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Cost effective design of RC building frame employing unified particle swarm optimization

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Abstract. Present paper deals with the cost effective design of reinforced concrete building frame employing unified particle swarm optimization (UPSO). A building frame with G+8 stories have been adopted to demonstrate the effectiveness of the present algorithm. Effect of seismic loads and wind load have been considered as per Indian Standard (IS) 1893 (Part-I) and IS 875 (Part-III) respectively. Analysis of the frame has been carried out in STAAD Pro software. The design loads for all the beams and columns obtained from STAAD Pro have been given as input of the optimization algorithm. Next, cost optimization of all beams and columns have been carried out in MATLAB environment using UPSO, considering the safety and serviceability criteria mentioned in IS 456. Cost of formwork, concrete and reinforcement have been considered to calculate the total cost. Reinforcement of beams and columns has been calculated with consideration for curtailment and feasibility of laying the reinforcement bars during actual construction. The numerical analysis ensures the accuracy of the developed algorithm in providing the cost optimized design of RC building frame considering safety, serviceability and constructional feasibilities. Further, Monte Carlo simulations performed on the numerical results, proved the consistency and robustness of the developed algorithm. Thus, the present algorithm is capable of giving a cost effective design of RC building frame, which can be adopted directly in construction site without making any changes.

Keywords: cost effective design; RC building frame; seismic load; STAAD Pro; unified particle swarm optimization; wind load

1. Introduction

Reinforced concrete is a dominant material for constructing various civil engineering structures due to its high compressive strength, durability and resistance to damage from fire and water. Conventional trial and error based design of RC structures is based on only safety criteria, and requires excessive materials. Thus construction costs of building increases. So, increase in use of reinforce concrete comes with the demand for economical design. Thus, from last few decades, researchers have proposed different method for cost optimization of RC frame. Increase in computational power of computers has enhanced the willfulness of such attempts drastically. Main challenge in optimizing the RC structures are number of optimizing variables in comparison with the optimal design of steel structures, where only material is considered for entire structures and

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cost of the entire structure is proportional to weight of that material. On the other hand, cost of RC structures consist of cost of concrete, cost of steel reinforcement and cost of formworks. Their unit costs differ from each other. Their inter-relations are not simple, because any two of them change significantly with slight changes in the quantity of the third factor. This in turn changes the total cost of RC frame to a great extent. Thus, the actual problem comes to finding appropriate combination of values for three aforementioned components of costs without violating the safety requirements so that the total cost becomes minimum. Again, RC sections are cast in situ. So, while designing cost effective sections, size of the sections and reinforcement detailing should be provided to meet the specific demands from architectural and construction point of view. Thus, presence of all these factors along with constraints regarding strength, serviceability, architectural demands and easiness of construction make cost optimization of real life RC structures a highly cumbersome task. However, in the same time an algorithm to optimize the cost of a RC building frame satisfying all the necessary criteria will be a valuable tool to the design practitioner.

Despite all these challenges many researchers have tried to find cost optimize designs of RC building components. Milajić *et al*. (2013) reviewed various methods available in literatures for optimal design of reinforced concrete structure. They focused on the problems of these existing methodologies, such as: gap between the theory and practice in the field, lack of universal criteria and standard benchmark problem etc. Also, when ones talks about of cost optimized design of a complete RC building project, it includes optimizing the topology of the building, material costs of the building components, reinforcement distribution, cost of shuttering, labor cost maintaining all the safety guidelines of competent authority.

Topology optimization of building means finding out optimum layout of the building for a particular project. Topology optimization is a very popular and common term in steel structures, where topology of truss is optimized to achieve cost optimum design (Hasencebi *et al*. 2013). However, researchers also tried to implemented the concept of topology optimization also in case of concrete structures by various means, such as windowed evolutionary structural optimization (WESO) (Wang *et al*. 2020), plastic design layout optimization technique (lu *et al*. 2019), dynamic programming utilizing genetic algorithm (GA) based multi objective optimization (Lee 2019). Apart from that, Zegard *et al*. (2020) have implemented three different methods of topology design optimization in building engineering. However, in real project topology is not decided solely based on cost only. There are other deciding factor such as utility of the building, aesthetic beauty or client's wish. Labor cost depend on the location of the construction and days required to finish the construction. Thus, present study is focused on the optimizing the design of the structural components of a building frame.

Structural components, such as, beams, columns, slab and foundation contribute most in the total cost of a project. Thus, cost optimum design of structural components are of utmost important for minimizing the project cost. Cost optimum design of RC beams have been attempted by researchers utilizing various methods, simplex and Lagrangian optimization method (Praksh *et al*. 1988), geometric programming (Chakrabarty 1988), GA based algorithm (Coello *et al*. 1997, Rajeev and Krishnamoorthy 1998), polynomial optimization technique (Dole *et al*. 2000), simulated annealing (De Medeiros and Kripka 2013), random search technique (RST) (Nigdeli and Bekdaş 2017) and charged system search (CSS) (Uz *et al*. 2018). Cost optimum design of T-beams (Ferreira 2003) and Cost optimum design of RC columns (Prakash *et al*. 1988, Preethi and Arulraj 2016, Bekdaş and Niğdeli 2016) also have been attempted. Apart from beam and column optimization, design of reinforced concrete (RC) flat slabs have also been optimized by Aldwaik and Adeli (Aldwaik and Adeli 2016) using Neural dynamic model for Adeli and Park (NDAP)

Table 1 Review of all the optimization techniques used by the researchers for solving design optimization problem of RC structures

Authors	Optimization algorithm
Prakash *et al*. 1988	Simplex and Lagrangian optimization method
Chakrabarty 1992	Geometric programming
Coello *et al*. 1997, Rajeev and Krishnamoorthy 1998, Chaudhuri and Maity 2020	Genetic algorithm (GA)
Dole *et al*. 2000	polynomial optimization technique
De Medeiros *et al*. 2013	Simulated annealing (SA)
Nigdeli and Bekdaş 2017	random search technique (RST)
Uz *et al*. [14], Kaveh and Behnam 2013	charged system search (CSS)
Preethi and Arulraj 2016	sequential quadratic programming (SQP)
Bekdaş and Niğdeli 2016	Teaching-learning-based-optimization (TLBO)
Bekdaş and Nigdel 2014	Harmony search (HS)
Aga and Adam 2015	Artificial neural network (ANN)
Gharehbaghi and Khatibinia 2015	intelligent regression model (IRM) combined with Particle swarm optimization (PSO)
Esfandiary *et al*. 2016, 2018	decision-making Particle Swarm Optimization (DMPSO)
Kulkarni and Bhusare 2017	Response Surface Method (RSM)
Tapao and Cheerarot 2017	Artificial bee colony (ABC)
RazmaraShooli *et al*. 2019	GA-PSO algorithm
Chaudhuri and Maity 2020	Unified particle swarm optimization (UPSO)

