Steel and Composite Structures, *Vol. 19*, *No. 2* (*2015*) *503-519*
DOI: http://dx.doi.org/10.12989/scs.2015.19.2.503

Optimum design of steel space frames with composite beams using genetic algorithm

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(*Received November 18, 2014, Revised January 17, 2015, Accepted August 13, 2015*)

Abstract. This paper presents an optimization process using Genetic Algorithm (GA) for minimum weight by selecting suitable standard sections from a specified list taken from American Institute of Steel Construction (AISC). The stress constraints obeying AISC-LRFD (American Institute of Steel Construction - Load and Resistance Factor Design), lateral displacement constraints being the top and inter-storey drift, mid-span deflection constraints for the beams and geometric constraints are considered for optimum design by using GA that mimics biological processes. Optimum designs for three different space frames taken from the literature are carried out first without considering concrete slab effects in finite element analyses for the constraints above and the results are compared with the ones available in literature. The same optimization procedures are then repeated for the case of space frames with composite (steel and concrete) beams. A program is coded in MATLAB for the optimization processes. Results obtained in the study showed that consideration of the contribution of the concrete on the behavior of the floor beams results with less steel weight and ends up with more economical designs.

Keywords: genetic algorithm; weight optimization; space frame; composite beams; fem analysis

1. Introduction

Weight minimization of steel structures is very important to reduce steel consumption. Therefore, optimization of structural systems with discrete design variables using various optimization methods such as genetic algorithm, harmony search algorithm, particle swarm optimization, ant colony optimization, tabu search optimization etc., 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. Daloglu and Aydın (1999) studied optimum design of plane trusses using GA. Erbatur *et al*. (2000) examined optimal design of planar and space structures with genetic algorithms. Sergeyev and Mroz (2000) investigated sensitivity analysis and optimal design of 3D frame structures for stress and frequency constraints. Tong and Liu (2001) developed an optimization procedure for truss structures with discrete design

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http://www.techno-press.org/?journal=scs&subpage=8 ISSN: 1229-9367 (Print), 1598-6233 (Online)

variables and dynamic constraints. They focused on planar and space truss systems. Isenberg *et al*. (2002) studied optimal design of planar steel frame structures. Togan and Daloglu (2006) investigated optimization of space trusses with adaptive approach in genetic algorithms. Degertekin and Hayalioglu (2009) examined optimum design of steel space frames using tabu search. They used strength constraints of AISC-LRFD specification, maximum drift, inter-storey drift and size constraints for columns. Gero *et al*. (2006) researched design optimization of 3D steel structures using genetic algorithms. Kaveh and Talatahari (2007) used a discrete particle swarm ant colony optimization for design of steel frame structures. Esen and Ülker (2008) researched optimization of multi storey space steel frames. Saka (2009) researched optimum design of steel sway frames according to BS5950 using harmony search algorithm. Hasancebi *et al*. (2011) investigated optimum design of high-rise steel buildings using an evolution strategy integrated parallel algorithm and examined several high-rise structures. Martini (2011) used harmony search method for multimodal size, shape and topology optimization of structural frame works. Rosca *et al*. (2012) examined practical optimization of composite steel and concrete girders. Aydogdu and Saka (2012) studied ant colony optimization of irregular steel frames according to LRFD-AISC. Carbas *et al*. (2013) examined a comparative study of three different metaheuristic search techniques namely the Firefly Algorithm (FFA), Artificial Bee Colony (ABC), and Cuckoo Search (CS) algorithms for optimum design of engineering structures. Hussein and Taysi (2013) focused on GA optimization of space frames and considered actual design constraints like, strength, lateral displacement, interstory drift according to LRFD. Degertekin (2012) examined optimum design of geometrically non-linear steel frames using artificial bee colony algorithm. Kaveh and Talatahari (2012) studied a hybrid CSS and PSO algorithm for optimal design of structures. Dede and Ayvaz (2013) studied structural optimization with teaching-learning-based optimization algorithm. Dede (2013) researched optimum design of grillage structures to LRFD-AISC with teaching-learning based optimization. Dede (2014) focused on application of teaching-learning-based-optimization algorithm for the discrete optimization of truss 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.

There are numerous studies published in the literature on the weight optimization of structural systems. But it is hard to see enough studies about optimization of space frames considering concrete slab effects on the behavior of beams. So, in this study, optimum design of space frames is studied with and without taking the effect of concrete slab into the consideration on the FEM analyses. Results obtained from the optimization of space 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. Genetic algorithm and FEM analyses

Genetic Algorithm was proposed by Goldberg (1989) conducting natural biological procedures such as reproduction, crossover and mutation. Genetic Algorithm is used for minimization of objective function in the optimal design of structures. The main purpose of genetic algorithm operators is to get a strong population for optimal solution of a structure. So, weak individuals are removed from population while the fittest individuals remain in population. A sequence code including 0 or 1 character can be used to represent a design variable in genetic algorithm. The genetic algorithm steps for an optimum design are listed as follows; GA analyses start with random initial population comprised of individuals which can be 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 according to the results obtained from finite element analyses. The individuals in population are arranged and reproduction, crossover and mutation operators are applied. Thus, the initial population is replaced by new population to complete one iteration. The procedures and iterations are repeated until convergence is obtained. Fig. 1 presents the flowchart of GA optimization. Several types of crossover operators are available in literature such as single-point, double-point and multi-point. The double-point crossover operator is adopted here. A computer program is coded in MATLAB for the optimization processes explained above.

