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Comparison of Optimization of Steel Frames Using the LRFD Method with Two Sections: I Section and W Section Using Genetic Algorithm and With The Help of SAP and Etabs Software

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Abstract

In the last few decades, the use of algorithms to optimize structures has received a lot of attention, and steel and concrete structures have been investigated with the aim of weight optimization. But the optimization of metal structures with new analyzes including LRFD analysis is one of the new challenges in this field. In this research, the objective function was the structure's weight, which was supposed to be directly correlated with its cost. Design models were optimized separately and investigated and compared using genetic algorithm (GA) and LRFD analyses with W sections for frames. The cross-section of beams and columns were studied as design variables, and the constraints specified in the Iranian National Building Code (Part 10) were considered design constraints. Ultimately, the graphs for GA and stress ratio in each condition of LRFD for the W sections were obtained and compared, followed by analyzing.

Keywords: Optimization, steel structures, genetic algorithm, LRFD analysis
2022 MSC: O20, O10

1 Introduction

Optimization is the process by which we improve something. Engineers and scientists use optimization to improve and advance their new ideas. In other words, optimization means testing different modes on an initial idea and using the obtained information to improve that idea. Also, considering that optimization on steel structures using meta-exploratory algorithms has been used less by researchers, it seems that more studies and researches are needed in this field in the coming years.

Optimizing structures by taking into account the existing conditions and limitations or the constraint functions has always been one of the concerns of engineers and designers. In general, the number of designs that provide acceptable design criteria is high, but choosing an economic design in such cases is not easily possible. Therefore, the design stage is a decision-making process, unlike the structural analysis stage, in which there is no choice.

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In a 2010 study, Kaveh et al. were concerned with the optimal design of 2-D steel frames with the improved ant colony optimization algorithm (ACO) [6]. Elsewhere, Kaveh et al. [8] studied the optimal design of 2-D moment steel frames with ACO and GA [8]. Sadik et al. [17] worked with the harmony search algorithm (HSA) to minimize the costs of 3-D steel frames with semi-rigid connections [17].

In a 2011 study, Kripakaran et al. operated with GA to optimize steel frames with two different methods [16]. Togan [19] employed teaching-learning-based optimization (TLBO) algorithms to minimize the weight of 2-D steel frames [19]. Saka et al. (2012)errorused PSO, GO, and HSA algorithms to optimize 2-D steel frames based on load and resistance factor design (LRFD) [2]. In a 2012 study, Kaveh et al. worked with the charged system search optimization algorithm (CSSOA) to minimize the weight of 2-D and 3-D frames [9].

Kaveh et al. [10] proposed an optimal design of 2-D frames using CSSOA and improved HAS algorithms [10]. Elsewhere, Gong et al. (2013)errorused GA to minimize the "weight of the frame" and "input seismic energy" and maximize the energy absorbed by the joint members in the braces. They further worked with the bee colony optimization algorithms (BCOA) to design the optimal capacity of a steel frame braced with divergent bracing [20].

Kociecki et al. [11] employed the two-phase GA to optimize 3-D free-form steel space-frame roof structures. These structures were two among all 13 train stations where the members were influenced by torsion and moment and axial loads. Using GA, they reduced the weight of these two structures by 12 and 7%, respectively [11]. Kaveh et al. [7] used CSSOA to optimize 2-D steel frames with semi-rigid connections [7]. Kao et al. [5] first designed several different buildings and then employed the obtained data to model a neural network that can predict the structure's behavior under design loads. Ultimately, they optimized the weight of this approximate model using evolutionary algorithms (EAs) [5]. In a 2014 study, Narimani et al. worked with optimization-based MHAs to minimize the weight of 2-D and 3-D steel frames [12]. Gholizadeh [4] optimized the study buildings using the modified firefly algorithm (MFA) [4]. Phan et al. [15] worked with GA to optimize cold-formed steel portal frames for stressed skin action. They reduced the weight of consumed materials by 53% with the help of GA [15].

Afzali et al. [1] operated optimization algorithms to minimize the weight of 2-D steel frames [1]. In a 2018 review, Ekichi et al. explored the use of collective and evolutionary intelligence algorithms in functional computing architecture. After the literature review, they proposed various statistical (e.g., the type of algorithms) and classified approaches in terms of sustainability, cost, performance, and structure. Ultimately, they reported sustainability as a factor that is at the attention center of most studies [3].

