Cost Effective Design of RC Building Frame Employing Unified Particle Swarm Optimization

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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). Two building frames with G+8 stories and G+10 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 frames 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 frames 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.

Key words: cost effective design, unified particle swarm optimization, STAAD Pro, RC building frame, wind load, seismic 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 reinforcement concrete comes with the demand for economical design. Thus, from last few decades, researchers have proposed different method for cost optimization of RC frames. 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 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. [1] 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 avery popular and common term in steel structures, where topology of truss is optimized to achieve cost optimum design [2]. 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) [3], plastic design layout optimization technique [4], dynamic programming utilizing genetic algorithm (GA) based multi objective optimization [5]. Apart from that, Zegard et al. [6] have implemented three different methods of topology design optimization in building engineering. However, in real project topology is not decided solely based oncost 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. This can be managed with proper constructionmanagement and planning strategies like Critical Path Method (CPM) and Project Evaluation and Review Technique (PERT). 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 [7], geometric programming [8], GA based algorithm [9, 10], polynomial optimization technique [11], simulated annealing [12], random search technique (RST) [13], charged system search (CSS) [14]. Cost optimum design of T-beams [15]andCost optimum design RC columns [7, 16, 17] 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 [18] using Neural dynamic model for Adeli and Park (NDAP) [19], Design optimization of RC foundation has been performed by Chaudhuri and Maity [20]. Besides these individual structural components researchers have also tried to optimize the beam-column frames in terms of weight [21], in terms of cost [22-29]. All these studies regarding design optimization of RC building components or frames are conformed to the

93 guidelines decided by the various standards of different countries such as Indian standard (IS
94 456)[7,11, 20, 26], American standard (ACI 318, ASCE7)[13,17, 21-25, 28, 29], European
95 standard (Eurocode 2) [15,27]and Australian standard (AS 3600) [14], Brazilian standard
96 (NBR 6118)[12]. Apart from these, RazmaraShooli et al. [30] have proposed a GA-PSO
97 based algorithm for performance-based design optimization of a special moment-resisting
98 frames based on guidelines provided by American standards (ATC-40, FEMA 356, ASCE-7
99 and ASCE-41).In these literatures the researchers used geometry of beam, column, amount of
100 reinforcement, cost of material and shuttering as their optimization variable.In the problems
101 regarding optimized design of RC components, optimizing the distribution of reinforcement
102 are also an important issue [31, 32].

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

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

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

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

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

119 iv) Usage of optimization techniques in this field is also very less compared to other
120 fields like structural health monitoring, travelling salesman problems, water
121 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 studyaims towards alleviate these shortcomings. Also Unified Particle Swarm Optimization has not been used asa cost optimization method for multistory building design in the previous studies, although it has been used very effectively in cost optimization of RC foundation [20], damage detection problems [33-38], magnetoencephalography problem [39] 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 frames following the safety and serviceability requirements of IS 456 [40] employing UPSO. Efficiency of the algorithm has been shown using two building frames (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 frames are not high-rise (Prakash et al. [7]). 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 favorablesolution of the developed optimization algorithm when a large number of experiments are carried out for cost optimization of both the buildings. 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 frames 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 frames. Detail procedure for analysis of building frames 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) [41]. 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) [42].

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 [43]. IS 875 (Part III) [44] 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.A typical velocity profile of wind load considered for analysis has been shown in Fig. 1. The dotted line represents boundary layer. In general velocity is zero at the ground and increases up to the boundary layer. Beyond boundary layer wind velocity does not changes with height. As the wind travel in upstream direction through a particular terrain, the wind profile changes and velocity and height of boundary layer increases. The distance travelled by a wind profile

on a particular terrain is called fetch length, i.e. x_1 and x_2 in Fig. 1. Upward penetration of the velocity profile at any fetch length is called developed height, i.e., h_1 and h_2 in Fig.1.Developed height increases up to gradient height, where velocity become maximum. After that even if wind travel upstream direction velocity profile remain unchanged. At this point it can said that velocity profile is fully developed for that particular terrain. In case of wind analysis of a structures generally it is assumed that wind profile is fully developed before hitting the structure. The values of a fully developed velocity profile depends on the terrain category. As the terrain category becomes rougher, gradient height increases. IS 875 (Part III) [44] have provided the multiplier for velocity profile for each terrain category.

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 $V_z$at 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.

2.1.3. Seismic loads

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

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

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. Equation 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)[45]. 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 (Fig. 2).

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 toIS 875 (Part V)[46] 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 frames.

2.2. Structural design

Beams and columns of all the frames were designed adopting Limit State Method as per the guidelines provided in the Indian Standard IS 456 [40] 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.

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

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.

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

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 and it is obtained from Table I (Appendix III) 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.

$$\tau_v = V_u / b d_e \tag{12}$$

The design shear strength $\tau_c$of concrete is calculated from IS 456 [40] or Appendix IIIfor 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 $V_{us} = V_u - \tau_c b d_e$. τ_{cmax} can be obtained from IS 456 depending on the strength of the concrete. (Table II, Appendix III) The spacing of the shear reinforcement 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.

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

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 [40] annex C-3.1. (Appendix III). $\in_{cs}$=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.

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

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.

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

$\alpha_n$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.

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

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

329 considering any particular direction bending two different cases can emerge based on the
330 position of the neutral axis as shown in Fig. 3.

331 Case 1: When the neutral axis lies within the column section (Fig. 3b), the axial load
332 carrying capacity and moment carrying capacity of the section can be calculated from Eq. 22
333 and Eq. 23 respectively.

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

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

334 where $k = x_u/D$;

335 Case 2: If neural axis lies outside the column section (Fig. 3c) the axial load carrying
336 capacity and moment carrying capacity of the section can be calculated from Eq. 24 and Eq.
337 25 respectively.

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

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

338 where,f_{ci} is stress in concrete at the level of i-th row of reinforcement and can be calculated
339 from Fig. 3d. f_{si} is stress in the i-th row of reinforcement can be obtained from Table III
340 (Appendix III). C_1 is stress co-efficient and C_2D is the distance of the centroid the concrete
341 stress block (Fig. 3d) measured from the highly compressed edge. C_1 and C_2can be obtained
342 from Eq. 26 and Eq. 27 respectively.

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

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

343 $A_{str}$is the area of the stress block (Fig.3d), and can be calculated from the Eq. 28.

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

344 M_c is the moment of the concrete stress block (Fig. 3d) about highly compressed edge is
345 obtained as per Eq. 29.

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

346 Where g is geometric properties of the parabola (Fig. 3d) obtained from Eq. 30.

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

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

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

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

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

349

350 2.3. Structural Design Optimization

351 *2.3.1. Unified Particle Swarm Optimization (UPSO)*

352 UPSO is a swarm based optimization proposed by Parsopoulos and Vrahitis [47] as a
353 upgraded version of Particle swarm optimization (PSO) [48, 49] based on the individual and
354 social behavior of flock of birds, school of fish etc. in their process of searching foods or
355 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.,thebest 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.

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

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

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

$\mu$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 $c_1 = c_2 = 2.05$[50].

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

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

1. 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}).
2. 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.
3. 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.
4. 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$\frac{l}{250}$.

Detailed algorithm of UPSO based RC beam design optimization have been presented in Algorithm 1 of Appendix II.

(B) UPSO based column design optimization

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

1. 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).

2. 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.
3. 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.

Detailed algorithm of UPSO based RC column design optimization have been presented in Algorithm 2 of Appendix II.

(C) Combined beam-column design optimization

Evemtually, all the individual optimization results of beams and columns are combined to obtain optimized design for the entire building frame in the third step. Detail algorithm for such action have been presented in Algorithm 3 of Appendix II. A flowchart for the entire developed program is presented in Fig. 4.

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 [51]).

Area of formwork (A_f) for beam and columns can be calculated from Fig. 5 as per Eq. 39 and Eq. 40 respectively

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

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

3.Numerical results and discussion

3.1. Problem definition

One G+8 L shaped building frame (Fig. 6a) and one G+10 U shaped building frame (Fig. 6b) are considered to demonstrate the efficacy of proposed algorithm. The structures were analyzed in STAADPro V8i [52] 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 [52] for critical load cases without disrupting the safety criteria. The designs ofeach beam and each columnare 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

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] (Appendix III)

$V_b = 50$m/s, $k_1 = 1$(50 years life span) (Table IV, Appendix III), k_2 is function of height (Table V of Appendix III) for terrain category 2, $k_3 = 1$ (for ground slope <3^o), $k_4 = 1$ (ordinary RC frame).

$K_d = 1$, $K_a = 1$, $K_c = 0.9$.The velocity profile of wind load analysis used in the analysis has been presented in Fig. 7.

vii) For seismic load (IS 1893(Part I)[45]

$Z = 0.16$ (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. 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 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.

x) Minimum and maximum diameter of main reinforcing bars for beams and columns are 12mm and 32mm. 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.

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. 9(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. 9(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 the frame for different grades of steel (Fe415, Fe500) and concrete (M20, M25, M30, M35) in Fig. 9(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 codalprovisions (IS 456 [40]) and the practical considerations.

3.4.2. U-shaped building frame

In this section developed UPSO based algorithm has been used to optimize the RC design of the G+10 U-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: $b \in [200,250]$, $D \in [500,300]$, $\varphi_t \in [12,16]$,$\varphi_b \in [12,16]$, $n_{cs} \in [2,5]$,$n_{cm} \in [2,5]$, $n_{ts} \in [2,5]$,$n_{tm} \in [2,5]$. Search space of design variables considered in the study in case of columns are: $b_c \in [250,400]$, $D_c \in [250,400]$, $k_x \in [0.5,1.2]$,$k_y \in [0.5,1.2]$, $\varphi_m \in [12,20]$,$n \in [2,3]$.Convergence curve for total cost for M20, M25, M30, and M35 grades of concrete along with Fe 415 steel have been plotted in Fig. 10(a). Total cost of the building frame is found to be varying within the range [2813601, 2898791] for Fe 415 steel and [2805123, 2813406] for Fe 500 steelfor different grades of concrete Hence, the maximum variation in total cost among different grades of concrete is found to be 3.03 % and 0.30 % for Fe 415 and Fe 500 steel respectively. Fig. 10(b) representsthe convergence curves of the total cost of concrete, total cost formwork and total cost of steel through all the iterations for M25 grade of concrete and Fe415 grade of steel.A bar diagram showing the total cost of frame was outlined in Fig. 10(c) for various grades of concrete (M20, M25, M30, M35) and steel (Fe415, Fe500). There is not much difference in the cost output for Fe 415 and Fe 500 as observed in the previous problem. It can be seen that for all concrete grades, Fe 500 steel yields lower cost than Fe415 steel. While beam design details of three different floors have been presented in Table 4, column design details has been presented in Table 5 for typical columns in all the floors. M25 concrete and Fe 415 steel are considered for design outputs of both the tables.The diameter and spacing of links in columns are similar to the results reported for L-shaped building frame.The present algorithm is adequately apt, suitably

adaptable and effectively potent to provide cost optimized design of reinforced concrete buildings U- shaped building frame considering all the codal provisions(IS 456 [40])and practical considerations.

3.5. Statistical Analysis

As optimization algorithms are based on random processes, possibility of coming up with a different solution is quite common when a large number of experiments are involved. However, the probability of the optimized objective function value to lie within an acceptable range decides the robustness of the optimization algorithm. For that, a large number of experiments are required and each and every solution should be checked whether it is in the acceptable range of the optimum solution. It is a very time consuming and rigorous to conduct a huge number of experiments (i.e. 5000 or 10,000) on the developed algorithm and check every solution manually. In that case, the Monte Carlo simulation (Metropolis and Ulam[53], Metropolis [54]) is used to assess the variations of the solutions of the developed algorithm for large of number of experiments. However, the variations in results should be within an acceptable range for an engineer to accept the results. Monte Carlo simulation is a mathematical numerical method that uses random draws to perform calculations and complex problems. The method was first introduced by Stanislaw Ulam (Eckhardt [55]) and is based on the game of dice, roulette etc. Monte Carlo simulation can be used create large number of solutions of the problem in hand numerically from a small available number of solutions based on a normal distribution curve or bell curve. Then the normal distribution curve can be used to obtain the forecast of the favorable solution and its probability. The probability of the occurrence of the favourable solution among a large number of experiments can be estimated in the normal distribution curve. This decides the robustness of the developed algorithm.

In the present study aMonte Carlo simulation for 5000 probable solutions (total cost of frames) of the developed algorithm was performed using MS excel.The expression for probable solution of each simulation was worked out using the following expression in Eq. 41.

$$tot\_cost_{i+1} = tot\_cost_i \times e^{D_r + R}, i = 1,2,\dots,5000 \tag{41}$$

where, D_r and R can be calculated from Eq. 42 and Eq. 43 respectively.

$$D_r = \mu_E - \sigma^2/2 \tag{42}$$

$$R = \sigma \times normsinv(rand(0,1)) \tag{43}$$

Where, $normsinv$ is an excel function which returns probability corresponding to the standard normal distribution with a mean of zero and a standard deviation of one. μ_E and σ are respectively mean and standard deviation of the 10 number of available total cost values. μ_E is considered as tot_cost_i for calculating the values for first simulation.