The formulations of objective, penalty and fitness functions for definition of fitness value of each individual in population are presented below (Daloglu and Aydın 1999)

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

$$C = \sum_{i=1}^{m} c_i \tag{2}$$

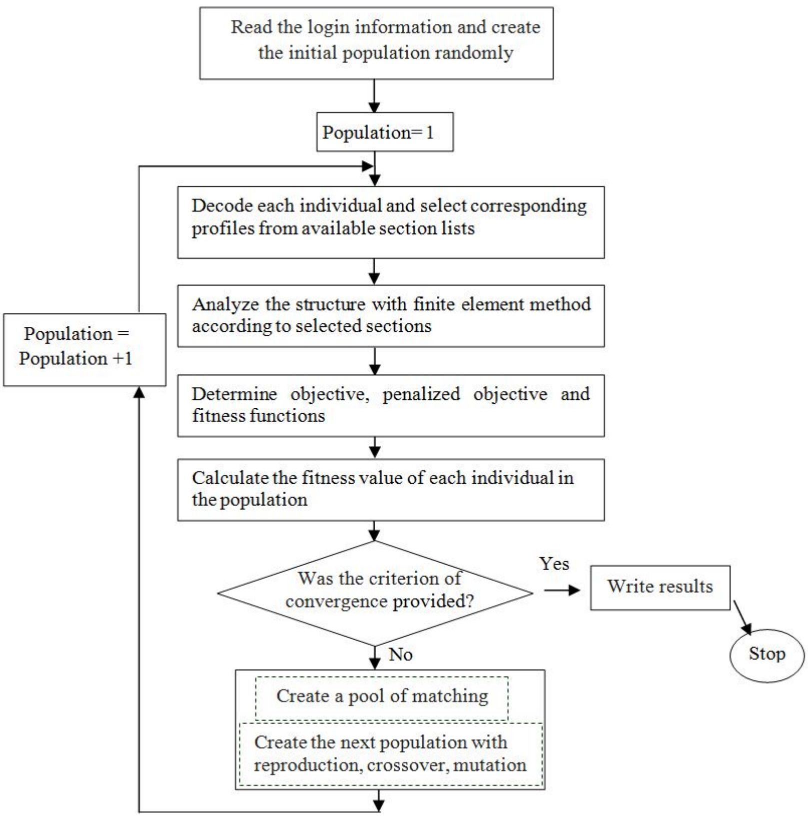


Fig. 1 GA flowchart

$$PC = P\sum_{i=1}^{m} c_i \tag{3}$$

$$\varphi(x) = W(x)(1 + PC) \tag{4}$$

$$F_i = (\varphi(x)_{\max} + \varphi(x)_{\min}) - \varphi(x)_i \tag{5}$$

$$F_{c,i} = \frac{F_i}{F_{\text{average}}} \tag{6}$$

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, c_i is constraint violations, C is penalty function, P is a penalty constant, $\varphi(x)$ is penalized objective function, F_i is fitness function and $F_{c,i}$ is fitness factor.

Element stiffness matrix in local coordinates for each member, (k), is calculated from Eq. (7). Then, stiffness matrix in global coordinates system for the member, (K), is obtained from Eq. (8) by using coordinate transformation matrix, (T). Stiffness matrix for whole structure is obtained by assembling element stiffness matrices in global coordinates. Then, FEM analyses are carried out and stresses and displacements of each element in the space frame are determined.

$$k = \begin{bmatrix}
\frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & -EA/L & 0 & 0 & 0 & 0 & 0 \\
 & \frac{12EI_Z}{L^3} & 0 & 0 & 0 & \frac{6EI_Z}{L^2} & 0 & \frac{-12EI_Z}{L^3} & 0 & 0 & 0 & \frac{6EI_Z}{L^2} \\
 & & \frac{12EI_Y}{L^3} & 0 & \frac{-6EI_Y}{L^2} & 0 & 0 & 0 & \frac{12EI_Y}{L^3} & 0 & \frac{6EI_Y}{L^2} & 0 \\
 & & & \frac{GJ}{L} & 0 & 0 & 0 & 0 & 0 & \frac{-GJ}{L} & 0 & 0 \\
 & & & & \frac{4EI_Y}{L} & 0 & 0 & 0 & \frac{6EI_Y}{L^2} & 0 & \frac{2EI_Y}{L} & 0 \\
 & & & & & \frac{4EI_Z}{L} & 0 & \frac{-6EI_Z}{L^2} & 0 & 0 & 0 & \frac{2EI_Z}{L} \\
 & & & & & & \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 \\
 & & & & & & & \frac{12EI_Y}{L^3} & 0 & 0 & 0 & \frac{-6EI_Y}{L^2} \\
 & & & & & & & & \frac{12EI_Y}{L^3} & 0 & \frac{6EI_Y}{L^2} & 0 \\
 & & & & & & & & & \frac{GJ}{L} & 0 & 0 \\
 & & & & & & & & & & \frac{4EI_Y}{L} & 0 \\
 & & & & & & & & & & & \frac{4EI_Z}{L}
\end{bmatrix} \tag{7}$$

$$K = T^t k T \tag{8}$$

3. The Formulations of constraints

In this study, the stress constraints obeying AISC-LRFD, lateral displacement constraints being the top-storey and the inter-storey drift, mid-span deflection constraints of beams and geometric constraints for column-to-column and beam-to-column are considered in the optimum design of space frames. The stress constraints taken from AISC–LRFD (1995) are presented in Eqs. (9) and (10).

