004a) (without composite beams)		Present study			
	Semi rigid		Semi rigid			
	End plate without column stiffeners	End plate with column stiffeners	End plate without column stiffeners		End plate with column stiffeners	
			Without composite beams	With composite beams	Without composite beams	With composite beams
1	21×62	21×62	21×68	21×68	16×67	16×67
2	21×62	21×68	24×94	24×76	21×101	21×83
3	16×67	21×62	12×45	16×50	14×48	14×38
4	21×62	14×61	16×57	14×48	16×50	21×68
5	12×53	14×61	8×28	10×22	8×24	12×26
6	14×61	10×54	8×31	10×22	14×30	21×68
7	18×60	21×62	21×62	18×71	24×68	18×60
8	12×50	21×62	16×57	18×46	21×50	21×50
9	8×40	14×53	18×35	16×31	18×35	16×26
Total weight (kg)	9831	10432	9646	9178	9616	9124
Top storey sway (cm)	4.48	3.30	3.28	2.56	2.75	2.21

6.1 Example 1: *Five-storey, two-bay frame*

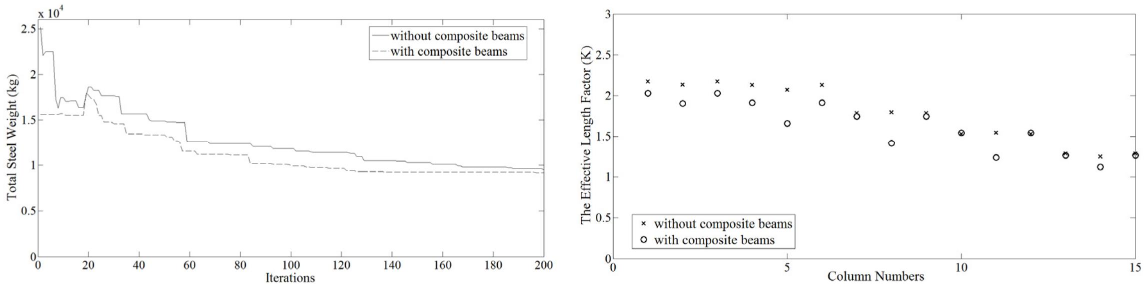
A five-storey, two-bay frame is grouped and loaded as seen in Fig. 5. The semi-rigid frame with regular beams was non-linearly analyzed by Hayalioglu and Degertekin (2004a) using genetic algorithm with and without P-Δ effects. In the present work, stress constraints of AISC-ASD, maximum lateral displacement constraints, column-to-column and beam-to-column size constraints used by Hayalioglu and Degertekin (2004a) are imposed on the semi rigid frame. Maximum top storey drift is restricted to 7.2 cm. Optimum design of full and semi rigid frames are performed for cases of the frame with and without composite beams. Minimum weights, maximum top story drifts, steel sections of optimum designs for full and semi rigid steel frames are presented in Tables 1 and 2, respectively. The results obtained by Hayalioglu and Degertekin (2004a) are also shown in the tables for comparison. Figs. 6(a), 7(a) and 8(a) show the variation of total steel weight with iterations for both cases and Figs. 6(b), 7(b) and 8(b) show the values of effective length factor (K) of the columns of full and semi rigid steel frame for both cases.

As shown in Tables 1 and 2, optimum design results of this study are very close to the results obtained by Hayalioglu and Degertekin (2004a) for frames without composite beams. Top story sway or maximum displacements are far less than upper limit. Therefore, it can be said that stress and size constraints play active role in optimum designs of full and semi rigid frames. As shown in the tables and the figures of fully rigid frame, minimum weight obtained for the fully rigid steel frame without composite beams is 8704 kg which is about 5% lighter compare to frames with



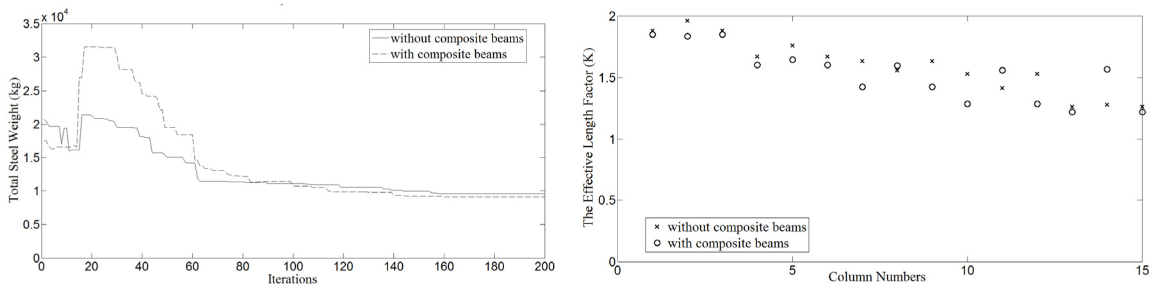
(a) Variation of total steel weight (b) Effective length factor (K) of columns

