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Optimum cost design of frames using genetic algorithms

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Abstract. The optimum cost of a reinforced concrete plane and space frames have been found by using the Genetic Algorithm (GA) method. The design procedure is subjected to many constraints controlling the designed sections (beams and columns) based on the standard specifications of the American Concrete Institute ACI Code 2011. The design variables have contained the dimensions of designed sections, reinforced steel and topology through the section. It is obtained from a predetermined database containing all the single reinforced design sections for beam and columns subjected to axial load, uniaxial or biaxial moments. The designed optimum beam sections by using GAs have been unified through MATLAB to satisfy axial, flexural, shear and torsion requirements based on the designed code. The frames' functional cost has contained the cost of concrete and reinforcement of steel in addition to the cost of the frames' formwork. The results have found that limiting the dimensions of the frame's beams with the frame's columns have increased the optimum cost of the structure by 2%, declining the re-analysis of the optimum designed structures through GA.

Keywords: optimum design; genetic algorithm; space frame; reinforced concrete

1. Introduction

Application of numerical methods has been used in many researches recently in different field of civil engineering (Hamidian *et al*. 2011, 2012, Aghakhani *et al*. 2015, Mohammadhassani *et al*. 2015, Toghroli 2015, Toghroli *et al*. 2014, 2016, 2018, Mansouri *et al*. 2016, Safa *et al*. 2016, Khorami *et al*. 2017a, b, Khorramian *et al*. 2017, Mansouri *et al*. 2017, Heydari and Shariati 2018, Chahnasir *et al*. 2018, Sedghi *et al*. 2018, Shariat *et al*. 2018, Zandi *et al*. 2018).

Genetic Algorithm as one of these methods and by way of a powerful tool is a method for solving both constrained and unconstrained optimization problems based on natural selection to drive biological evolution, also to modify a population of individual solution. GA is also used to find the optimum cost and variables of a space frame subjected to different load cases.

The optimum dimensions of the beams have been unified through the solution procedure of GAs, so that the optimum results would be applicable for the whole span length, thereafter, a sub-optimum procedure has occurred to choose the nearest sub-optimum dimensions to the optimum ones including the lowest cost to the designed section subjected to many design constraints (Shah *et al*. 2016, Andalib *et al*. 2018, Bazzaz *et al*. 2018, Hosseinpour *et al*. 2018, Nasrollahi *et al*. 2018, Paknahad *et al*. 2018, Zandi *et al*. 2018). The difference in the cross-sectional areas between the optimum and the suboptimum solution has been added or subtracted from the reinforcement ratio after transforming it into an equivalent area. Then, the suboptimum solution has been selected from a predetermined database containing all the available cross sections resisting on the applied loads. Also, the effect of materials' price has revealed the efficiency of GAs with a highly constrained problem (Toghroli *et al*. 2014, 2016, Fanaie *et al*. 2015, Shariati *et al*. 2016, Khorramian *et al*. 2017).

GAs has already used in many studies to find an optimum solution for many structural members (concrete of steel), used by Perera and Varona (2009) to find an optimum solution in designing FRP as strength reinforced concrete beams subjected to many design constraints such as (1) moment capacity, (2) maximum plate width and (3) peeling off fiber at shear cracks (by limiting the acting of shear force to the shear resistance for members without shear reinforcement).

Two different databases (flexural and shear) have been

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used to represent FRP plates and sheets with standard sizes and properties, because it has dealt with discrete design variables and the market size supplied by the manufacturers. The constraints have been incorporated in the optimization problem through the inclusion of a penalty in the objective function used to solve two examples such as (1) flexural strengthening essentiality while the shear strengthening is used to avoid concrete cover rip – off, (2) both flexural and shear strengthening have been required in the design.

In 2010, GAs is used for weight minimization of steel trusses by MATLAB (Hultman 2010). Constraints regarding material strength and buckling stability are taken from "Eurocode 3: Design of steel structures" and implemented in the algorithm. A simultaneous optimization has been carried out for size, shape, and topology by providing basic and arbitrary positioned nodes. It is found that the reduced method for topology optimization has offered a relief to the number of possible solutions and should generally be considered in the range of 6-64 nodes.

In 2011, GAs has been applied to achieve the optimum cost design of reinforced concrete beams and restressed concrete beams (Alqedra *et al*. 2011). The results have shown 27.9 % and 16.7 % savings cost for the 4m and 8m Span of RC beam. On the other hand, the saving cost for PC beams are about 29.8 % and 17.8 % for the 10m and 20m. Also, it has proved that the cost of the whole section has been increased by the compressive strength increment.

GAs has also been used by Augusto *et al*. (2012) to find the optimum cost of pre-cast concrete floors. The objective function is to account the cost of materials consumption, labour, manufacture, indirect costs, storage, transport, assembly, taxes and profits. In this study, two new genetic operators have been proposed. The first one as *Transgenic* automatically has modified the number of strands to keep the number of first layer larger than the second layer, because it has resisted on a smaller bending moment. The second one as *Twins* has been implemented to check if the individuals from elitism are the same or not, so that one of the twins is placed to crossover and the next one in the rank is taken to the elitism.

In 2017, GA has been used to perform optimum design for cantilever retaining walls of different heights (Hasanipanah *et al*. 2017, Mahdiyar *et al*. 2017, Armaghani *et al*. 2018). The conventional design method has been used to compare the results showing the efficiency of GA over the conventional design method.

Another GA has been developed by MATLAB for optimum design of reinforced concrete slabs (Sahab *et al*. 2005). Two types of reinforced concrete slabs, simply supported one-way slab and cantilever slab, have been designed. Cost reduction of 18.92% and 6.78% are observed for reinforced cantilever and one-way slab, according to literature.

In the present study, an appropriate objective function has been explained, and then the design constraints and the design procedure curbs have been demonstrated in order to get the optimum design of frames after clarifying the optimized usage of method. Finally, few space frame examples have been solved to check the efficiency of the design procedure with specific views to widen the horizon of related works.