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

$$\text{for} \quad \frac{P_u}{\phi P_n} < 0.2 \quad g_u(x) = \left(\frac{P_u}{2\phi P_n}\right)_{il} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) - 1.0 \leq 0 \quad \begin{matrix} i = 1,\ldots,nm \\ l = 1,\ldots,nl \end{matrix} \tag{10}$$

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_{uy} is the required flexural strength about minor axis, M_{nx} is the nominal flexural strength about major axis, M_{ny} is the nominal flexural strength about minor axis, ϕ is resistance factor for compression (0.85) and for tension (0.90), ϕ_b is resistance factor for flexure (0.90).

The nominal compressive strength is calculated from Eqs. (11)-(14)

$$P_n = A_g F_{cr} \tag{11}$$

$$\text{for} \quad \lambda_c \leq 1.5 \quad F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y \tag{12}$$

$$\text{for} \quad \lambda_c > 1.5 \quad F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right) F_y \tag{13}$$

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} \tag{14}$$

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, F_{cr} is critical stress, λ_c is slenderness ratio. The effective length factor K for unbraced frames is determined from Eq. (15); (Dumonteil 1992, Degertekin 2007)

$$K = \sqrt{\frac{1.6 G_A G_B + 4.0(G_A + G_B) + 7.50}{G_A + G_B + 7.50}} \tag{15}$$

where G_A and G_B are the relative stiffness factors at Ath and Bth ends of columns. The factors are calculated from Eq. (16).

$$G = \left(\frac{\sum I_c / L_c}{\sum (I_g / L_g)} \right) \tag{16}$$

where I_c is moment of inertia of column cross section in plane of buckling, L_c is unbraced length of column, I_g is moment of inertia of beam cross section in plane of bending, L_g is unbraced length of beam.

Other constraints are as below: (Aydogdu and Saka 2012)

- Displacement constraints are shown in Eq. (17)

$$g_{jl}(x) = \frac{\delta_{jl}}{\delta_{ju}} \leq 0 \qquad \begin{aligned} i &= 1, \ldots, m \\ l &= 1, \ldots, nl \end{aligned} \tag{17}$$

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

- Inter-storey drift constraints are shown in Eq. (18)

$$g_{jul}(x) = \frac{\Delta_{jil}}{\Delta_{ju}} - 1 \leq 0 \qquad \begin{aligned} i &= 1, \ldots, ns \\ l &= 1, \ldots, nsc \\ l &= 1, \ldots, nl \end{aligned} \tag{18}$$

where Δ_{jil} is the inter-storey drift of ith column in the jth storey under load case l, Δ_{ju} is the limit value (story height/300), ns is the number of storey, nsc is the number of columns in a storey.

- Deflection constraints are presented in Eq. (19)

$$g_{dl}(x) = \frac{\delta_{dl}}{\delta_{du}} - 1 \leq 0 \qquad \begin{aligned} i &= 1, \ldots, nb \\ l &= 1, \ldots, nl \end{aligned} \tag{19}$$

where δ_{dl} is the maximum deflection of dth beam under load case l, δ_{du} is the upper bound of deflection (span/360), nb is the total number of beams.

- Column-to-column geometric constraints (size constraints) are expressed in Eqs. (20)

$$g_n(x) = \frac{D_{un}}{D_{\ln}} - 1 \leq 0 \qquad n = 2, \ldots, ns \tag{20}$$

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 Eqs. (21) and (22)

$$g_{bf,i}(x) = \frac{b_{fbk,i}}{d_{c,i} - 2t_{fl,i}} - 1 \leq 0 \qquad i = 1, \ldots, n_{bw} \tag{21}$$

where n_{bw} is number of joints where beams are connected to web of column, $b'_{fbk,i}$ is flange width of beam, $d_{c,i}$ is depth of column, $t_{fl,i}$ is flange thickness of column.

$$g_{bb,i}(x) = \frac{b_{fbk,i}}{b_{fck,i}} - 1 \leq 0 \qquad i = 1,\ldots,n_{bf} \tag{22}$$

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 the beam and column, respectively.