(Aldwaik and Adeli 2014), Design optimization of RC foundation has been performed by Chaudhuri and Maity (Chaudhuri and Maity 2020). Besides these individual structural components researchers have also tried to optimize the beam-column frame in terms of weight (Kaveh and Behnam 2013), in terms of cost (Bekdaş and Nigdel 2014, Aga and Adam 2015, Gharehbaghi and Khatibinia 2015, Esfandiary *et al*. 2016, Kulkarni and Bhusare 2017, Bekas and Stavroulakis 2017, Tapao and Cheerarot 2017, Esfandiari *et al*.). All these studies regarding design optimization of RC building components or frame are conformed to the guidelines decided by the various standards of different countries such as Indian standard (IS 456) (Prakash *et al*. 1988, Dole *et al*. 2000, Chaudhuri 2020 *et al*., Kulkarni and Bhusare 2017), American standard (ACI 318, ASCE7) (Nigdeli and Bekdaş 2017, Bekdaş and Niğdeli 2016, Kaveh and Behnam 2013, 2014, Aga and Adam 2015, Gharehbaghi and Khatibinia 2015, Esfandiary *et al*. 2016, Tapao and Cheerarot 2017, Esfandiari 2018], European standard (Eurocode 2) (Ferreira 2003, Bekas and Stavroulakis 2017) and Australian standard (AS 3600) (Uz *et al*. 2018), Brazilian standard (NBR 6118) (De Medeiros and Kripka 2013). Apart from these, RazmaraShooli *et al*. (2019) have proposed a GA-PSO based algorithm for performance-based design optimization of a special moment-resisting frame based on guidelines provided by American standards (ATC-40, FEMA 356, ASCE-7 and ASCE-41).In these literatures the researchers used geometry of beam, column, amount of reinforcement, cost of material and shuttering as their optimization variable. In the problems regarding optimized design of RC components, optimizing the distribution of reinforcement are also an important issue (Milajić *et al*. 2014, 2015).

While solving any optimization problems choosing proper optimization techniques is utmost important, every optimization techniques have their strengths and weaknesses. In this regard, it is important to have an idea about the optimization techniques used by the predecessors to solve a particular class of problems. The cost optimization design problems of RC frame have been tackled by the researchers by using various optimization techniques. Few of them have already been discussed earlier in this section. However, reviews of the optimization algorithms used by researchers have been presented in Table 1 for clarity.

Thus, the key points, which have been observed from the above literature are as follows:

i) Literature studying the cost-optimum design of RC building are quite less in number.

ii) Most of the literatures are focused on the optimization of only a particular member (beam or column), and reinforcement detailing pattern of beam or column sections.

iii) Also, no algorithm from above literature have been tested for large scale multi-storey building frame to provide cost optimized design accompanied by construction friendly reinforcement detailing.

iv) Usage of optimization techniques in this field is also very less compared to other fields like structural health monitoring, travelling salesman problems, water resource etc.

Apart from above points another important issue is the shortcomings of commercial design software. Although, design performed in the software is correct in term of safety criteria, they provide the reinforcement amount in terms area instead construction friendly reinforcement detailing. Thus, the method developed in the present study aims towards alleviate these shortcomings. Also Unified Particle Swarm Optimization has not been used as a cost optimization method for multistory building design in the previous studies, although it has been used very effectively in cost optimization of RC foundation (Chaudhuriand Maity 2020), damage detection problems [33-38], magnetoencephalography problem (Parsopoulos *et al.* 2009) etc. Hence, UPSO have been found to the appropriate optimization method for multistory building design and cost optimization. Thus, main objective of the present paper is to develop cost-optimized design algorithm for RC frame following the safety and serviceability requirements of IS 456 (Part I) (1987) employing UPSO. Efficiency of the algorithm has been shown using a building frame (G+8, and G+10) of different planner configurations. Effects of seismic and wind load also have been considered. Optimization has been performed based on minimum cost rather than minimum weight, as the considered building frame are not high-rise (Prakash *et al.* 1988). Final optimized sections and reinforcement details obtained from the present algorithm can be used without altering in the actual site during construction phase. The efficiency of the developed design optimization algorithm has been investigated using Monte Carlo simulation. Monte Carlo method estimates the probability of favorable solution of the developed optimization algorithm when a large number of experiments are carried out for cost optimization of the building. This consequently evaluates the robustness of the developed algorithm in case of huge number of experiments and different types of buildings.

2. Mathematical formulation

An UPSO based algorithm has been developed in the present study to obtain cost effective design of the multi-storeyed reinforcement concrete frame. Details of structural analysis, design criteria, optimization algorithm, objective functions have been discussed in a brief manner in this section.

2.1 Structural analysis

In general building frame are subjected to gravity load (dead load (DL) and live load (LL)), wind load (WL) and seismic load (SL). Designers use all these loads in various combinations to calculate design forces such as axial loads, bending moments and shear forces for beams and columns. The moments, shear forces and axial loads for the critical load combination are used to design the beams and columns of the building frame. Detail procedure for analysis of building frame under aforementioned loads is mentioned briefly in the subsequent sections.

2.1.1 Gravity loads

Gravity loads consist of dead load and live load. Dead load of different components of a building can be considered as per IS 875 (Part I) (1987). Dead loads constitute of the following loads.

1. Self-weight of beams and columns.
2. Self-weight of internal and external wall and parapet wall.
3. Self-weight of floor slabs.
4. Dead load coming from floor finish and plastering

Any temporary or transient loads which act on the building can be defined as live loads. People, furniture, vehicles, and almost everything else that can be moved throughout a building come under live loads. Live loads can be provided to any structural element (floors, columns, beams, even roofs). Appropriate amount and type of live load can be decided based on the specification given on IS 875 (Part II) (1987).

2.1.2 Wind loads

Wind load should be considered for designing a multi-storey building frame. The nature of flow of wind past a body resting on a surface depends on the conditions of the surface, the shape of the body, its height, velocity of wind flow and many other factors SP 64 (Part III) (2001). IS 875 (Part III) (2015) contains the guidelines for considering the effects of wind on a multistorey building. The shape of wind load velocity profile is similar to boundary layer flow profile.

Initially wind analysis of any structure starts with selecting proper basic wind speed V_b depending on the location of the structure. The design wind speed V_z height z in m/s can be calculated mathematically as per Eq. 1.

$$V_z = V_b k_1 k_2 k_3 k_4 \tag{1}$$

Here, k_1 is risk factor or probability factor. It is decided based on design life of structures. k_2 is the velocity profile multiplier based on the terrain category. k_3 is topography factor, decided based on the ground slope of the site. k_4 is a factor based on the cyclonic importance of the structure.

The wind pressure in N/m^2 at any height z above the mean ground level can be obtained from the following Eq. 2.

$$p_z = 0.6{V_z}^2 \tag{2}$$

Finally design wind pressure (p_d) in N/m^2 at any height z above the mean ground level can be obtained from the following Eq. 3.

$$p_d = K_d K_a K_c p_z, p_d \geq 0.7 p_z \tag{3}$$

Design moment, shear and axial forces in beams and columns for wind load are calculated by applying the design wind pressure on the frame.

When, any structure is subjected to seismic load dynamic equation of the structure can be represented as Eq. 4.

$$[M]\ddot{x}(t) + [C]\dot{x}(t) + [K]x(t) = -[M]\ddot{u_g}(t) \tag{4}$$

where, $[M]$, $[C]$ and $[K]$ are mass, damping and stiffness matrix of the structure. $\ddot{u_g}(t)$ is the acceleration time history of the induced earthquake. $x(t)$ is the time history response of the structure due to the induced earthquake force. Eq. 4 can be solved using numerical approaches for time history responses, such as displacement $x(t)$, velocity $\dot{x}(t)$ and acceleration $\ddot{x}(t)$. Eventually stress time history also can be obtained. But, there are some difficulties to do an actual seismic analysis for every structure to be designed. They are

i) It is not always possible to have the earthquake acceleration time history of the exact location of the structure.

ii) Analysis of the structure cannot be carried out solely considering the peak ground acceleration (PGA) of the earthquake, as the response of the structure depends on the frequency component of the earthquake and its own dynamic properties.

