Optimization of truss industrial sheds with sloping and arched roofs using the force method and a meta-heuristic algorithm

Ali Kaveh* and Neda Khavaninzadeh

School of Civil Engineering, Iran University of Science and Technology, PO Box 16846-13114, Iran

alikaveh@iust.ac.ir *(corresponding author)*

Abstract

Due to technological advances, efficient structural systems are increasingly required in modern industries. Many industrial buildings, such as warehouses, hangars, fire stations, and sports halls, are designed with large spans and non-flat roofs, often sloped or curved. This paper presents a comparative optimal design of steel truss sheds with two roof types (sloping and curved) and two column types (rectangular and circular). The Enhanced Colliding Body Optimization (ECBO) algorithm is employed, offering a balanced mechanism of exploration and exploitation, which improves efficiency compared to algorithms like PSO and GA. The design process follows AISC specifications, considering stress, displacement, and slenderness limitations. Structural members are modeled with discrete cross-sectional variables, and the trusses are analyzed under dead, live, snow, wind, and earthquake loads. Results indicate that the truss with a sloping roof and rectangular columns achieved the minimum weight while maintaining uniform stress distribution. This configuration demonstrated superior structural performance regarding strength and serviceability, highlighting its suitability for industrial applications. Overall, the findings emphasize the role of optimization methods in enhancing structural efficiency and provide guidance for the practical design of lightweight and resilient steel sheds.

Keywords: Industrial building, Matrix force method, Metaheuristic optimization algorithm, Enhanced Colliding Body Optimization algorithm.

1. Introduction

In today's world, with the advancement of technology and science, the need for modern and efficient structures in various industries is required. The use of truss structures is known as one of the main methods in the design and construction of industrial structures and has become the first choice of engineers and architects due to their special features, such as lightness, strength and flexible design. After World War II, the need for rebuilding infrastructure led countries toward using modern construction methods. In this regard, truss structures have played a special role in industrial buildings by reducing construction costs, increasing implementation speed, and optimizing material usage [1]. Pitched roof frames are constructed in various types such as gable frames, saw tooth frames, and mono slope frames, T-shape frames, lean-to frames, and domed frames [2]. Achieving maximum efficiency and reducing costs in the design and construction of industrial truss structures are the key challenge. Accurate analysis and optimization can serve as crucial strategies to address this. Gradient-based and stochastic optimization techniques have been widely used in the optimal design of structural systems [3–9]. Since the 1960s, considerable research has focused on minimizing the weight of structures through nonlinear mathematical programming [10–13]. Over the past two decades, metaheuristic algorithms have been widely applied in optimizing 2D and 3D structures, including trusses and frames [14–17]. Although much research has focused on frame structures, limited studies exist on the optimal design of truss sheds with inclined or non-prismatic members [18]. Several types of truss sheds are shown in Fig. 1. Given the significant effect of structure weight on project costs potentially up to 30–40% accurate design of trusses is essential. The force method is a classical technique based on satisfying equilibrium first, followed by compatibility. Recent developments have enabled graph-theoretical approaches for analyzing both planar and space structures using this method [19-21].

Algebraic methods have been developed and extended by Denke [22], Robinson and Haggenmacher [23], Topçu [24], Kaneko et al. [25], Soyer and Topçu [26] and Kaveh and Shabani [27]. Mixed algebraic-topological methods have been performed by Gilbert and Heath [28], Coleman and Pothen [29, 30], and Pothen [31]. Simultaneous analysis and design can be found the work of Kaveh and Rahami [32] and Kaveh and Bijari [33].

Due to the limited number of studies focusing on truss sheds with non-prismatic members, here the effectiveness of graph-theoretic force method is integrated with a metaheuristic algorithm known as the ECBO.

After this introduction, Section 2 provides the force method using the associated graph of the model. Section 3 presents the ECBO algorithm, including the objective function and constraints. Section 4 outlines the structural loading conditions, while Section 5 provides the design examples. Finally, Section 6 concludes the present study.

2. Methodology

2.1 Force Method of Structural Analysis

The force method, also known as the flexibility method, is an analytical approach for evaluating the internal forces of structures. In contrast to the stiffness method, which directly solves for displacements, the force method treats selected redundants as unknowns. These are determined by enforcing compatibility conditions that ensure consistent displacements in the structure.

The basic steps include:

- Reducing the structure to a statically determinate system by removing the constraints corresponding to the selected redundants.

- Writing compatibility equations relating displacements due to the applied loads and redundants.

- Solving these equations to compute the redundant forces.

- Evaluating internal forces and displacements using superposition.

The use of this method is efficient for structures with a lower degree of static indeterminacy compared to the degree of kinematical indeterminacy, and offers computational advantages, especially when utilized in optimal design, where the analysis should be performed thousands of times.

2.2 Graph-Theoretic Formulation

In this study, a graph-theoretic approach is employed to construct the required matrices B_0, B_1, and matrix G. This is carried out via the associate graphs of the geometry of the planar trusses [34].

2.2.1 Associate Graph Formation

Given a truss structure S, the associate graph A(S) of its truss model S is formed by representing each triangular panel as a node in A(S). Two nodes are connected if their corresponding panels share a common edge [35]. This representation helps to identify a generalized cycle basis (GCB), which supports the formation of selfequilibrating stress systems (SESs). As an example, consider a planar truss as shown in Figure 2 supported in a statically determinate fashion.

2.2.2 Cycle Basis Selection

The associate graph A(S) is utilized for the formation of a cycles basis. These cycles are then used to construct a statical basis for the formation of an efficient B_1, which is crucial in applying the force method.

2.3 Formulation of the Optimization Problem

In this optimization process, the purpose is to minimize the weight of the steel used, while satisfying the member tension and slenderness constraints. Apart from the stress, there are other constraints that must be considered using the design code. This process is computationally expensive, in particular for large structures, when no analytical method can be used directly. However, when the existing structural profiles are limited to a specific discrete list, which significantly increases the complexity of the discrete optimization problems. For this purpose, the problem has been specialized with discrete variables, i.e. the cross-section number of the truss members, and programmed using MATLAB environment. The continuous values which may appear during the execution are rounded to correct variables and decoded before analyzing the structure.