4. Composite beams

Concrete slab and steel floor beams act together to carry loads. In this study, contribution of concrete slabs on behavior of steel beams is taken into account in the analysis. Possible advantages of considering the contribution of concrete slab on beams may be listed as; greater stiffness, greater bending strength, less lateral displacements, less mid-span deflection. These advantages result with more economical steel structures designs. Although various types of composite steel-concrete sections are constructed, most commonly used composite section type in practice is shown in Fig. 2. For practical calculation of composite section properties, the effective width of concrete slab is determined from Eq. (23) or (24) (Salmon and Johnson 1980)

$$\text{for an interior beam,} \quad \begin{aligned} b_E &\leq \frac{L}{4} \\ b_E &\leq b_0 \\ b_E &\leq b_f + 16t_s \end{aligned} \tag{23}$$

$$\text{for an exterior beam,} \quad \begin{aligned} b_E &\leq \frac{L}{12} + b_f \\ b_E &\leq \frac{1}{2}(b_0 + b_f) \\ b_E &\leq b_f + 6t_s \end{aligned} \tag{24}$$

The ultimate moment capacity M_u is calculated according to neutral axis within the concrete slab or within the steel beam. The cases of neutral axis are named as slab adequate and slab

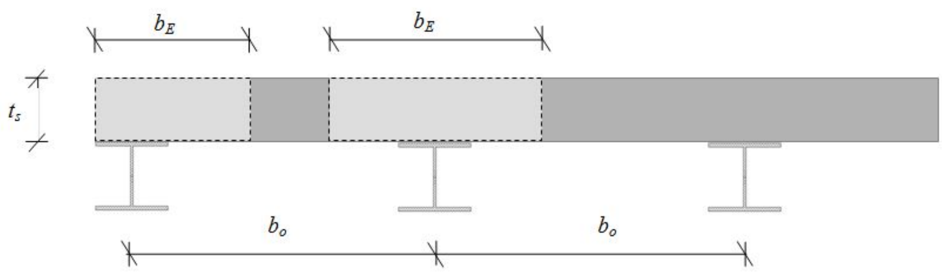


Fig. 2 Effective width of composite beam

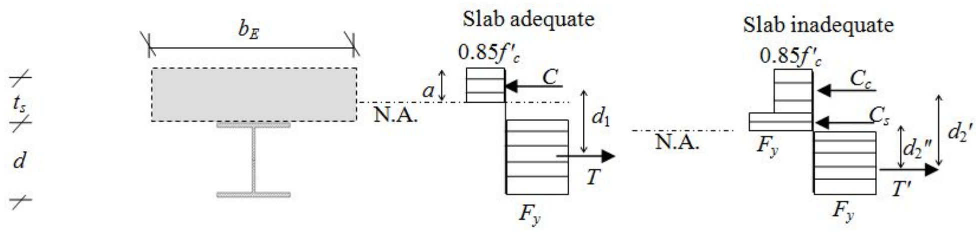


Fig. 3 The stress distribution of composite beam

inadequate. Stress distribution and calculation of M_u for two cases are presented below; (Salmon and Johnson 1980).
for the case of slab adequate

$$C = 0.85 f_c' a b_E \tag{25}$$

$$T = A_s F_y \tag{26}$$

$$a = \frac{A_s F_y}{0.85 f_c' b_E} \tag{27}$$

$$M_u = C d_1 \text{ or } T d_1 \tag{28}$$

$$M_u = A_s F_y \left(\frac{d}{2} + t_s - \frac{a}{2} \right) \tag{29}$$

for the case of slab inadequate

$$C_c = 0.85 f_c t_s b_E \tag{30}$$

$$T' = C_c + C_s \text{ or } A_s F_y - C_s \tag{31}$$

$$M_u = C_c d_2' + C_s d_2'' \tag{32}$$

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_0 is interval between two beams, t_s is thickness of concrete slab, d is beam depth, a is a depth, C is the ultimate compressive force, C_c is compressive force in the slab, C_s is compressive force in the steel beam, T and T' are the ultimate tensile forces, M_u is the ultimate moment capacity, d_1, d_2, d_3 are the moment arms, f'_c is the compressive strength, A_s is the beam area.

5. Design examples

Three different space frames taken from the literature are designed for comparison purposes. Optimum designs of these examples are studied for two different cases. Case 1 is the optimization of space frames without considering concrete slab effects on FEM analyses which introduces the space frames with steel beams. Case 2 is the optimization of space frames considering concrete slab effects which means the frames with composite (steel and concrete) beams. In Case 2, concrete slab is placed as seen in Fig. 2 in each example. The 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, W8×21, W8×24, W8×28, W8×31, W8×35, W8×40, W10×15, W10×22, W10×26, W10×33, W10×39, W10×54, W10×77, 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, W143×70, 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: Design of single-storey, 8-member space frame

A single-storey and 8 member space frame system in Fig. 4 is studied first. The space frame members collected into three groups as two groups for the beams and one for the columns as shown in Fig. 4. The beams in x- direction are collected in the 1-st group, 2-nd group is for the beams in y-direction and the 3-rd group is for all the columns. The space frame is subjected to dead load, D = 3.12 kN/m^2, live load of L = 2.40 kN/m^2, roof live load L_r = 2.40 kN/m^2 and wind pressure $p = C_e C_q q_s I_w$, where p is design wind pressure; C_e is combined height, exposure and guest factor coefficient; C_q is pressure coefficient and equal to 0.8 and 0.5 for windward and leeward faces respectively, q_s is wind stagnation pressure and equal to 0.785 kN/m^2, the wind importance factor I_w is 1. Also the design parameters are elastic modulus E = 200 GPa, shear modulus G = 83 GPa, yield stress f_y = 248.2 MPa and material density ρ = 7.85 ton/m^3. The wind loads are applied to structure in the x-direction. The maximum top and inter-storey drifts are restricted to 1.3. Load combinations used in this example are as follows:

