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# Optimum design of steel frames with semi-rigid connections and composite beams

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**Abstract.** In this paper, an optimization process using Genetic Algorithm (GA) that mimics biological processes is presented for optimum design of planar frames with semi-rigid connections by selecting suitable standard sections from a specified list taken from American Institute of Steel Construction (AISC). The stress constraints as indicated in AISC-LRFD (American Institute of Steel Construction - Load and Resistance Factor Design), maximum lateral displacement constraints and geometric constraints are considered for optimum design. Two different planar frames with semi-rigid connections taken from the literature are carried out first without considering concrete slab effects in finite element analyses and the results are compared with the ones available in literature. The same optimization procedures are then repeated for full and semi rigid planar frames with composite (steel and concrete) beams. A program is developed in MATLAB for all optimization procedures. Results obtained from this study proved that consideration of the contribution of the concrete on the behavior of the floor beams provides lighter planar frames.

**Keywords:** genetic algorithm, weight optimization, planar frame, composite beams, semi-rigid connection

## 1. Introduction

Optimum design of steel structures is quite important to obtain more economical designs. Minimum weight design of steel structural systems involving discrete design variables are quickly carried out by numerous methods such as Genetic Algorithm (GA), Harmony Search Algorithm (HAS), Ant Colony Algorithm (ACA), Particle Swarm Optimizer (PSO) and Artificial Bee Colony Algorithm (ABC). Optimal design of frames with fully rigid or semi-rigid connections via these algorithm methods based on mathematical programming has been widely studied by many researchers in recent years.

Rajeev and Krishnamoorthy (1992) researched discrete optimization of structures using genetic algorithms. They studied various planar and space truss systems. Simoes (1996) focused on optimization of frames with semi-rigid connections. Daloglu and Armutcu (1998) studied optimum design of plane steel frames using genetic algorithm. Erbatur *et al.* (2000) examined optimal

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design of planar and space structures with genetic algorithms. Kameshki and Saka (2001) researched optimum design of nonlinear steel frames with semi-rigid connections using a genetic algorithm. Hayalioglu and Degertekin (2004) focused on genetic algorithm based optimum design of non-linear steel frames with semi-rigid connections. Filho *et al.* (2004) studied wind pressures in framed structures with semi-rigid connections. Choi and Kim (2006) studied optimal design of semi-rigid steel frames using practical nonlinear inelastic analysis. Wang and Li (2007) studied stability analysis of semi-rigid composite frames. Kaveh and Talatahari (2007) used a discrete particle swarm ant colony optimization for design of steel frame structures. Esen and Ulker (2008) researched optimization of multi storey space steel frames. Degertekin *et al*. (2009) researched optimum design of geometrically non-linear steel frames with semi-rigid connections using a harmony search algorithm. Degertekin and Hayalioglu (2010) focused on harmony search algorithm for minimum cost design of steel frames with semi-rigid connections and column bases. Degertekin *et al.* (2011) researched optimum design of geometrically nonlinear steel frames with semi-rigid connections using improved harmony search method. Gorgun and Yılmaz (2012) researched geometrically nonlinear analysis of plane frames with semi-rigid connections accounting for shear deformations. Togan *et al*. (2011) researched optimization of trusses under uncertainties with harmony search. Aydogdu and Saka (2012) studied ant colony optimization of irregular steel frames according to LRFD-AISC. Kaveh and Talatahari (2012) studied a hybrid CSS and PSO algorithm for optimal design of structures. Rafiee *et al.* (2013) focused on optimum design of steel frames with semi-rigid connections using Big Bang-Big Crunch method. Hadidi and Rafiee (2014) researched harmony search based, improved Particle Swarm Optimizer for minimum cost design of semi-rigid steel frames.

In the literature, there are numerous studies on the weight optimization of steel frames with fully rigid or semi-rigid connections. However, it is hard to see enough studies about optimization of semi-rigid steel frames considering concrete slab effects on the behavior of beams. So, in this study, optimum design of semi-rigid planar frames is studied with and without taking the effect of concrete slab into the consideration on FE analyses. Results obtained from the optimization of the frames with composite beams showed that the consideration of the concrete slabs contribution on the behavior of beams ended up with less steel weight.

## 2. Semi-rigid connections and FEM

Moment capacity changes between full rigid and pin connections (Dogan 2010). As shown in