Reza Najafi, Reza Ansari, Abolfazl Darweizah and Vahid Nuraifar [14] have worked on evolutionary optimization such as genetic algorithm. In the research, the optimization starts from a fixed initial shape and with its evolution, it approaches its optimal shape. Their main emphasis in this research is to optimize the size, shape and topology of the structure at the same time. . Considering design constraints including kinematic stability of the structure, tensile and compressive stresses in elements, displacement in nodes, buckling, Euler buckling for compression members, and natural frequency constraints, to optimize the size, shape and topology of the two-dimensional truss and 3D has been used. [14].

Mostafa Sepehari Menesh and colleagues used genetic algorithm in 2019 with the aim of reducing the weight of astronaut structures by using reliability constraints [18].

Genetic algorithm is an evolutionary algorithm based on intelligent search. The purpose of this article is to find the minimum cost and minimum displacement of the structure with two sections I and W using the genetic algorithm, which will be obtained according to the limitations of the LRFD design method, the results of each of which are presented. It is worth mentioning that the optimization of steel structures with different algorithms and using limit state analysis has been common, and we have used these methods in this article.

2 Optimizations

2.1 Definition of the objective function

In this research, the objective function was the weight of the structure, which is directly correlated with the structure's cost. Thus, the structure's cost was calculated based on the profile weight parameter.

The design variables intended in this research were the dimensions of the cross-section of the frame's beams and columns. Importantly, this study reflects constraints and items specified in Standard 2800 and the Iranian National Building Code (Part 10), such as stress and displacement. The beams were subjected to the bending moment and the columns were under the synergistic effect of bending moment and axial force. The constraints of the optimization

model were the design criteria based on the analytical criteria of LRFD and ASD.

$$\text{Minimize } F = f(\text{cost}) \tag{1}$$

$$f(\text{cost}) \propto \text{Weight} \tag{2}$$

2.2 Penalty function

The objective function should be defined to allow satisfying all constraints such as the stresses in each of the elements and lateral displacements. It further should cover constraints imposed on the deformation of the structure. For best functioning, the columns of the lower stories should be of larger dimensions than those of the upper stories. Other constraints specified in the building codes need to be further reflected. In case of failure to satisfy these constraints, the objective (cost) functions will be penalized. This method simplifies the objective function and further allows to apply required constraints to increase the structure's cost or displacement, thereby automatically preventing the choice of penalized answers as the optimal solution. A multitude of methods have so far been proposed regarding the penalty function. However, the penalty function in this study is as follows:

$$C_i = \alpha_i V_i \tag{3}$$

$$\phi = \sum_{i=1}^{n} C_i \tag{4}$$

Where (C_i) is the penalty function for each constraint, (α_i) is the penalty factor for each constraint, (V_i) is the amount of penalty for each constraint, and (ϕ) is the total of penalties applied. The penalty factor is obtained by trial and error and according to the model and takes a unique for each structure.

$$\text{Penalty functions:} \quad \begin{array}{ll} \text{Tension} & \text{if} \sigma > 1 \rightarrow \alpha_1 \times (1 - \sigma) \\ \text{Shape} \longrightarrow & \alpha_3 \times (\text{shape penalti}) \\ \text{Disp} & \text{displacement} > 6.4 \rightarrow \alpha_4 \times (\text{displacement} - 6.4) \end{array}$$

3 GA

GAs are a group of search and optimization methods inspired by the principles of natural selection and genetics. These algorithms imitate the theory of natural evolution to find optimal solutions to complex problems. This evolutionary algorithm searches among a population of candidate solutions called chromosomes or individuals.

This algorithm randomly generates a number of solutions for the problem where each of these solutions is called a "chromosome" and the set of attempted (generated) solutions are called the "population". After the random population was generated, GA investigates all the chromosomes based on the merit function. Indeed, GA employs a variety of methods such as combination and mutation to select the most qualified individuals to make the next generation made up of better individuals. The combination operator selects some individuals in the population and combines them to integrate their characteristics and allow them to generate better offspring for the next generation. Likewise, the mutation operator selects a number of individuals and then alters some of their characteristics to upgrade the chosen individual to a better (improved) individual.