2. Optimum design for space frame

MATLAB by using GA has been developed to find the optimum cost design of continuous beams and columns with all its loading conditions (axially, uniaxial and biaxial loaded), also to find the optimum cost of plan and space frames and conduct the effect of the material- price on the optimum design variables.

The program is to check whether the designed columns are axial, uniaxial or biaxial loaded, introducing a new parameter (e_{all}) that separate each of those solutions based on its value.

2.1 Objective function

Besides the material costs, the cost function of the space frame has included the cost of the formwork. In this study, according to the market, the ratio r representing the cost of steel C_s to the cost of concrete C_c is about 75, and the ratio r_f representing the cost of formwork C_f to the cost of concrete C_c is about 0.4. The cost function for this case is as Eqs. (1) to (4).

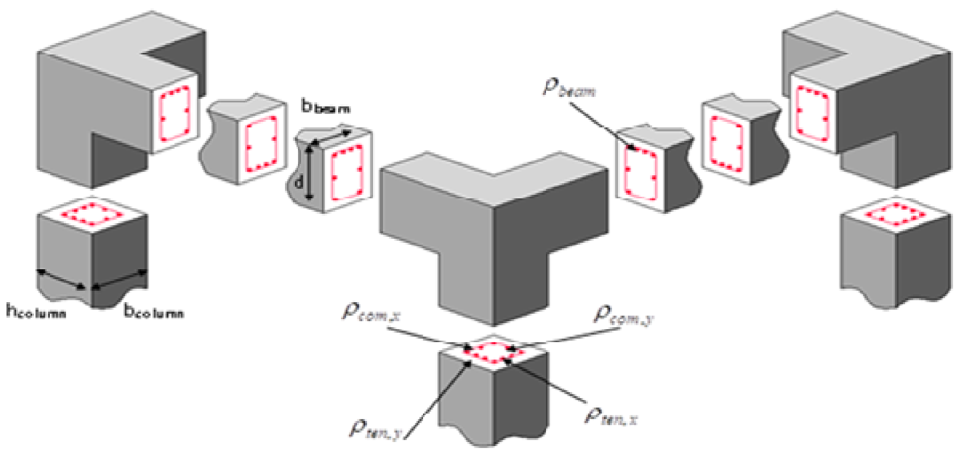

Fig. 1 Space frame design variables

The program has combined the cost function of continuous beams with any cost function of the three types of loaded columns depending on e_{all} value in order to get the cost function of the whole space frame.

The design variables optimized by using this program (Fig. 1) have included the design variables of the continuous beam and the columns for any load case.

2.2 Design constraints

The design constraints for the space frame have contained the design constraints of both continuous beam constraints (beams cross-sectional dimensions and reinforcement ratio at three sections along the beam span length) and column constraints (column cross-sectional dimensions and the reinforcement ratio at each face of the cross-section), subsequently, a constraint has been introduced as the control dimensions of the designed members. This constraint has limited the dimensions of the beams within the dimensions of the column, unless leads to a problem in finding the minimum cost of the beams combined with the minimum cost of columns, which has not necessarily produced a minimum cost of the frame. If the mentioned procedure is adopted to find the minimum cost design of a space frame, some problems would be occurred to make the designed frame with inapplicable dimensions.

Solving this problem has adopted a trading process between the optimum designed beam and the optimum designed column till the finding of a new optimum designed dimension for both beam and column. The new dimensions have provided a minimum cost for the whole structure and mightily altered from a minimum combining cost of the optimum beam cost and the optimum column cost, like to the pareto optimal principle of increasing one cost against decreasing the other one to find a compromised solution for the both (Fig. 2).

- Continuous beams

$$C_t = C_c \times b_{beam} \times \{ (d + t) + r \times \rho_{beam} \times d \} + C_f \times \{ (2 \times (d + d')) + b_{beam} \} \quad (1)$$

- Axially loaded columns

$$C_t = C_c \times b_{column} \times h \times \{ 1 + (r \times \rho_{column}) \} + C_f \times \{ 2 \times (b_{column} + h) \} \quad (2)$$

- Uniaxially loaded columns

$$C_t = C_c \times b_{column} \times h \times \{ 1 + (r \times (\rho_{ten} + \rho_{com})) \} + C_f \times \{ 2 \times (b_{column} + h) \} \quad (3)$$

- Biaxially loaded columns

$$C_t = C_c \times b_{column} \times h \times \{1 + (r \times (\rho_{ten,x} + \rho_{com,x} + \rho_{ten,y} + \rho_{com,y}))\} + C_f \times \{2 \times (b_{column} + h)\} \quad (4)$$

The structure, supposedly designed optimally, has been analyzed linearly at the beginning with the assumed dimensions and applied loads through STAAD Pro. 2006, in order to get the data required for the optimum design such as the moments, shear and torsion related to the beam, and axial loads with moments in different directions like columns. The eccentricity of the loads on columns e is calculated in two directions (X and Y), then the value e is compared to the parameter e_{all} separating the solutions of the differently loaded columns. The value of this parameter is (0.1 × h), if the value of e is less than e_{all} in any direction, neglecting the effect of the moment in that direction and designing uniaxially of the column.

If the value e in the other direction is less than e_{all}, then the moment's effect in that direction has been discarded, therefore, the column has been designed axially, unless the column has been designed biaxially.

When the optimum frame cost is found, the optimum dimensions of the frame members are used to analyze the structure to ensure that the designed sections of the beams and columns are capable in resisting on the applied loads within the code limits fulfilling the purpose.