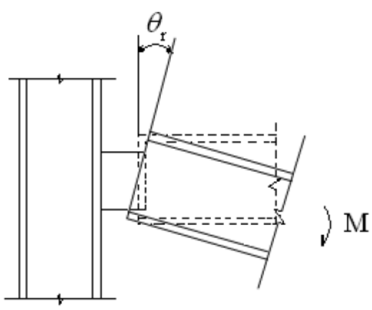



Fig. 1 Rotation of a semi-rigid connection

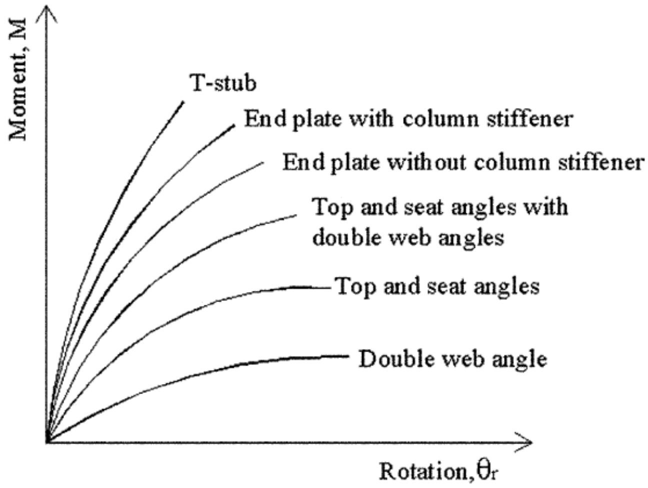



Fig. 2 Moment-rotation curves of semi-rigid connections

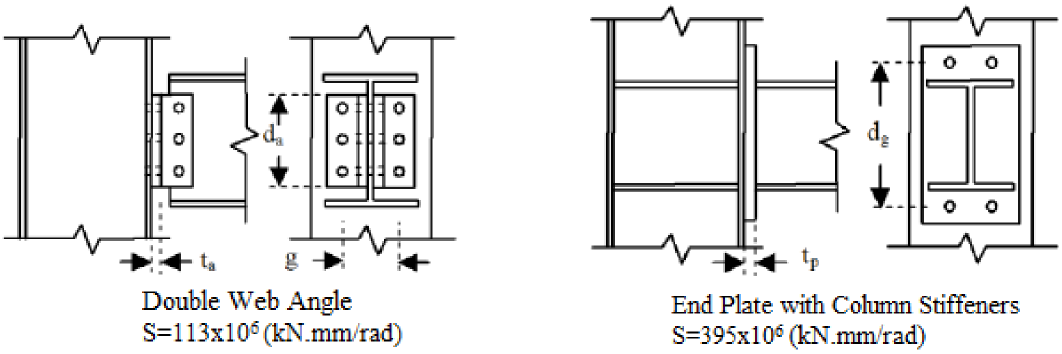



Fig. 3 The two types of connections and rotational stiffness values

Fig. 1, in the semi-rigid connections, bending moment (M) at the beam-to-column connection leads to some rotation of the joint depending on types of semi-rigid connections. And, the types of semi rigid connections play a crucial role in the amount of moment capacity as seen in Fig. 2 (Hayalioglu *et al*. 2004).

In this study, the examples of the planar frames with two types of these semi-rigid connections, previously studied by Hadidi and Rafiee (2014), are carried out and the results are compared with the ones available in literature. The cases are studied with and without considering concrete slab effects in finite element analyses. Connection details and rotational stiffness values of these two types are shown in Fig. 3.

Stresses and displacements of each element in the semi-rigid frame are determined by Finite Element Method (FEM). Local stiffness matrix of each member, ($k_l$), is calculated first. According to first-order analysis, the local stiffness matrices of beams with semi-rigid end connections are defined by Eq. (1) (Simoes 1996, Filho *et al*. 2004). Then, global stiffness matrix for whole structure, ($K$), is obtained from Eq. (3) by using coordinate transformation matrix, ($T$). Thus, the stresses and displacements of each element are defined.

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

where $E$ is the elastic modulus, $L$ is the length of member, $A$ is the cross-section area of member, $I$ is the inertia moment of member, $\alpha_1$ and $\alpha_2$ are fixity factors as defined in Eq. (2).

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

where $S_1$ and $S_2$ are rotational spring stiffness values of the ends of the semi-rigid connected beams.

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

## 3. Formulation of optimum design

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

$$
\min W = \sum_{k=1}^{ng} A_k \sum_{i=1}^{nk} \rho_i L_i \tag{3}
$$

$$
g_i(x) > 0 \rightarrow c_i = g_i(x) \tag{4}
$$

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

$$
\varphi(x) = W(x)\left(1+P\sum_{i=1}^{m} c_i\right) \tag{6}
$$

$$
F_i = (\varphi(x)_{max} + \varphi(x)_{min}) - \varphi(x)_i \tag{7}
$$

where $W$ is the weight of the frame, $A_k$ is cross-sectional area of group $k$, $\rho_i$ 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.

The objective function is subjected to the stress constraints of AISC-LRFD (1995), maximum lateral displacement constraints geometric constraints for column-to-column and beam-to-column as follows,

The stress constraints taken from AISC–LRFD (1995) are presented in Eqs. (8) and (9).

$$\text{for } \frac{P_u}{\phi P_n} \geq 0.2 \qquad g_{il}(x) = \left(\frac{P_u}{\phi P_n}\right)_{il} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right)_{il} - 1.0 \leq 0 \qquad \begin{array}{l} i = 1,\ldots,nm \\ l = 1,\ldots,nl \end{array} \tag{8}$$

$$\text{for } \frac{P_u}{\phi P_n} < 0.2 \qquad g_{il}(x) = \left(\frac{P_u}{2\phi P_n}\right)_{il} + \left(\frac{M_{ux}}{\phi_b M_{nx}}\right)_{il} - 1.0 \leq 0 \qquad \begin{array}{l} i = 1,\ldots,nm \\ l = 1,\ldots,nl \end{array} \tag{9}$$

where $nm$ is the total number of members, $nl$ is the total number of loading conditions, $P_u$ is the required axial strength, $P_n$ is the nominal strength, $M_{ux}$ is the required flexural strength about major axis, $M_{nx}$ is the nominal flexural strength about major axis, $\phi$ is resistance factor for compression (0.85) and for tension (0.90), $\phi_b$ is resistance factor for flexure (0.90).