Summary of results of 10 numbers of experiments obtained employing the optimization algorithm along with the key solution of Monte Carlo simulation is presented in Table 6 and Table 7 for L-shaped building frame and U-shaped building frame respectively. Total cost of L-shaped and U-shaped building frame are found to be lying in the ranges of [3792373.03, 3923637.97] and [2833889.28, 2887321.15] respectively after 5000 simulations. These ranges are divided into 20 equal divisions. The probability of total cost falling within each interval was estimated considering 5000 forecasted solutions. These probability values obtained for 20 intervals are plotted in Fig. 11 and Fig. 12 for L-shaped and U-shaped frames respectively. Both the plots clearly depict the nature of a normal distribution curve.The most likely solution is at the middle of the curves, meaning there is an equal chance that the actual solutionwill be higher or lower than that value. The key percentile shows the likely values of the total cost of frames.For example, the key percentile value against the 25% shows that there is 25% chance that the developed algorithm will provide this total cost. Similarly, key percentiles for 50% and 75% chance have also been evaluated. Key percentile values are presented in Table 6 and Table 7 for L-shaped and U-shaped building frame respectively.Also, the total cost corresponding to 99% for both the frames also proves that there is negligiblescope for design failure by the developed algorithm. Loss percentile is calculated keeping the maximum total cost to be Rs. 39, 00,000 and Rs. 28, 80,000 for L-shaped and U-shaped frame respectively. The loss percentile shows that there is only 2% chance of the total cost to exceed Rs. 39, 00,000 (Table. 6) for L-shaped frame and 0.6% chance to exceed Rs. 28, 80,000 (Table 7) for U-shaped frame, which are very close to first simulation values. The loss percentages are found to be very negligible. Therefore, the optimum design of the frames using the developed algorithm is not only found safe in every experiments but it also provide an optimum overall cost almost every time.

3.6. Feasibility analysis in terms of safety

In the present algorithm beams and columns are optimized separately and added later to obtain the optimized design for the entire frame. Therefore, the stiffness redistribution among beams and columns due to varying sizes of them in each iteration of the optimization have not been taken into account. Thus, from mathematical point of view slight error has been imparted into the analysis. In the present algorithm a few measures have been taken to ensure the safety of the structures. The analysis and preliminary design of the buildings are performed in STAAD Pro, in such way that sizes of all beams and columns has been chosen in a conservative manner. It was then made sure that design was safe as per design standard. Next, design forces (shear force, moment and axial forces) have been taken from the conservative STAAD Pro model and fed into the developed optimization algorithm in MATLAB. In that way the sizes of beams and columns only reduced during optimization procedure as can be seen from Table 2-5. The design force is kept same as the preliminary analysis of STAAD Pro. In that way, it has been made sure that design was always carried out for design forces higher than the actual design forces obtained during the course of the optimization program. After optimization is done, optimized member sizes are incorporated in STAADPro model to compare the design forces of optimized structures with actual STAADPro model. Table 8-9 have been presented showing the comparison between the design forces of the structures before and after optimization for a typical fourth floor of one building (L-shaped). Actual design forces and moments of optimized structure are observed to be significantly lower than the design forces and moments of the structure before optimization. So, the optimized design has been indeed carried out for higher design forces and moments than the actual. Therefore, the aforementioned error has not influence the safety of the buildings significantly enough to violate the safety criteria provided by the design standard. So, from practical civil engineering point of view the design results obtained from present algorithm is feasible and safe enough to be used for cost-optimum design of RC building frame.

4.Conclusion

In the present study, an UPSO based optimization algorithm has been developed in MATLAB [56]environment to find cost optimum design of reinforcement concrete building frame considering the codal specifications of safety and serviceability of IS 456 [33] along with the consideration for the construction requirements in practical field.

Two building frames namely G+8 L-shaped frame and G+10 U-shaped frame have been adopted to demonstrate the efficacy of the developed algorithm. Popular design and analysis software STAADPro. V8i [52] 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 frames hasbeen 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 frames of any shape and height with profound accuracy.Further, Monte Carlo simulationperformed assures the consistency and robustness of the developed algorithm with almost hundred percent design success. This further confirms that the algorithm can be effectively used for optimal design of any type of reinforced concrete buildings having different design constraints. 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.

Data availability statement

All the required data presented in the manuscript itself. Any further dataimportant to the readers can be made available as per their request.

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Conflict of interest

On behalf of all authors, the corresponding author states that there is no conflict of interest.

Replication of results

The authors hereby statethat they are willing to share all the codes and numerical data needed to reproduce the figures.

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APPENDIX I. Notation

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

870 $E_c =$ Short term elasticity modulus for beam

871 $I_{eff} =$ Effective moment of inertia for short term deflection of beam

872 $I_r =$ Moment of inertia of cracked section of beam

873 $I_{gr} =$ Gross moment of inertia of beam

874 $M_r =$Cracking moment

875 $f_{cr} =$ Modulus of rupture of concrete

876 $y_t =$ Distance from the centroidal axis of gross section, neglecting the
877 reinforcement, to extreme fibre in tension

878 $M =$ Maximum moment under service load for beam

879 $z =$ Lever arm of the beam section

880 $x =$ Depth of the neutral axis for beam

881 $b_w =$ Breadth of web for beam

882 $b_b =$ Breadth of compression face for beam

883 $f_3 =$ Constant depending upon the support condition of beam

884 $\varphi_{cs} =$ Shrinkage curvature for beam

885 $f_4 =$ Factor depending on percentage of tensile and compressive
886 reinforcement for beam

887 $\in_{cs} =$ Ultimate shrinkage strain of concrete for beam

888 $l =$ Length of the span of beam

889 $a_{i,cc} =$ Initial plus creep deflection of beam due to permanent loads

890 $E_{ce} =$ 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

b_c =width of column

l_c =Length of the column.

A_c = Area of the concrete in column section

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.

APPENDIX II. Algorithm of the developed program

Algorithm1 Beam design optimization

Input: SS=10, dim=8 (b, D, φ_t, φ_b, n_{cs}, n_{cm}, n_{ts}, n_{tm}), GM=1, $MaxIt$=50, $nexp$=10, $Smin = [b_{min}, D_{min}, \varphi_{min}, \varphi_{min}, n_{min}, n_{min}, n_{min}, n_{min}]$, $Smax = [b_{max}, D_{max}, \varphi_{max}, \varphi_{max}, n_{max}, n_{max}, n_{max}, n_{max}]$

Output: Optimized variable $S_{opt,beam}$, Optimized cost $F_{opt,beam}$

1: **for** $Ex=1:nexp$
2: Initialization: $t \leftarrow 0$
3: Initialization: Swarm position $S(t)$: $b \in [b_{min}, b_{max}]$, $D \in [D_{min}, D_{max}]$
$\varphi_t, \varphi_b \in \varphi$, $\varphi = [12,16,20,25,32]$, $\varphi_t, \varphi_b \in [\varphi_{min}, \varphi_{max}]$, $n_{cs}, n_{cm}, n_{ts}, n_{tm} \in [n_{min}, n_{max}]$
Swarm velocity: $H(t) \in [-H_{max}, H_{max}]$
4: Maintain: Constraint for beam design of Section 2.3.2
5: Perform: Beam design (flexure and shear), deflection check (Section 2.2.1)
6: Calculate: Cost of beam F_{beam} (Eq. 38)
7: Calculate: best positions $p(t)$, $g(t)$ and $l(t)$
8: Calculate: best particle $x_{opt,beam}$, corresponding objective function $f_{xopt,beam}$
9: Initialization: $STOP$=0
10: **while** $STOP$=0 **do**
11: Update: $t \leftarrow t+1$
12: Calculate: $\mu(t)$ (Eq.37)
13: Update: $H(t)$ (Eq.34) and $S(t)$ (Eq.33)
14: Maintain: Constraint for beam design of Section 2.3.2
15: Perform: Beam design (flexure and shear), deflection check (Section 2.2.1)
16: Calculate: Cost of beam F_{beam} (Eq.38)
17: Update: $p(t)$, $g(t)$ and $l(t)$
18: Update: $x_{opt,beam}$, $f_{xopt,beam}$
19: **if** $f_{xopt} \leq GM$ or $t=MaxIt$ **then**
20: $STOP$=1
21: **else**
22: Go to step 11
23: **end if**
24: **end while**
25: Calculate: Optimize variable S_{opt}, Optimized cost F_{xopt}
26: **end for**
27: Calculate: Final Optimize variable $S_{opt,beam}$, Optimized cost $F_{xopt,beam}$ (best out of $nexp$ experiments)

Algorithm2 Column design optimization

Input: SS=10, dim= 6 (b_c, D_c, k_x, k_y, φ_m, n),GM=1,$MaxIt$=50, $nexp$=10, $Smin = [b_{c,min}, D_{c,min}, k_{x,min}, k_{y,min}, \varphi_{min}, n_{min}]$, $Smax = [b_{c,max}, D_{c,max}, k_{x,max}, k_{y,max}, \varphi_{max}, n_{max}]$

Output: Optimized variable $S_{opt,column}$, Optimized cost $F_{opt,column}$

1: **for** $Ex=1:nexp$
2: Initialization:$t \leftarrow 0$
3: Initialization:Swarmposition$S(t)$:
$b_c \in [b_{c,min}, b_{c,max}]$, $D_c \in [D_{c,min}, D_{c,max}]$, $k_x \in [k_{x,min}, k_{x,max}]$, $k_y \in [k_{y,min}, k_{y,max}]$
$\varphi_m \in \varphi$, $\varphi = [12,16,20,25,32]$, $\varphi_m \in [\varphi_{min}, \varphi_{max}]$, $n \in [n_{min}, n_{max}]$

Swarm velocity$H(t) \in [-H_{max}, H_{max}]$
4: Maintain: Constraint for column design of Section 2.3.2
5: Perform: Column design for longitudinal reinforcement and lateral ties (Section 2.2.2)
6: Calculate: Cost of column F_{column} (Eq.38)
7: Calculate: best positions $p(t)$, $g(t)$and $l(t)$
8: Calculate: best particle $x_{opt,column}$, corresponding objective function $f_{xopt,column}$
9: Initialization: $STOP$=0
10: **while** $STOP$=0 **do**
11: Update: $t \leftarrow t+1$
12: Calculate: $\mu(t)$ (Eq.37)
13: Update: $H(t)$ (Eq.34) and $S(t)$ (Eq.33)
14: Maintain: Constraint for column design of Section 2.3.2
15: Perform: Column design for longitudinal reinforcement and lateral ties (Section 2.2.2)
16: Calculate: Cost of column F_{column} (Eq.38)
17: Update: $p(t)$,$g(t)$and$l(t)$
18: Update: $x_{opt,column}$, $f_{xopt,column}$
19: **if** $f_{xopt} \leq GM$or$t=MaxIt$ **then**
20: $STOP$=1
21: **else**
22: Gotostep11
23: **end if**
24: **end while**
25: Calculate: Optimize variable S_{opt}, Optimized cost F_{xopt}
26: **end for**
27: Calculate: Final Optimize variable $S_{opt,column}$, Optimized cost $F_{xopt,column}$ (best out of $nexp$ experiments)

Algorithm3 Combined optimized cost of frame

Input: X_{min},X_{max} (as per Section.3.4), Design forces: Moment and shear force for beams, Axial forces and biaxial moments for columns from STAAD

Output: Optimized designs: $X_{opt,beam}$, $X_{opt,column}$, Optimized cost: $F_{opt,beam}$, $F_{opt,column}$

1: Beam design optimization (**Algorithm 1**): One by one from ground floor to top floor. Update X_{min},X_{max} for each floor (Section 3.3).

2: Column design optimization (**Algorithm 2**): One by one from top floor to ground floor. Update X_{min},X_{max} for each floor (Section 3.3).

3: Find out optimum beam design and column design

3: Calculate total optimize cost of frame=$\sum F_{opt,beam} + \sum F_{opt,column}$

APPENDIX III

Parameters used in design of beam:

For doubly reinforced beam

Table I Stress in compression reinforcement (f_{sc}), (N/mm^2) in doubly reinforced beams with cold worked bars

SP16 Table F		
d'/d	f_y =415 N/mm^2	f_y =500 N/mm^2
0.05	355	424
0.1	353	412
0.15	342	395
0.2	329	370

For design of beam in shear

Design shear strength $\tau_c = \frac{0.85\sqrt{0.8f_{ck}}(\sqrt{1+5\beta}-1)}{6\beta}$, $\beta = \frac{0.8f_{ck}}{6.89P_t} \geq 1$, $P_t = \frac{100A_{st}}{bd}$, A_{st} is area of tensile reinforcement, b, d are width and effective depth.f_{ck}= Grade of concrete

Table II Maximum Shear Stress ($\tau_{c,max}$), N/mm^2 (IS 456)

Table 24 (Clauses B-2.3, B-5.2.3. B·5.2.3. t, B.5.5,1 and B-6.3.1)						
f_{ck} (N/mm^2)	15	20	25	30	35	40
$\tau_{c,max}$(N/mm^2)	2.5	2.8	3.1	3.5	3.7	4

For deflection check in beam

f_3 =0.5 for cantilevers, 0.125 for simply supported members, 0.086 for continuous at one end and 0.063 for fully continuous members

$f_4 = 0.72\frac{P_t-P_c}{\sqrt{P_t}} \leq 1.0$ for$0.25 \leq P_t - P_c \leq 1.0$

$= 0.65\frac{P_t-P_c}{\sqrt{P_t}} \leq 1.0$ for $P_t - P_c \geq 1.0$, $P_t = \frac{100A_{st}}{bd}$, $P_c = \frac{100A_{sc}}{bd}$, A_{sc} is area of compressive reinforcement.