I: $1.4D$;
II: $1.2D + 1.6L + 0.5L_r$;
III: $1.2D + 1.6L_r + 0.5L$;
IV: $1.2D + 1.3W + 0.5L_r + 0.5L$ (W: wind load)

This steel frame system was previously studied by Degertekin (2007), Degertekin and Hayalioglu (2009) using Genetic Algorithm (GA), Tabu Search (TS) and Simulated Annealing (SA). They used stress constraints of AISC-LRFD, size constraints of columns, maximum (lateral displacement) and inter-storey drift constraints. They conducted geometrically nonlinear analysis of space frames. On the other hand, in this study, the same example is carried out by using GA with linear analysis for Case 1 and Case 2 subjected to all the constraints mentioned above in Section 3. Fig. 5 presents the variation of the total steel weight with the number of iterations for both cases. The minimum weights, maximum top-storey drifts, steel sections of optimum designs are presented in Table 1.

As seen in Fig. 5, although the results of optimum design are determined after about 40 iterations, the analysis is repeated up to 100 iterations to escape from getting trapped at local minima for respective run. The results obtained in this study agree well with the ones published by other researchers as can be seen in Table 1. However, the larger steel section (W18 × 35) is obtained for the members in the 1st group being the beams in x- direction. The possible reason for this is that mid-span deflection constraints of beams and beam-to-column geometric constraints are

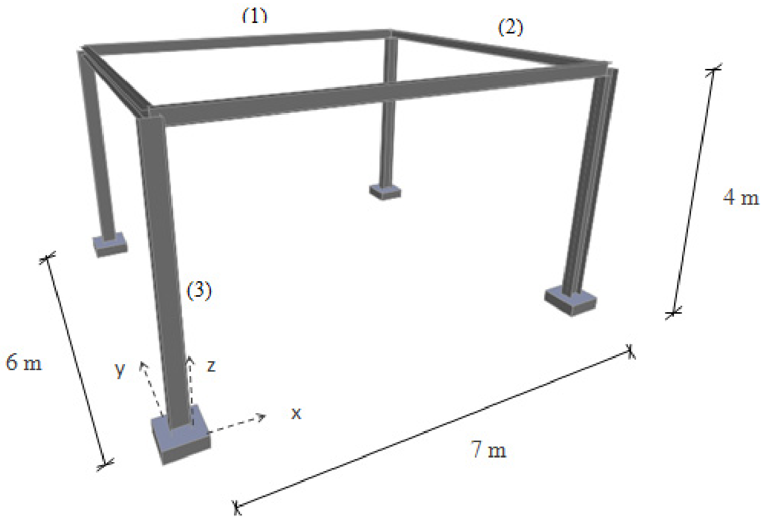


Fig. 4 Single-storey, 8-member space frame

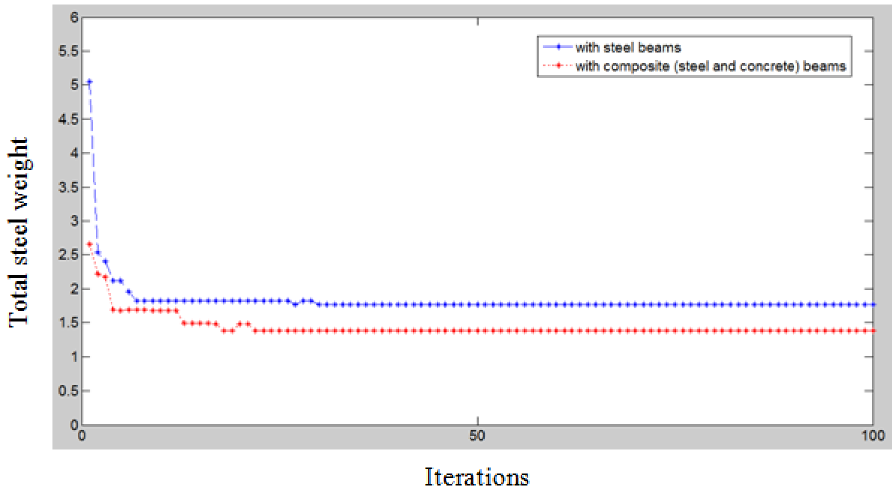


Fig. 5 The variation of the total steel weight with iterations

not included in the previous studies (Degertekin 2007, Degertekin and Hayalioglu 2009) while they are incorporated here. It is obviously seen from Table 1 that the optimum design with composite (steel and concrete) beams as in the Case 2 ended up with smaller beam cross sections and so the weight in this case is least of the all solutions.