The nominal compressive strength is calculated as below

$$P_n = A_g F_{cr} \tag{10}$$

$$\text{for } \lambda_c \leq 1.5 \qquad F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y \tag{11}$$

$$\text{for } \lambda_c > 1.5 \qquad F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right) F_y \tag{12}$$

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} \tag{13}$$

where $A_g$ is the cross-sectional area; $K$ is the effective length factor; $E$ is the elastic modulus; $r$ is the governing radius of gyration; $L$ is the member length; $F_y$ is the yield stress of steel. The effective length factor $K$ for unbraced frames is determined as follows (Dumonteil 1992)

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

where $G_A$ and $G_B$ are the relative stiffness factors at $A^{th}$ and $B^{th}$ ends of columns.

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

$$\alpha_{uf} = \frac{1}{\left(1 + \dfrac{6EI}{Lk_\theta}\right)} \tag{16}$$

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 inertia of beam corresponding to plane of bending, $L_g$ is unbraced length of beam, $S$ is rotational spring stiffness of corresponding end, $\alpha_{uf}$ is a coefficient which shows the connection condition and it is equal to 1 for rigid connections. It is calculated by Eq. (16) (Dhillon and O'Malley1999, Degertekin *et al*. 2011) if the beams are not rigidly connected to columns. $k_\theta$ in the related equation is corresponding spring stiffness, and expressed as $M/\theta_r$. However, in this

study, estimated rotational stiffness of connection, $S$, is used instead of $k_\theta$ in Eq. (16).
Other constraints are as below: (Hadidi and Rafiee 2014)

• Displacement constraints are shown in Eq. (17)

$$g_{jl}(x) = \frac{\delta_{jl}}{\delta_{ju}} - 1 \le 0 \qquad \begin{array}{l} j = 1,\ldots,m \\ l = 1,\ldots,nl \end{array} \tag{17}$$

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

• Column-to-column geometric constraints (size constraints) are expressed in Eq. (18)

$$g_n(x) = \frac{D_{un}}{D_{ln}} - 1 \le 0 \qquad n=2,\ldots,ns \tag{18}$$

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

• Beam-to-column geometric constraints are shown in Eq. (19)

$$g_{bb,i}(x) = \frac{b_{fbk,i}}{b_{fck,i}} - 1 \le 0 \qquad i= 1,\ldots,n_{bf} \tag{19}$$

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

## 3. Genetic algorithm

Genetic Algorithm is used conducting natural biological procedures such as reproduction, crossover and mutation as proposed by Goldberg (1989). In this study, double point crossover is applied. Optimum design steps for the frames are listed below:

1. Start with random initial population comprised of individuals which are coded as binary digits.
2. Decode each individual and select corresponding profiles from available section lists.
3. Analyze with finite element method (FEM) according to selected profiles.
4. Determine objective, penalized objective and fitness functions.
5. Apply reproduction, double-point crossover and mutation operators.
6. Replace the initial population with the new population.
7. Repeat all steps until the convergence is obtained.

Detailed information about GA steps can be found in the literature Daloglu and Armutcu (1998), Kameshki and Saka (2001), Hayalioglu and Degertekin (2004), Degetekin *et al*. (2011).

## 4. 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),
for an interior beam

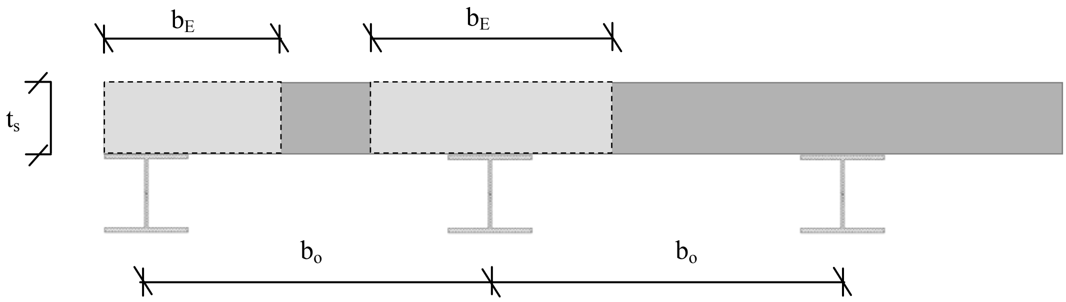



Fig. 4 Effective width of composite beam

$$\begin{aligned} b_E &\le \frac{L}{4} \\ b_E &\le b_0 \\ b_E &\le b_f + 16t_s \end{aligned} \tag{20}$$

for an exterior beam

$$\begin{aligned} b_E &\le \frac{L}{12} + b_f \\ b_E &\le \frac{1}{2}\left(b_0 + b_f\right) \\ b_E &\le b_f + 6t_s \end{aligned} \tag{21}$$

where $b_E$ is effective width of concrete slab, $L$ is the span length of steel beam, $b_f$ is the flange width of steel beam, $b_o$ is the interval between two beams, $t_s$ is the thickness of concrete slab. The effective width of concrete slab is transformed by Eq. (24)

$$b_{E(transformed)} = b_E \frac{E_c}{E_s} \tag{22}$$

where $E_c$ is the elastic modulus of concrete, $E_s$ is the 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.