Parameters used in design of column:

Table III Salient points on the design stress-strain curve for cold-worked bars

SP 16 (Table A)			
f_y =415 N/mm^2		f_y=500 N/mm^2	
Strain	Stress (N/mm^2)	Strain	Stress (N/mm^2)
0.00144	288.7	0.00174	347.8
0.00163	306.7	0.00195	369.6
0.00192	324.8	0.00226	391.3
0.00241	342.8	0.00277	413
0.00276	351.8	0.00312	423.9
0.0038	360.9	0.00417	434.8

Parameters used in wind analysis:

Table IV Risk coefficient (k_1) for structures in different wind speed zones

IS 875 part 3 2015 Table 1 (Clause 6.3.1)							
Class of structures	Design life of structures (years)	k_1 factor for basic wind speed (m/s)					
		33	39	44	47	50	55
All general structures	50	1.00	1.00	1.00	1.00	1.00	1.00
Temporary structures	5	0.82	0.76	0.73	0.71	0.70	0.67
Structures with low degree of hazards	25	0.94	0.92	0.91	0.90	0.90	0.89
Important building	100	1.05	1.06	1.07	1.07	1.08	1.08

Table V Factors to obtain design wind speed variation with height in different terrains (k_2)

IS 875 part 3 2015 Table 2 (Clause 6.3.2.2.)				
Height (m)	Height multiplier (k_2) for terrain categoty			
	1	2	3	4
10	1.05	1.00	0.91	0.80
15	1.09	1.05	0.97	0.80
20	1.12	1.07	1.01	0.80
30	1.15	1.12	1.06	0.97
50	1.2	1.17	1.12	1.1
100	1.26	1.24	1.2	1.2
150	1.3	1.28	1.24	1.24
200	1.32	1.3	1.27	1.27
250	1.34	1.32	1.29	1.28
300	1.35	1.34	1.31	1.3
350	1.35	1.35	1.32	1.31
400	1.35	1.35	1.34	1.32
450	1.35	1.35	1.35	1.33
500	1.35	1.35	1.35	1.34

List of Tables

1077 **Table 1**. Review of all the optimization techniques used by the researchers for
1078 solving design optimization problem of RC structures

Authors	Optimization algorithm
Prakash et al.[7]	Simplex and Lagrangian optimization method
Chakrabarty [8]	Geometric programming
Coello et al.[9], Rajeev and Krishnamoorthy[10], Chaudhuri and Maity[20]	Genetic algorithm (GA)
Dole et al. [11]	polynomial optimization technique
De Medeiros et al. [12]	Simulated annealing (SA)
Nigdeli and Bekdaş [13]	random search technique (RST)
Uz et al. [14], Kaveh and Behnam [21]	charged system search (CSS)
Preethi and Arulraj [16]	sequential quadratic programming (SQP)
Bekdaş and Niğdeli [17]	Teaching-learning-based-optimization (TLBO)
Bekdaş and Nigdel[22]	Harmony search (HS)
Aga and Adam[23]	Artificial neural network (ANN)
Gharehbaghi and Khatibinia[24]	intelligent regression model (IRM) combined with Particle swarm optimization (PSO)
Esfandiary et al. [25, 29]	decision-making Particle Swarm Optimization (DMPSO)
Kulkarni and Bhusare[26]	Response Surface Method (RSM)
Tapao and Cheerarot[28]	Artificial bee colony (ABC)
RazmaraShooli et al. [30]	GA-PSO algorithm
Chaudhuri and Maity [20]	Unified particle swarm optimization (UPSO)

1079
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Tables 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
			Top	Bottom	Top	Bottom	8mm φ
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
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

Table 3. TypicalColumndesign 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

1086 **Table 4.**Beam design details for three different floors- U shaped building frame

Floors	Beam types	Size (mm x mm)	Support (mm)		Mid-span (mm)		Stirrups spacing
			Top	Bottom	Top	Bottom	8mm φ
Top	B1,B2, B3, B4, B5, B6, B7, B8	200x 380	2-12φ	2-16 φ	2 -12 φ	2-16φ	300
9th	B1	220x 430	5- 12φ	2- 16 φ	2 -12 φ	3-16φ	300
	B2, B3	220x 440	5- 12φ	2- 16 φ	2 -12 φ	3-16φ	300
	B4	230x 450	5- 12φ	2- 16 φ	2 -12 φ	3-16φ	300
	B5	220x 430	4- 16φ	2- 16 φ	2 -16 φ	3-16φ	300
	B6, B7	200x 370	4- 16φ	2- 16 φ	2 -16 φ	2-16φ	300
	B8	220x 350	4- 16φ	2- 16 φ	2 -16 φ	3-16φ	300
1st	B1	240x 470	5- 16 φ	3-16 φ	2 -16 φ	4-16φ	300
	B2	240x 480	4- 16φ	2- 16 φ	2 -16 φ	3-16φ	300
	B3	250x 480	4- 16φ	2- 16 φ	2 -16 φ	4-16φ	300
	B4	230x 450	5- 16 φ	3-16 φ	2 -16 φ	3-16φ	300
	B5	250x 500	4- 16φ	2- 16 φ	2 -16 φ	4-16φ	300
	B6	240x 430	5- 16 φ	3-16 φ	2 -16 φ	3-16φ	300
	B7	240x 440	5- 16 φ	3-16 φ	2 -16 φ	3-16φ	300
	B8	240x 470	5- 16 φ	3-16 φ	2 -16 φ	3-16φ	300

1087

1088 **Table 5.**Typicalcolumn design details for all floor –U shaped building frame

Floors	Column numbers							
	C1		C3		C5		C9	
	Size (mmxmm)	Reinforcement	Size (mmxmm)	Reinforcement	Size (mmxmm)	Reinforcement	Size (mmxmm)	Reinforcement
10	330x320	4-12 φ	330x260	4-12 φ	270 x310	4-12 φ	310x310	4-12 φ
9	380x350	4-16 φ	350x360	4-12 φ	330x350	4-16 φ	390x320	4-16 φ
8	390x370	4-20 φ	390x370	4-20 φ	350x370	4-20 φ	400x380	4-20φ
7	400x390	4-20 φ	400x380	4-20φ	360x390	4-20φ	400x390	4-20φ
6	400x400	4-20 φ	400x400	4-20 φ	380x400	4-20φ	400x400	4-20φ
5	400x400	4-20 φ	400x400	4-20 φ	390x400	8-20φ	400x400	4-20φ
4	400x400	4-20 φ	400x400	4-20 φ	390x400	8-20φ	400x400	4-20φ
3	400x400	4-20 φ	400x400	4-20 φ	390x400	8-20φ	400x400	4-20φ
2	400x400	4-20 φ	400x400	8-20 φ	400x400	8-20 φ	400x400	8-20 φ
1	400x400	4-20 φ	400x400	8-20 φ	400x400	8-20 φ	400x400	8-20 φ
G	400x400	8-20 φ	400x400	8-20 φ	400x400	8-20 φ	400x400	8-20 φ

1089

Table 6. Monte Carlo Result summary (L shaped framed)

RESULT SUMMARY				
Result after 10 number of experiments				
	Total concrete Rs.	**Total formwork** Rs.	**Total steel** Rs.	**Total cost** Rs.
Mean	1044706.57	883111.02	1928855.23	3856672.82
Standard Deviation (S)	8777.08	4033.53	20228.66	20861.87
Max value	1057596.73	888445.8	1954894.32	3885445.32
Min value	1026706.01	875166.6	1894407.89	3827581.44
MONTE CARLO simulation				
First simulation	1033501.77	875617.80	1934664.40	**3885044.13**

After 5000 Simulation	
Total cost (Rs.)	
Mean	**3856471.18**
Standard Deviation (S)	**22264.92**
Min	**3792373.03**
Max	**3923637.97**
Difference	131264.93
Increment	6563.24
Design failure	0
Key Percentile	
25%	**3841858.21**
50%	**3856822.15**
75%	**3872005.02**
99%	**3926568.36**
Loss percentile	**2**

Table 7. Monte Carlo Result summary (U shaped framed)

Result Summary				
Results after 10 number of experiments				
	Total concrete Rs.	**Total formwork** Rs.	**Total steel** Rs.	**Total cost** Rs.
Mean	830104.70	751285.78	1277901.51	2859292.00
Standard Deviation (S)	4042.70	2251.79	6651.91	9282.60
Max value	836727.36	754809.66	1287597.09	2875499.09
Min value	824728.59	747885.6	1270301.75	2847013.56
MONTE CARLO simulation				
First simulation	820513.55	754301.59	1272679.41	**2863711.65**

After 5000 Simulation	
Total cost (Rs.)	
Mean	**2859591.13**
Standard Deviation (S)	**7680.10**
Min	**2833889.28**
Max	**2887321.15**
Difference	53431.86
Increment	2671.59
Design failure	0
Key Percentile	
25%	2853784.32
50%	2859260.04
75%	2864541.17
99%	2881806.97
Loss percentile	**0.6**

1096 **Table 8.**Comparison of beam design forces before and after optimization(L shaped frame)

Fourth floor	Beam No.	Size (mm x mm)	Bending moment (kN-m)			Shear force (KN)
			Support top	Support bottom	Midspan bottom	
Before	B1	270 x 530	249	117	51	155
After			233	111	50	144
Before	B2	280 x 490	255	130	50	150
After			242	122	60	144
Before	B3	240 x 400	260	133	61	152
After			243	125	60	144
Before	B4	240 x 440	263	137	61	153
After			260	143	64	144
Before	B5	220 x 430	260	131	60	153
After			260	130	54	144

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Table 9. Comparison of column design forces before and after optimization(L shaped frame)

Fourth floor	Column No.	Size (mm x mm)	Axial force (kN)	Bending moment (kN-m)	
				M_z	M_y
Before	C1	450 x 450	1587	254	9
After			1575	207	3
Before	C2	450 x 450	1262	250	37
After			1231	216	35
Before	C3	450 x 450	1125	41	184
After			1096	39	173
Before	C4	420 x 450	1537	6	232
After			1505	4	205
Before	C5	450 x 450	1565	1.49	250
After			1507	1.3	224
Before	C6	430 x 450	1217	40	227
After			1210	37	219
Before	C7	450 x 450	1523	221	6
After			1508	204	3.6
Before	C8	400 x 400	1262	181	40
After			1247	175	38
Before	C9	450 x390	1245	184	41
After			1230	178	39
Before	C10	450 x430	1123	148	39
After			1106	144	38
Before	C11	450 x 400	1246	230	43
After			1186	227	41
Before	C12	450 x 450	1110	220	43
After			997	199	40

List of Figures

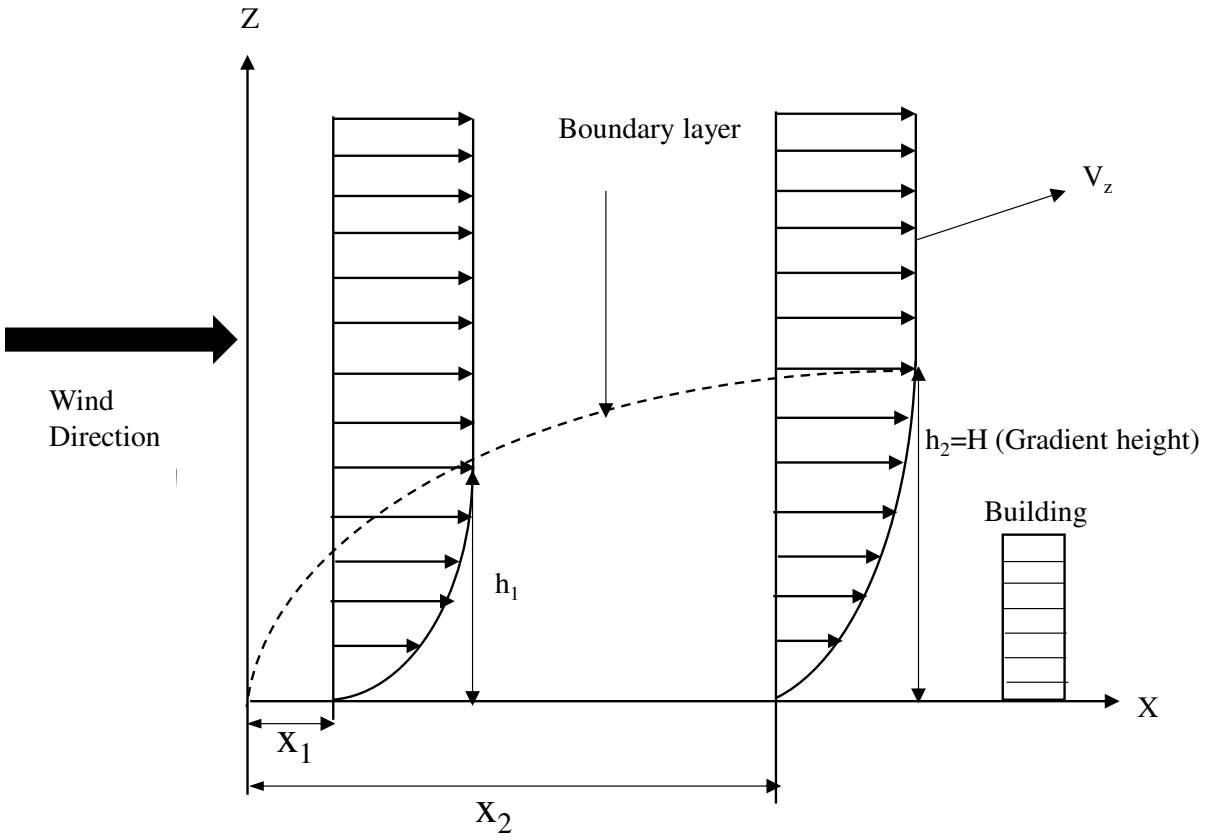


Fig.1. Typical velocity profile of wind moving through a particular terrain (x_1, x_2=fetch length; h_1, h_2=developed height)

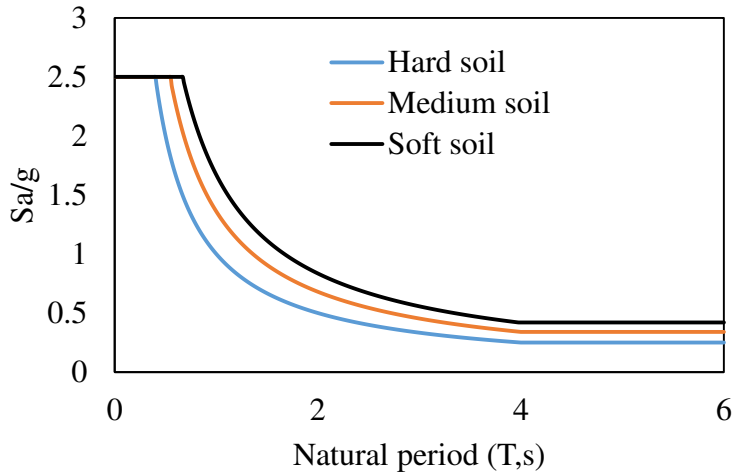


Fig.2. Design acceleration response spectrum of IS 1893-2016(Part-III)[43]