These difficulties can be overcome by using the response spectrum of the earthquake instead of using the acceleration time history as input. Response spectrum represents the maximum response of damped single degree of freedom (SDOF) system for a particular input earthquake motion at different natural period. Maximum response is more important to a designer than the entire time history. One can also obtain mean response spectrum for a particular location using more than one data of past earthquakes of the location. Also, use of different damping value will give different response spectrum for same earthquake response. Once acceleration response spectrum is known, one can easily obtain the maximum base shear simply by multiplying the spectral acceleration obtained from the response spectrum with the seismic mass of the structure. Every country develops their own design response spectrum based on the past earthquake happenings in the region. In general seismic design practice the structure should prevent non-structural damage for minor earthquake, prevent structural damage with minimum non-structural damage for moderate earthquake and avoid collapse to save lives in case of a major earthquake. Thus, no one use the spectral acceleration associated with PGA (maximum considered earthquake (MCE)) for seismic design of structure. Rather, the analysis is carried out for a much reduced value of spectral acceleration (design basis earthquake (DBE)).

The guidelines for analyzing a building frame for seismic loading is mentioned in IS 1893 (Part-I) (2016). Instead of solving rigorous dynamic equations, Indian standard has provided simple linear static approach for simple regular structures utilizing the response spectrum. Entire country has been divided into four seismic zone (II, III, IV, V) based on the PGA. Seismic forces has two horizontal and one vertical components. For simple regular building frame vertical component is ignored. The design horizontal seismic coefficient (A_h) can be calculated as Eq. 5.

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)} \tag{5}$$

Here, Z is seismic zone factor depend on the seismic zone of the location of the structures. In the term $\left(\frac{Z}{2}\right)$, $\left(\frac{1}{2}\right)$ factor used to reduce the MCE to DBE. $I$is the importance factor, decided based on the occupancy and use of the structure. $R$is the response reduction factor depends on the ductility, redundancy and overstrength of the structure. A structure with good ductility will have high value of R, i.e., that structure will be designed for low seismic force. $\left(\frac{S_a}{g}\right)$ is normalize spectral acceleration coefficient, which can be calculated based on the natural period of the structure and soil type from the design response spectrum presented in the standard.

The natural period of ordinary RC building can be calculated from Eq. 6.

$$T_a = \frac{0.09H_{bl}}{\sqrt{D_{bl}}} \tag{6}$$

Now, the base shear can be computed for the building as a whole from the following equation (Eq. 7).

$$V_B = A_h W \tag{7}$$

where, W is the seismic weight of the building which is calculated by adding full dead load and 25 percent of the liveload.

The base shear in Eq. 7 is distributed at the center of mass of all the floor levels of the frame. These forces are distributed to the individual lateral load resisting elements through structural analysis considering floor diaphragm action.

2.1.4 Load combinations

All the above loads are considered for suitable load combinations according to IS 875 (Part V) (2015) with appropriate load factors. The load combinations have been mentioned below

1. 1.5(DL + LL)
2. 1.5(DL± WL)
3. (0.9DL±1.5WL)
4. 1.5(DL± SL)
5. (0.9DL±1.5SL)
6. 1.2(DL+LL±WL)
7. 1.2(DL+LL±SL)

Among all these load combinations only most critical load combination has been used for designing each beam and column of the building frame.

2.2 Structural design

Beams and columns of all the frame were designed adopting Limit State Method as per the guidelines provided in the Indian Standard IS 456 (2000) in the present study. The design procedure has been described briefly in the following sections.

2.2.1 Beam design

The design of beam is carried out based on limit state of collapse in flexure considering plane section normal to the axis remains plane after bending. Longitudinal reinforcements in beams are provided to carry bending moments, whereas stirrups are provided to carry shear forces. Design bending moment (M_u) and shear forces (V_u) for all beam sections are obtained from the structural analysis. At first the beam is designed to carry design bending moment. The limiting moment of resistance of balanced singly reinforced beam section due to flexure can be calculated as per Eq. 8.

$$M_{u,lim} = 0.36\frac{x_{umax}}{d_e}\left(1 - 0.42\frac{x_{umax}}{d_e}\right)bd_e{}^2 f_{ck} \tag{8}$$

If $M_u \leq M_{u,lim}$, then the beam section is designed as singly reinforced section. The required area of tensile reinforcement can be calculated from equation Eq. 9.

$$M_u = 0.87 f_y A_{st} d_e \frac{x_{umax}}{d_e}\left(1 - \frac{f_y A_{st}}{bd_e f_{ck}}\right), \qquad A_{st} \geq 0.85 bd_e/f_y \tag{9}$$

If $M_u > M_{u,lim}$, then the beam is designed as doubly reinforced section. Area of tensile reinforcement (A_{st1}) required for $M_{u,lim}$ is calculated from equation Eq. 9. The area of compression reinforcement for the excess moment i.e. ($M_u - M_{u,lim}$) is obtained from Eq. 10.

$$M_u - M_{u,lim} = f_{sc} A_{sc}(d_e - d') \tag{10}$$

where , f_{sc} = design stress in compression reinforcement corresponding to a strain of $0.0035\frac{(x_{umax} - d')}{x_{umax}}$, $\frac{x_{umax}}{d_e}$ =0.53, 0.48, 0.46 for $f_y = 250, 415, 500$ respectively. The area of corresponding tensile reinforcement (A_{st2}) for the excess moment$(M_u - M_{u,lim})$ is calculated in Eq. 11.

$$A_{st2} = f_{sc} A_{sc}/0.87 f_y \tag{11}$$

longitudinal reinforcement of beams are provided based on A_{st}, A_{st1}, A_{st2}.Next, the provide beam section is checked for design shear force. Nominal shear stress for beam section shall be obtained from the following Eq. 12.

$$\tau_v = V_u/bd_e \tag{12}$$

The design shear strength τ_c of concrete is calculated from IS 456 (2000) for grade of concrete and percentage of total tensile reinforcement provided. If $\tau_v \leq \tau_c$, minimum shear reinforcement shall be provided as per in Eq. 13.

$$s_v = \min\left(\frac{0.87 f_y A_{sv}}{0.4b}, 3d_e, 300\ mm\right) \tag{13}$$

If $\tau_c < \tau_v < \tau_{cmax}$, The shear reinforcement should be designed to carry a shear force $V_{us} = V_u - \tau_c bd_e$. τ_{cmax} can be obtained from IS 456 depending on the strength of the concrete. The spacing of the shear reinforcement s_v shall be provided as obtained from the following equation. Eq. 14.

$$s_v = \min\left(\frac{0.87 f_y A_{sv} d_e}{V_{us}}, \frac{0.87 f_y A_{sv}}{0.4b}, 3d_e, 300\ mm\right) \tag{14}$$