5.2 Example 2: Design of 2-storey, 21-member irregular space frame

A 21-member irregular space frame shown in Fig. 6 was previously studied by Aydogdu (2010) using ant colony optimization and harmony search algorithms based on FEM analyses regardless of concrete slab. 21 members of the space frame are organized in two beam and three column

Table 1 Optimum cross sections of 8-member space frame

Group no.	This study (with linear analysis)		Previous studies (with geometric nonlinear analysis)		
	Case 2: with composite beams (steel and concrete)	Case 1: with non-composite beams (with steel beams)	with non-composite beams (with steel beams)		
	(GA)	(GA)	(Degertekin 2007) (GA)	(Degertekin 2007) (SA)	(Degertekin and Hayalioglu 2009) (TS)
1	W16 × 26	W18 × 35	W14 × 30	W12 × 30	W16 × 31
2	W10 × 15	W14 × 26	W14 × 30	W12 × 30	W16 × 26
3	W8 × 24	W8 × 24	W8 × 28	W8 × 24	W8 × 24
Minimum weight(ton)	1.386	1.7712	1.830	1.728	1.687
Maximum top storey drift (cm)	0.87	0.90	1.27	1.19	1.24

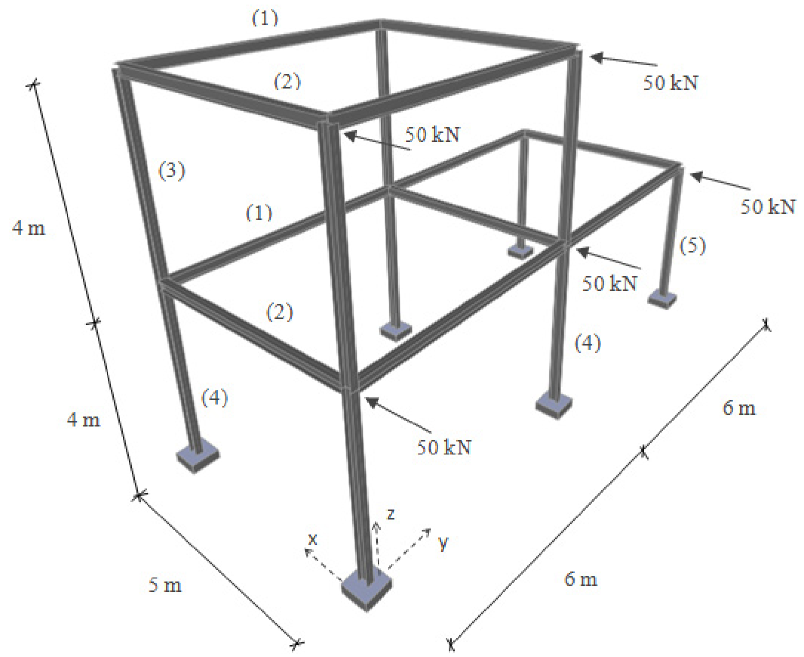


Fig. 6 2-storey and 21-member irregular space frame

groups as shown in Fig. 6. Vertical (gravity) loads on each beam are defined as 20 kN/m and wind load of 50 kN is applied as seen in Fig. 6. The top and inter-storey drift constraints are 4.0 cm and 1.0 cm, respectively. Also, maximum deflection of beams is constrained to 1.39 cm. The design parameters are elastic modulus E = 200 GPa, shear modulus G = 80 GPa, yield stress f_y = 250 MPa and material density ρ = 7.85 ton/m^3.

All the constraints mentioned in Section 3 of this study were applied in the optimization process by Aydogdu (2010) and results are shown in Table 2. In this study, this example is designed by using GA based FEM analyses considering concrete slab (with composite beams, Case 2) and without considering concrete slab (with steel beams, Case 1). The results of both cases are shown in Table 2. The results of present study for Case 1 are suitable and close to previous results obtained by Aydogdu (2010). Fig. 7 illustrates the variation of the total steel weight with iterations for both cases. The minimum weights, maximum top-storey and inter-storey drift values, steel sections of optimum designs are also presented in Table 2. It is obviously seen from Table 2, GA designs 14% lighter space frame for Case 2 where concrete slabs are taken into consideration.

5.3 Example 3: Design of 4-storey, 84-member space frame

The 4-storey space frame system in Fig. 8 was previously studied by Degertekin (2007), Degertekin and Hayalioglu (2009) using various optimization algorithms along with geometrically nonlinear analysis. The space frame members (84 members) divided into 10 groups as follows:

Table 2 Optimum design results of 21-member space frame and comparison with literature results

Group no.	This study (with linear analysis)		Previous study (Aydogdu 2010) (with linear analysis)	
	Genetic Algorithm (GA) Case 2: with composite beams (with steel and concrete beams)	Genetic Algorithm (GA) Case 1: with non-composite beams (with steel beams)	Ant Colony Optimization (ACO) with non-composite beams (with steel beams)	Harmony Search Algorithms (HS) with non-composite beams (with steel beams)
1	W12 × 19	W16 × 26	W460 × 60 (W18 × 40)	W460 × 60 (W18 × 40)
2	W14 × 26	W18 × 35	W360 × 32.9 (W14 × 22)	W310 × 28.3 (W12 × 19)
3	W18 × 35	W18 × 35	W460 × 52 (W18 × 35)	W410 × 60 (W16 × 40)
4	W21 × 50	W21 × 50	W460 × 68 (W18 × 46)	W460 × 60 (W18 × 40)
5	W14 × 30	W14 × 30	W310 × 44.5 (W12 × 30)	W410 × 38.8 (W16 × 26)
Minimum weight (ton)	4.37	5.088	4.96	4.75
Maximum top storey drift (cm)	1.67	1.75	1.82	1.917
Maximum inter-storey drift (cm)	0.87	0.97	0.95	0.956