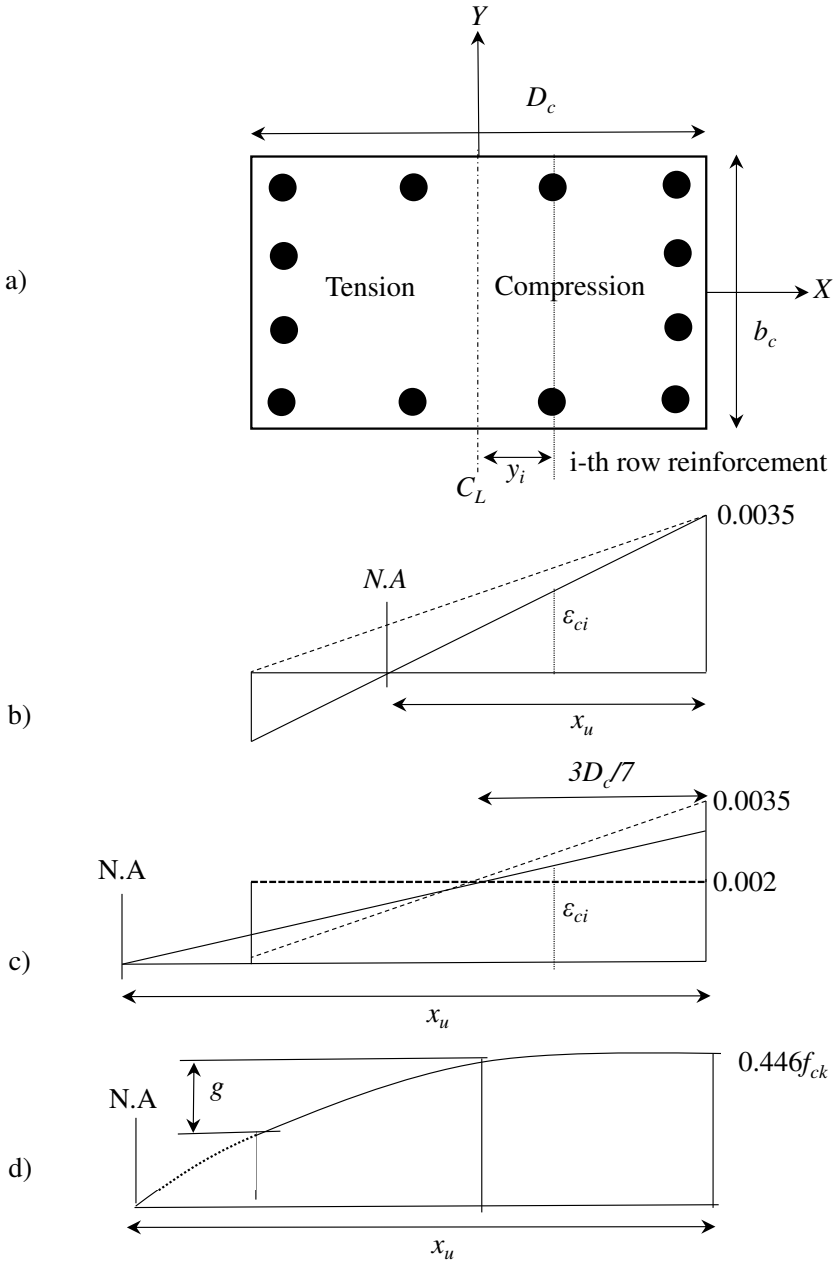
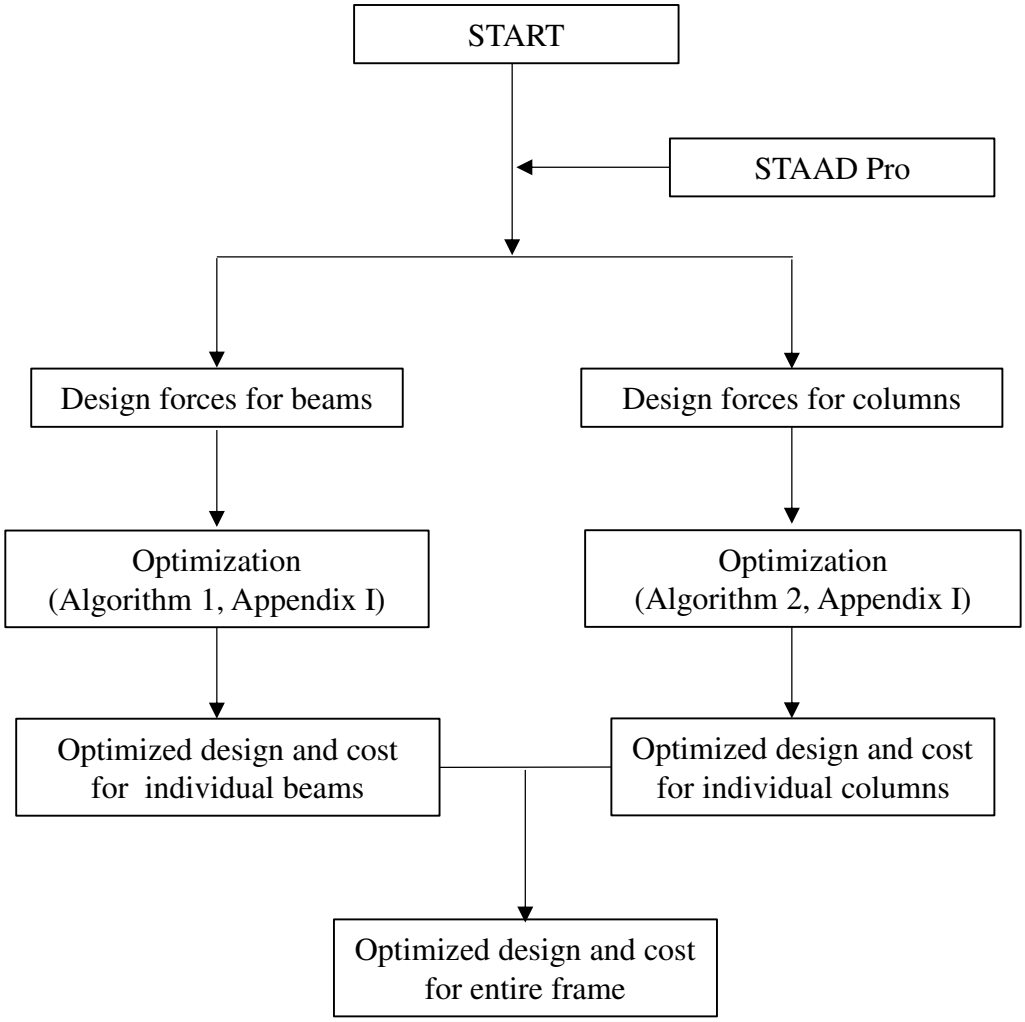


Fig. 3.a) Typical reinforcement detail of column section b) Strain diagram of concrete section when neutral axis lies inside the section c) Strain diagram of concrete section when neutral axis lies outside the section d) Stress diagram of concrete section when neutral axis lies outside the section



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1156 **Fig.4.** Flowchart for cost optimization of whole frame

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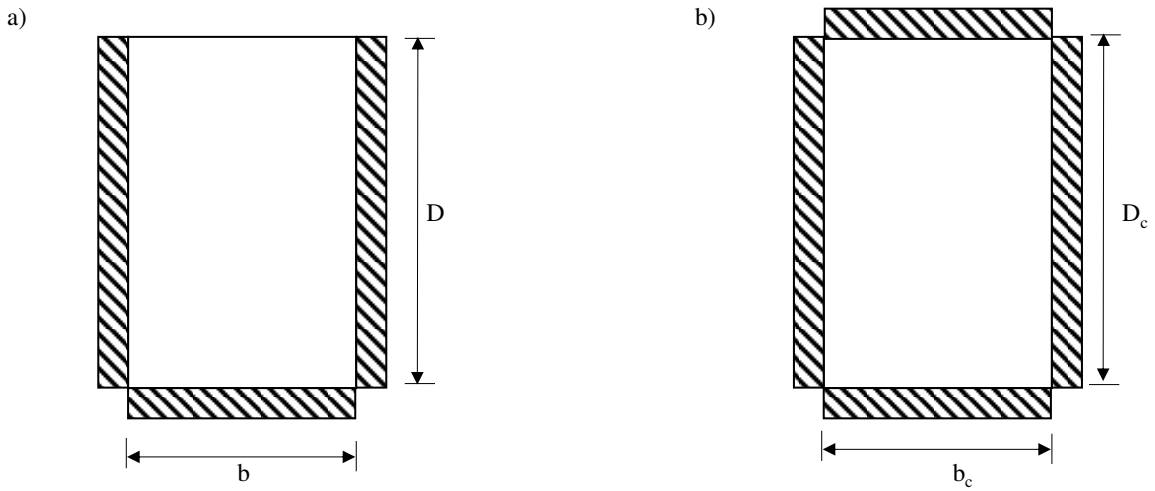
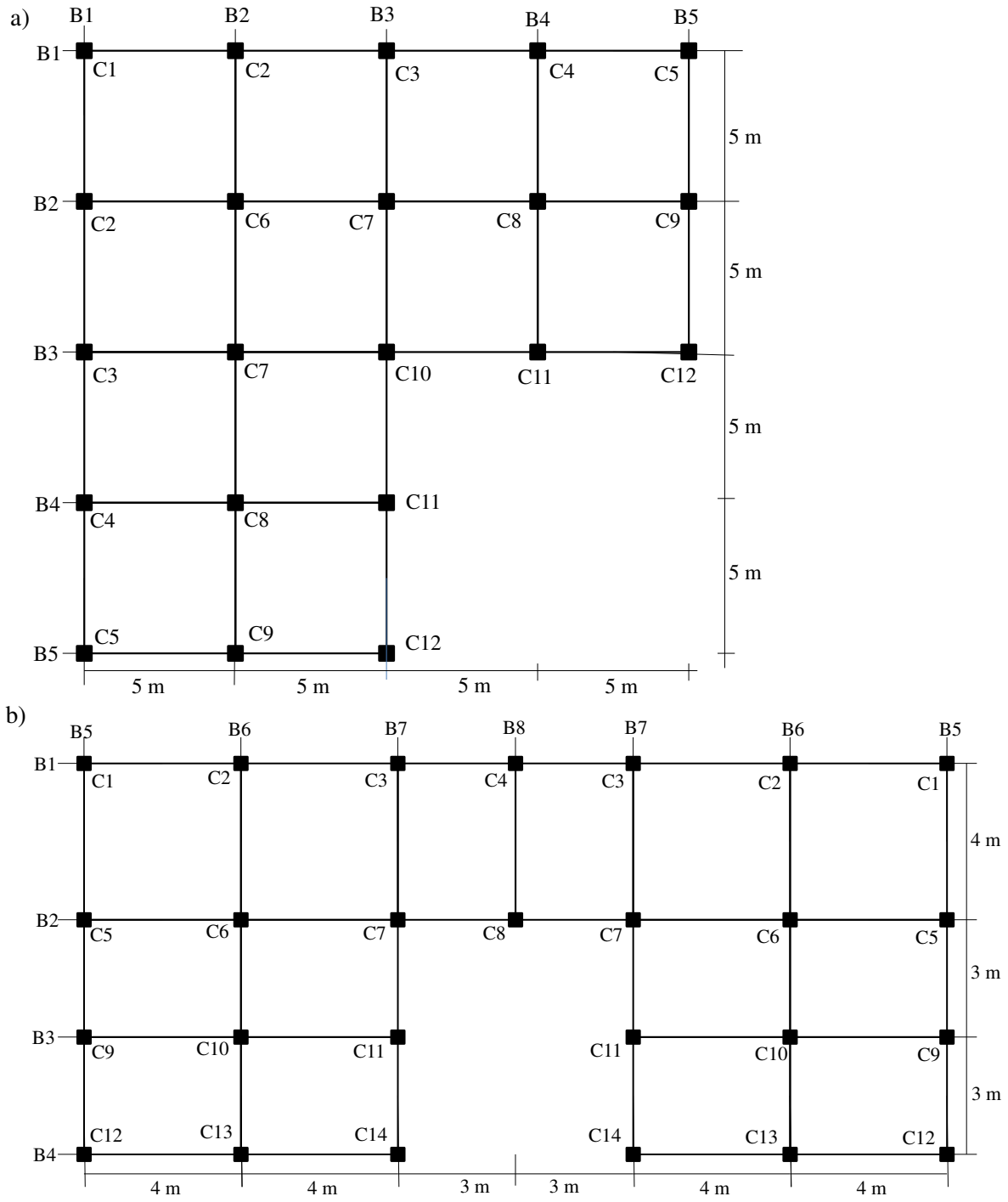


Fig.5. Formwork profile for member cross section: a) Beam b) Column



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1163 **Fig. 6**. Typical floor grid plan of the building frames showing beams and column positions a)