2.2.1 Design constraints for beams

A reinforced concrete beam should have a structural capacity greater than the factored applied loading based on ACI Code facing some restrictions on (1) the beam's cross-

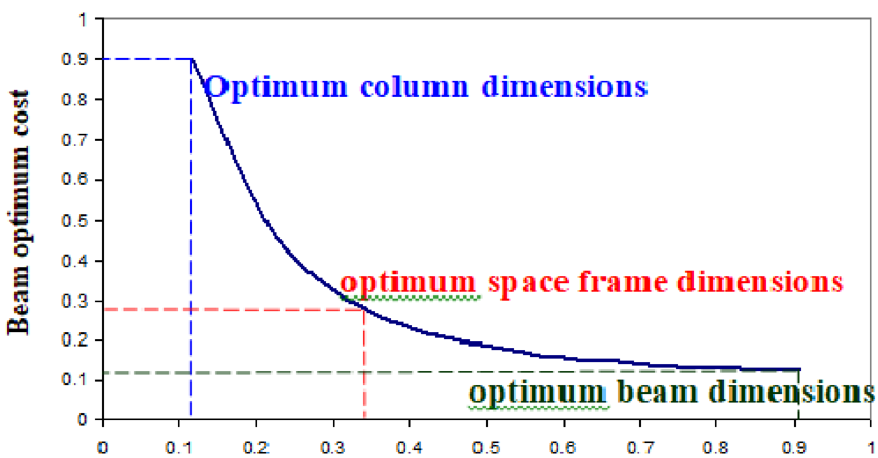

Fig. 2 Compromising the solution of the space frame

sectional geometry, (2) the position and (3) quantity of steel reinforcement for all kinds of loading.

So far, dimensions have been highly used as design variables, then the reinforcement ratio has been calculated depending on these variables and topology optimized. However, this study has used the reinforcement ratio as a design variable beside the dimensions (offering a minimum cost) and also included the effect of shear and torsion on these optimum dimensions beside other constraints (Mohammadhassani *et al*. 2015, Mansouri *et al*. 2016, Safa *et al*. 2016, Toghroli *et al*. 2016, Mansouri *et al*. 2017, Chahnasir *et al*. 2018). These constraints have been used in order to specify the main variables to have them resist on the applied loads (in many ways), and also stay within the limits of the used code in order to make the optimal solution more realistic and applicable.

The first constraint Eq. (5) has been used to make the three variables ρ, b and d (reinforcement ratio, beam width and beam effective depth) of the section carrying the smallest values resisting on the applied moment of the section.

Eqs. (6) and (7) have represented the constraints prevented the reinforcement ratio to exceed the maximum value or below the minimum value which is defined based on ACI Code.

Considering the effects of cracking and reinforcement on member, Eq. (8) is used to guarantee the optimum section has no depth less than the one (depth) controlling the elastic deflection, ACI code (9.5.2.2), Building Code Requirements 2011, and stiffness (Adeli and Sarma 2006).

In order to make the dimensions more realistic, Eqs. (9) and (10) are used to keep the ratio of the optimum depth to the optimum width as 1.5 and 2.5 (specified by the designer).

Dimensions of the optimum width in the range of 200 mm and 500 mm and the optimum depth in the range of 300 mm and 1250 mm have been used through the Eqs. (11) and (12), (specified by designer).

To reduce unsightly cracking and prevent the crushing of surface concrete due to the inclined compressive stresses caused by shear and torsion, Eq. (13) has been used to limit the optimum dimensions within this condition. No more specifications could be achieved for the case of limiting the reinforcing steel for shear and torsion, because it has depended on the section dimensions before found optimally. Also, if the steel area has been used as a constraint, the solution direction would reinforce the section with minimum reinforcement or without reinforcement. Therefore, this solution wouldn't be a general optimum but an optimum design for a special case approved before starting the solution.

Accordingly, in terms of shear and torsion, the right decision for enormously optimizing of the section is the limiting of the cross section dimensions through the code specifications and leaving the reinforced area of steel to be defined and optimized through the bar selection procedure by the designer.

Finally, Eqs. (14) and (15) have been used for the reinforcement topology through the section, considering the minimum spacing between the chosen bars (Adeli and Sarma 2006).

$$\frac{k\times w\times L^2}{0.9(\rho\times b\times d\times f_y \times(d-\frac{(\rho\times b\times d\times f_y/0.85\times f_c'\times b)}{2}))}-1\le 0 \tag{5}$$

$$1-\frac{\rho}{\rho_{\min}}\le 0 \tag{6}$$

$$\frac{\rho}{\rho_{\max}}-1\le 0 \tag{7}$$

$$1-\frac{h}{h_{\min}}\le 0 \tag{8}$$

$$1.5-\frac{h}{b}\le 0 \tag{9}$$

$$\frac{h}{b}-2.5\le 0 \tag{10}$$

$$\left(1-\frac{b}{200mm}\le 0\right) \text{ and } \left(\frac{b}{500mm}-1\le 0\right) \tag{11}$$

$$\left(\frac{h}{1250mm}-1\le 0\right) \text{ and } \left(1-\frac{h}{300mm}\le 0\right) \tag{12}$$

$$\frac{\sqrt{\left(\frac{V_u}{bd}\right)^2+\left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2}}{\phi\left(\frac{V_c}{bd}+0.66\sqrt{f_c'}\right)}-1\le 0 \tag{13}$$

$$\left(1-\frac{Bars\_Spacing}{Bars\_Diameter}\le 0\right) \text{ or } \left(1-\frac{Bars\_Spacing}{25mm}\le 0\right) \tag{14}$$

$$\left(1-\frac{Layers\_Spacing}{25mm}\le 0\right) \tag{15}$$

$$\frac{b}{column\_width}-1\le 0 \tag{16}$$

2.2.2 Design constraints for columns

In this study, following the core idea of the load contour method is the transforming of biaxial problem into an equivalent uniaxial one through Eq. (26) introduced as a new design constraint, so the problem has been solved uniaxially with M_{nx} considering (e_x = 0) and uniaxially with M_{ny} considering (e_y = 0). After that, the new constraint has transformed the effect of the solved procedure into a biaxial bending problem for both M_{nx} and M_{ny} (Wight, James K. and MacGregor, James G,).

The first two constraints have been used to confined the applied force with the balanced force of the section, also the applied moment has been limited to the balanced moment of the section means (*e* is less or equal to balanced moment).

Also, the plastic centroid in these equations (x^- and y^-) is found in two directions (X and Y) without any interaction of the bars' positioning (explained earlier). Regarding the slender column constraint, two directions have been considered by replacing the height with the width in other direction.