## 5. Design examples

Two different semi-rigid planar frames from literature are designed for comparison purposes. Minimum weight optimizations of the frames with and without considering concrete slab effects in FEM analyses are carried out. Concrete slab is placed as seen in Fig. 4 in numerical examples of optimum design of semi-rigid frames with composite beams. 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 W-section list which consists of 64 sections (W8×15, W 8×21, W8×24, W8×28, W8×31, W8×35, W8×40, W10×15, W10×22, W10×26, W10×33, W10×39, W10×54, W10×77,

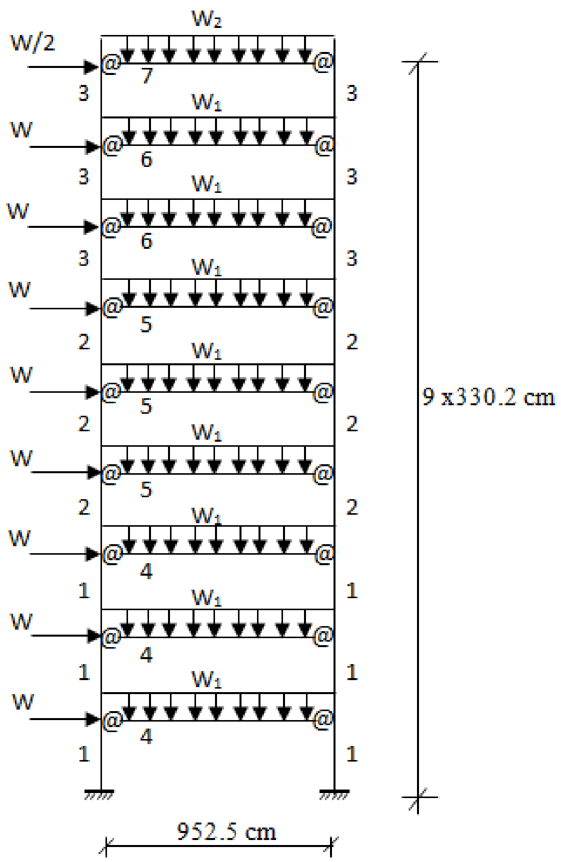



Fig. 5 Nine-storey, single-bay frame

W12×19, W12×26, W12×30, W12×35, W12×40, W12×45, W12×50, W12×53, W12×58, W12×72, W12×96, W14×26, W14×30, W14×34, W14×38, W14×43, W14×48, W14×53, W14×61, W14×68, W14×74, W14×82, W14×90, W14×120, W14×159, W14×193, W14×257, W14×311, W14×370, W14×426, W16×26, W16×31, W16×36, W16×40, W18×35, W18×40, W18×50, W18×76, W21×50, W21×62, W21×132, W24×68, W24×103, W27×94, W27×161, W30×108, W30×148, W30×191, W33×221, W36×194).

### *5.1 Example1: Nine-storey, single-bay frame*

A nine-storey, single-bay frame shown in Fig. 5 was previously studied by Hadidi and Rafiee (2014) using Harmony search based, improved Particle Swarm Optimizer (HS-PSO) for minimum steel weight without composite beams incorporating stress constraints of AISC-LRFD, maximum lateral displacement constraints, column-to-column and beam to column size constraints. In this study, the frame is studied under the same constraints. Also, the frame is grouped and loaded as seen in Fig. 5. The loads $W$, $W_1$ and $W_2$ are 17.8 kN, 27.14 kN/m and 24.51 Kn/m, respectively and the design parameters are $E$=200 GPa, yield stress $f_y$=248.2 MPa, material density $\rho$=7.85 ton/m$^3$ (Hadidi and Rafiee 2014). The maximum top storey drift is restricted to 154 mm.

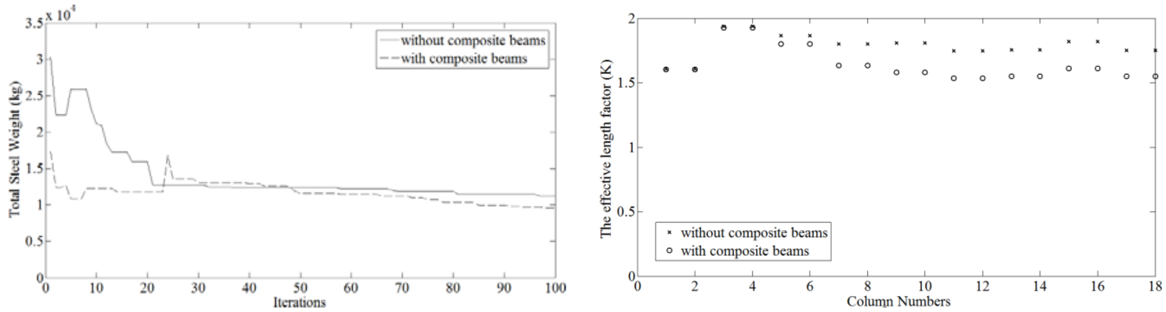
Table 1 Optimum cross sections of the full rigid frame