1164 L shaped building b) U shaped building

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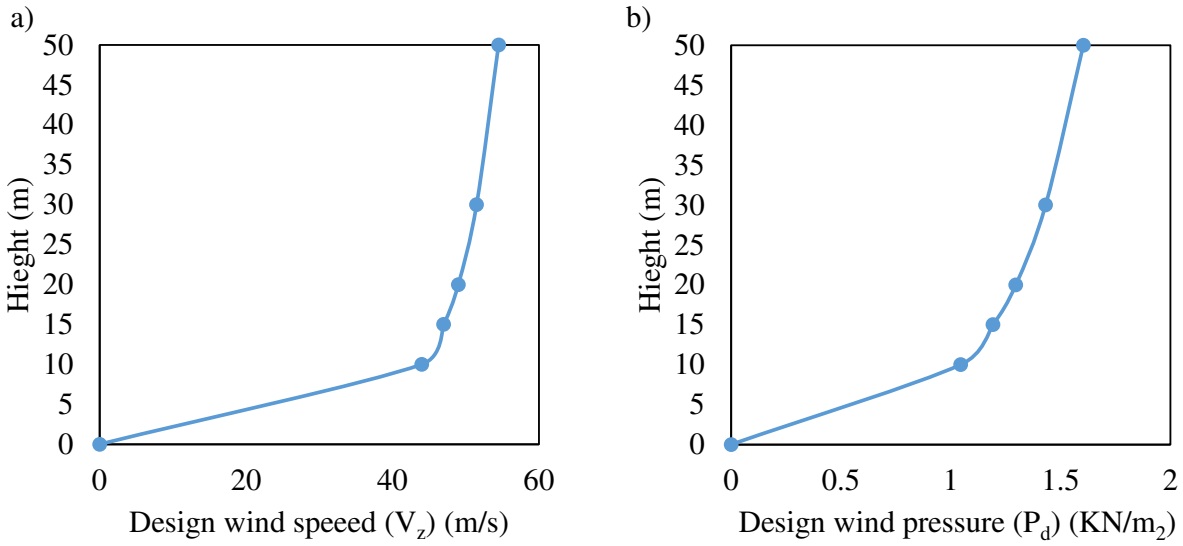

Fig. 7.Wind profile used wind load analysis a) Velocity profile b) Pressure profile

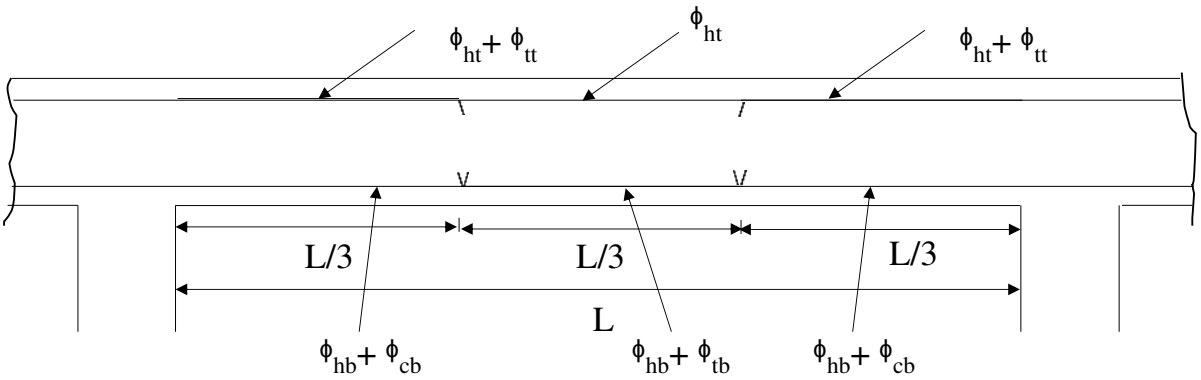

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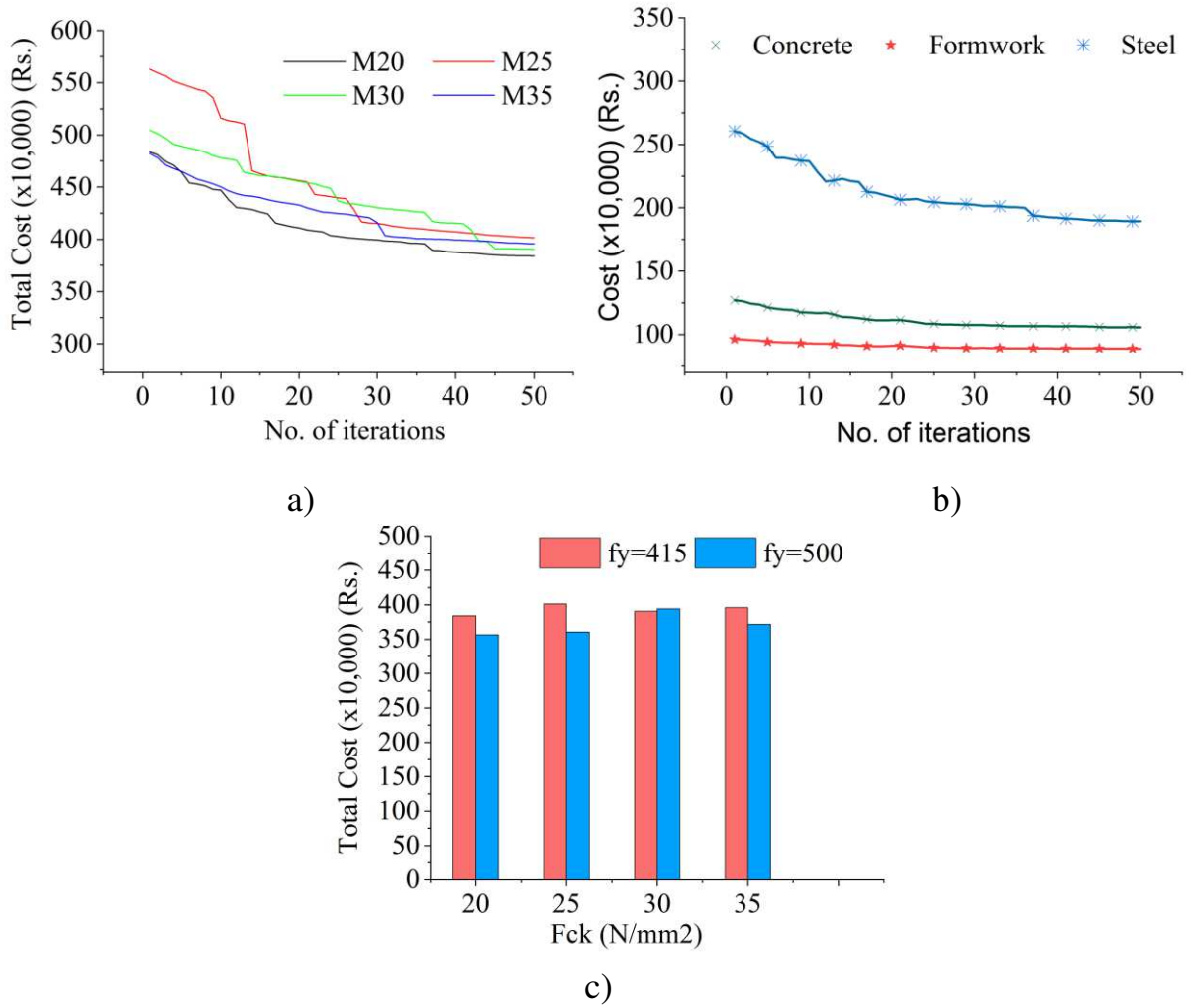
Fig. 8. Schematic diagram of continuous beam showing the details of curtailments in bars



1179

1180 **Fig. 9.** Design optimization results for L-shaped building frame. (a) convergence curve of
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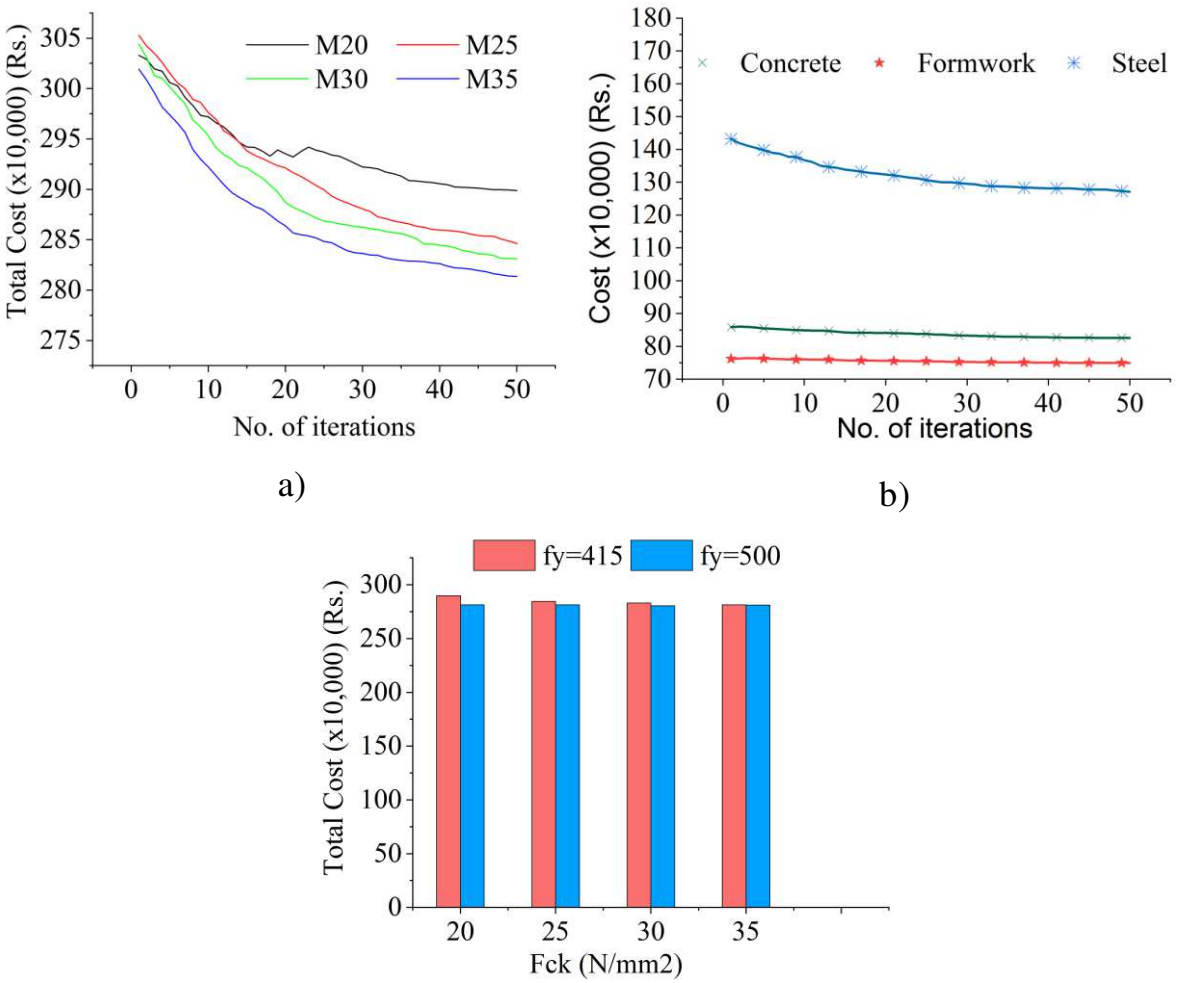

1185

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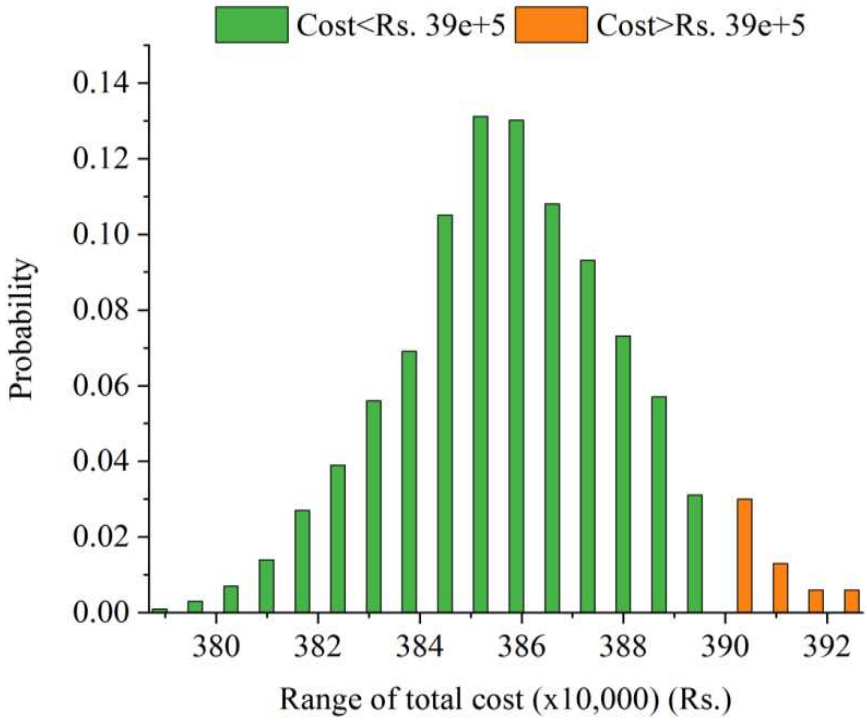

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1192 **Fig.11.** Probability distribution for 5000 simulated solutions (L shaped frame)