The reinforcement ratio constraint has four parameters (two reinforcement ratios for each direction – for tension and compression face) as shown in Eqs. (31) and (32).

$$P - P_{bal} = 0 \tag{17}$$

$$\left(\frac{e}{e_{bal}} - 1\right) \le 0 \tag{18}$$

$$\begin{aligned} P_{bal} &= 0.85 \times f_c' \times a \times b + f_{s,com}' \times \rho_{com} \\ &\times b \times h - f_{s,ten} \times \rho_{ten} \times b \times h \end{aligned} \tag{19}$$

$$\begin{aligned} M_{bal} &= 0.85 \times f_c' \times a \times b \times \left(y^- - \frac{a}{2}\right) + f_{s,com}' \times \rho_{com} \\ &\times b \times h \times \left(y^- - d^-\right) + f_{s,ten} \times \rho_{ten} \times b \times h \times \left(\left(h - d^-\right) - y^-\right) \end{aligned} \tag{20}$$

$$a = \beta 1 \times c_{bal} = \beta 1 \times \left(\frac{0.003 \times E_s}{(0.003 \times E_s) + f_y} \times \left(h - d^-\right)\right) \tag{21}$$

$$y^- = \frac{\begin{array}{c} 0.85 \times f_c' \times b \times (h^2/2) + \rho_{com} \times b \times h \times \\ f_{s,com}' \times d^- + \rho_{ten} \times b \times h \times f_{s,ten} \times (h - d^-) \end{array}}{\begin{array}{c} 0.85 \times f_c' \times b \times h + \rho_{com} \times b \times h \times \\ f_{s,com}' + \rho_{ten} \times b \times h \times f_{s,ten} \end{array}} \tag{22}$$

$$e_{bal} = M_{bal} / P_{bal} \tag{23}$$

$$e = M_u / P_u \tag{24}$$

$$f_{s,com} = f_y - 0.85 f_c' \tag{25}$$

$$f_{s,ten}^- = f_y$$

Where

E_s = 200000 MPa (26)

P_u : Nominal strength

M_u : Nominal bending strength

$$\begin{aligned} M_{ox,bal} &= 0.85 \times f_c' \times a \times b \times \left(x^- - \frac{a}{2}\right) \\ &+ f_{s,com}' \times \rho_{com} \times b \times h \times \left(x^- - d^-\right) \\ &+ f_{s,ten} \times \rho_{ten} \times b \times h \times \left(\left(h - d^-\right) - x^-\right) \end{aligned} \tag{27}$$

$$\begin{aligned} M_{oy,bal} &= 0.85 \times f_c' \times a \times h \times \left(y^- - \frac{a}{2}\right) \\ &+ f_{s,com}' \times \rho_{com} \times b \times h \times \left(y^- - d^-\right) \\ &+ f_{s,ten} \times \rho_{ten} \times b \times h \times \left(\left(b - d^-\right) - y^-\right) \end{aligned} \tag{28}$$

$$\left(\frac{M_{nx}}{M_{ox,bal}}\right)^\alpha + \left(\frac{M_{ny}}{M_{oy,bal}}\right)^\alpha - 1.0 \le 0$$

$$\left[\left\{\frac{\left((k_b \times l_u)/(\sqrt{(b \times h^3/12)/(b \times h)})\right)}{22}\right\} - 1\right] \le 0 \tag{29}$$

$$\left[\left\{\frac{\left((k_b \times l_u)/(\sqrt{(h \times b^3/12)/(b \times h)})\right)}{22}\right\} - 1\right] \le 0 \tag{30}$$

$$\left(\frac{(\rho_{ten,x} + \rho_{com,x} + \rho_{ten,y} + \rho_{com,y})}{0.08}\right) - 1 \le 0 \tag{31}$$

$$1 - \left(\frac{(\rho_{ten,x} + \rho_{com,x} + \rho_{ten,y} + \rho_{com,y})}{0.01}\right) \le 0 \tag{32}$$

In the last two equations, the reinforcement ratio in one direction has been excluded if the column would be designed uniaxially or represented the whole ratio of the section by one value if the column would be designed axially. Considering the cross section dimensions, minimum and maximum dimensions are specified as the following four equations for both width and height without limiting them by any ratio between them.

$$1 - \frac{b}{0.3 or 0.25} \le 0 \tag{33}$$

$$1 - \frac{h}{0.3 or 0.25} \le 0 \tag{34}$$

$$\frac{b}{1.0} - 1 \le 0 \tag{35}$$

$$\frac{h}{1.0} - 1 \le 0 \tag{36}$$

$$1 - \frac{b}{beam\_width} \le 0 \tag{37}$$

After finding the optimum design variables, the same steps in finding the suboptimal solution used for the beam section have been applied for the suboptimal column section.

2.2.3 Design of plane frame

This example has checked the validity of the written program concerning the multi-objective optimization. Due to the limitations between the adjacent members (explained earlier), combining the optimum beam and column together have not always provided the optimum design of a plane frame.A single bay one story plane frame with 3.5 m height

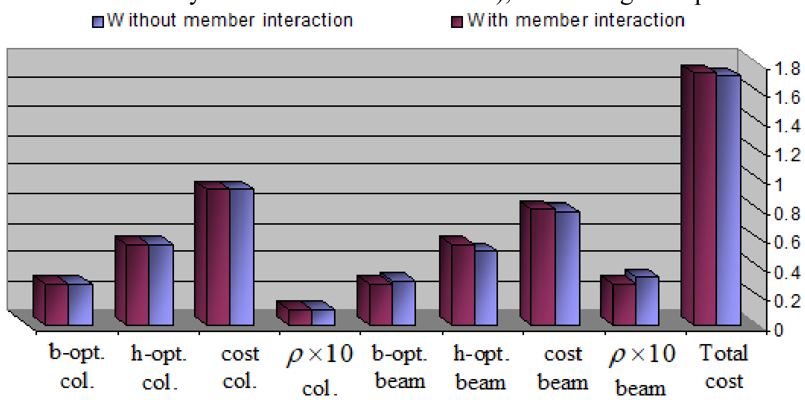

Fig. 3 The difference of design variables of plane frame with / without member interaction