The number of individuals selected for combination and/or mutation will depend on the type of problem and the size of the population. Similarly, the method of choosing individuals for combination and mutation is a key factor to consider. The sought-after methods for choosing the best individuals are the roulette wheel, tournament selection, and random methods, which can be performed by trial and error according to the type of problem. For the problem concerned here, the roulette wheel outperforms other methods. GAs are usually run as a computer simulator in which the population of candidate solutions (i.e., chromosomes) for an optimization problem results in a better solution for the problem.

According to the regulatory constraints, this study introduces some cross-sections for beams and columns, where GA tries to possibly minimize the cost by choosing the best sections.

4 Regulatory constraints

The Iranian National Building Code (Part 10) has defined some constraints for the resistance of each of the elements, positioning of the elements, and examining the stress in each element.

In this study, the constraints set to the resistance of the columns are investigated based on the synergistic effect of the axial force and flexural moment of the same column. Indeed, when tensile or compressive axial force and the flexural moment in the study column exceed the values specified in the building codes and the stress ratio is greater than one, the column will be penalized. The equations are as follows:

A) If $\frac{P_r}{\phi_c P_n} \geq 0.2$
Then,

$$\frac{P_r}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{rx}}{\phi_b M_{nx}} + \frac{M_{ry}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{1}$$

B) If $\frac{P_r}{\phi_c P_n} \leq 0.2$
Then,

$$\frac{P_r}{2\phi_c P_n} + \left(\frac{M_{rx}}{\phi_b M_{nx}} + \frac{M_{ry}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{2}$$

Where P_r is the compressive strength required, P_n is the nominal compressive strength for the given section, (ϕ_c) is the compressive strength coefficient that takes a value of 0.9, M_{rx} and M_{ry} are the required flexural strength around strong and weak axes (respectively), Mnx and Mny are the nominal flexural strength around the strong and weak axes (respectively), and (ϕ_b) is flexural strength coefficient taking a value of 0.9.

The regulatory constraints set columns are also applied to beams, but the difference is that there are no interaction between axial and flexural forces in the beams and the only determinant factor will be "flexural moment". Here, according to the calculated reduction coefficients of the allowable flexural strength, and when the above condition is not satisfied, the cost function will be penalized to some values.

$$C_b = \frac{12/5\ M_{\max}}{2/5 M_{\max} + 3M_A + 4M_B + 3M_C} \tag{3}$$

A) Limit state of surrender:

$$M_{n=} M_{p=} F_{y=} Z_x \tag{4}$$

B1) If $L_b \leq L_p$ It is not necessary to consider lateral torsional buckling.

B2) For $L_p < L_b \leq L_r$:

$$M_n = C_b\left[M_p - (M_p - ./7\ F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \tag{5}$$

B3) For $L_p > L_r$:

$$M_n = F_{cr} S_x \leq M_p \tag{6}$$

$$L_p = 1.67 r_y \sqrt{\frac{E}{F_y}} \tag{7}$$

$$L_r = 1.95 r_{ts} \frac{E}{.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{.7F_y}{E}\right)^2}} \tag{8}$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + .078 \frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{9}$$

Importantly, besides the constraints mentioned above, some other constraints have also been regraded in this research. In turn, the cross-section of columns of the lower stories should definitely be larger than that of the columns of the upper stories. Therefore, if these conditions are not satisfied, or if the cross-section of columns of the lower stories are smaller than that of the columns of the upper stories, the model under consideration will be further penalized beyond the penalty set on the stress ratio. Accordingly, such problems will be automatically omitted from the optimization cycle [13].

5 The study samples

This research investigated two distinct samples to evaluate the performance of the algorithm. For this, two structures were studied. These were two-bay structures, one with three stories and the other with six stories, where the former represents a short-height and the latter represent a medium-height structure. The height of the stories was $3m$ and the length of the bays was $4m$. The live and dead loads of the stories were widely applied to the beams of the stories. Similarly, the lateral point loads were applied to the joints of the beams and columns. According to the building codes, the loads were synergistically applied as follows:

$$\begin{aligned} U &= D + 1.2L + 1.2E \\ U &= 0.85D + 1.2E \end{aligned} \tag{1}$$

Furthermore, uniform dead and live loads were equal to $Dl = 550\ kg/cm^2$ and $Ll = 200\ kg/m^2$, respectively. For the roof, these values were $500kg/m^2$ and $150kg/m2$, respectively. The compressive strength of steel was equal to $F_y = 2400\ kg/cm^2$.