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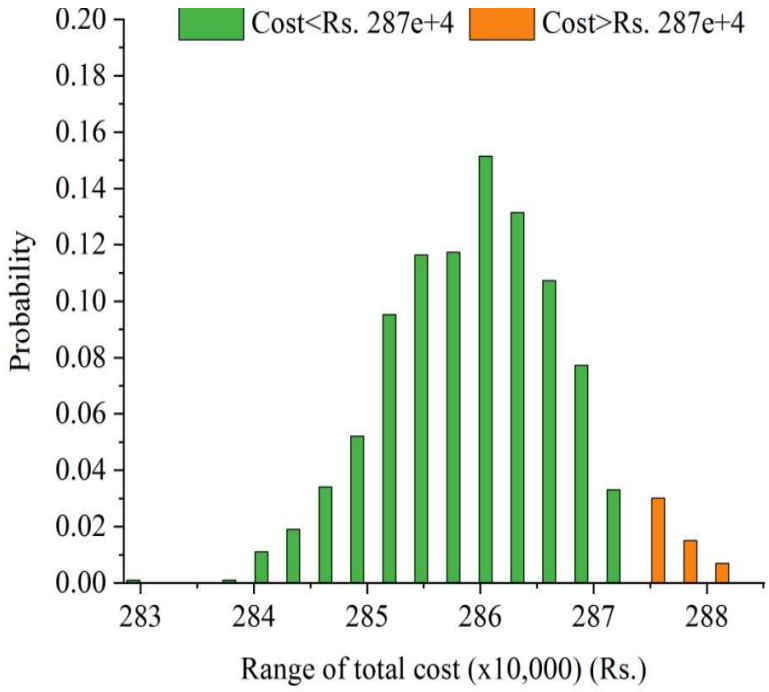

1194

1195 **Fig. 12.**Probability distribution for 5000 simulated solutions (U shaped frame)

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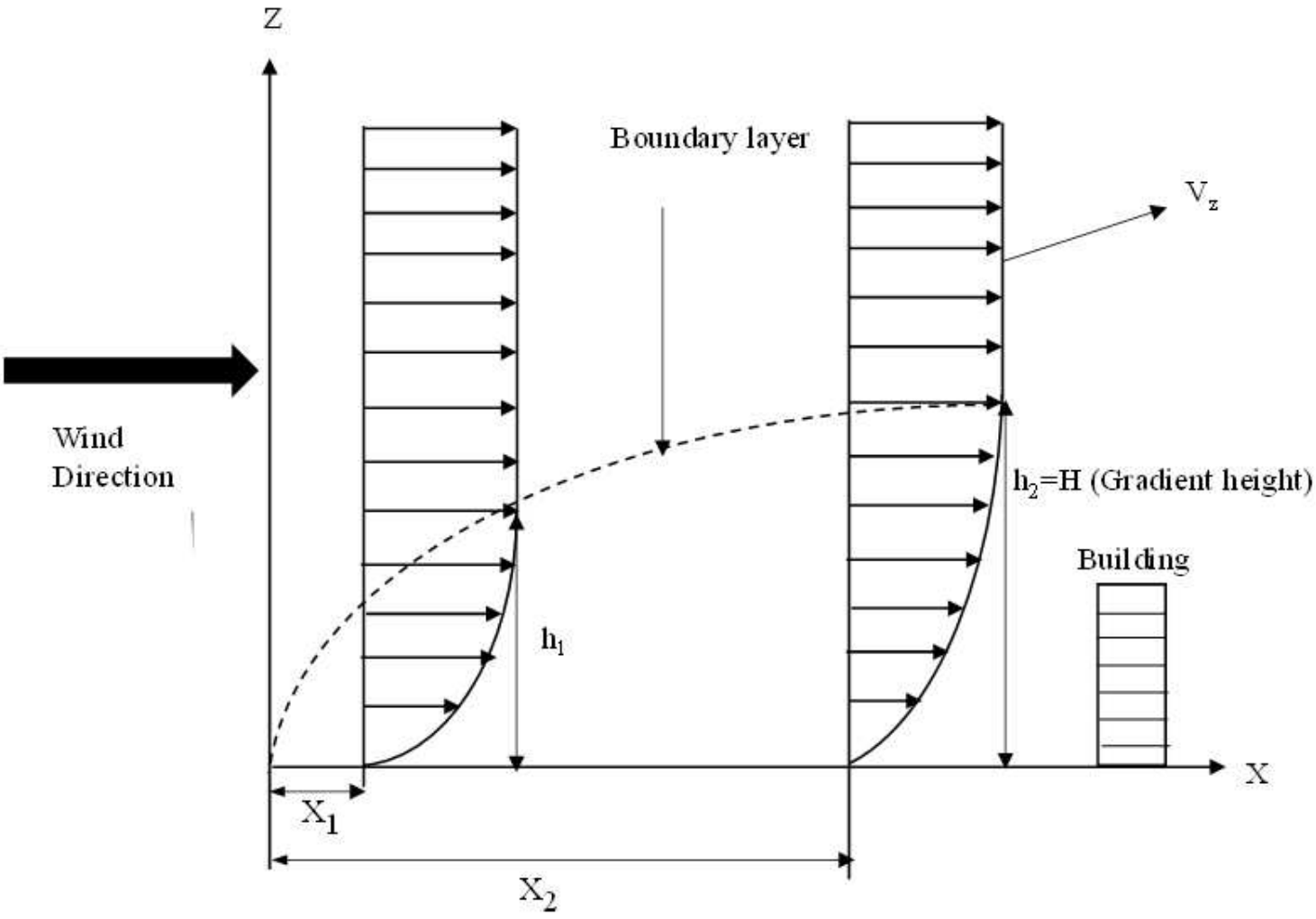
Figures



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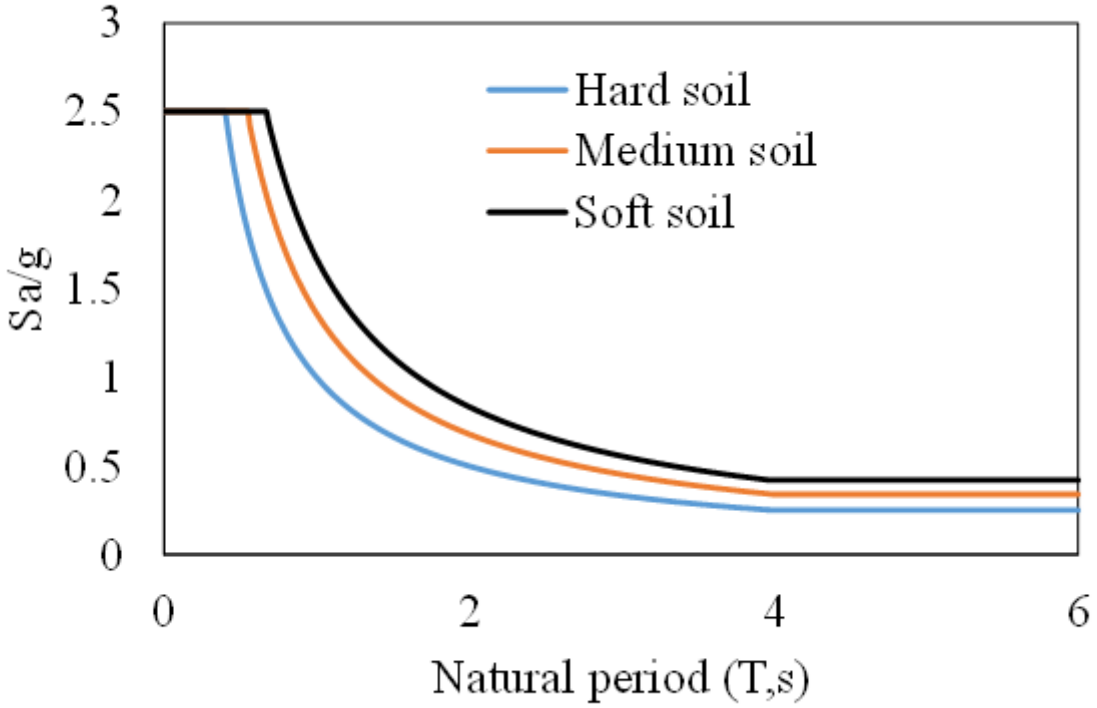
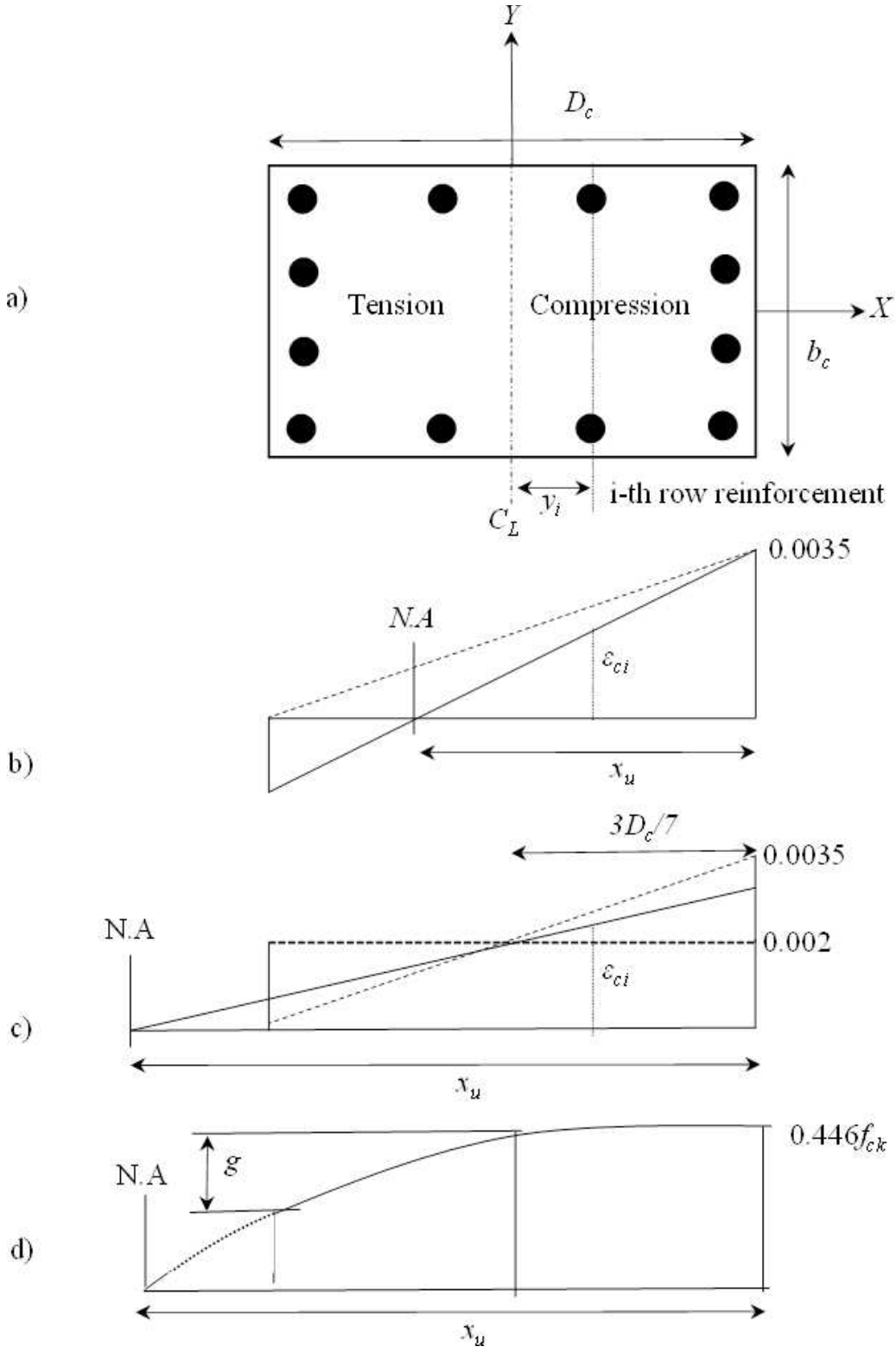


Figure 2

Design acceleration response spectrum of IS 1893-2016(Part-III)[43]