1-st group: outer beams of 4-th storey; 2-nd group: outer beams of 3-rd, 2-nd and 1-st storeys; 3-rd group: inner beams of 4-th storey; 4-th group: inner beams of 3-rd, 2-nd and 1-st storeys; 5-th group: corner columns of 4-th storey; 6-st group: corner columns of 3-rd, 2-nd and 1-st storeys;

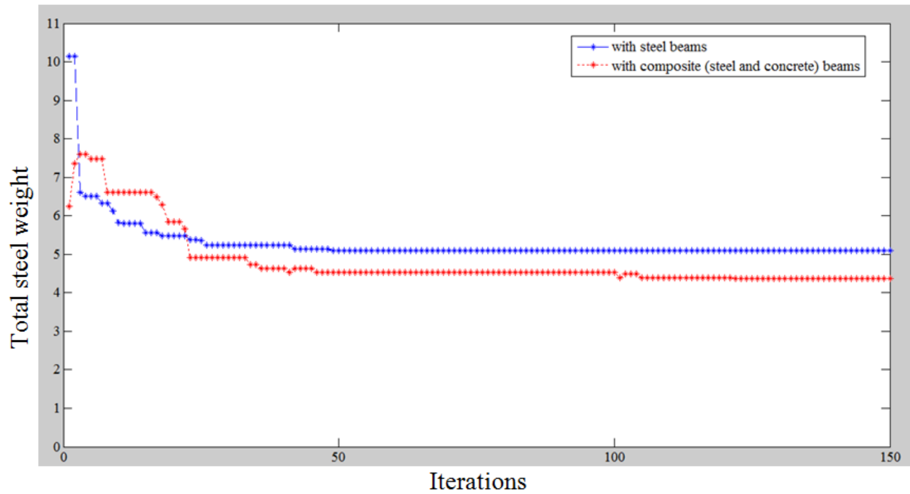
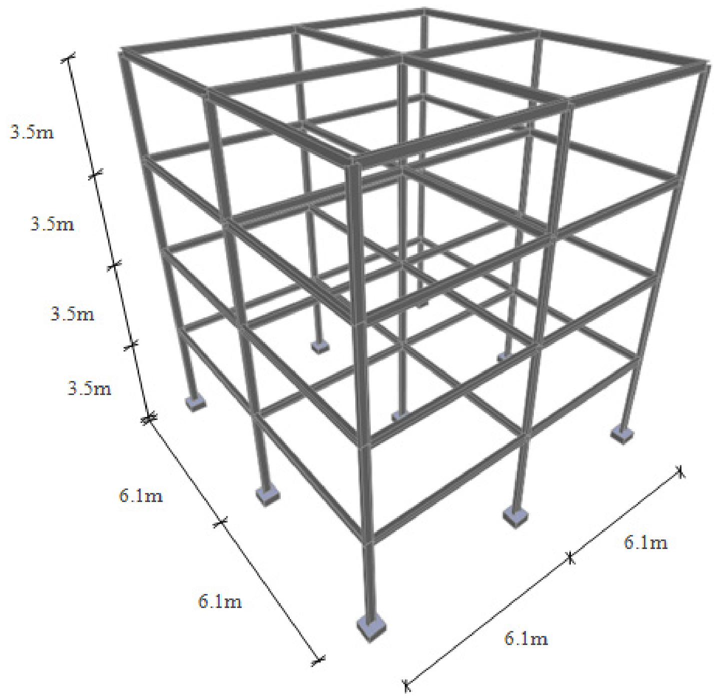
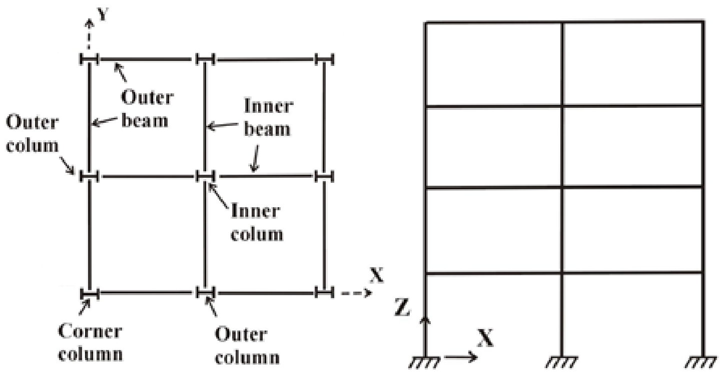


Fig. 7 The variation of the total steel weight with iterations



(a) 3D view

Fig. 8 4-storey, 84-member space frame



(b) Floor plan and side view

Fig. 8 Continued

7-th group: outer columns of 4-th storey; 8-th group: outer columns of 3-rd, 2-nd and 1-st storeys; 9-th group: inner columns of 4-th storey; 10-th group: inner columns of 3-rd, 2-nd and 1-st storeys. Dead, live, roof live, wind loads and design parameters (E, G, f_y, ρ) used in Example 1 are also applied for this example. The wind loads act in the x-direction. The top and inter-storey drifts are constrained to 4.55 cm and 1.52 cm, respectively. Load combinations used in this example are as follows; I: $1.4D$; II: $1.2D+1.6L+0.5L_r$; III: $1.2D+1.6L_r+0.5L$; IV: $1.2D+1.3W+0.5L_r+0.5L$ (W: wind load).