Once the longitudinal reinforcement and shear reinforcement had been designed, the beam should be checked to be safe against the serviceability criteria of limit state method. Thus, maximum deflection of the beam should be within the limit provided by the design standard. The total deflection of beam is thus calculated as per Eq. 15.

$$a_{td} = a_s + a_{cs} + a_{cc} \tag{15}$$

where, a_s is calculated for the usual method for elastic deformation theory using short term elasticity modulus E_c and effective moment of inertia I_{eff} given in Eq. 16.

$$\frac{I_r}{1.2-\frac{M_r}{M}\frac{z}{d_e}(1-\frac{x}{d_e})\frac{b_w}{b_b}} \text{ , } I_r < I_{eff}\ I_{eff} =< I_{gr} \tag{16}$$

where, $M_r = f_{cr}I_{gr}/y_t$, $a_{cs}$is the deflection due to shrinkage and it is calculated according to the equation Eq. 17.

$$a_{cs} = f_3\varphi_{cs}l^2 \tag{17}$$

where, $\varphi_{cs} = f_4\frac{\in_{cs}}{D}$, where, f_3, f_4 are calculated from IS 456 (2000) annex C-3.1. $\in_{cs}$=0.0003. a_{cc} is the deflection due to creep for permanent loads and is defined in Eq. 18.

$$a_{cc} = a_{i,cc} - a_i \tag{18}$$

where, $a_{i,cc}$ is the initial plus creep deflection due to permanent loads obtained using an elastic analysis with an effective modulus of elasticity ($E_{ce} = E_c/(1+\theta)$).

2.2.2 Column design

The design of column is done based on the same assumption for limit state of collapse in flexure i.e. plane section normal to the axis remains plane after bending. All compression members should be designed for a minimum eccentricity of load in two principal direction. Minimum eccentricity in design of columns can be obtained from Eq. 19.

$$e_{min} = \min\left(\frac{l_c}{500} + \frac{D_c}{30}, 20\ mm\right) \tag{19}$$

All the column sections are designed considering combined effects of axial load and biaxial bending moments. Thus, minimum eccentricity should be checked for both x and y direction bending separately. If the column is subjected to axial load P_u, biaxial moments M_{ux} and M_{uy}, the column section thus designed should satisfy for interaction ratio given by Eq. 20.

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1.0 \tag{20}$$

α_n is the exponent component whose value depends on P_u/P_{uz}. $\alpha_n = 1$ for $P_u/P_{uz} \leq 0.2$. $\alpha_n = 2$ for $P_u/P_{uz} \geq 0.8$. Linear interpolation should be used for intermediate values. P_{uz} is calculated from Eq. 21.

$$P_{uz} = 0.45f_{ck}\,\mathrm{A_c} + 0.75f_yA_{scc} \tag{21}$$

M_{ux1}, M_{uy1} are the maximum uniaxial moment carrying capacity of the column section combined with the axial load P_u respectively for about x and y direction. Now, while considering any particular direction bending two different cases can emerge based on the position of the neutral axis as shown in Fig. 1.

Case 1: When the neutral axis lies within the column section (Fig. 1b), the axial load carrying capacity and moment carrying capacity of the section can be calculated from Eqs. 22 and 23 respectively.

$$P_{u1} = 0.36kf_{ck}b_cD_c + \sum_{i=1}^{n}\frac{p_ib_cD_c}{100} * (f_{si} - f_{ci}) \tag{22}$$

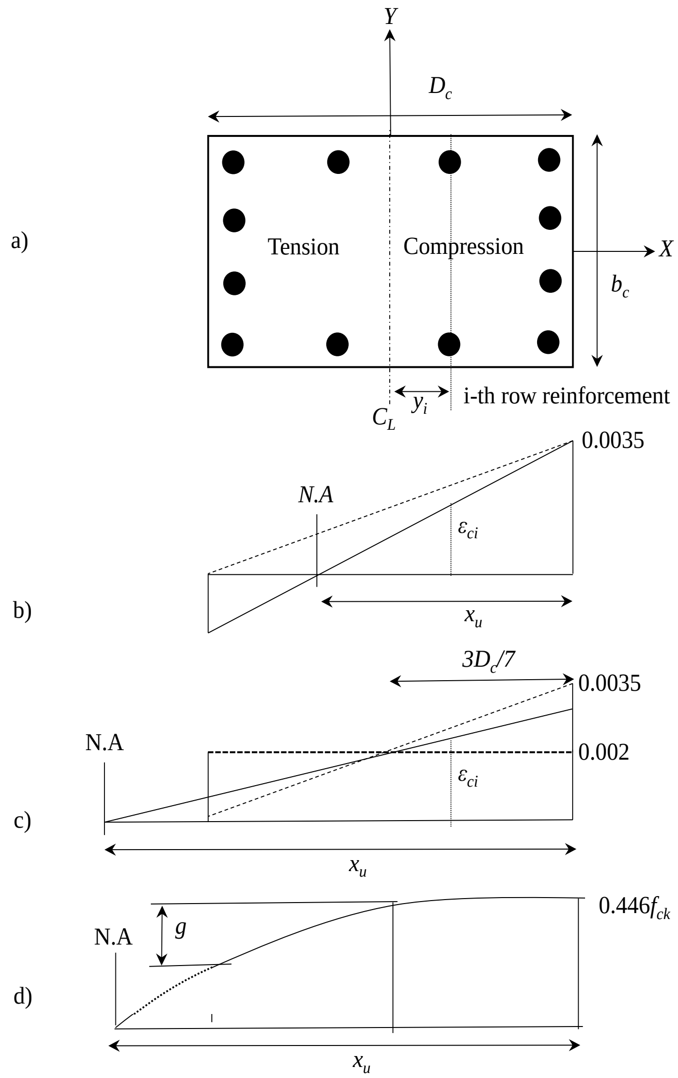

Fig. 1 a) Typical reinforcement detail of column section b) Strain diagram of concrete section whe n neutral axis lies inside the section c) Strain diagram of concrete section when neutral axis lies o utside the section d) Stress diagram of concrete section when neutral axis lies outside the section

$$M_{u1} = 0.36 k f_{ck} b_c {D_c}^2 (0.5 - 0.416k) + \sum_{i=1}^{n} \frac{p_i b_c D_c}{100} * (f_{si} - f_{ci})\, y_i \tag{23}$$

where $k = x_u/D$;

Case 2: If neural axis lies outside the column section (Fig. 1c) the axial load carrying capacity and moment carrying capacity of the section can be calculated from Eqs. 24 and 25 respectively.

$$P_{u1} = C_1 f_{ck} b_c D_c + \sum_{i=1}^{n} \frac{p_i b_c D_c}{100} * (f_{si} - f_{ci}) \tag{24}$$

$$M_{u1} = C_1 f_{ck} b_c D_c (0.5D_c - C_2 D_c) + \sum_{i=1}^{n} \frac{p_i b_c D_c}{100} * (f_{si} - f_{ci})\, y_i \tag{25}$$

where, f_{ci} is stress in concrete at the level of i-th row of reinforcement and can be calculated from Fig. 1d. f_{si} is stress in the i-th row of reinforcement. C_1 is stress co-efficient and $C_2 D$ is the distance of the centroid the concrete stress block (Fig. 1d) measured from the highly compressed edge. C_1 and C_2 can be obtained from Eqs. 26 and 27 respectively.