Table 1 The optimum design results of the frame for the two cases (with / without members' interaction)

Member	Variables	Without member interaction	With member interaction
Column	b – optimum	275.6 mm	275.6 mm
	h – optimum	551.1 mm	551.1 mm
	ρ tension - optimum	0.0025	0.0014
	ρ compression – optimum	0.0075	0.0086
	Cost value	$0.9271C_c$	$0.9271C_c$
	Iteration number	6	2
	b – suboptimum	275 mm	275 mm
	h – suboptimum	550 mm	550 mm
	ρ tension – rounded	0.0027	0.0015
	ρ compression – rounded	0.0078	0.009
	A_s tension	452 mm^2	226 mm^2
	Tension bar no.	4 φ 12	2 φ 12
	A_s compression	1232 mm^2	1473 mm^2
	Compression bar no.	2 φ 28	3 φ 25
Beam	b – optimum rounded	300 mm	275 mm
	h – optimum rounded	500 mm	550 mm
	ρ1 – rounded	0.005	0.0044
	ρ2 – rounded	0.0105	0.0092
	ρ3 – rounded	0.0167	0.0144
	Cost value	$0.7759C_c$	$0.7949C_c$
	A_{s1} optimum rounded	741 mm^2	603 mm^2
	Bar no. - 1	2 φ 20 + 1 φ 12	3 φ 16
	A_{s2} optimum rounded	1388 mm^2	1232 mm^2
	Bar no. – 2	2 φ 22 + 2 φ 20	2 φ 28
	A_{s3} optimum rounded	2214 mm^2	2099 mm^2
	Bar no. – 3	2 φ 28 + 2 φ 25	2 φ 32 + 1 φ 25
	Stirrups – zone 1	2700(0–1.2379 m)	2400(0-1.1915m)
	Stirrups – zone 2	262.25(1.2379-1.4)	240(1.1915-1.35m)
	Stirrups – zone 3	0.0 (1.4 - 2.0)	0.0 (1.35 m - 2 m)
	Total cost value	1.7030	1.7219

and 4.0 m span length has been designed optimally with r = 75, r_f = 0.4, when the properties of material are f'_c = 30 MPa and f_y = 400 MPa. The beam is under three critical moments as M1 = 100 kN.m, M2 = 200 kN.m and M3 = 300 kN.m. Also, a maximum shear V_u is about 500 kN without torsion, when the column is under P_n = 2000 kN and M_n = 300 kN.m. The formwork has attained a significant role in finding the optimum frame design cost, and when the long column constraint is used with k_b = 1. The frame is designed twice, accordingly, at first time, a separate solution has been adopted without any interaction between the members, while at the second time, and this interaction has been used as a new constraint in the design procedure. Comparing the results has shown the effect of optimally designing of the frame as one unit is not separated. The differences between two cases in optimally designing the frame have been depicted in Table 1. Considering the limitation of the designed beams' width occurred by the designed column's width, the cost of the frame has been increased about (2%) compared to the separate optimum solution, subsequently, this ratio has certainly been raised on more limitations such as the presence of torsion.

The major differences of two cases in terms of cost is that the column optimum design has not been affected by the member limitations, whilst the difference in the total cost is related to the beam optimum design, because of the shear effect on the cross-sectional dimensions (Fig. 3). Therefore, in case of any torsion applied to the section, the difference would be greater than this, mightily due to the more effects of torsion on the cross-sectional dimensions of the beam.

3. Space Frame

3.1 One story two bays

One story two bays frame designed by Gas has been represented in Fig. 4. The frame has been loaded with a uniformly distributed floor load of 12 kN / m^2 in addition to its self-weight with a uniform line load of 20 kN / m on the beams named B1, B2, B4 and B6. Also, a concentrated load of 200 kN has been applied to the same beams, adding that the material properties were f'_c = 28 MPa and f_y = 400 MPa. At first, the frame is analyzed linearly with STAAD Pro. 2006. In the following, to get the moments, shear and torsion have been applied to each member section needed in the optimum design with GAs. All the column sections are assumed to be 400 × 400 mm with a height of 3.5 m, while the beam sections are 300 × 600 mm at the beginning of the analysis.

The initial dimensions used in the frame analysis have been replaced by the dimensions of GAs led to the whole frame's re-analyzing through the new optimum dimensions with the same applied load to check the capacity of the optimum designed section of the members against the applied loads, respectively, the process has been repeated till the no change in the design variables of optimum section.