2.3.1 Design Variables

The vector of design variables for a structure with member groups is given according to Eq. (1):

$$X = \langle x_i \rangle, i = 1, 2, \ldots, m \quad (1)$$

The design variables are the discrete cross-sectional area indices of truss members. Each x_i corresponds to a standard profile selected from a predefined list based on design codes.

2.3.2 Objective Function

The objective is to minimize the total weight w of the structure.The weight optimization is formulated with the aid of Eq. (2) using the density.

$$Min\, w(x) = \rho L' \times A(x) \tag{2}$$

2.3.3 Constraints

The optimization is subject to the following constraints Eq. (3):

$$S.t. \begin{cases} g_j(x) \le 0 \\ x_{i\,\min} \le x_i \le x_{i\,\max} \end{cases} \tag{3}$$

The functions containing the limits of stresses and the allowable slenderness are as the Eq. (4) and Eq. (5):

$$g_\sigma^k = \frac{\sigma_k}{(\sigma_k)_{allowable}} - 1 \le 0 \quad k = 1, 2, \ldots, N_d \tag{4}$$

$$g_\lambda^k = \frac{\lambda_k}{(\lambda_k)_{allowable}} - 1 \le 0 \quad k = 1, 2, \ldots, N_d \tag{5}$$

σ is member tension, , N_d being the number of members, and λ is member slenderness.

3. Optimization Algorithms, ECBO

In the field of optimization, there are various metaheuristic algorithms, each offering its own superiors to solve different optimization problems. In another study, we used four important algorithms including Particle Swarm Optimization (PSO), Genetic

Algorithm (GA), colliding Body Optimization (CBO), and Enhanced colliding Body Optimization (ECBO) to optimize our objective function, which was the mean square error. According to the conclusions of the analysis and as shown in Figure 3, it was found that the ECBO algorithm shows better convergence performance than the other three algorithms. Therefore, in this research, due to this superiority, the ECBO algorithm was chosen as the main optimization method.

Colliding Bodies Optimization (CBO) has been developed by Kaveh and Mahdavi [36], and it is improved by Kaveh and Ilchi Ghazaan [37] as the Enhanced Colliding Bodies Optimization (ECBO) using a memory to save the number of historically best CBs. ECBO and also utilizes a mechanism to escape from local optima.

The initial positions of all the CBs in an n-dimensional space are randomly determined as Eq. (6):

$$x_k^0 = x_{MIN} + rand \times (x_{MAX} - X_{MIN}) \qquad k = 1, 2, \ldots, n \qquad (6)$$

Every colliding object has a momentum as Eq. (7):

$$Mass_k = \frac{\frac{1}{func(k)}}{\sum_{k=1}^{n} \frac{1}{func(k)}} \qquad k = 1, 2, \ldots, n \qquad (7)$$

In order to choose the objects, these are sorted in descending order and divided into 2 equal groups; a “Stationary objects” and “Moving objects”.

Thus the objects move to a better position. For all the objects prior the moving and collision, the speed is taken $V(i)$ .Eq. (8) and Eq. (9) show this.

$$V_k = 0, \qquad k = 1, 2, \ldots, \frac{n}{2} \qquad (8)$$

$$V_k = x_{k-\frac{n}{2}} - x_k, \qquad k = \frac{n}{2}+1, \frac{n}{2}+2, \ldots, n \tag{9}$$

The velocity of the static and moving objects are as Eq. (10) and Eq. (11) after collision:

$$V_k' = \frac{(Mass_{k+\frac{n}{2}} + \varepsilon Mass_{k+\frac{n}{2}})V_{k+\frac{n}{2}}}{Mass_k + Mass_{k+\frac{n}{2}}} \qquad k = 1, 2, \ldots, \frac{n}{2} \tag{10}$$

$$V_k' = \frac{(Mass_k - \varepsilon Mass_{k-\frac{n}{2}})V_k}{Mass_k + Mass_{k-\frac{n}{2}}} \qquad k = \frac{n}{2}+1, \frac{n}{2}+2, \ldots, n \tag{11}$$

$$\varepsilon = 1 - \frac{iter}{iter_{Max}} \tag{12}$$

The new position of objects is given by Eq. (13) and Eq. (14)

$$x_k^{new} = x_k + random \times V_{k'}' \qquad k = 1, 2, \ldots, \frac{n}{2} \tag{13}$$

$$x_k^{new} = x_{k-\frac{n}{2}} + random \times V_{k'}' \qquad k = \frac{n}{2}+1, \frac{n}{2}+2, \ldots, n \tag{14}$$

The parameter $\Pr o$ is chosen from the interval (0, 1) that specifies if a component of CB should be altered or not. For each colliding object, $\Pr o$ is compared to rn_i. rn_i, a random number uniformly distributed between (0, 1). If $rn_i < \Pr o$, a dimension of the *ith* CB is randomly chosen and the corresponding value is taken as Eq. (15).

$$x_{ij} = x_{j,\min} + rand \times (x_{j,MAX} - x_{j,MIN}) \quad (15)$$

4. Structural loading

According to 6th Iranian National Building Code [38], the structure must withstand the following combination of loadings, including dead, live, wind, snow, and earthquake:

$$1.4D$$

$$1.2D + 1.6L + 0.5(L_r \ or \ S)$$

$$1.2D + 1.6(L_r \ or \ S) + [L \ or \ 0.5(1.6W)]$$

$$1.2D + 1.6W + L + 0.5(L_r \ or \ S)$$

$$1.2D + E + L + 0.2S$$

$$0.9D + 1.6W$$

$$0.9D + E$$

where D, E, W, L, S are dead, earthquake, wind, live, snow loads, respectively. L_r means L, where L is roof live load.

The dead loads

In order to consider the dead and lateral loads, the roof type is considered as a metal sandwich panel with a mass of 14.65 kg/m^2. This load includes the purlins on the roof and there is no false ceiling. The collateral load is assumed to be zero. The information about the dead load is given in Table 1.

The live loads

According to 6th edition of the Iranian National Building Code [38], the live load for a lightweight roof is 50.98 kg/m^2 and there is no concentrated load for it. It is also assumed that the live load cannot be reduced. The live load information is illustrated in Table 2.