| Group no | Hadidi and Rafiee (2014) (without composite beams) | | Present Study | |
|---|---|---|---|---|
| | Full rigid | | Full Rigid | |
| | | | without composite beams | with composite beams |
| 1 | 24×62 | 14×68 | 14×74 | 21×62 |
| 2 | 24×55 | 14×48 | 14×48 | 14×48 |
| 3 | 14×30 | 14×30 | 12×40 | 12×40 |
| 4 | 24×55 | 24×68 | 21×62 | 21×50 |
| 5 | 21×50 | 24×55 | 21×50 | 18×40 |
| 6 | 21×44 | 21×44 | 18×40 | 14×34 |
| 7 | 18×35 | 18×35 | 14×38 | 12×26 |
| Total weight kg | 10.529 | 11.281 | 11230 | 9594 |
| Top storey sway mm | 73 | 74 | 86 | 61 |

Table 2 Optimum cross sections of the semi rigid frame

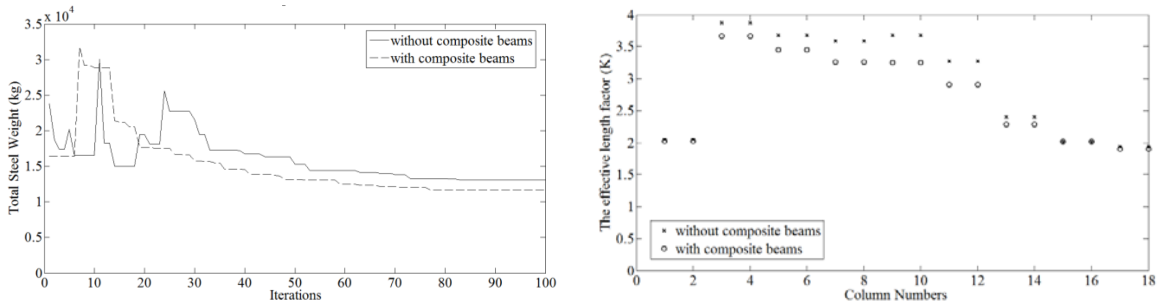
| Group no | Hadidi and Rafiee (2014) (without composite beams) | | | | Present Study | | | |
|---|---|---|---|---|---|---|---|---|
| | Semi Rigid | | | | Semi Rigid | | | |
| | Double Web Angle | | End Plate with Column Stiffeners | | Double Web Angle | | End Plate with Column Stiffeners | |
| | | | | | without composite beams | with composite beams | without composite beams | with composite beams |
| 1 | 40×149 | 14×109 | 33×118 | 14×99 | 24×103 | 27×94 | 24×68 | 21×62 |
| 2 | 24×62 | 14×74 | 24×55 | 14×74 | 24×68 | 24×68 | 14×61 | 12×58 |
| 3 | 12×40 | 14×30 | 14×30 | 14×30 | 16×36 | 16×36 | 14×61 | 12×58 |
| 4 | 21×44 | 24×68 | 21×48 | 24×68 | 24×68 | 21×62 | 21×62 | 21×50 |
| 5 | 21×50 | 24×55 | 24×55 | 24×55 | 21×50 | 18×40 | 14×53 | 14×48 |
| 6 | 16×45 | 18×46 | 21×44 | 18×46 | 21×50 | 14×38 | 12×58 | 12×58 |
| 7 | 18×35 | 18×35 | 18×35 | 18×35 | 18×35 | 12×26 | 18×35 | 18×35 |
| Total weight kg | 13182 | 13281 | 12136 | 12983 | 13072 | 11665 | 12652 | 11552 |
| Top storey sway mm | 73 | 76 | 69 | 77 | 75 | 63 | 98 | 69 |

The example is carried out 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 Table 1 and Table 2, respectively. Also in this table, the results obtained twice by Hadidi and Rafiee (2014) are presented for comparison. Figs. 6(a), 7(a) and 8(a) present the variation of the total steel weight with iterations for both cases (with and without composite beams) and Figs. 6(b), 7(b) and 8(b) show the values of the effective length factor ($K$) of the columns of steel frame for both cases.