We can apply various conditions to stop the algorithm, including the "condition of convergence" or "the number of iterations". By examining various "trials and errors" and choosing the best stopping strategy, we selected the condition of "the number of iterations" for this algorithm. To find the best population and number of iterations, we evaluated various populations with different iterations and ultimately picked up a population with 45 members and 250 iterations.

5.1 Study samples

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In this article, two different samples have been examined to measure the performance of the algorithm. The two investigated models of two-bay structures with three and six floors are considered so that each of the samples are representative of low and medium height structures. The height of the floors is 3 meters and the length of the openings is 4 meters.

We further considered 150 and 78 distinct cross-sections for "columns" and "beams", respectively. We further considered 267 distinct cross-sections for "W sections" according to the building codes for both the beam and column.

Sample 1

In this frame, the stories are each 3 m in height and the length of the bays is $4m$. The values of gravity loads have been reported previously, and the values of lateral loads are depicted in Figure 1. There are two types of beams and four types of columns.

$$\begin{cases} \Delta m = 0.7R\Delta w \\ \Delta m < 0.025h \end{cases} \longrightarrow \Delta w < \frac{0.025 \times 900}{0.7 \times 5} = 6.4\ cm$$

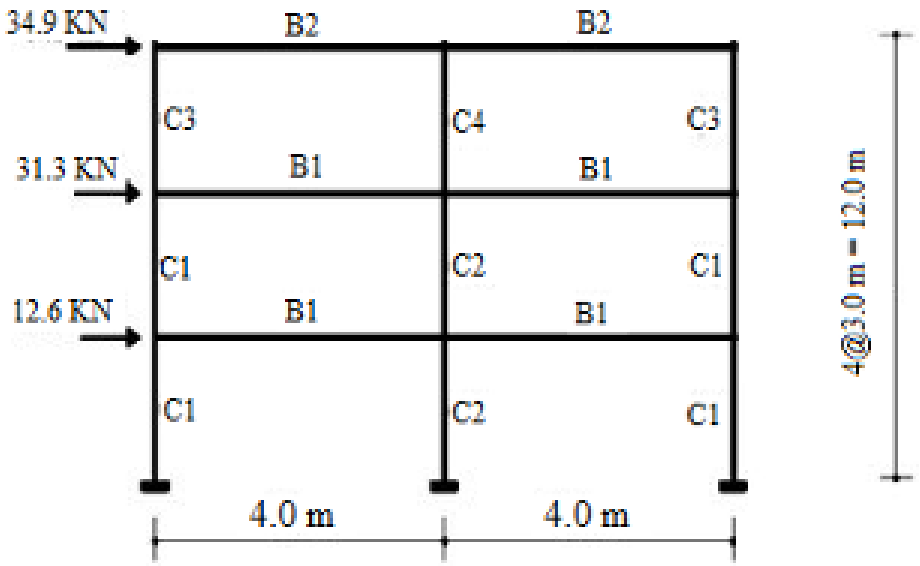


Figure 1: Sample 1

Information		
$DL = 550kg/m$	$LL = 200kg/m$	$E = 2E6$
$F = 2400kg/cm2$	$LLforroof = 250kg/m$	
Loads on beams $= 1.2 \times 550 + 200 = 860\frac{kg}{m} = 8.6\ kg/cm$		
Loads on the beams of the roof $= 1.2 \times 500 + 150 = 750\frac{kg}{m} = 7.5\ kg/cm$		

Model 1: Two-bay, three-story structure

In this model, each story is $3m$ in height and the length of each bay is $4m$. The seismic load is calculated as follows:

$H = 9m$: $T = 0.08H^{0.75} = 0.08 \times 9^{0.75} = 0.4157\ s$

$A = 0.3$, $I = 1$, II Type of ground

$T_0 = 0.1$, $T_S = 0.5$, $S = 1.5$, $S_0 = 1$

$B_1 = 2.5$, $N = 1$, $B = B.N = 2.5$, $R_U = 5$

$C = \frac{ABI}{R} = \frac{0.3 \times 2.5 \times 1}{5} = 0.15$

$W = (0.2 \times 200 + 550) \times 2 \times 8 \times 8 + (0.2 \times 150 + 550) \times 8 \times 8 + (3 \times (3-1)) \times 4 \times 8 \times 250 = 157.44\ ton$