The snow load

Snow load is one of the types of environmental loads acting on a building. When it snows, snowflakes accumulate on the roof. The accumulation of snowflakes increases the weight of the mass and exerts downward vertical pressure on the roof of the building. For sloping roofs, balanced and unbalanced loading is performed as shown in Figure 4.

The flat roof snow load, P_r , is evaluated by using the following Eq. (16) :

$$P_r = I_s C_n C_h C_s P_s \quad (16)$$

Where C_n is the snow removal coefficient, C_h is the temperature condition coefficient, C_s is the slope coefficient, P_s is the base snow load, and I_s is the snow load importance coefficient according to the 6th edition of the Iranian National Building Code [28]. To obtain the unbalanced snow load, γ and h_d are obtained as follows Eq. (17) and Eq. (18):

$$\gamma = 0.43 P_s + 2.2 \quad (17)$$

$$h_d = 0.12 \sqrt[3]{l_u} \sqrt[4]{100 P_s + 50} - 0.5 \quad (18)$$

So: $P_{r=0.96}, h_d = 0.3468, \gamma = 2.845$

For arched roofs, balanced and unbalanced loading is performed as shown in Figure 5.

The seismic load

The seismic base shear, V_u is determined using the following Eq. (19):

$$V_u = CW \tag{19}$$

Where C and W are the seismic response coefficient and the effective seismic weight, respectively. The coefficient, C , is evaluated as Eq. (20)

$$C = \frac{ABI}{R_u} \tag{20}$$

Where A is the design basis acceleration, B is the building reflection coefficient, I is the building importance coefficient, and R_u is the building behavior coefficient.

Since the location of this study is Tehran, Iran, the above values are obtained from the standard No. 2800, 4th edition [39].

The wind loads

For calculating the wind load, for a low rise structure, the wind pressure is obtained using Eq. (21)

$$P = I_w q C_e C_t C_g C_p C_d \tag{21}$$

where q is the base wind pressure taken by Eq. (22)

$$q = 0.000613V^2 \tag{22}$$

Also, I_w is the wind load importance coefficient, C_e is the velocity change effect coefficient, C_t is the land slope and elevation coefficient, C_g is the wind gust effect coefficient, C_p is the pressure coefficient, and C_d is the wind alignment coefficient. These coefficients are obtained from the 6th Iranian National Building Code [38].

5. Design examples

In this study, the analysis and optimal design for four different cases of truss sheds have been carried out by the force method using the associate graph method and utilizing the ECBO metaheuristic algorithm. In these four studied examples, the height and width of the sheds are taken the same. The truss members are divided into 16 groups. The list of profiles used for optimization is given in Table 3. The sections used are equal in angle and leg. Figure 6 shows the general view and schematic of the trusses.

All four models shared identical geometric configurations, as shown in Table 4 for their general geometric characteristics. Two types of supports (one hinged and one fixed) are utilized for representing realistic structural conditions. In practice, many trusses have a fixed support and a hinged support to enable the transmission of horizontal and vertical forces on the one hand and to allow for thermal deformation and partial settlements on the other.

The values of R_u for the different structures in this study have been determined based on the design standards for structures under earthquake loads, particularly ASCE 7 and Eurocode 8. According to these sources, the values of R_u for the seismic design of trusses in cases 2 and 4 are 5-6, while for cases 1 and 3, they are 4-5. In this study, we considered the R_u values to be 5 for all cases.

Regarding the empirical period of vibration of the trusses, there are various methods including empirical formulas, numerical modeling, and modal analysis, and in this research, we used the empirical formula from Eq. (23).

$$T = C.(\frac{H}{L})^n \quad (23)$$

where T is the period, H is the height of the truss, L is the span of the structure, C and n are empirical parameters that can vary depending on the type of structure (for steel structures, $C \approx 0.1 - 0.15$ and $n \approx 0.5$ are usually used).

Figure 7 illustrates the grouping scheme of truss members for all 4 cases. Also, Figure 8 shows the associate graph formed, which is the same for all 4 cases.

The convergence results with the ECBO algorithm after performing 250 iterations and using a population size of 50 for all the studied cases are compared in Figure 9. Also the optimal cross section comparison is provided in Table 5. The results indicate that the minimum weight was achieved in the second case, i.e. to a shed truss with a sloping roof having rectangular columns. The comparison of the results generally shows that even with more members in the trusses with rectangular columns, they have better results than the conical column cases. In general, it can be stated that rectangular columns led to lower optimal values than the conical columns in both types of roofs.

A comparison of the CPU times for performing the optimization is illustrated in Figure 10. It is observed that the optimization time in the case with rectangular columns is shorter than in the case with conical columns. Figure 11 shows the comparison of the best cost values, or the lowest optimal weight values, for each of the cases examined.

The stress analysis of the truss members of the shed in Figure 12 shows that the stress of all members is almost less than unity and has an acceptable value. Also, the comparison of the maximum stresses of the members is given in Table 6. This comparative stress analysis determines the structural efficiency of each truss model and helps in selecting the most appropriate configuration in terms of performance and optimal material consumption. Case 2 has a lower maximum stress distribution and also the stress distribution according to Figure 12 is more uniform in Case 2 than other models, which shows that it remains safer under the applied forces and has better performance in terms of safety and economy.

6. Conclusions

The comparative analysis of the four studied truss shed configurations revealed valuable insights into the structural efficiency associated with different roof shapes and column types. While conical columns are often used for architectural and aesthetic purposes, the findings of this study indicate that rectangular columns lead to lighter structural designs under the same loading conditions. This study presented the analysis and optimal design of four steel truss sheds with varying roof shapes and column types, employing the force method and a graph-theoretical approach for structural analysis, and the Enhanced Colliding Bodies Optimization (ECBO) algorithm for design optimization. Two types of roof geometries (arched and sloped) and two types of columns (rectangular and conical) were considered to investigate their influence on structural performance and optimal weight. The results demonstrated that truss sheds with sloping roofs and rectangular columns consistently achieved the lowest optimal weights, highlighting their structural efficiency and material economy. Despite having more members, configurations with rectangular columns outperformed those with conical columns in both roof types. Furthermore, the use of the force method, combined with the associate graph formulation, resulted in efficient structural analysis with reduced computational effort, particularly advantageous for repetitive analyses in optimization loops. Stress distributions across the optimized trusses were found to be within allowable limits, confirming the validity of the design. Among the four cases, the truss with a sloping roof and rectangular columns not only had the lightest structure but also exhibited a more uniform stress distribution, indicating better performance in terms of strength and serviceability. This study highlights the potential of combining graph-theoretical formulations with advanced metaheuristic algorithms for the optimal design of industrial steel structures. Future research can extend this approach to consider dynamic loads, nonlinear material behavior, multi-objective optimization (e.g., cost vs. weight), or real-world constraints such as fabrication limits and environmental factors.