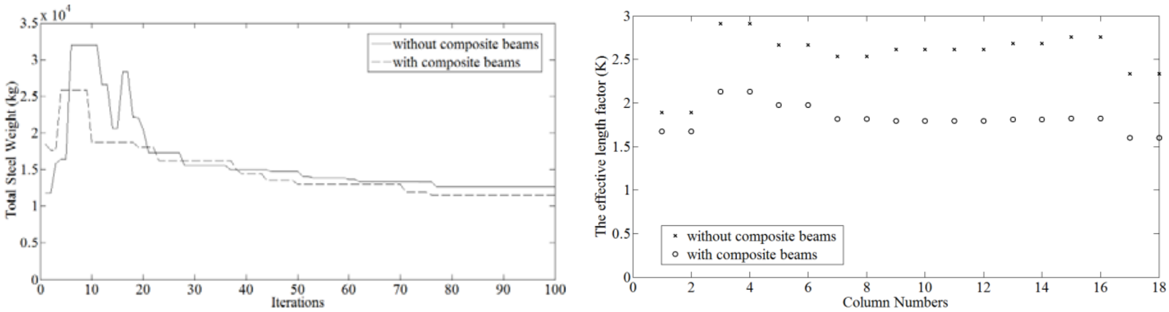

(a) The variation of the total steel weight (b) The effective length factor (K) of the columns

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




(a) The variation of the total steel weight (b) The effective length factor (K) of the columns

Fig. 7 Semi rigid steel frame with Double Web Angle connection with and without composite beams




(a) The variation of the total steel weight (b) The effective length factor (*K*) of the columns

Fig. 8 Semi rigid steel frame with end Pate with Column Stiffness connection with and without composite beams

It is observed from Table 1 and Table 2 that the optimum design results of Genetic Algorithm are very close to the ones available in literature. Top storey sway values in Table 1 and Table 2 are far below the limit. Therefore, stress and size constraints play active roles in determining the optimum design of the frames. As seen in the figures and the tables above, the minimum weights obtained for the fully rigid steel frame without composite beams is 16% and 12.6% lighter than the semi-rigid frames with Double Web Angle and End Plate with Column Stiffeners connections, respectively. Rotational spring stiffnesses of these two types of semi rigid connections are shown in Fig. 3. As regards the figures of the effective length factor (*K*) of the columns, a decrease in

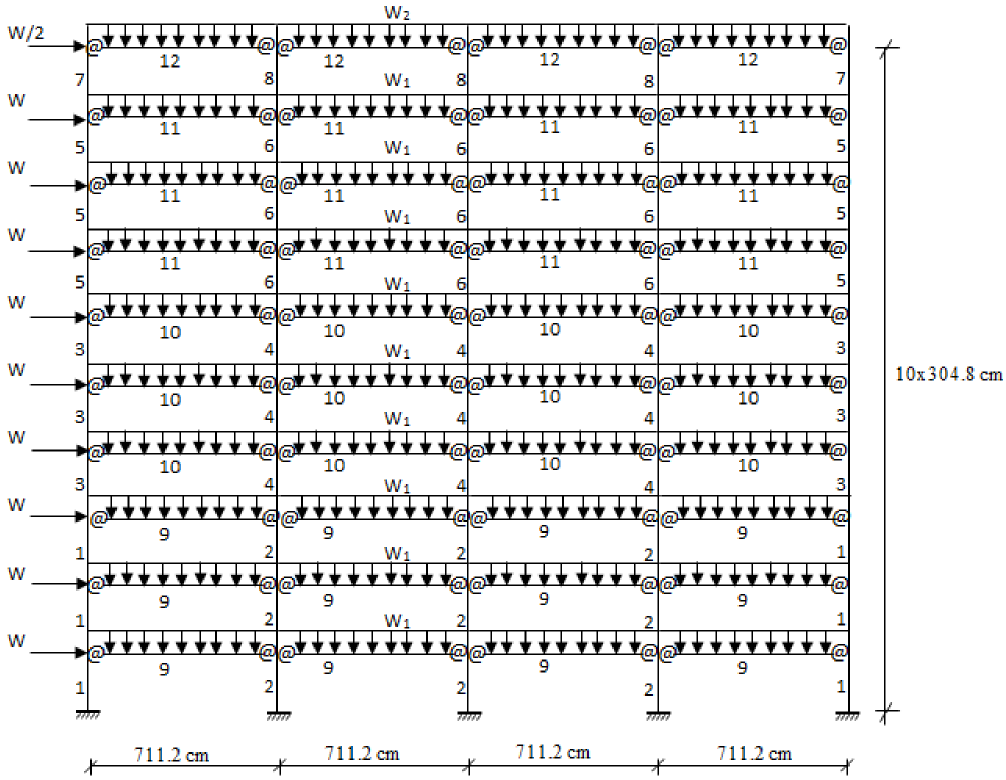



Fig. 9 Ten-storey, four-bay frame

rotational spring stiffness of semi rigid connection significantly increases *K* values of the columns and so, this situation increases the design weight.

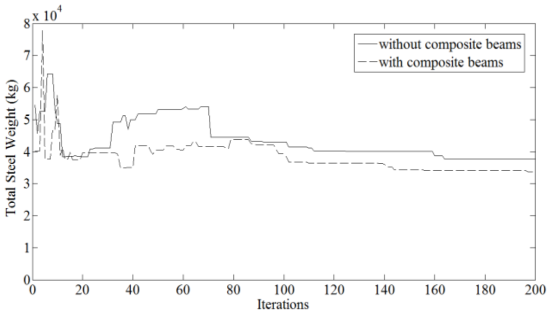
It is apparently seen from the tables above, the minimum weight obtained for the full rigid frame with composite (steel and concrete) beams is 14.6% lighter compare to the weight of the same frame with regular beams without considering composite effect. Furthermore, in the optimal designs of fully rigid frame, maximum top storey displacement decreases from 86 mm to 61 mm by considering concrete slab effects. The design weights of semi rigid frames for the case with composite beams are 8-10% lighter than the case without composite beams. As regards the figures above, in the optimal design of the case with composite beams, considering concrete slab effects in finite element analyses significantly reduces the effective length factor values of the columns, and so design weight decreases.