Fig. 6 Fully rigid steel frame with and without composite beams



(a) Variation of total steel weight (b) Effective length factor (K) of columns

Fig. 7 Semi rigid steel frame with end plate without column Stiffeners connection with and without composite beams



(a) Variation of total steel weight (b) Effective length factor (K) of columns

Fig. 8 Semi rigid steel frame with end plate with column stiffeners connection with and without composite beams

regular beams or without composite beams. This is also valid in optimum designs of semi-rigid frames (End Plate without Column Stiffeners and End Plate with Column Stiffeners connections) shown in Table 2. The weight, 8704 kg, is also about 9-10% lighter than the minimum weights 9646 kg and 9616 kg obtained from optimum designs of both semi rigid frames. Besides it can clearly be observed from Figs. 6(b), 7(b) and 8(b) that the effective length factor K for columns depends on fixity factors based on rotational spring stiffness of semi rigid connections. So a reduction in rotational spring stiffness and hence in the fixity factors results with an increase in K. This situation leads to an increase in the buckling lengths of columns and the selection of larger cross-section profiles for columns. Also, these figures and tables show that consideration of concrete slab effects on the behavior of beams significantly reduces the effective length factor of columns and maximum top story sways.

6.2 Example 2: Ten-storey, single-bay frame

A ten-storey, single-bay frame is grouped and loaded as seen in Fig. 9. This semi-rigid frame without composite beams was non-linearly researched by Hayalioglu and Degertekin (2004a) for different section lists with and without P-Δ effects. The constraints applied in this example are the same as those of the previous example. The maximum top storey drift is restricted to 12.0 cm. Fully rigid frames and semi rigid frames with End Plate without Column Stiffeners and End Plate with Column Stiffeners connections are performed for cases of regular and composite beams. Minimum weights of optimum designs for full and semi rigid steel frames are presented in Table 3 for comparison with the results of Hayalioglu and Degertekin (2004a). Maximum top story drifts and steel sections of optimum designs for full and semi rigid steel frames are presented in Table 4. Figs. 10(a), 11(a) and 12(a) show the variation of total weight with iterations for both cases and Figs. 10(b), 11(b) and 12(b) show the values of effective length factor (K) of columns of full and semi rigid steel frames for both cases.