Authors' Biographies

Ali Kaveh

Kaveh is a distinguished professor of structural engineering at Iran University of Science and Technology. He obtained his PhD from Imperial College, London in 1974. He has been teaching at IUST, TU-Wien, Sharif University and some other universities in Iran. He is a fellow of the Iranian Academy of Sciences, fellow of The World Academy of Sciences (TWAS), fellow of the European Academy of Sciences and Arts, and member of 8 scientific societies, founder and editor of more than 5 scientific journals and advisory member of many journals and conferences. He has published 800 scientific journal papers, 160 conference papers, 23 books in Farsi and 18 books in English published by Springer, Wiley and Research Studies Press.

Neda Khavaninzadeh

She obtained her M.Sc from Kharazmi University and Ph.D. from Iran University of Science and Technology in 2020 and 2024, respectively. Dr. Khavaninzadeh published many scientific papers during her study. She then obtained Postdoc. Research Fellowship and continued her research at Iran University of Science and Technology extending the use of force method in metaheuristic algorithms.

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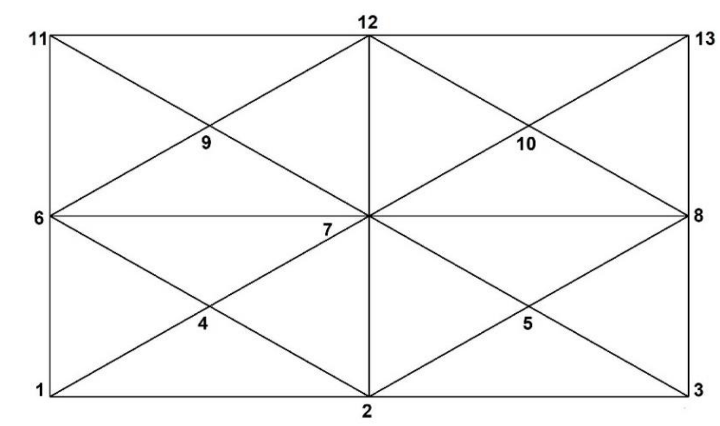
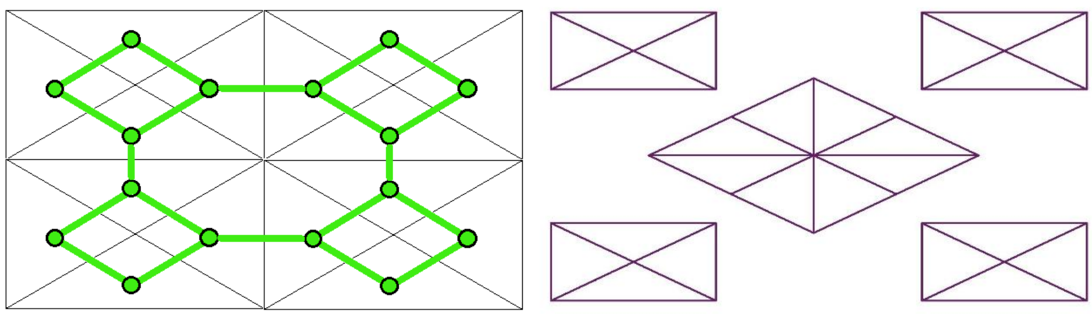
Fig. 1. Different types of truss sheds



(a) The graph model of a planar truss S



(b) Associate graph of S **(c) The elements of a GCB of S**

Fig. 2. Planar truss S, its associate graph, and the corresponding GCB elements

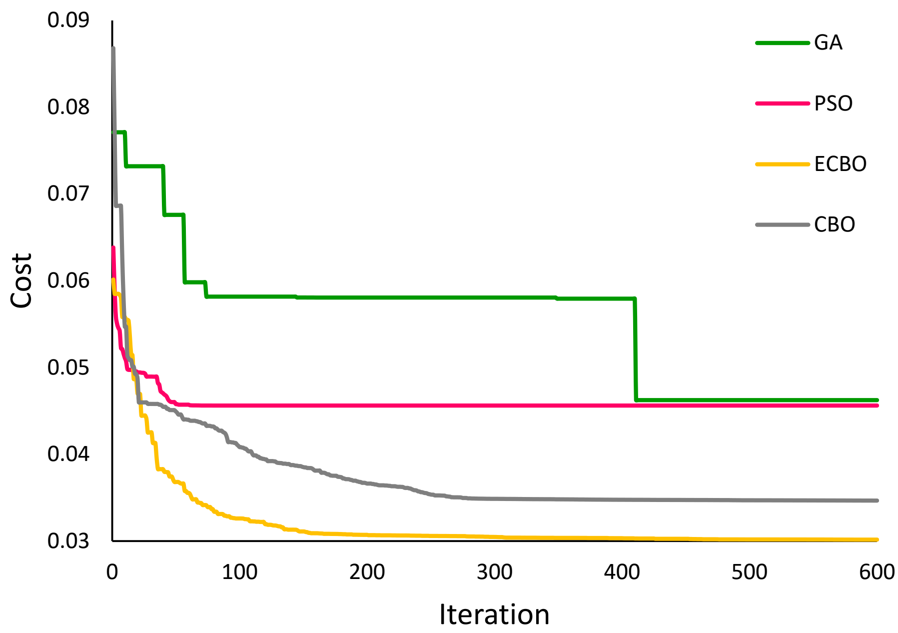


Fig. 3. Comparison of the convergence performance of four different algorithms in mean square error