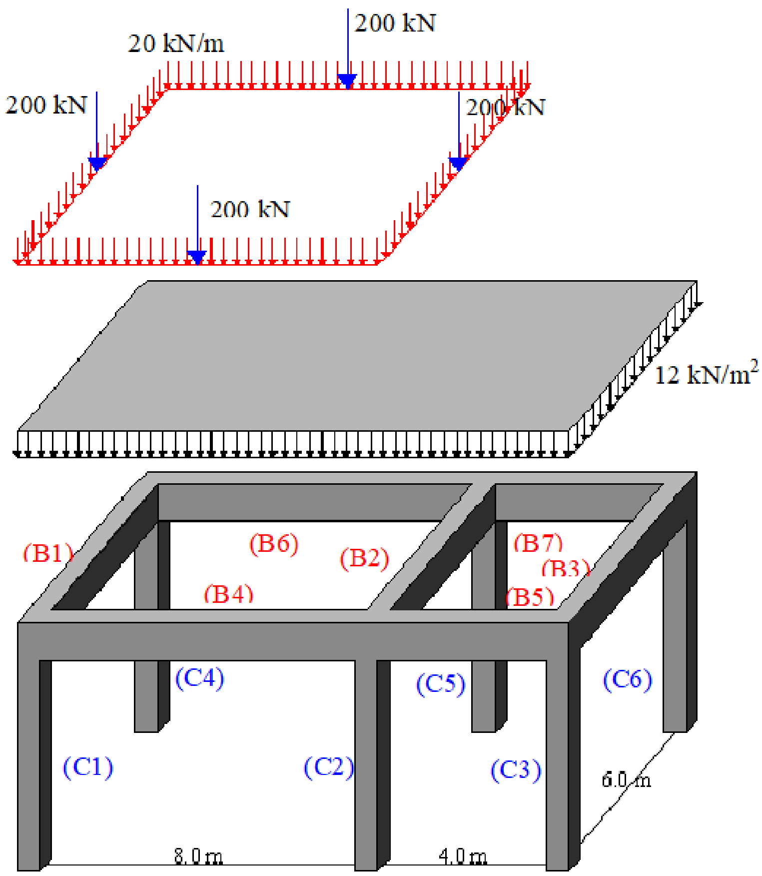

Fig. 4 One story two bays of space frame example

The constraints used in this example have contained the long column constraint with kb = 0.6 and the plastic centroid adjustment.

The cost values of the members in the frame are shown in Fig. 5, accordingly, half of the space frame has been designed by GAs to simplify the solution. Following the figure shows that the members with the lower cost have shown more stability in the final optimum designed section within an earlier iteration of the analysis steps. While the sections with the higher cost are wobbled through the numbers of analysis iterations, they still stay within narrow limits.

By focusing on the design variables controlling the cost value of column C1 have been represented in Figs. 5-6. Indeed, the design variables causing this variation are the reinforcement ratios in x-direction due to the large moment in this direction. So, the resistance of the column to the applied loads is distributed between these columns through the analysis iterations by providing the best applicable optimum design to the space frame and handling the applied

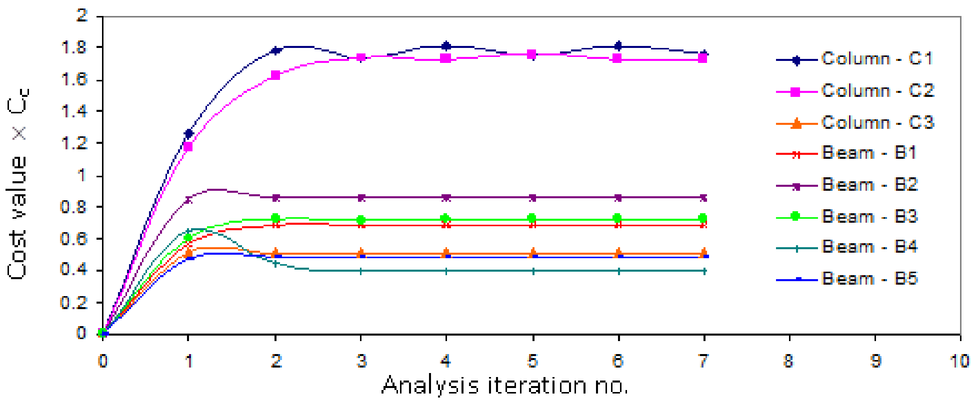

Fig. 5 Variation of the optimum sections cost value with the analysis iteration number

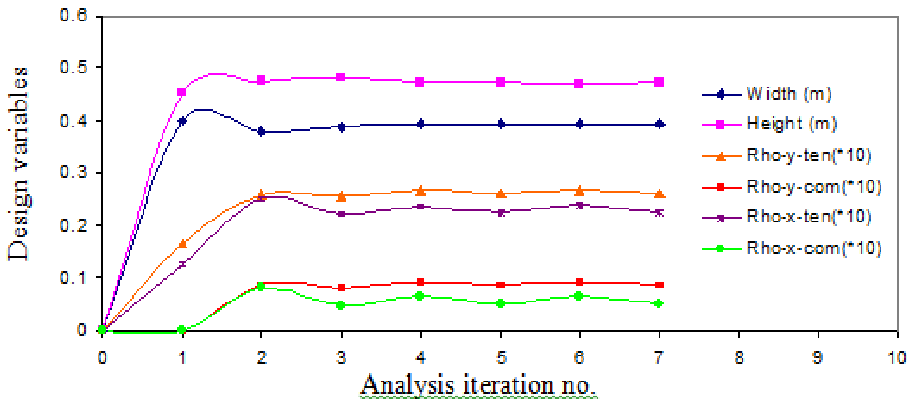

Fig. 6 The variation of design variables of column C1 with the analysis iteration number

Table 2 Suboptimum results for beams for analysis iteration - 7 (1 story–2 bays)

Design variables	Beam B1 Suboptimum results	Beam B2 Suboptimum results	Beam B3 Suboptimum results	Beams B4 and B6 Suboptimum results	Beams B5 and B7 Suboptimum results
Width (mm)	225	225	225	300	200
Height (mm)	425	450	350	475	300
Steel reinf. - 1	2 φ 35	'2 φ 35"2 φ 12'	3 φ 12	2 φ 32+1 φ 25	2 φ 22
Steel reinf. - 2	2 φ 35	'2 φ 35"2 φ 12'	3 φ 16	3 φ 35	2 φ 12
Steel reinf. - 3	2 φ 35	'2 φ 35"2 φ 12'	3 φ 12	2 φ 35+2 φ 32	2 φ 12
Torsion level-3	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10
Torsion level-2	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10
Stirrups(0-1.9)	1200	1500	197	1300	175
Stirrups(1.9-2.3)	197	197	197	263	175
Stirrups(2.3-mid)	0.0	0.0	0.0	0.0	0.0

Table 3 Suboptimum results for columns for analysis iteration - 7 (1 story–2 bays)

Design variables	Columns C1 and C4 Suboptimum results	Columns C2 and C5 Suboptimum results	Columns C3 and C6 Suboptimum results
Width (mm)	400	450	300
Height (mm)	450	500	300
Tens.rein. (y)	'4 φ 35' '3 φ 25'	5 φ 35	$\rho = 0.0025$
Comp.rein. (y)	2 φ 25 + 2 φ 22	3 φ 16	$\rho = 0.0025$
Tens.rein. (x)	'4 φ 35' '2 φ 20 + 2 φ 12'	3 φ 35 + 2 φ 32	$\rho = 0.0025$
Comp.rein. (x)	2 φ 22 + 1 φ 20	3 φ 12	$\rho = 0.0025$

loads together.