$$C_1 = \frac{\mathrm{A_{str}}}{f_{ck} D_c} \tag{26}$$

$$C_2 = M_c / \mathrm{A_{str}} \tag{27}$$

A_{str} is the area of the stress block (Fig.1d), and can be calculated from the Eq. 28.

$$A_{str} = 0.446 f_{ck} D_c * (1 - (\frac{4}{21})(\frac{4}{(7k-3)})^2) \tag{28}$$

M_c is the moment of the concrete stress block (Fig. 1d) about highly compressed edge is obtained as per Eq. 29.

$$M_c = 0.446 f_{ck} D_c * (0.5 * D_c) - (8/49) * g * {D_c}^2 \tag{29}$$

where g is geometric properties of the parabola (Fig. 1d) obtained from Eq. 30.

$$g = (0.446 f_{ck} * (\frac{4}{7k-3})^2) \tag{30}$$

Diameter of tie bar of a column section (φ_{tie}) should be decided according to Eq. 31.

$$\varphi_{tie} = \max\,(\frac{\varphi_m}{6}, 6\,mm\,) \tag{31}$$

Spacing of tie bar (S_{tie}) can be calculated according to Eq. 32.

$$S_{tie} = \min\,(16\varphi_{tie}, 300\,mm) \tag{32}$$

2.3 Structural Design Optimization

2.3.1 Unified Particle Swarm Optimization (UPSO)

UPSO is a swarm based optimization proposed by Parsopoulos and Vrahitis (2005) as a upgraded version of Particle swarm optimization (PSO) (Kennedy and Eberhert 1995, Perez and Behdinan 2007) based on the individual and social behavior of flock of birds, school of fish etc. in their process of searching foods or avoiding predators.

The algorithm begins with each particle assuming random position $S(t)$ and velocity $H(t)$. The position of each particle in the swarm represents a possible solution of the optimization problem. During the search process position of each particle gets updated with every iteration through a velocity update rule. Velocity update of each particle in every iteration depends on the three factor, such as, $a(t)$, i.e., the best position visited by the particle itself, $u(t)$, i.e., the best position ever visited by all the particles and $n(t)$, i.e., the best position visited by the neighbors of that particle. Thus, new position for each particle $S(t+1)$ can be obtained by adding the updated velocity $H(t+1)$ with previous position $S(t)$ (Eq. 33).

$$S(t+1) = S(t) + H(t+1),\ S(t+1) \in [S_{min}, S_{max}] \tag{33}$$

The updated velocity can be obtained from Eq. 34.

$$H(t+1) = \mu G(t+1) + (1-\mu)L(t+1),\ H(t+1) \in [-H_{max}, H_{max}] \tag{34}$$

$$G(t+1) = \chi\big[H(t) + c_1 rand(0,1)\big(a(t) - S(t)\big) + c_2 r\ and\ (0,1)(u(t) - S(t))\big] \tag{35}$$

$$L(t+1) = \chi\big[H(t) + c_1 rand(0,1)\big(a(t) - S(t)\big) + c_2 r\ and\ (0,1)(n(t) - S(t))\big] \tag{36}$$

μ is unification factor increasing from 0 to 1 exponentially according to Eq. 37

$$\mu(t) = \exp\left(\frac{t * \log 2}{MaxIt}\right) - 1 \tag{37}$$

Also, S_{max} and S_{min} are upper and lower limit for position respectively. V_{max} is upper limit for velocity = $(S_{max} - S_{min})/2$, $\chi = 0.729$, and $c_1 = c_2 = 2.05$ (Parsopoulos and Vrahatis 2010).

2.3.2 Beam and column design optimization

Beam and column design optimizations algorithm are developed by making suitable changes in the above mentioned UPSO algorithm. In the present study beams and columns has been optimized separately. The entire algorithm is divided into three steps accordingly.

(A) UPSO based beam design optimization

1. In the first steps all beams are optimized separately In case of beam design optimization

2. The input variables are considered as width (b), overall depth (D), diameter of main bar at top (φ_t) and bottom (φ_b),no of compression bar support(n_{cs}) and mid span (n_{cm}), no of tension bar at support (n_{ts}) and mid span (n_{tm}).

3. Only longitudinal reinforcement are optimized along with the cross sectional area. Shear reinforcement are designed based on the optimized cross section and longitudinal reinforcement amount.

4. The output variables are width, overall depth, diameter of bar at top and bottom of section, number of bars at top and bottom for mid-span and support of section, spacing of shear reinforcement at support and mid-spanof section.

5. The following constraints are considered

a. $1.5b \leq D \leq 2b$

b. $b$and$D$ are assumed in the multiple of 10 to take into account the practicality aspects of construction.

c. Clear spacing of the bars should exceed the maximum aggregate size.

d. Moment and shear capacity of section should exceed the design moment and shear of that section.

e. Total deflection (Eq. 14) should be less than $\frac{l}{250}$.

(B) UPSO based column design optimization

1. In the second step all column are optimized separately. In case of column design optimization

2. The input design variables considered are as follows: width (b_c), depth (D_c), k_x, k_y, diameter of main bars (φ_m), number of main bars of columns in x and y directions (n).

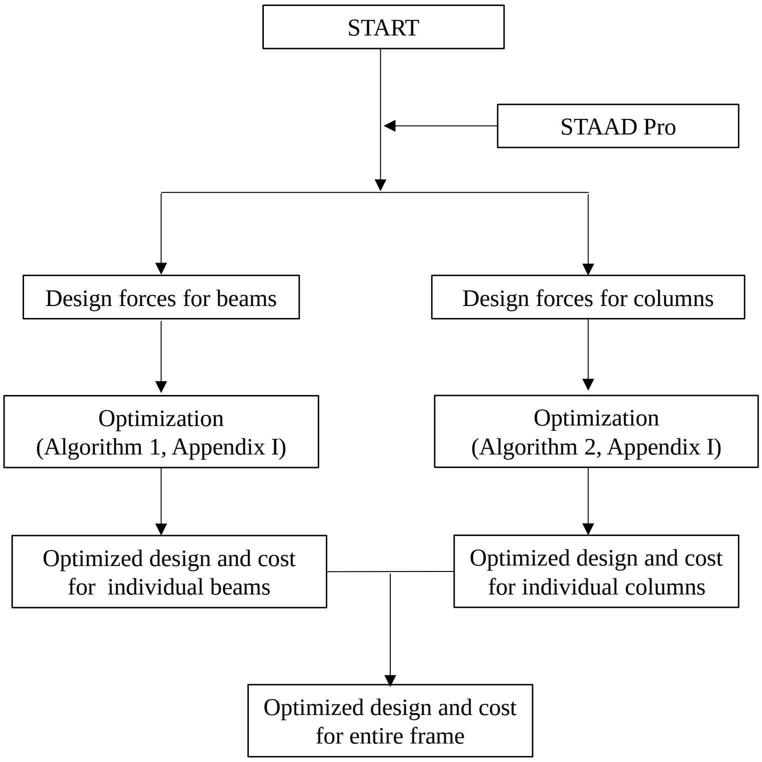

Fig. 2 Flowchart for cost optimization of whole frame

3. The output variables are considered as width (b_c), depth (D_c), kx, ky, diameter of main bars (φ_m), number of main bars of columns in x and y directions (n), diameter of tie bars and their spacing.