### *5.2 Example2: Ten-storey, four-bay frame*

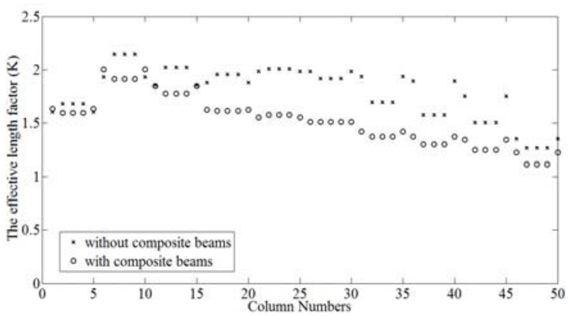
A ten-storey, four-bay frame with 90 members shown in Fig. 9 was previously studied by Hadidi and Rafiee (2014) using HS-PSO for minimum steel weight. The frame members are collected in 12 groups and the loads are imposed as shown in Fig. 9. Loads $W$, $W_1$ and $W_2$ are 44.49 kN, 47.46 kN/m and 42.91 kN/m, respectively and material properties are $E$=200 GPa, yield stress $f_y$=248.2 MPa, material density $\rho$=7.85 ton/m$^3$ (Hadidi and Rafiee 2014). Maximum top storey drift is restricted to 158 mm. The example is carried out for optimum weight considering the

Table 3 Optimum cross sections of the full and semi rigid frame

| Group no | Hadidi and Rafiee (2014) (without composite beams) | | | Present Study | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Semi Rigid | | Full Rigid | | Semi Rigid | | | |
| | Full rigid | Double Web Angle | End Plate with Column Stiff. | without composite beams | with composite beams | Double Web Angle | | End Plate with Column Stiffeners | |
| | | | | | | without composite beams | with composite beams | without composite beams | with composite beams |
| 1 | 14×74 | 14×132 | 14×132 | 14×68 | 24×68 | 24×103 | 30×108 | 14×74 | 21×62 |
| 2 | 14×132 | 14×120 | 14×120 | 14×159 | 21×132 | 30×148 | 21×132 | 21×132 | 30×108 |
| 3 | 14×61 | 14×61 | 14×99 | 12×53 | 12×58 | 14×90 | 14×53 | 14×61 | 21×62 |
| 4 | 14×82 | 14×109 | 14×82 | 14×82 | 14×90 | 27×94 | 21×132 | 14×82 | 14×74 |
| 5 | 14×43 | 14×48 | 14×68 | 12×53 | 12×40 | 14×53 | 14×48 | 14×48 | 12×58 |
| 6 | 14×48 | 14×61 | 14×48 | 14×48 | 14×48 | 14×61 | 14×43 | 14×48 | 12×50 |
| 7 | 14×43 | 14×43 | 14×68 | 8×31 | 10×39 | 8×35 | 12×40 | 14×48 | 8×24 |
| 8 | 14×43 | 14×48 | 14×30 | 10×39 | 8×31 | 12×58 | 10×33 | 8×31 | 8×31 |
| 9 | 21×44 | 21×48 | 21×44 | 21×50 | 21×50 | 21×50 | 21×50 | 24×68 | 21×62 |
| 10 | 21×44 | 16×50 | 21×44 | 14×53 | 16×40 | 21×50 | 18×35 | 14×53 | 18×40 |
| 11 | 21×44 | 16×45 | 16×45 | 18×40 | 18×35 | 16×40 | 14×34 | 18×40 | 14×38 |
| 12 | 18×40 | 18×40 | 18×46 | 18×50 | 18×35 | 14×38 | 14×30 | 14×38 | 18×35 |
| Total weight kg | 35125 | 39435 | 38288 | 37653 | 33718 | 39611 | 35433 | 38669 | 34400 |
| Top storey sway mm | 56 | 72 | 68 | 61 | 35 | 54 | 45 | 56 | 38 |