Fig. 9 shows the variation of the total steel weight with iterations for both cases. The minimum weights, maximum top-storey and inter-storey drift values, steel sections of optimum designs are illustrated in Table 3. The minimum weight obtained in this study for the space frame with steel

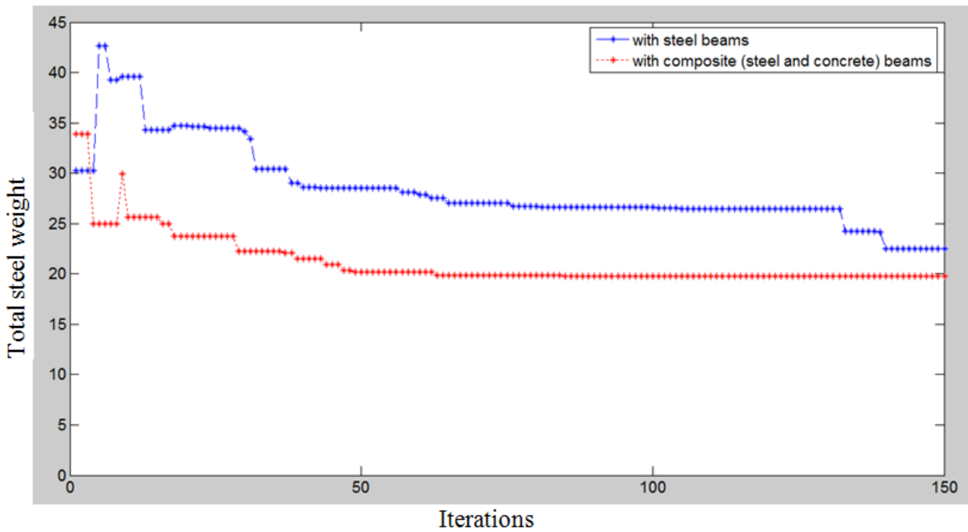


Fig. 9 The variation of the total steel weight with iterations

Table 3 Optimum design of 84-member space frame

Group no.	This study (with linear analysis)		Previous study (Aydogdu 2010) (with linear analysis)	
	Genetic Algorithm (GA) Case 2: with composite beams (with steel and concrete beams)	Genetic Algorithm (GA) Case 1: with non-composite beams (with steel beams)	Ant Colony Optimization (ACO) with non-composite beams (with steel beams)	Harmony Search Algorithms (HS) with non-composite beams (with steel beams)
1	W12 × 19	W12 × 30	W 16 × 31	W 18 × 35
2	W16 × 26	W16 × 36	W 16 × 31	W 18 × 35
3	W 18 × 40	W18 × 35	W 18 × 40	W 18 × 35
4	W14 × 34	W16 × 36	W 18 × 35	W 18 × 35
5	W12 × 26	W8 × 31	W 8 × 35	W 8 × 31
6	W14 × 43	W10 × 39	W 14 × 53	W 12 × 40
7	W12 × 30	W12 × 30	W 8 × 31	W 10 × 39
8	W16 × 40	W14 × 48	W 8 × 35	W 12 × 45
9	W12 × 30	W8 × 24	W 8 × 31	W 8 × 28
10	W14 × 68	W14 × 68	W 14 × 68	W 12 × 58
Minimum weight (ton)	19.81	23.139	22.405	23.105
Maximum top storey drift (cm)	3.14	4.23	4.33	4.43
Maximum inter-storey drift (cm)	1.01	1.31	1.30	1.52

beams without considering concrete slab effects on FEM analyses are very close to the results given in the literature. Also, it is apparently seen from this table, the minimum weight obtained for the frame with composite (steel and concrete) beams is 14.3% lighter than the others.

6. Conclusions

A cooperative study is presented for the optimum design of the steel space frames using GA. The effects of the concrete slab on the behavior of steel beams are also considered. Numerical examples taken from the technical literature are resolved with or without the effect of concrete slab as considering floor beams plain steel and composite. All the results obtained in the study agree very well with the ones obtained previously by other researchers as presented in tabular and graphical formats. According to all results obtained in this paper, GA method is also a very suitable method for optimum design of space frames as the other algorithms such as ACO, TS, HS, SA. Moreover, it can also be said for the case of composite beams that consideration of the contribution of concrete slabs on behavior of beams ended up with less steel weight. Minimum steel weights are reduced by about 20% for first example and %14 for second and third examples here. Also, selected sections of the beams are usually smaller and maximum top and inter storey drifts significantly decrease.

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