The cost value and constraints history of column C1 are shown in Figs. 7 and 8. Accordingly, the optimum solution has been found after the 9th iteration with almost zero constraints violation. Regarding other members of the frame, there would be few constant optimum design variables through analysis iterations Table 2-3.

3.2 Three stories two bays

The three stories space frame (Fig. 9) have been loaded with a floor load of 12 kN / m^2 on all the stories, also all the beams have been loaded with a line load of 15 kN / m except the ones of the roof, loaded with a line load of 8 kN / m, besides the frame's self-weight. The concrete compressive strength is f'_c = 28 MPa, and the yield stress is f_y = 400 MPa. At first, all the column sections are assumed with dimensions of 400 × 400 mm with a height of 4 m, and all the beam sections are assumed with dimensions of 300 × 500 mm, changed according to the optimum design results by GAs, in the following, a reanalysis with the new optimum designed section is carried out until the optimum sections are converged.

The cost of the frame has included the cost of the formwork with the cost of the materials. Also, another design constraint has been introduced according to ACI – Code (10.3.6): Design axial strength φP_n of compression members would not be taken greater than $\varphi P_{n,\max}$, computed by the following equation for non - prestressed members with tie reinforcement.

$$\varphi P_{n,\max} = 0.8\varphi \,\{0.85\, f_c^{'}\,(A_g - A_s) + f_y A_s\} \tag{38}$$

The frame has been reanalyzed three times, and the final results of the optimum design are shown in Tables 4 and 5, the history of columns C1 and C4 through generations is shown in Fig. 10.

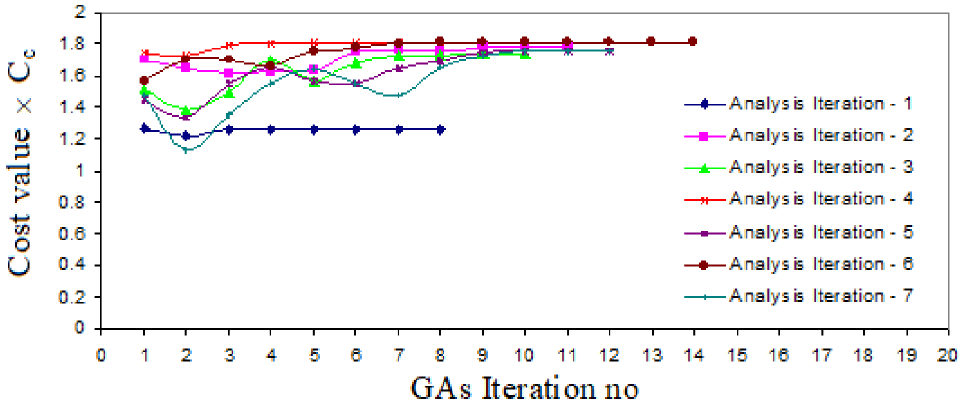

Fig. 7 Cost function scaling through iterations for column C1 for all analysis iterations