4. Constraints applied for columns are

a. b_c and D_c are assumed in the multiple of 10 to take into account the practicality aspects of construction

b. Clear spacing of the bars should exceed the maximum aggregate size.

c. Limit for interaction ratio as given in equation Eq. 19 should be satisfied.

(C) Combined beam-column design optimization

Eventually, all the individual optimization results of beams and columns are combined to obtain optimized design for the entire building frame in the third step. A flowchart for the entire developed program is presented in Fig. 2.

2.3.3 Cost based objective function

Cost based objective function for beams and columns have been presented in Eq. 38.

$$F(design\_var) = V_c C_c + V_s \rho_s C_s + A_f C_f \tag{38}$$

where $C_c = Rs. 5844/m^3$; $C_s = Rs. 68.508/Kg$; $C_f = 225/m^2$ (WB PWD schedule 2017).

Area of formwork (A_f) for beam and columns can be calculated from Fig. 3 as per Eq. 39 and Eq. 40 respectively.

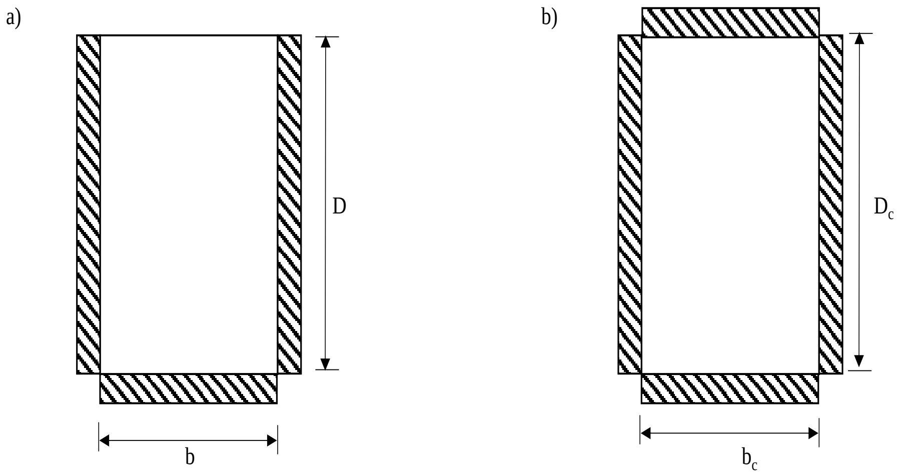

Fig. 3 Formwork profile for member cross section: a) Beam b) Column

$$(A_f)_{beam} = (b + 2D) * length\ of\ beam \tag{39}$$

$$(A_f)_{column} = (2b_c + 2D_c) * length\ of\ column \tag{40}$$

3. Numerical results and discussion

3.1 Problem definition

A G+8 L shaped building frame (Fig. 4) is considered to demonstrate the efficacy of proposed algorithm. The structures were analyzed in STAAD Pro V8i (2017) considering seismic loads, wind loads and gravity loads to determine the most critical load case for each member. Now, an UPSO based algorithm has been developed to obtain cost optimum design for beams and columns utilizing the design loads obtained from STAAD Pro (2017) for critical load cases without disrupting the safety criteria. The designs of each beam and each column are optimized separately to consider the contribution of all the design parameters responsible for beams and columns. The optimized cost of all beams and columns are summed up to get the total optimized cost for each frame. Efficient cost optimization algorithm of RC building frame depends on appropriate choice of design parameters and internal parameters for optimization algorithm.

3.2 Design parameters

Important design parameter considered in the present study has been mentioned below:

i) Location of the structure: Kolkata, India (assumed)

ii) Thickness of outer wall and inner wall =250mm and 125 mm respectively (outer walls and inner walls are assumed to be on outer and inner beam respectively).

iii) Unit weight of brick wall= 20 KN/m^3

Unit weight of reinforced concrete= 25 KN/m^3

iv) Floor finish load on roof= 1 KN/m^2

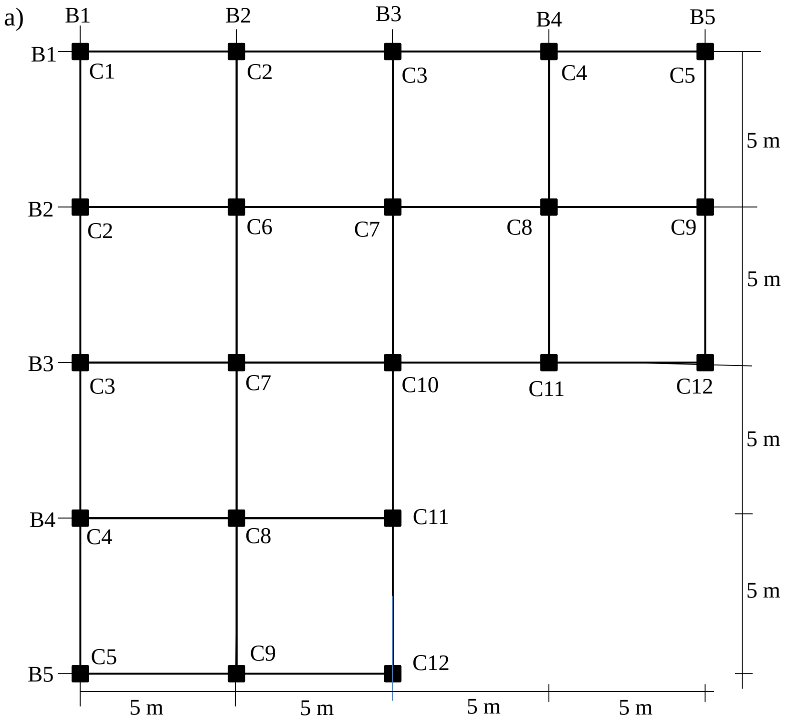

Fig. 4 Typical floor grid plan of the building frame showing beams and column positions a) L shaped building

v) Live on all roof= 1.5 KN/m^2

Live load on all other floor= 3 KN/m^2

vi) For wind load (IS 875(Part III) 2015) [44] $V_b = 50$m/s, $k_1 = 1$ (50 years life span), k_2 is function of height for terrain category 2, $k_3 = 1$ (for ground slope <3°), $k_4 = 1$ (ordinary RC frame).

$K_d = 1,\ K_a = 1,\ K_c = 0.9$.

vii) For seismic load (IS 1893 (Part I) 2016)

$Z = 0.16$ (Zone III), $I = 1$ (Ordinary building), $R = 3$ (ordinary moment resisting frame), soil type: medium, $\frac{S_a}{g}$ can be calculated as per Fig. 2.