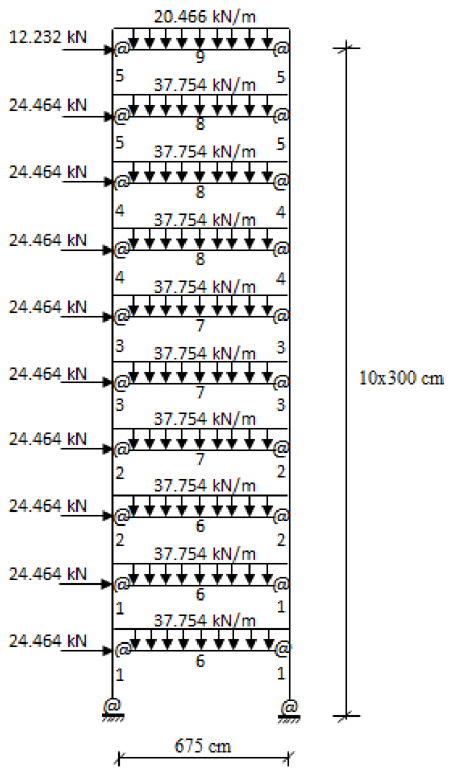


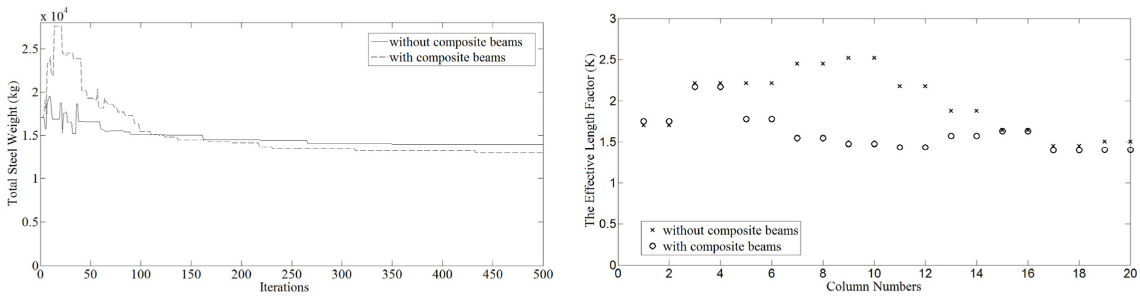
Fig. 9 Ten-storey, single-bay frame

Table 3 Minimum weights (kg) of optimum designs

The frames according to connection types		This study: Without composite beams	This study: With composite beams	Hayalioglu and Degertekin 2004a (Without composite beams)	
Full rigid steel frames		13951	12988	12119	14067
Semi rigid steel frames	End plate without column stiffeners connection	15016	14091	12691	15422
	End plate with column stiffeners connection	14245	13584	15053	15756

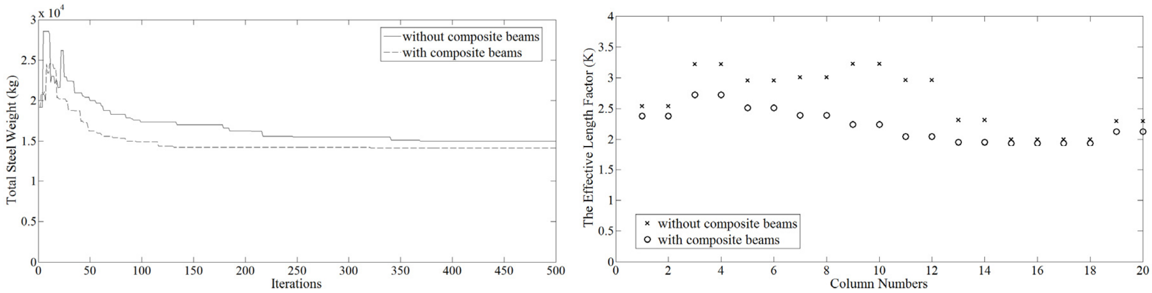
Table 4 Optimum designs for full and semi rigid steel frames

Group no	Full rigid: Without composite beams	Full rigid: With composite beams	Semi rigid, End plate without column stiffeners connection: Without composite beams	Semi rigid, End plate without column stiffeners connection: With composite beams	Semi rigid, End plate with column stiffeners connection: Without composite beams	Semi rigid, End plate with column stiffeners connection: With composite beams
1	24×117	30×108	24×117	24×117	21×101	21×101
2	24×117	16×100	21×101	21×101	21×101	16×100
3	24×68	16×77	21×101	21×68	18×76	16×89
4	16×57	16×67	16×67	16×67	18×60	16×67
5	12×40	14×30	16×67	16×67	16×36	14×30
6	27×94	21×83	24×76	24×84	30×108	21×93
7	21×68	24×68	18×76	21×68	18×76	24×76
8	24×55	18×46	24×68	18×60	21×57	21×44
9	16×26	10×22	16×26	12×19	14×30	10×22
Top storey way (cm)	5.57	4.76	8.46	6.80	7.55	6.48



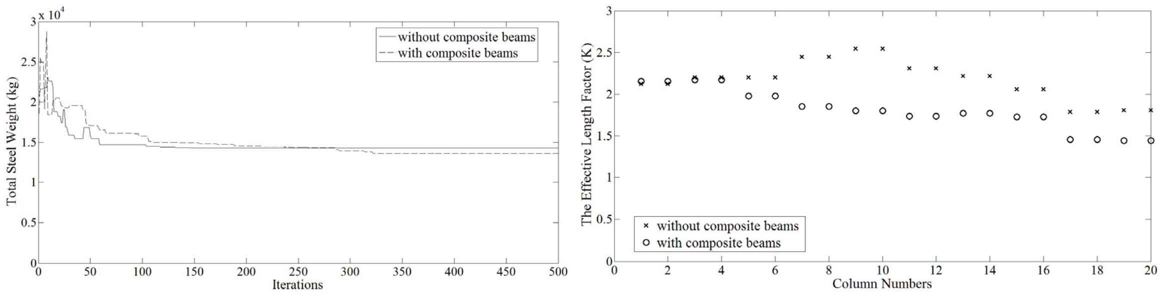
(a) Variation of total weight (b) Effective length factor (K) of columns