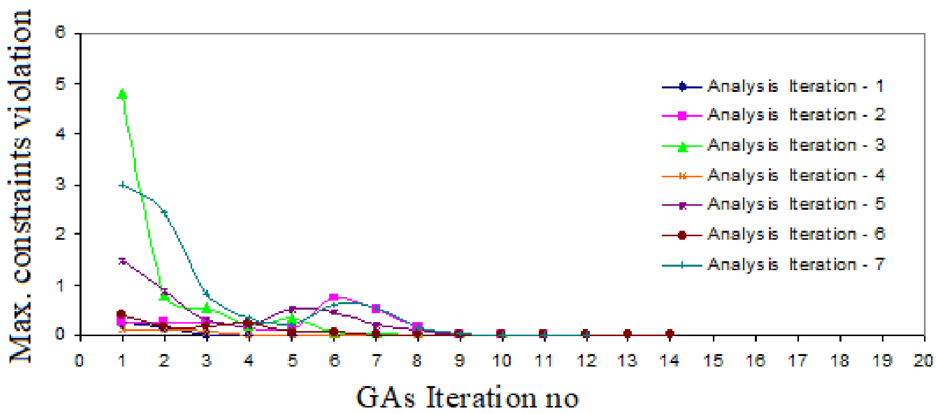

Fig. 8 Maximum constraints violation through iterations for column C1 for all

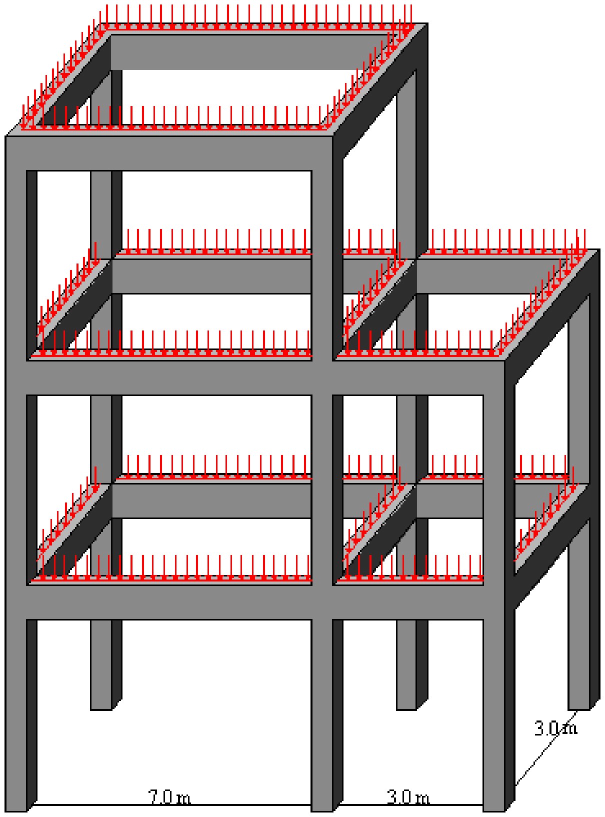

Fig. 9 Multi-story two bays of space frame example

4. Conclusions

The results of the study have revealed that there is no need to any reanalysis (linear or non-linear) to check the capacity of the designed section, as long as the design constraints of the applied algorithm are sufficient and capable to attain reliable results. Meanwhile, there would be no violations through the design procedure, unless using a penalty function to bring the solution to the closest optimum would be recommended. By increasing the applied torsion on beams, the optimum reinforcement ratio has not been decreased with some increment of steel price, also the optimum dimensions wouldn't be increased by the steel price increment. This is because when a design variable has reached its limits, the other design variable has handled the applied torsion at that level, even if it would be more expensive to use the first design variable to resist the applied torsion. More design charts have been found for different moments' values in order to cover as much as possible for the optimum design charts of beam sections, columns, different loading conditions and material properties. By limiting the width of the designed beams with the width of the designed column, the cost of the optimum frame has been increased about (2%) compared to the separate optimum solution.

As any further study suggestion: more design constraints should be considered as more complex engineering problems (non-linear relations) to achieve more accurate results. Also, other artificial intelligence methods

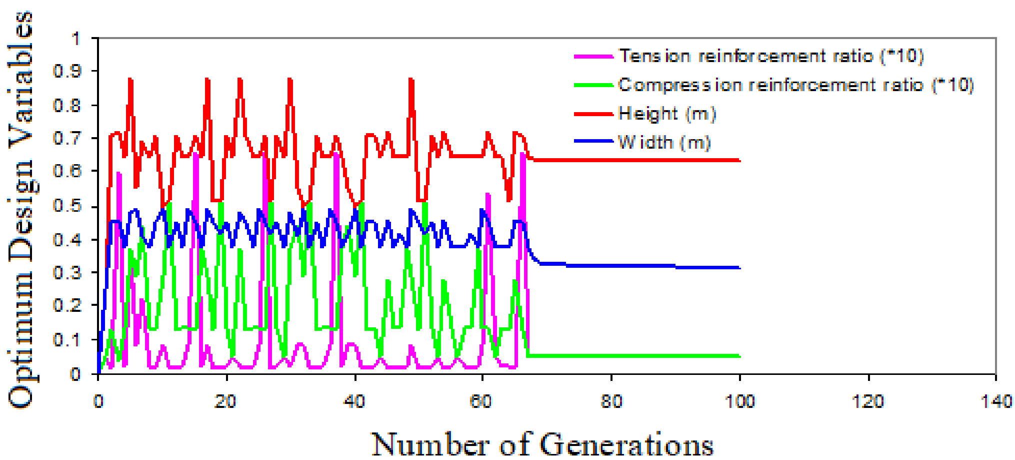

Fig. 10 Optimum design variables history through generations

Table 4 Suboptimum results for columns of analysis iteration - 3 (3 story – 2 bays)

Design variables	Column C1 and C4	Column C2 and C5	Column C3 and C6	Column C7 and C10	Column C8 and C11	Column C9 and C12	Column C13 and C15	Column C14 and C16
Width (mm)	300	325	250	300	300	250	250	250
Height (mm)	650	625	250	650	650	250	250	250
Tens. rein. (y)	2 φ 25 + 1 φ 20	2 φ 35		2 φ 32 + 1 φ 25	2 φ 25 + 1 φ 22	0.0025	0.0025	0.0025
Comp.rein. (y)	3 φ 22	2 φ 20 + 2 φ 12	0.01	4 φ 12	2 φ 22 + 1 φ 20	0.0025	0.0025	0.0025
Tens.rein.(x)						0.0025	0.0025	0.0025
Comp.rein.(x)						0.0025	0.0025	0.0025

Table 5 Suboptimum results for beams of analysis iteration - 3 (3 story–2 bays)

Design variables	Beam B1	Beam B2	Beam B3	Beam B4 and B6	Beam B5 and B7	Beam B8
Width (mm)	200	200	200	275	200	200
Height (mm)	300	300	300	425	300	300
Steel reinf. - 1	2 φ 12	2 φ 12	2 φ 12	4 φ 16	2 φ 20	2 φ 12
Steel reinf. - 2	2 φ 12	2 φ 12	2 φ 16	2 φ 16 + 2 φ 12	2 φ 12	2 φ 12
Steel reinf. - 3	2 φ 12	2 φ 12	2 φ 12	2 φ 20 + 2 φ 12	2 φ 12	2 φ 12
Torsion level-3	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10
Torsion level-2	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10
Stirrups(0-1.9)	175	175	175	250	175	175
Stirrups(1.9-2.3)	0.0	0.0	0.0	240	175	0.0
Stirrups(2.3-mid)				0.0	0.0	
Design variables	**Beam B9**	**Beam B10**	**Beam B11 and B13**	**Beam B12 and B14**	**Beam B15 and B16**	**Beam B17 and B18**
Width (mm)	200	200	275	200	200	275
Height (mm)	300	300	425	300	300	425
Steel reinf. - 1	2 φ 12	2 φ 12	4 φ 16	2 φ 12	2 φ 12	2 φ 16
Steel reinf. - 2	2 φ 16	2 φ 12	2 φ 20 + 1 φ 12	2 φ 12	2 φ 12	3 φ 20
Steel reinf. - 3	2 φ 12	2 φ 12	2 φ 20 + 1 φ 16	2 φ 12	2 φ 12	2 φ 16
Torsion level-3	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10
Torsion level-2	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10	2 φ 10
Stirrups(0-1.9)	175	175	251	175	175	240
Stirrups(1.9-2.3)	175	0.0	240	0.0	0.0	240
Stirrups(2.3-mid)	0.0		0.0			0.0

could be used like a neural network or pattern search, etc.

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