Seismic weight is calculated considering full dead load, no live load on roof and half live load on all other floors

viii) Yield strength of reinforcement bar =415MPa and Characteristic strength of concrete =25MPa (assumed)

ix) Beams and columns are designed considering load reversal due to seismic and wind load cases. Reinforcement is placed only along the outer periphery of the sections. Equal number of reinforcement is placed on all the four sides of the columns. The space between two consecutive bars should be greater than maximum aggregate size for convenience of the construction.

x) Curtailment of extra tensile bars in beam has been considered in present study (Fig. 8). Development length of reinforcement of beam and columns at the end supports has not been considered. In case of beam, different shear reinforcement spacing has been used for support and

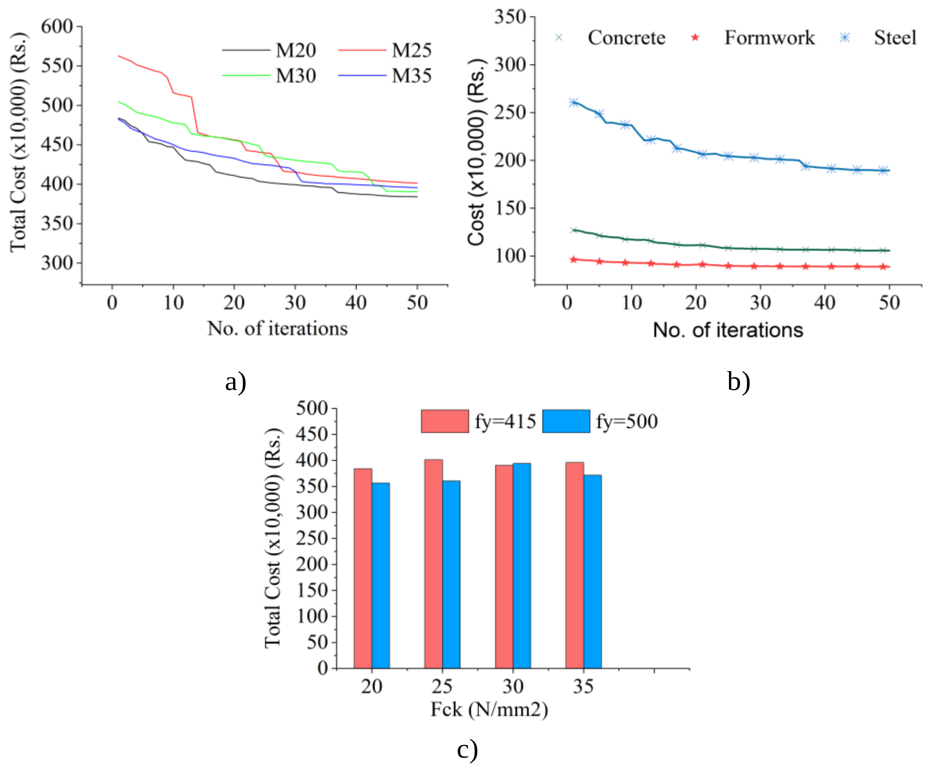

Fig. 5 Design optimization results for L-shaped building frame. (a) convergence curve of total cost for different grade concrete along with Fe 415 steel.(b) convergence curve for cost of different parameters for M20 concrete and Fe 415 steel. (c) Variation of total cost for different concrete and steel grades

mid span as required as all the beams are continuous with fixed supports. Shear reinforcement for support has been designed for maximum shear force of beam, while shear reinforcement at the mid span designed to carry minimum shear.

xi) Minimum and maximum diameter of main reinforcing bars for beams and columns are 12 mm and 32 mm. The diameter of tie bar for shear in beams is 2 legged 8 mm and for columns are 8 mm.

3.3 Optimization parameters

1. Beams are optimized starting from bottom floor to top floor. The search space of the beam design optimization has been restricted in such a way that the maximum value of optimization design variables for a particular floor shall be equal to the optimized design values obtained for the beams in the subsequent bottom floor (except for ground floor). The minimum range of the design variables of all beams have been kept same for all floors.

2. Columns are optimized starting from top floor to bottom floor. The search space of the column design optimization has been restricted in such a way that the minimum value of optimization design variables for a particular floor shall be equal to the optimized design values obtained for the columns in the subsequent top floor (except for topmost floor). The maximum range of the design variables of all beams have been kept same for all floors.

Table 2 Beam design details for three different floors - L shaped building frame

Floors	Beam types	Size (mm x mm)	Support (mm)		Mid-span (mm)		Stirrups spacing (8mm φ)
			Top	Bottom	Top	Bottom	
Top	B1	210x 410	3-12φ	3-12 φ	2 -12 φ	3-12φ	300
	B2, B3,B4	210x 420	2-16φ	2-12φ	2 -16 φ	3-12φ	300
	B5	210x 420	3-12φ	3-12 φ	2-12φ	3-12 φ	300
	B1	210x 410	3-12φ	3-12 φ	2-12 φ	3-12φ	300
6th	B1	210x 390	3- 20φ	2- 20 φ	2- 20 φ	2- 20 φ	300
	B2, B3	270x 530	6-16φ	2- 20 φ	2-16φ	2- 20 φ	300
	B4	240x 470	4- 20 φ	2- 20 φ	2- 20 φ	3- 20 φ	300
	B5	240x 470	5-16φ	2- 20 φ	2-16φ	3- 20 φ	300
1st	B1	240x 460	5- 20 φ	2- 20 φ	2- 20 φ	3- 20 φ	300
	B2	280x 530	6-16φ	2- 20 φ	2-16φ	4- 20 φ	300
	B3	270x 510	5- 20 φ	2- 20 φ	2- 20 φ	4- 20 φ	300
	B4	270x 520	5- 20 φ	3- 20 φ	2- 20 φ	2- 20 φ	300
	B5	240x 470	5- 20 φ	2- 20 φ	2- 20 φ	3- 20 φ	300

3. Individual beam and column design optimization are performed separately for 10 numbers of experiments considering maximum iteration and swarm size 50 and 10 respectively. The experiment which exhibit minimum cost has been considered as the optimized design for the respective beam and column.

4. In case of continuous beams, all spans have been designed considering the maximum design moments and shear for the beam.

3.4 Numerical Results

3.4.1 L-shaped building frame

In this section developed UPSO based algorithm has been used to optimize the RC design of the G+8 L-shaped building frame to have minimum cost. Search space of design variables should be decided appropriately depending on the experience of the designer, as inappropriate choice of search space can lead to large computational effort. Search space of design variables considered in the study in case of beams are: $b \in [200,400]$, $D \in [300,600]$, $\varphi_t \in [12,20], \varphi_b \in [12,20]$, $n_{cs} \in [2,5]$, $n_{cm} \in [2,5]$, $n_{ts} \in [2,5]$, $n_{tm} \in [2,5]$. Search space of design variables considered in the study in case of columns are: $b_c \in [250,500]$, $D_c \in [250,500]$, $k_x \in [0.5,1.2]$, $k_y \in [0.5,1.2]$, $\varphi_m \in [12,25]$, $n \in [2,4]$.Convergence curve for total cost for M20, M25, M30, and M35 grades of concrete along with Fe 415 steel have been plotted in Fig. 5(a). Total cost of the building frame is found to be varying within the range [3840101, 4014875] for Fe 415 steel and [3564230, 3942723] for Fe 500 steel for different grades of concrete. It can be observed that variation among different grades of concrete is 4.5% and 10.7% for Fe 415 and Fe 500 steel. Next, Fig. 5(b) has been plotted showing the convergence curves of the total cost of concrete, total cost formwork and total cost of steel through all the iterations for M20 grade of concrete and Fe415 grade of steel. This will give designers a good insight regarding the inter-relationship among these three parameters. Also, a bar diagram has been presented showing the comparisons of total cost of