Fig. 10 Fully rigid steel frame with and without composite beams



(a) Variation of total weight (b) Effective length factor (K) of columns

Fig. 11 Semi rigid steel frame with End Plate without Column Stiffeners connection with and without composite beams



(a) (b)

Fig. 12 Semi rigid steel frame with end plate with column stiffeners connection with and without composite beams

As it is observed from Table 3 that minimum weights obtained from this study are very close to ones obtained by Hayalioglu and Degertekin (2004a) for the frames with regular beams. Maximum displacement of all three optimum designs is 8.46 cm which is far below the limit. It indicates that stress and size constraints are important determinants of optimal designs for full and semi rigid frames. Also, if the semi-rigid frames is connected to full rigid column bases instead of semi rigid column bases used in the present study, the value of maximum top storey displacement of the semi-rigid frame decreases from 8.46 cm to 7.63 cm. It is observed from Table 3 that minimum weights of all three optimum designs for the case of frame with composite beams are about 5-6% lighter than the case without composite beams. Also, minimum weights obtained for fully rigid steel frames without composite beams is 7.6% and 2.1% lighter than those of semi-rigid frames with end plate without column Stiffeners connections and End Plate with Column Stiffeners connections, respectively. It is apparently seen from Figs. 10(b), 11(b) and 12(b) that a decrease in rotational spring stiffness and so fixity factor of semi rigid connection increases the effective length factor (K) and so the buckling lengths of columns. This situation leads to the selection of larger cross-section profiles for columns and so the heavier designs. It is also observed from Table 4 that selected sections of beams in the optimum designs of frame with composite beams are commonly smaller than those of frames without composite beams.