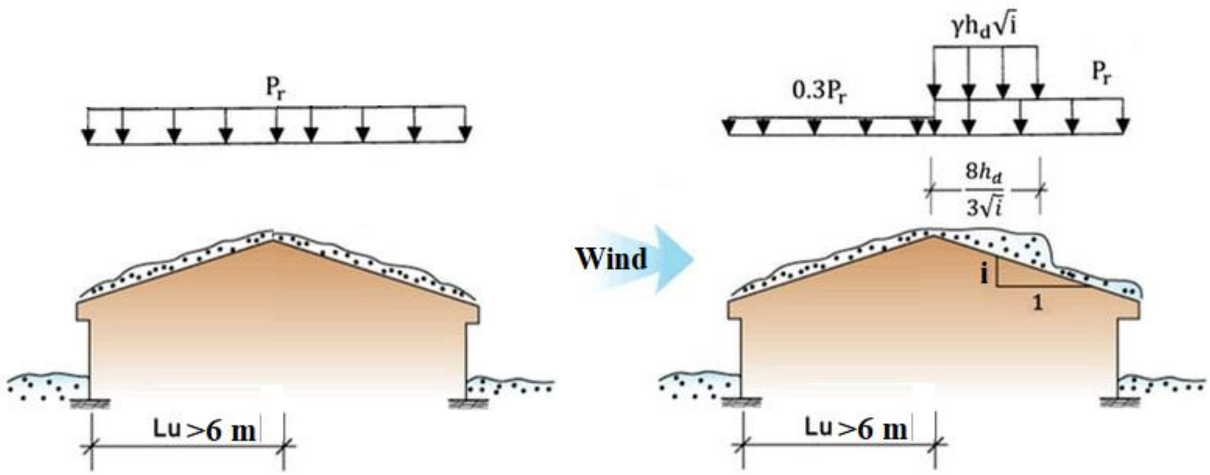


Fig. 4. A balanced and unbalanced loading for sloping roofs,

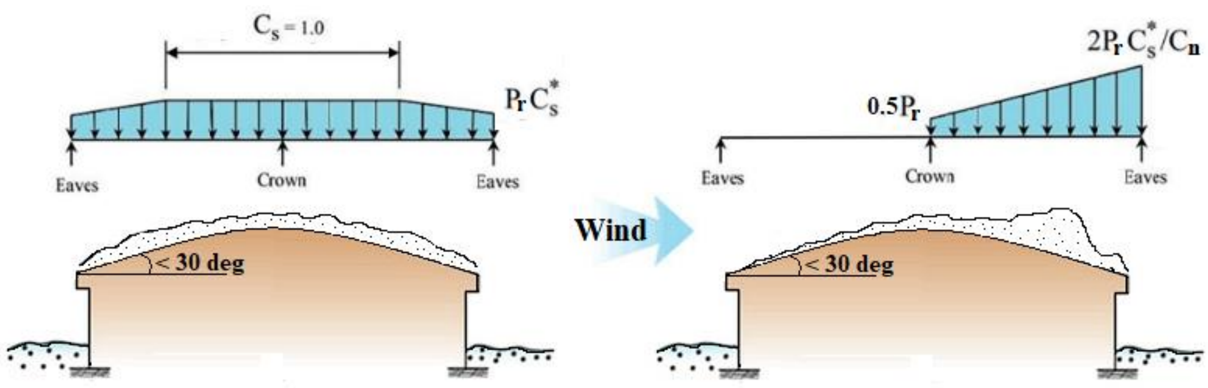


Fig. 5. A balanced and unbalanced loading for arched roofs,

Table1. Summary of the dead loading

Dead load kg/m^2	14.64
Loading per (m)	6
Uniform dead load (kg/m)	87.84

Table2. Summary of the live loading

Live load (kg/m^2)	50.98
Loading per (m)	6
Uniform live load (kg/m)	305.88

Table 3. List of the available profile for truss models

Prf. ID	Prf. Name	Area (cm^2)	Prf . ID	Prf. Name	Area (cm^2)	Prf. ID	Prf. Name	Area (cm^2)	Prf. ID	Prf. Name	Area (cm^2)
1	20x3	1.12	24	50x7	6.56	47	75x10	14.1	70	120x13	29.7
2	20x4	1.45	25	50x8	7.41	48	75x12	16.7	71	120x15	33.9
3	25x3	1.42	26	50x9	8.24	49	80x7	10.8	72	130x12	30
4	25x4	1.85	27	55x5	5.32	50	80x8	12.3	73	130x14	34.7
5	25x5	2.26	28	55x6	6.31	51	80x10	15.1	74	130x16	39.3
6	30x3	1.74	29	55x8	8.23	52	80x12	17.9	75	140x13	35
7	30x4	2.27	30	55x10	10.1	53	80x14	20.6	76	140x15	40
8	30x5	2.78	31	60x5	5.82	54	90x8	13.9	77	150x12	34.8

9	35x3	2.04	32	60x6	6.91	55	90x9	15.5	78	150x14	40.3
10	35x4	2.67	33	60x8	9.03	56	90x11	18.7	79	150x15	43
11	35x5	3.28	34	60x10	11.1	57	90x13	21.8	80	150x16	45.7
12	35x6	3.87	35	65x6	7.53	58	90x16	26.4	81	150x18	51
13	40x3	2.35	36	65x7	8.7	59	100x8	15.5	82	150x20	56.3
14	40x4	3.08	37	65x8	9.85	60	100x10	19.2	83	160x15	46.1
15	40x5	3.79	38	65x9	11	61	100x12	22.7	84	160x17	51.8
16	40x6	4.48	39	65x11	13.2	62	100x14	26.2	85	160x19	57.5
17	45x4	3.49	40	70x6	8.13	63	100x16	29.6	86	180x16	55.4
18	45x5	4.3	41	70x7	9.4	64	100x20	36.2	87	180x18	61.9
19	45x6	5.09	42	70x9	11.9	65	110x10	21.2	88	180x20	68.4
20	45x7	5.86	43	70x11	14.3	66	110x12	25.1	89	180x22	74.7
21	50x4	3.89	44	75x6	8.75	67	110x14	29	90	200x16	61.8
22	50x5	4.8	45	75x7	10.1	68	120x11	25.4	91	200x18	69.1
23	50x6	5.69	46	L75x8	11.5	69	120x12	27.5	92	200x20	76.4