Table 3 Typical Column design details for all floor - L shaped building frame

Floors	Column numbers							
	C1		C3		C5		C9	
	Size (mmxmm)	Reinforcement	Size (mmxmm)	Reinforcement	Size (mmxmm)	Reinforcement	Size (mmxmm)	Reinforcement
8	340x450	12-16φ	320x380	8-16 φ	370x280	8-16 φ	430x370	8-12 φ
7	410x450	12-16φ	330x420	8-20 φ	380x330	8-20 φ	450x450	8-12 φ
6	440x450	12-16 φ	350x440	8-20 φ	410x370	8-20 φ	450x450	8-20 φ
5	450x450	12-20 φ	370x450	8-20 φ	440x410	8-20 φ	450x450	8-20 φ
4	450x450	12-20 φ	410x500	8-20 φ	450x430	8-20 φ	450x450	8-20 φ
3	460x 480	12-20 φ	420x500	8-20 φ	480x460	8-20 φ	470x470	8-20 φ
2	500x 500	12-20 φ	470x500	8-20 φ	490x500	8-20 φ	500x500	8-20 φ
1	500x 500	12-20 φ	500x500	8-20 φ	500x500	8-20 φ	500x500	8-20 φ
G	500x 500	12-20 φ	500x500	8-25 φ	500x500	8-25 φ	500x500	8-25 φ

the frame for different grades of steel (Fe415, Fe500) and concrete (M20, M25, M30, M35) in Fig. 5(c). It can be seen that for all concrete grades, Fe 500 steel yields lower cost than Fe415 steel. Optimized design output for beams and columns obtained from the algorithm has been reported respectively in Table 2 and Table 3 for M20concrete and Fe 415 steel. Beam design details of only three floors have been presented for brevity (Table 2), whereas typical column design details for all the floors have been presented (Table 3). In case of columns6 mm diameter of links are considered throughout and the spacing is 190mm c/c, 255mm c/c and 300mm c/c for 12 mm, 16 mm, and 20 mm diameter main bars respectively.

Thus, the present algorithm has been found to be very flexible and effective to provide cost optimized design for multistoried L shaped building, considering all the codal provisions (IS 456 2000) and the practical considerations.

4. Conclusions

In the present study, an UPSO based optimization algorithm has been developed in MATLAB (2015) environment to find cost optimum design of reinforcement concrete building frame considering the codal specifications of safety and serviceability of IS 456 (2014) along with the consideration for the construction requirements in practical field.

A framed structure G+8 L-shaped frame have been adopted to demonstrate the efficacy of the developed algorithm. Popular design and analysis software STAADPro. V8i (2017) has been used to obtain the design forces (bending moments, shear forces and axial forces) in critical sections of all the beams and columns considering the effects of gravity loads, wind loads and seismic loads as per the specifications of the respective Indian Standards. Next, each beam and column are optimized separately employing UPSO based algorithm. Thus, total optimized cost of the frame has been obtained by adding up all the optimized costs of these beams and columns. Numerical results have revealed that the present algorithm is capable of providing cost optimized design of RC frame of any shape and height with profound accuracy. Only the design variables and constraints need to be modified to adapt to the particular building problem. Overall, the present

UPSO based algorithm has been found to be very effective in finding cost optimum design of RC frame having any planner irregularity and any number of floors. The positive findings of the research will encourage the future researchers to improve the present algorithm to incorporate more minute reinforcement details such as development length, ductile detailing etc. Also, finite element method can be incorporated within the algorithm to obtain the design forces directly instead of relying on commercial design software. In that way accuracy of the results can be improved further.

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Appendix

The following symbols have been used in this paper:
$A_h =$ Horizontal acceleration coefficient
$W =$ Seismic weight of building
$Z =$ Seismic zone factor
$\frac{S_a}{g} =$ Design acceleration coefficient
$T_a =$ Natural period of building
H_{bl} = Height of the building from plinth level
$D_{bl} =$ Base dimension of the building in the direction of earthquake shaking
$R =$ Response reduction factor
$I =$ Importance factor
$p_d =$ Design wind pressure
$K_d =$ Wind directionality factor
$K_a =$ Area averaging factor
$K_c =$ Combination factor
$V_z =$ Design wind speed
$V_b =$ Basic wind speed
$k_1 =$ Probability factor
$k_2 =$ Terrain roughness and height factor
$k_3 =$ Topology factor
$k_4 =$ Importance factor for cyclonic region
$b =$ Width of the beam
$D =$ Overall depth of the beam
$d_e =$ Effective depth of the beam
d'= depth of compression reinforcement from compression face of beam.
l_c= effective length of column
D_c= width/ depth of the column
$\frac{x_{umax}}{d_e} =$ Limiting neutral axis depth factor for beam.
$f_{ck} =$ Grade of concrete
$f_y =$ Grade of steel reinforcement
$A_{st} =$ Area of tensile reinforcement for beam
$A_{sc} =$ Area of compressive reinforcement for beam
$\tau_v =$ Nominal shear strength
$\tau_c =$ Shear strength of concrete

$\tau_{cmax} =$ Maximum shear strength of concrete
$s_v =$ Spacing of shear reinforcement for beam
$A_{sv} =$ Total cross sectional area of the stirrup legs
$a_s =$ Short term deflection of beam
$a_{cs} =$ Deflection of beam due to shrinkage
$a_{cc} =$ Deflection of beam due to creep
$E_c =$ Short term elasticity modulus for beam
$I_{eff} =$ Effective moment of inertia for short term deflection of beam
$I_r =$ Moment of inertia of cracked section of beam
$I_{gr} =$ Gross moment of inertia of beam
$M_r =$ Cracking moment
$f_{cr} =$ Modulus of rupture of concrete
$y_t =$ Distance from the centroidal axis of gross section, neglecting the reinforcement, to extreme fibre in tension
$M =$ Maximum moment under service load for beam
$z =$ Lever arm of the beam section
$x =$ Depth of the neutral axis for beam
$b_w =$ Breadth of web for beam
$b_b =$ Breadth of compression face for beam
$f_3 =$ Constant depending upon the support condition of beam
$\varphi_{cs} =$ Shrinkage curvature for beam
$f_4 =$ Factor depending on percentage of tensile and compressive reinforcement for beam
$\in_{cs} =$ Ultimate shrinkage strain of concrete for beam
$l =$ Length of the span of beam
$a_{i,cc} =$ Initial plus creep deflection of beam due to permanent loads
$E_{ce} =$ Young's modulus of concrete to calculate $a_{i,cc}$
E_c = Actual Young's modulus of concrete to calculate short term deflection
θ = Creep coefficient
$D_c =$ Depth of the column
$\mathrm{b_c} =$ width of column
$l_c =$ Length of the column.
$\mathrm{A_c} =$ Area of the concrete in column section
$\mathrm{A_{scc}} =$ Area of reinforcement in column
φ_m = Diameter of the main bar of the column
iter = number of iterations in each experiment for the developed MATLAB program.
max_iter = maximum number of iterations in each experiment for the developed MATLAB program.
exp = number of experiments, i.e., 1,2,3,….
$nexp$ = maximum number of experiments considered.
V_c= Volume of gross concrete work of beam / column in cubic meters.
V_s= Volumeof steel reinforcement of beam / column in cubic meters.
ρ_s= Density of steel i.e. 7850 Kg/m^3.
C_c= Cost of reinforced concrete work per cubic meters.
C_s= Cost of steel reinforcement per Kg.
C_f= Cost of formwork per square meters.