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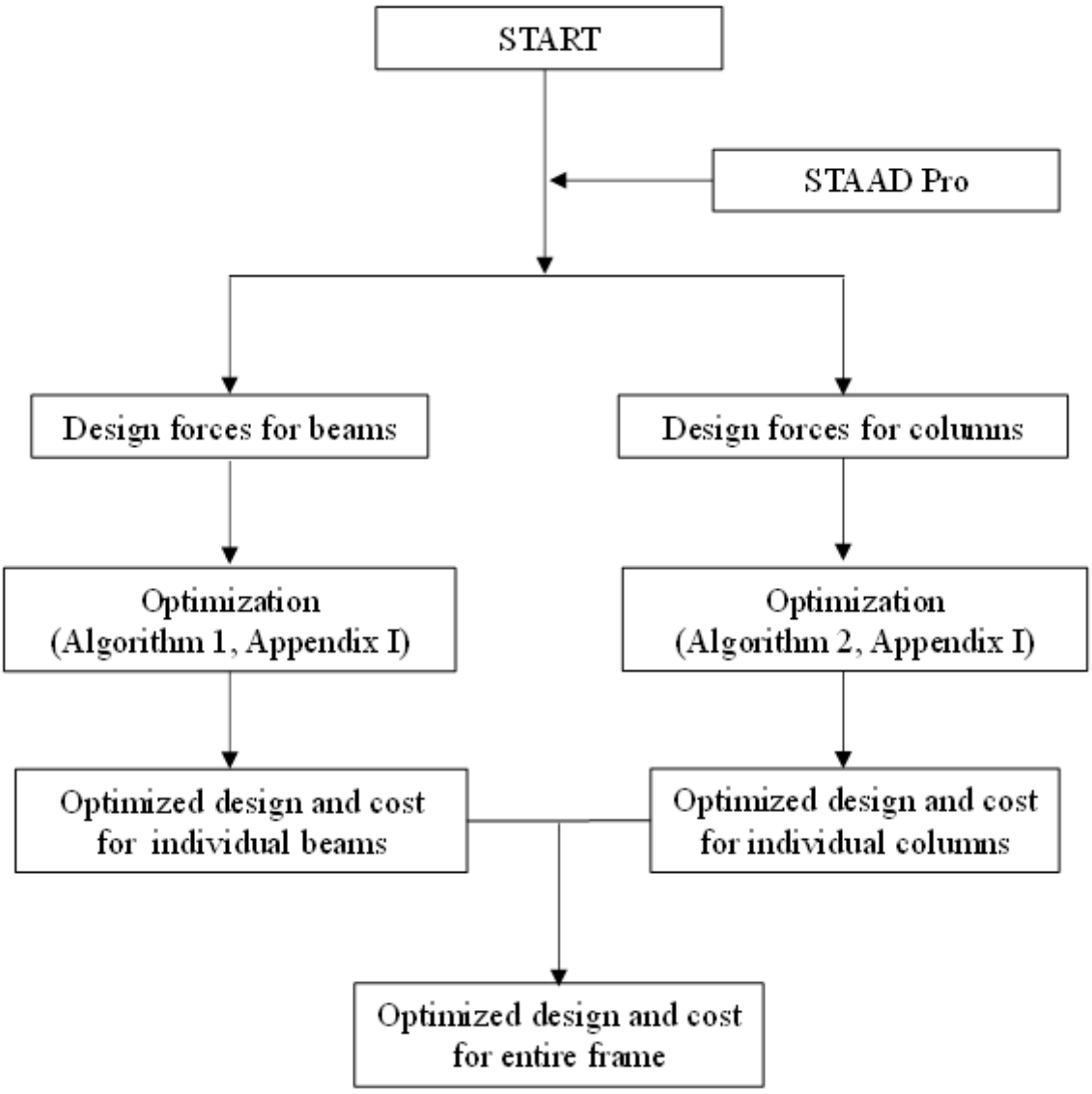
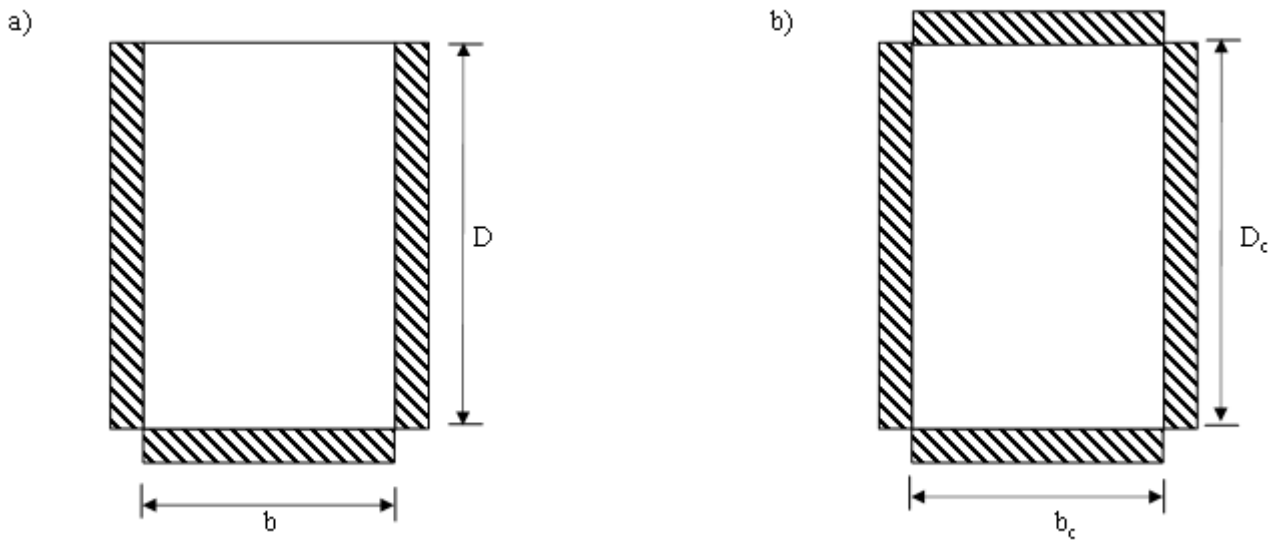


Figure 4

Flowchart for cost optimization of whole frame



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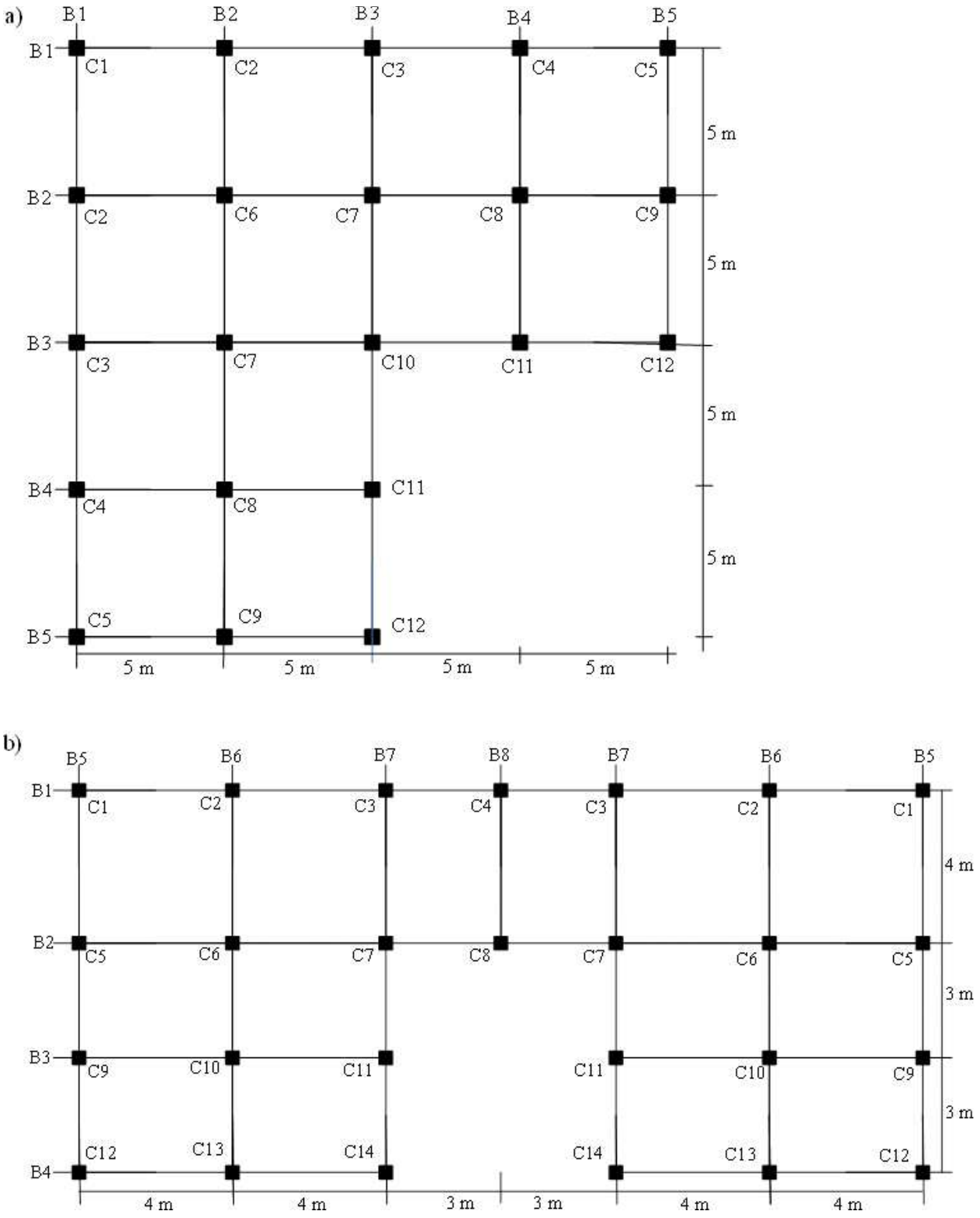

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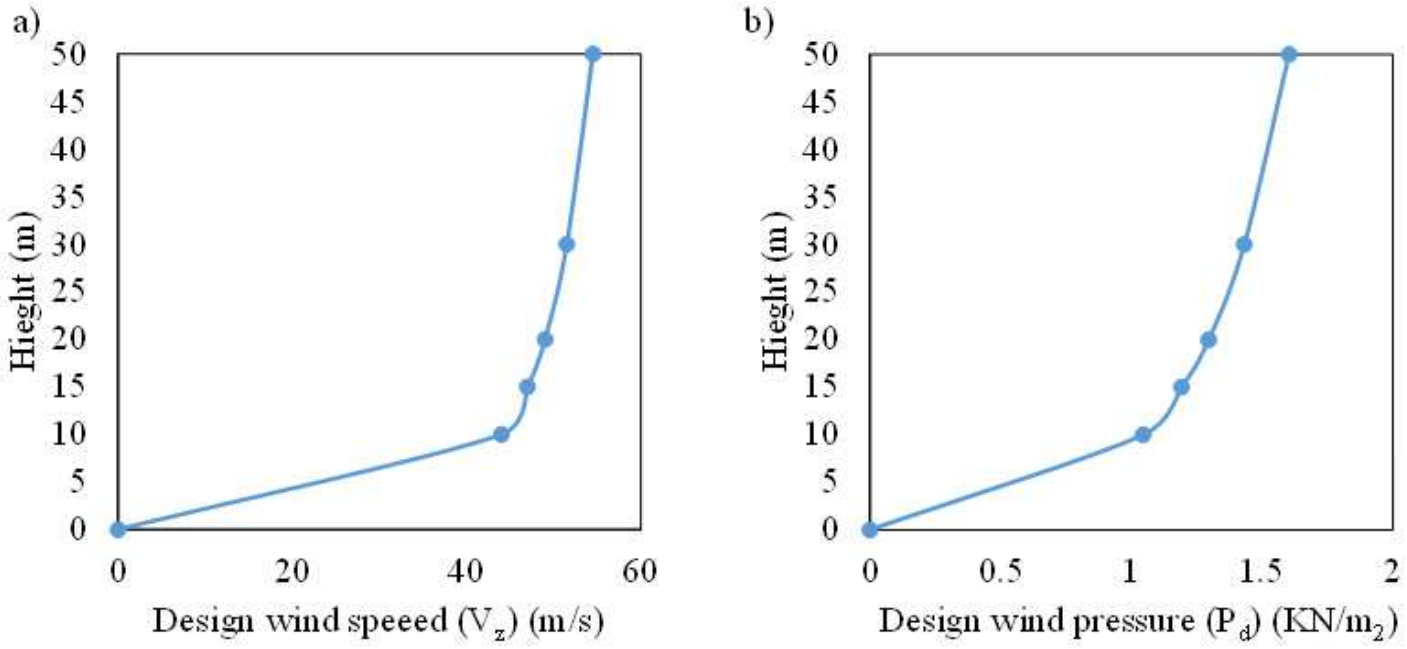

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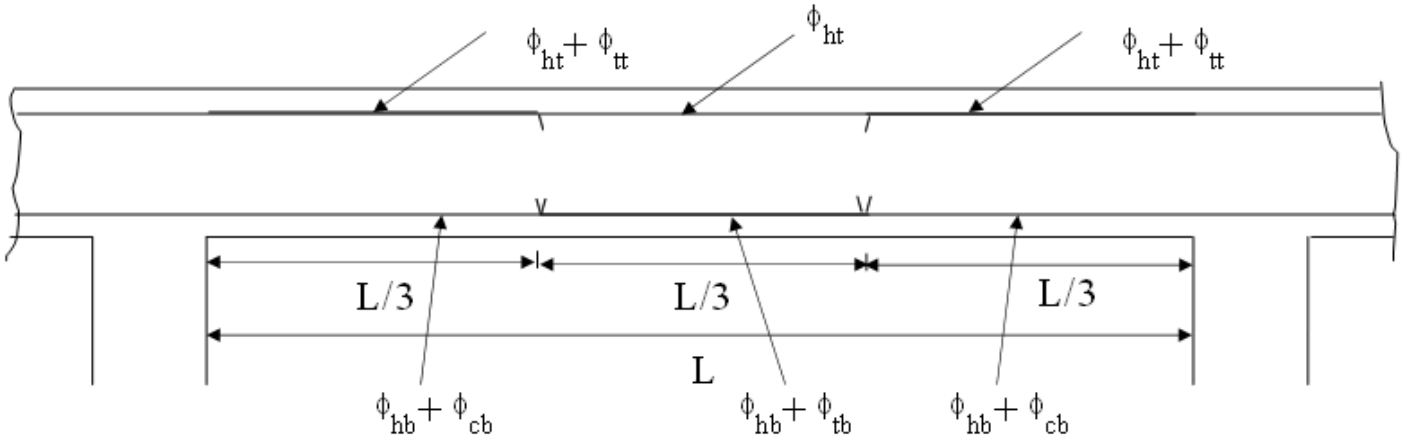

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Schematic diagram of continuous beam showing the details of curtailments in bars