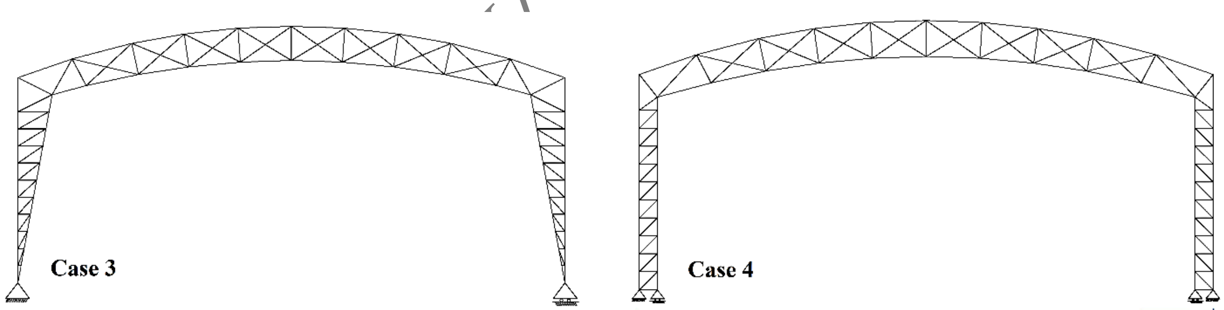


Fig. 6. General view and schematic of the trusses

Table 4. The geometrical information of building shape

Eave height	6 m
Crown height	7.5 m
Width	16 m
Length	18 m
Bay spanning	3@ 6 m

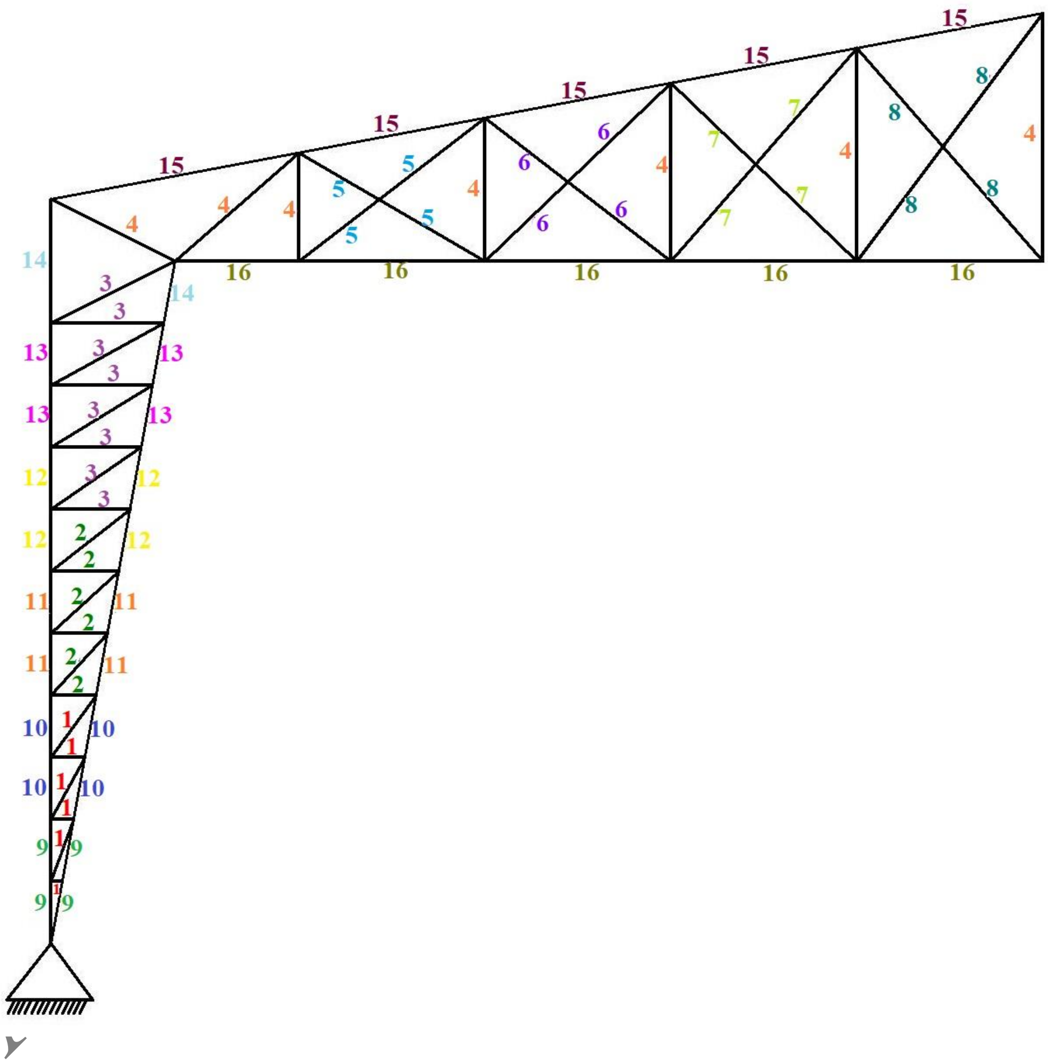


Fig. 7. Geometry and member grouping

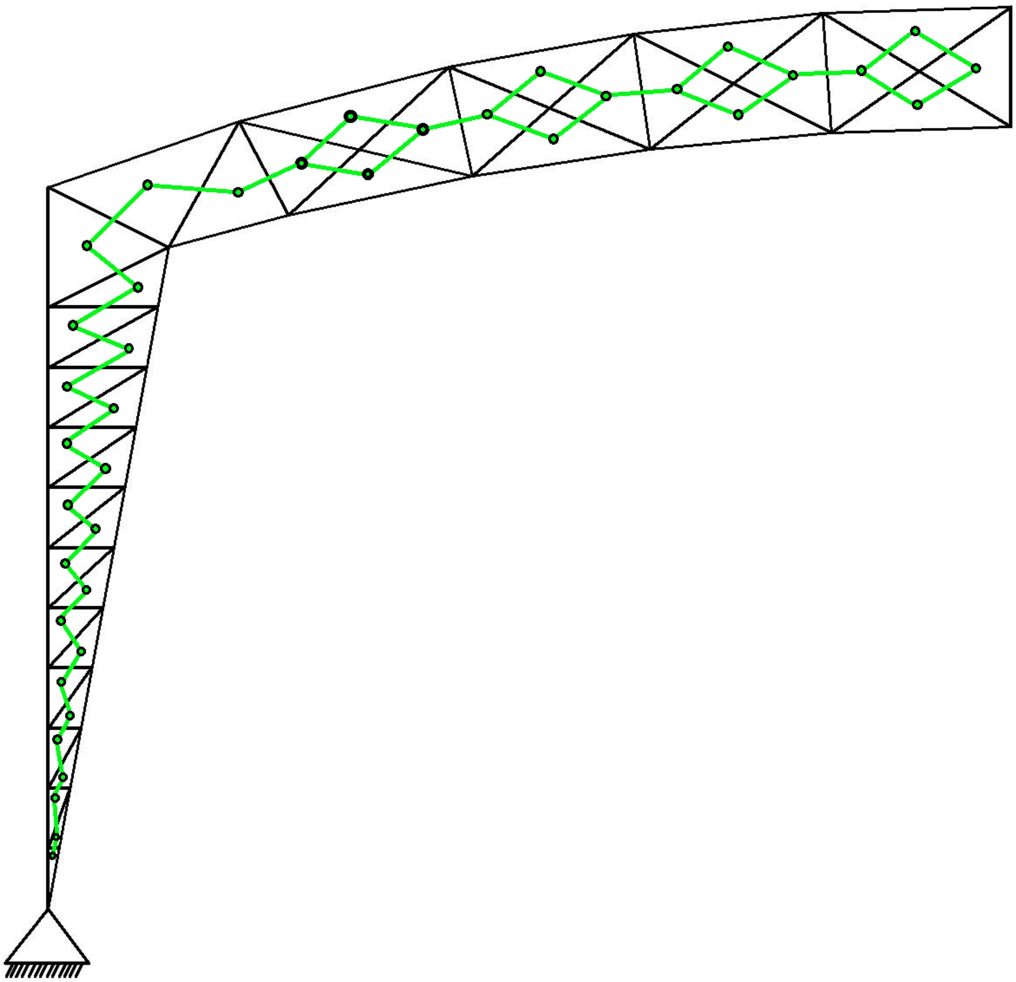


Fig. 8. The associate graph of the considered sheds truss

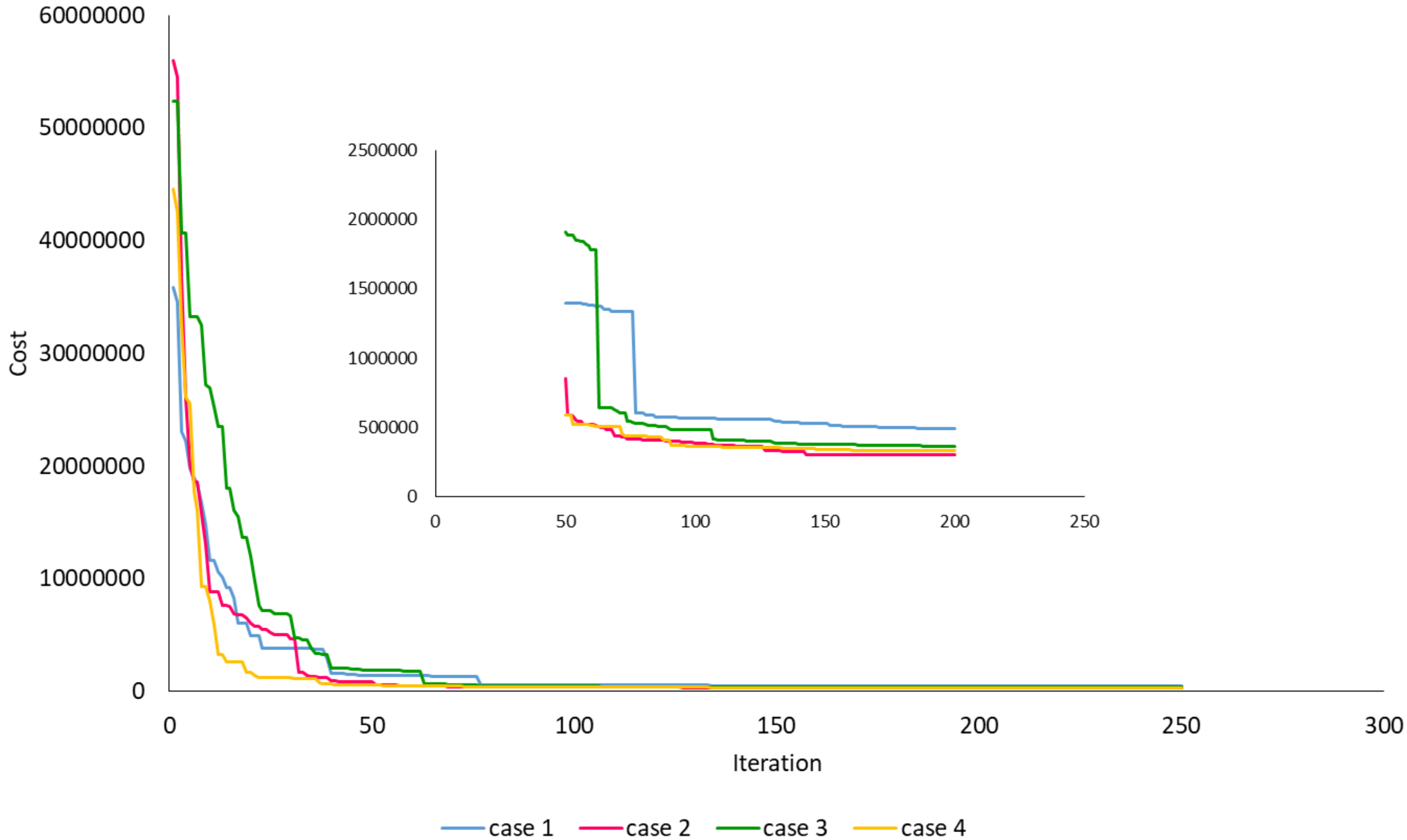


Fig. 9.The convergence diagrams of the ECBO