$K = 1$, $V = C.W = 0.15 \times 157.44 = 23.616$

$f_i = \frac{w_i.h_i}{\sum w_i.h_i}$, $F = f_i \cdot V$

Table 1: Lateral loads on the structure

Story	**Load applied to the story**	**Load applied to the frame**	**Load applied to the node**
1	3.73	1.26	0.42
2	9.38	3.13	1.64
3	10.46	3.46	1.16

Since the optimization method is a MHA, we cannot consider the answers obtained after the first optimization as the best solutions. Thus, to achieve the best solution, the optimization needs to be repeated in several iterations. In this research, each model was optimized within 11 separate iterations. Figure 2 illustrates the values obtained for the best answer and the average value of answers.

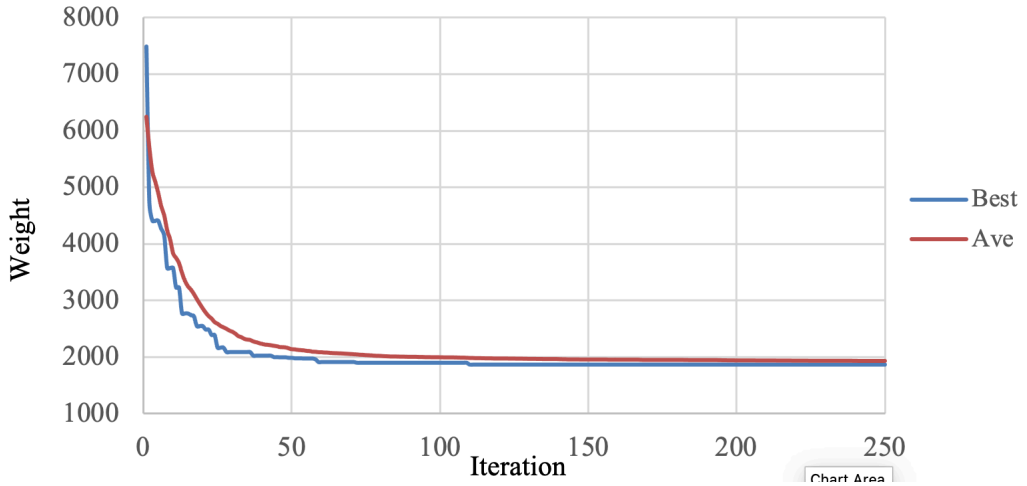


Figure 2: The optimization process for sample 1 based on the method of ultimate strength (W sections)

Regarding ultimate strength, the weight of a 3-story structure is reduced from 7491.80 kg to 1868.4 kg at best, and its average weight is reduced from 6250.9 kg to 1930.2 kg.