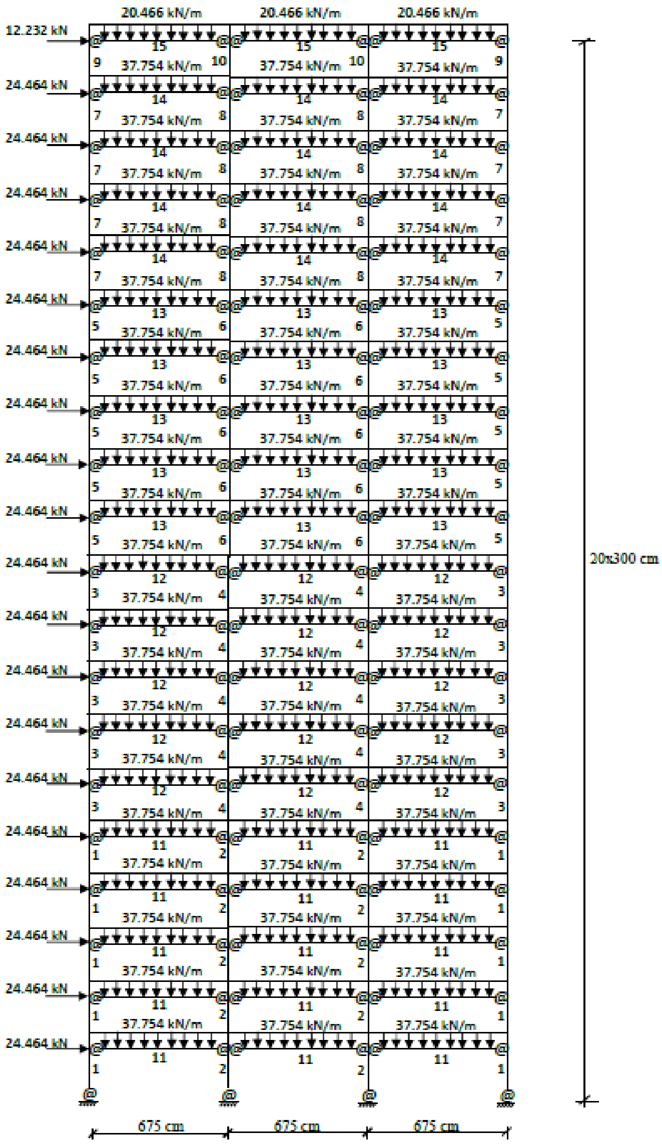


Fig. 13 Twenty-storey, three-bay frame

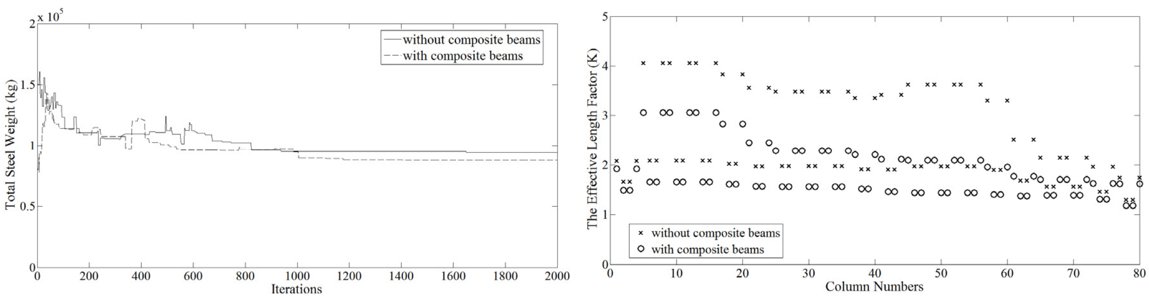
6.3 Example 3: Twenty-storey, three-bay frame

A twenty-storey, three-bay frame is grouped and loaded as seen in Fig. 13. The maximum top storey drift is restricted to 24.0 cm (H/250). The full and semi rigid frames are performed for the cases of frame with and without composite beams. Minimum weights, maximum top story drifts, steel sections of optimum designs are presented in Table 5. The constraints applied in this example are the same as those of the previous examples. Figs. 14(a), 15(a) and 16(a) show the variation of the total weight with iterations for both cases and Figs. 14(b), 15(b) and 16(b) show the values of the effective length factor (K) of the columns of fully and semi rigid steel frame for both cases.

As shown in Table 5, maximum top storey displacements are significantly less than upper limit. So, stress and size constraints are important determinants of optimal designs for full and semi rigid

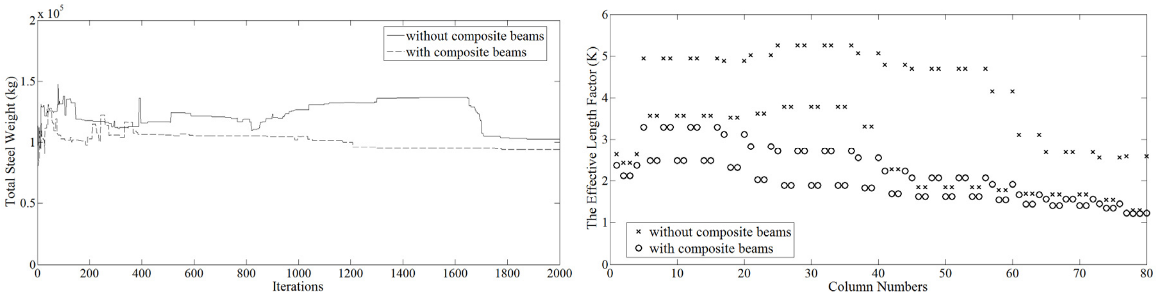
Table 5 Optimum designs for full and semi rigid steel frames

Group no	Full rigid		Semi rigid			
			End plate without column stiffeners connection		End plate with column stiffeners connection	
	Without composite beams	With composite beams	Without composite beams	With composite beams	Without composite beams	With composite beams
1	27×194	30×173	30×211	24×162	27×161	36×194
2	14×257	14×257	30×211	14×370	30×211	24×207
3	24×146	24×117	30×191	21×122	27×161	30×132
4	14×193	14×193	30×191	14×193	30×211	18×175
5	21×132	21×101	30×148	18×86	27×94	24×117
6	14×132	14×132	14×132	14×132	14×176	18×119
7	18×71	16×67	21×73	14×48	16×67	14×53
8	12×106	12×106	12×106	12×79	14×132	18×65
9	14×30	16×36	16×67	8×24	16×36	8×21
10	10×33	12×19	8×31	12×30	12×30	8×21
11	24×68	21×83	27×94	24×94	30×108	30×108
12	21×68	21×68	24×76	24×68	21×68	24×76
13	16×67	21×57	21×83	24×68	21×68	24×76
14	18×71	18×50	21×62	21×44	14×53	21×44
15	14×30	12×19	12×35	16×26	12×30	14×26
Total weight (kg)	94120	88070	10251	93590	97210	92090
Top storey sway (cm)	9.61	6.52	9.79	9.13	9.86	7.10