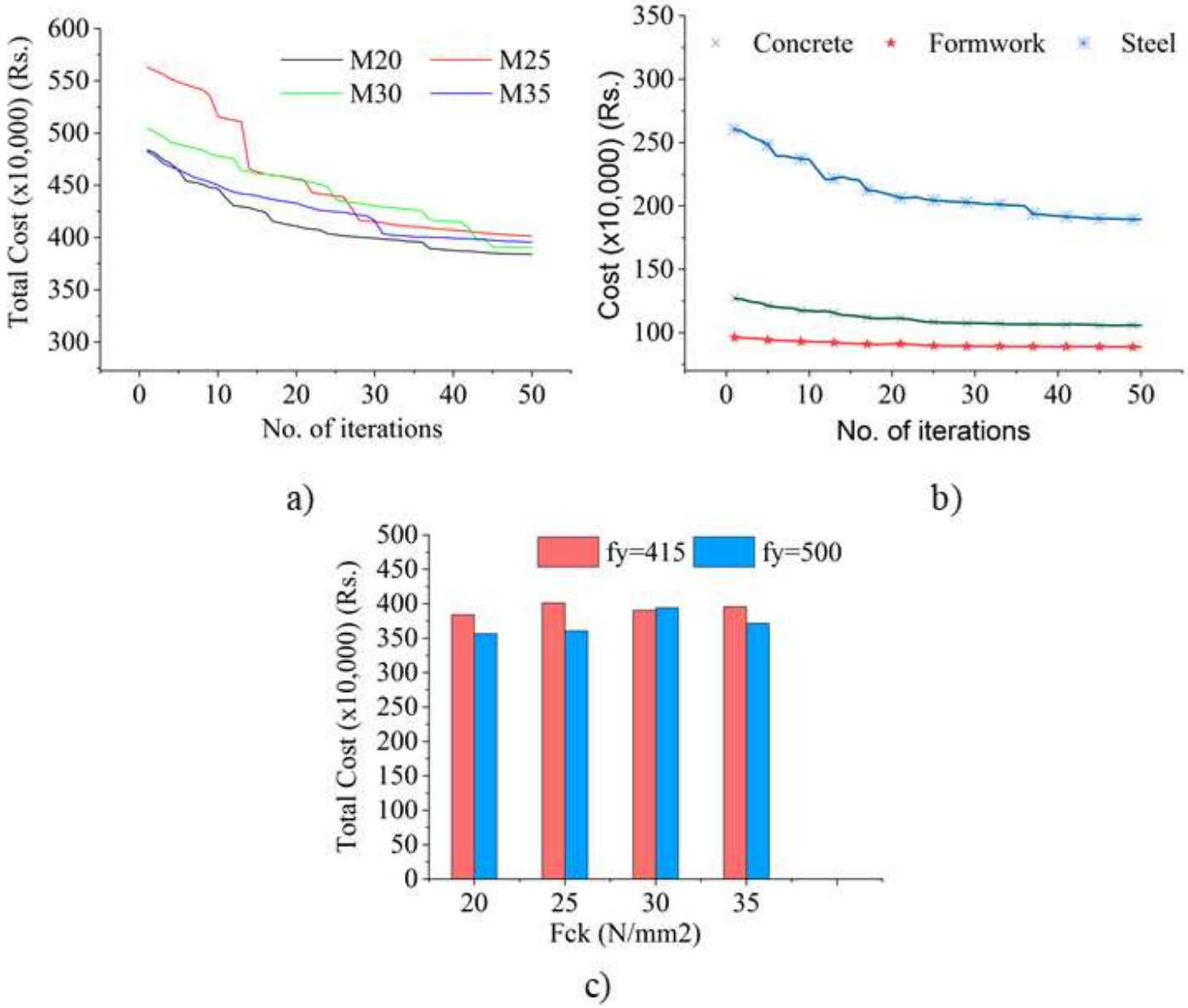

Figure 9

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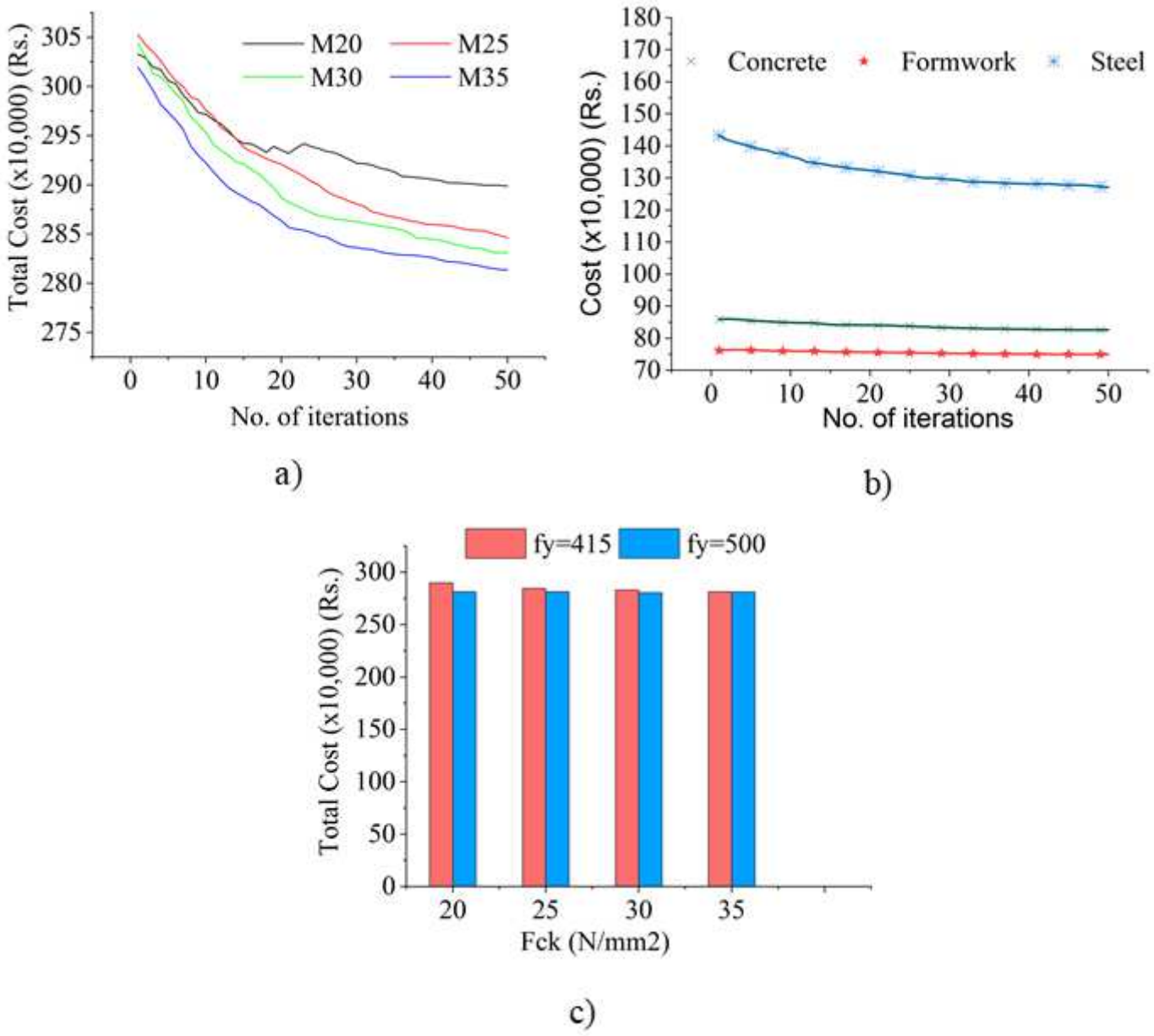

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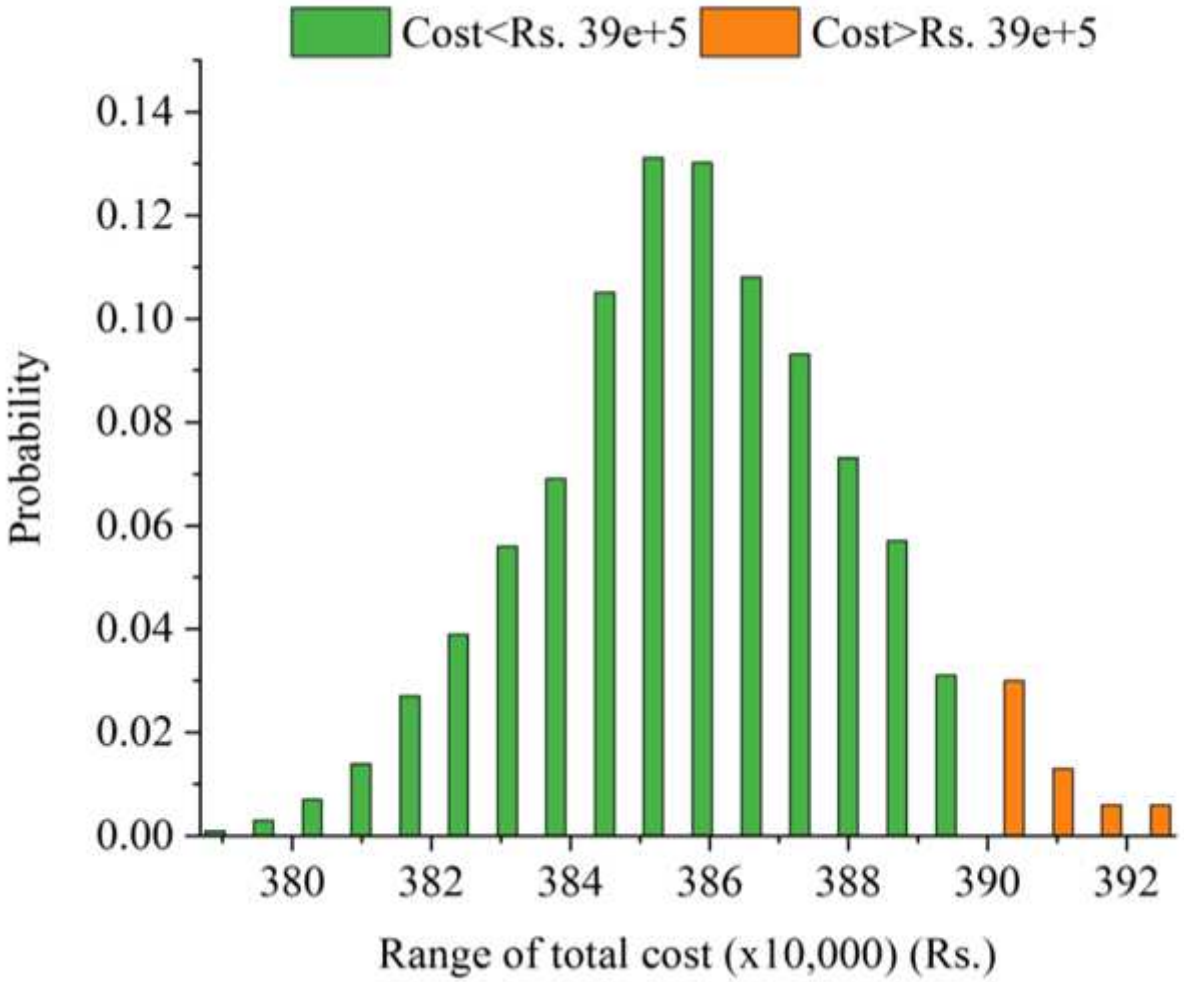

Figure 11

Probability distribution for 5000 simulated solutions (L shaped frame)

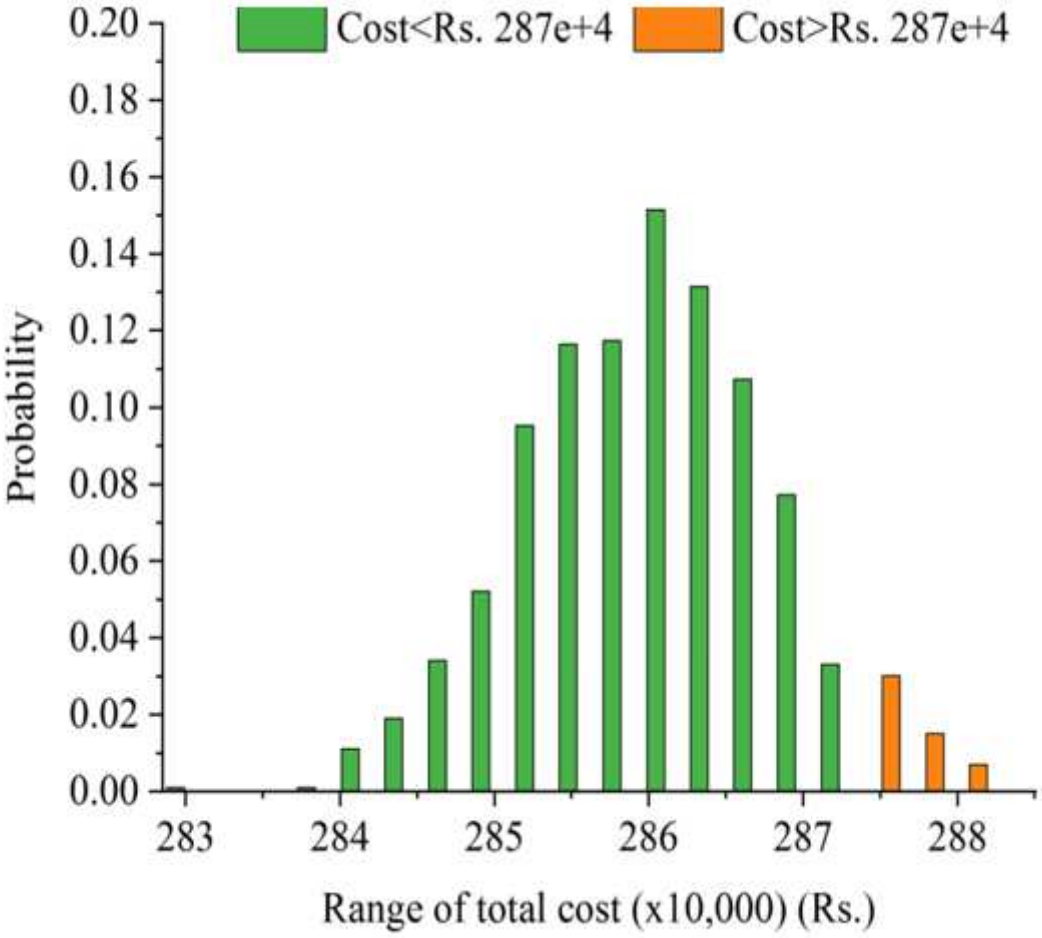

Figure 12

Probability distribution for 5000 simulated solutions (U shaped frame)

Supplementary Files

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