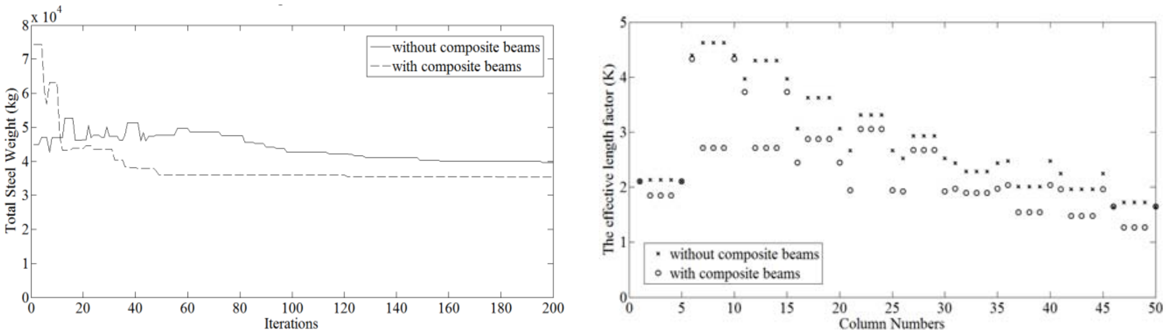

(a) The variation of the total steel weight




(b) The effective length factor (K) of the columns

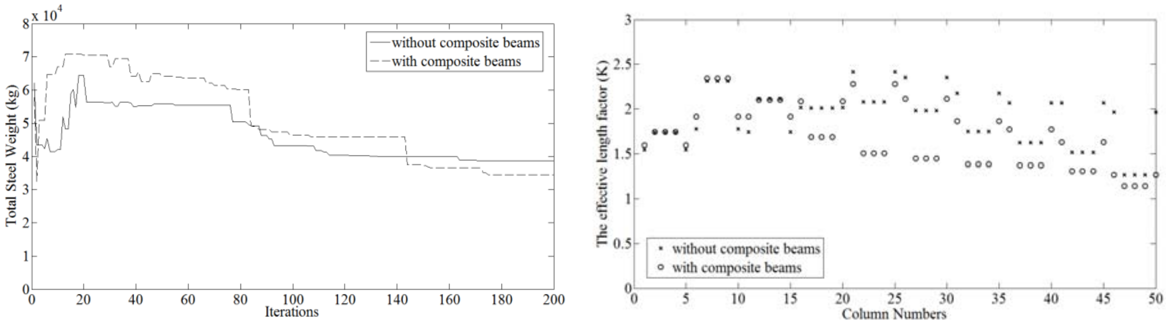
Fig. 10 Fully rigid steel frame with and without composite beams

frame with and without composite beams. Minimum weights, maximum top story drifts, steel sections of optimum design for fully rigid and semi rigid steel frames are presented in Table 3. Also in this table, results obtained by Hadidi and Rafiee (2014) are presented for comparison. Figs. 10(a), 11(a) and 12(a) show the variation of total steel weight with iterations for both cases and




(a) The variation of the total steel weight (b) The effective length factor (K) of the columns

Fig. 11 Semi rigid steel frame with Double Web Angle connection with and without composite beams




(a) The variation of the total steel weight (b) The effective length factor (K) of the columns

Fig. 12 Semi rigid steel frame with End Plate with Column Stiffness connection with and without composite beams

Figs. 10(b), 11(b) and 12(b) present the values of effective length factor (*K*) for the columns of steel frame for both cases.

As shown in Table 3, the optimum design results of present study are very close to the results obtained by Hadidi and Rafiee (2014) for the case of frames with regular beams or without composite beams. Maximum top storey displacements are significantly less than upper limit. Therefore, stress and size constraints are important determinants of optimal designs for full and semi rigid frames. As seen in Table 3 and Figs. 10, 11 and 12, minimum weight for the fully rigid steel frame without composite beams is about 10% heavier than the design with composite beams and it is about 5% and 2.6% lighter than the optimum designs of semi rigid frames (Double Web Angle and End Plate with Column Stiffeners connections) without composite beams. Studying Figs. 11b and 12b it can clearly be observed that the effective length factor, K, for columns depend on rotational spring stiffness of semi rigid connections and a decrease in the rotational spring stiffness results with an increase in K and so the buckling lengths of columns. So, this situation leads to the selection of larger cross-section profiles for columns, Table 3. Consideration of concrete slab effects on the story beams significantly reduces the effective length factor of columns and maximum top storey displacements. Also in the optimal design of frames with composite beams, selected sections of beams are usually smaller and minimum steel weight is reduced by about %10.

## 6. Conclusions

In the present study, Genetic Algorithm is used for optimum design of full and semi rigid frames with and without composite beams. Stress constraints of AISC-LRFD, maximum lateral displacement constraints and geometric constraints are applied. Also, in the optimum design of plane frames with composite beams, concrete slab effects are considered in finite element analyses. All procedures are also repeated for the designs of semi rigid frames with Double Web Angle and End Plate with Column Stiffeners connections. Two different examples taken from literature are resized for the cases of plane frames with and without composite beams. Results obtained from analyses are presented in tabular and graphical formats.

• Consideration of the contribution of concrete slabs on behavior of beams ended up with less steel weight. Minimum steel weight is reduced by about %10 or both examples here.

• Effective length factor (*K*) of columns depend on rotational spring stiffness of semi rigid connections and a decrease in rotational spring stiffness results with an increase in *K*, and buckling length of columns. This situation increases design weight. In the first example, the design weight of fully rigid frame without composite beams is 16% and 12.6% lighter than the semi-rigid frames with Double Web Angle and End Plate with Column Stiffeners connections, respectively. These values become about 5% and 2.6% in the second example,

• In the optimum design of plane frames with composite beams, consideration of concrete slab effects in finite element analyses significantly reduces the effective length factor values of the columns and maximum top storey displacements. Furthermore, selected sections of the beams are usually smaller.

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