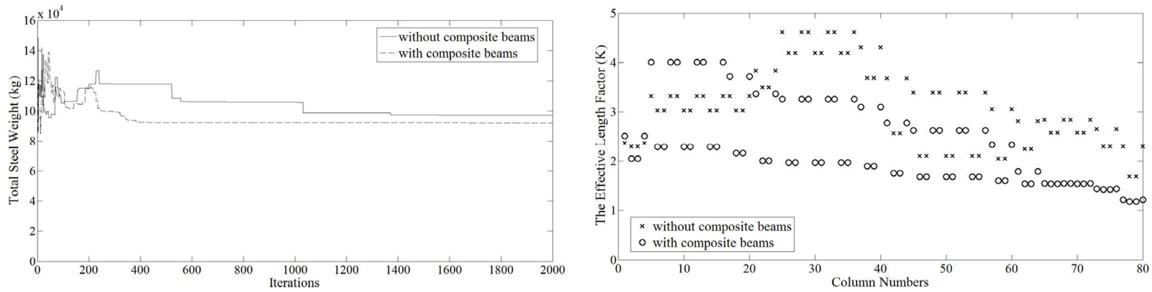
(a) Variation of total weight (b) Effective length factor (K) of columns

Fig. 14 Fully rigid steel frame with and without composite beams



(a) Variation of total weight (b) Effective length factor (K) of columns

Fig. 15 Semi rigid steel frame with end plate without column stiffeners connection with and without composite beams



(a) Variation of total weight (b) Effective length factor (K) of columns

Fig. 16 Semi rigid steel frame with end plate with column stiffeners connection with and without composite beams

frames as in the previous examples. It can apparently observed from the comparison of Figs. 14(b), 15(b) and 16(b) that the effective length factor K for columns depends on the fixity factors based on rotational spring stiffness of connection types, and an increase in the rotational spring stiffness provides fixity factors to get closer to 1 (fully rigid). Moreover, this situation results with a reduction in K. So, minimum weight (94120 kg) for fully rigid steel frame without composite beams is about 8% and 3% lighter than the optimum designs of semi rigid (End Plate without Column Stiffeners connections and End Plate with Column Stiffeners connections) for the case of the frame without composite beams.

As regards the figures above (Figs. 14(b), 15(b) and 16(b)), in the optimal design of frame with composite beams, considering concrete slab effects in finite element analyses significantly reduces the effective length factor of columns and so the buckling lengths decrease. Furthermore, selected sections of the beams are usually smaller and minimum weights for all full and semi rigid frames are reduced by about 5-8%. Furthermore, considering concrete slab effects in finite element analyses substantially reduces the values of maximum.

7. Conclusions

Main purpose of the present work is to consider concrete slab effects on behavior of steel floor beams and optimum design of semi rigid multistorey frames with composite beams. The stress constraints of AISC-ASD, maximum lateral displacement constraints and geometric constraints are imposed on full and semi rigid frames. Genetic Algorithm incorporating reproduction, crossover and mutation operators are selected as the method for minimum weight design of steel structural systems involving discrete design variables. All procedures are repeated for the optimum designs of full and semi rigid frames. Two of the examples taken from literature are resized for the cases of the frame with and without composite beams. Another multistorey frame is studied as third examples. Results obtained from analyses are presented in tabular and graphical formats. Most important conclusions drawn from the study are briefly summarized below:

- While first example is carried out with 200 iterations, third example is solved with 2000 iterations.
- A decrease in the rotational spring stiffness or fixity factor of frames increases the values of effective length factor K, and so the buckling lengths of columns. In the first example, while maximum K value for fully rigid frame without composite beams is about 1.9, this value for semi rigid frame reaches to 2.2. This situation is also valid for the other examples. In the third example, this value increases from about 4 to 5. Therefore the optimum design weights increase. In the second example, while design weight of for fully rigid frame without composite beams is 13951 kg, this weight is 15016 kg for the semi rigid frame.
- In the optimum designs of frames with composite beams, consideration of concrete slab effects in finite element analyses significantly reduces the effective length factor of columns and maximum top storey displacements. In the first example, while the maximum K values for fully rigid frame decrease from about 1.9 to 1.7 for the fully rigid frame and from about 2.2 to 2.0 for the semi rigid frame. This situation is also valid for the second and third examples. Therefore optimum weight of the steel frames decreased by about 5-8% when the effect of concrete slab on behavior of beams is considered in all three frame examples studied. Furthermore, selected sections of the beams are usually smaller.

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