Table 5. Results of the optimization with 250 iterations

Group number of Element	Case 4 Profile ID	Case 3 Profile ID	Case 2 Profile ID	Case 1 Profile ID
1	71	68	65	50
2	86	34	29	60
3	91	49	17	54
4	87	70	79	75
5	54	48	65	54
6	41	44	38	49
7	51	31	33	45
8	23	29	40	29
9	65	87	65	90
10	59	70	83	71
11	54	61	87	65
12	41	75	88	71
13	54	83	88	84
14	59	80	90	86
15	83	74	71	78
16	70	50	81	66
Best weight (kg)	327880	356220	300080	485410
Average weight (kg)	1448611	3126832	1957834	2058739
Std (kg)	5147709	8101679	6212349	4601257

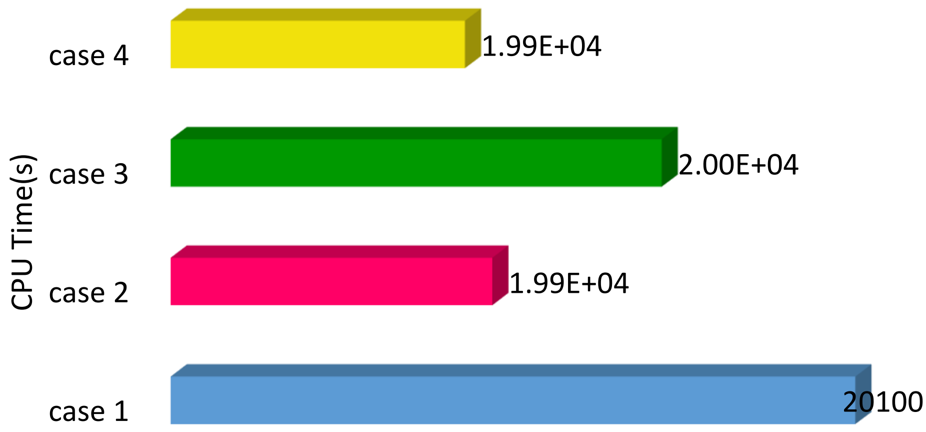
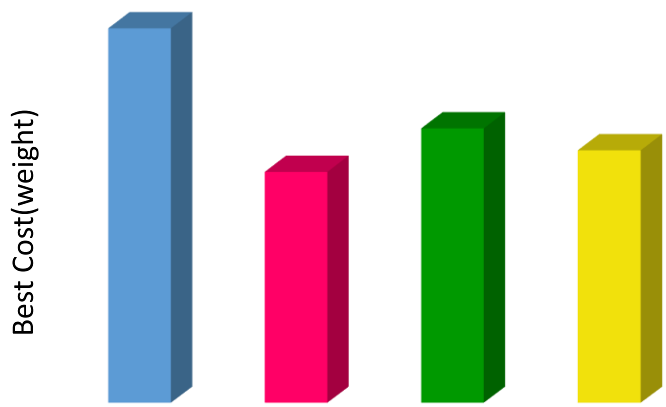


Fig.10. Comparison of the optimization time



	case 1	case 2	case 3	case 4
Series1	485410	300080	356220	3.28E+05

Fig.11. Comparison of the best costs

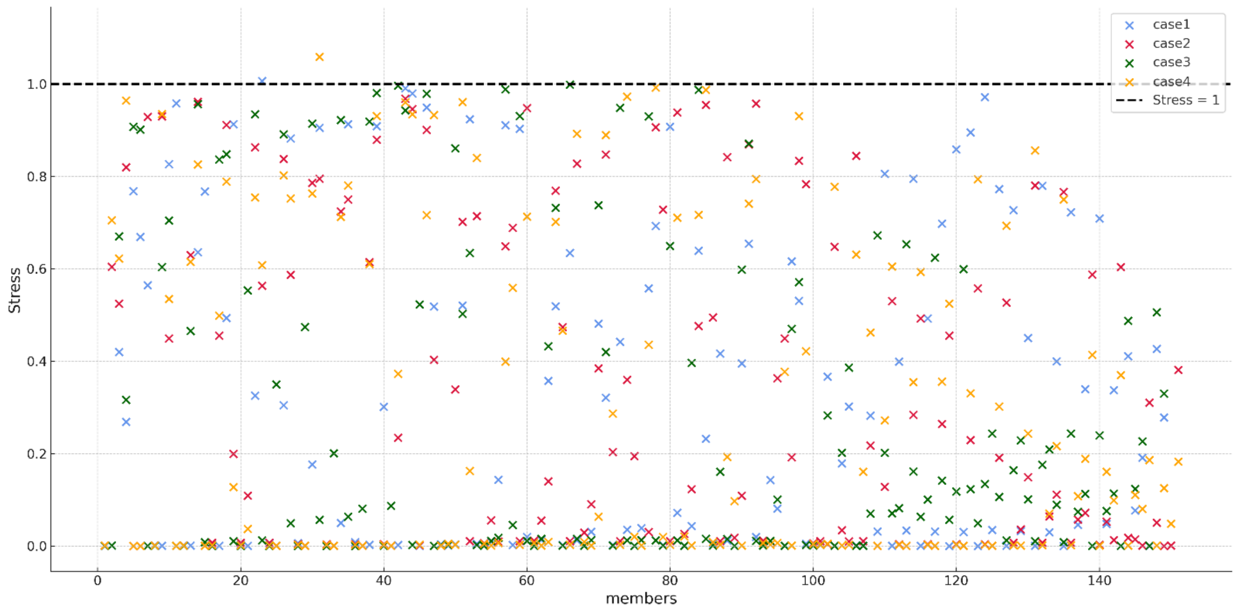


Fig. 12. The stress values of the shed truss members in optimal design

Table 6. Comparison of Maximum Member Stresses

	Case1	Case2	Case3	Case4
Max stress	1.0064	0.9684	0.9987	1.0808