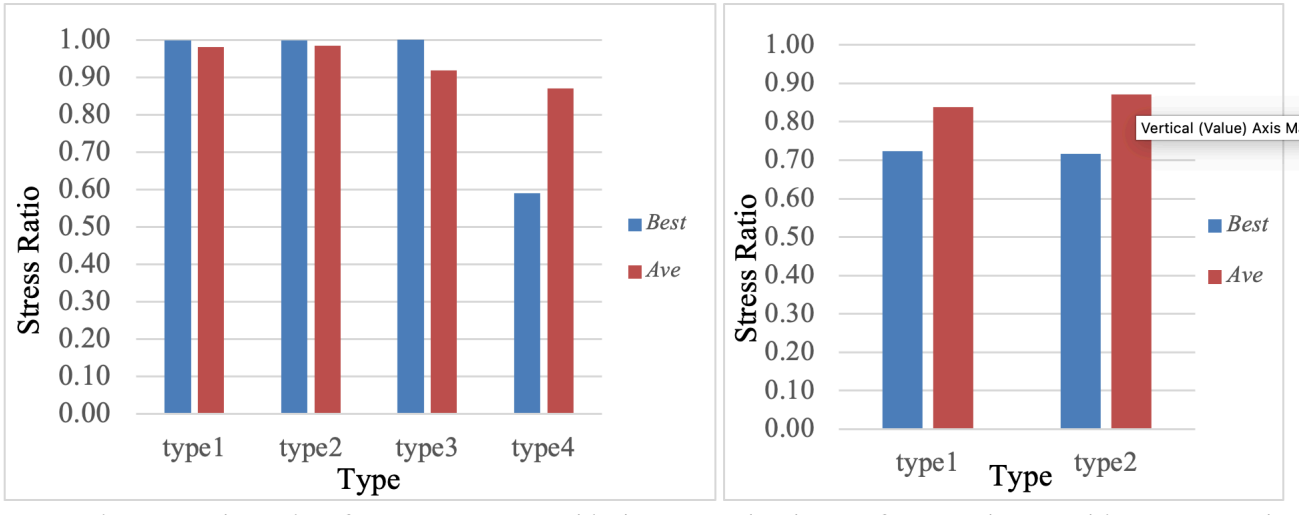


Figure 3: The ratio of stresses generated in beams and columns for sample 1 at ultimate strength (W section)

To determine the efficiency of the algorithm, we can pay attention to the values of the ratio of the stresses in the sections to their tolerable stress. Considering that six beams with two different types are considered in this model, the average amount of the existing stress ratio to the tolerable stress of the beams is shown in Figure 3.

Also, the maximum stresses created in the columns in four different types are shown in Figure 3. The remarkable thing is the very good performance of the algorithm in finding the right sections of the columns. Because the stress ratio created in the columns confirms the maximum use of the capacity of these sections, which is desirable for any optimization operation.

Table 2: Specifications of the beams (3-story) – ultimate strength

Type	W section	Stress ratio
1	W14×22	0.72
2	W12×14	0.72

Table 3: Specifications of the columns (3-story) – ultimate strength

Type	W section	Stress ratio
1	W16×26	1.00
2	W18×50	1.00
3	W12×16	1.00
4	W16×26	0.59
Best Weight		1868.40 kg
Worst Weight		7491.80 kg
Ave Best Weight		1930.21 kg
Displacement		1.61 cm

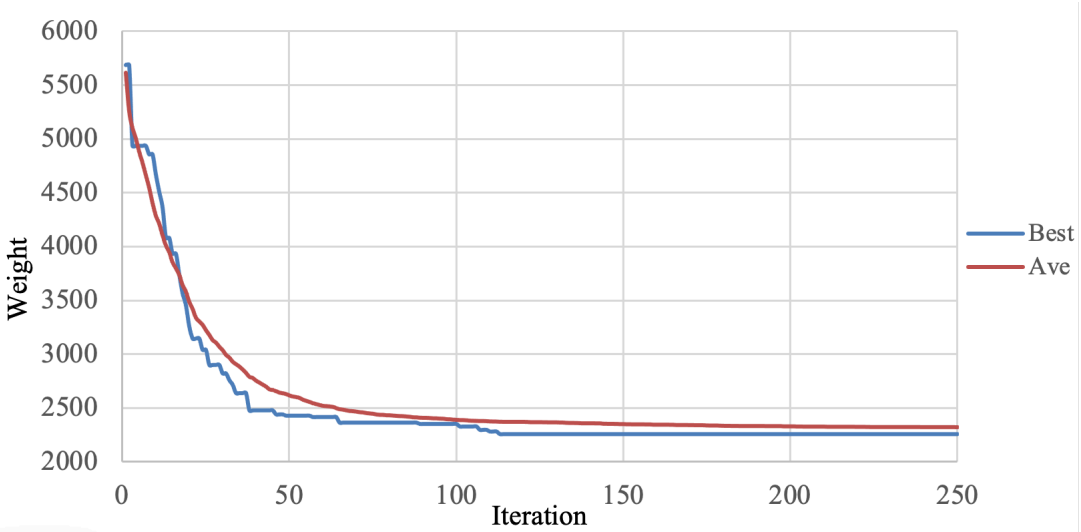


Figure 4: Optimization process of the first model using the ultimate strength method (I section)

In the ultimate strength case with I-sections in a 3-story structure, the weight decreases from 5687.8 kg to 2260.30 kg in the best case and also on average, the weight of the structure decreases from 5618 kg to 2319 kg.

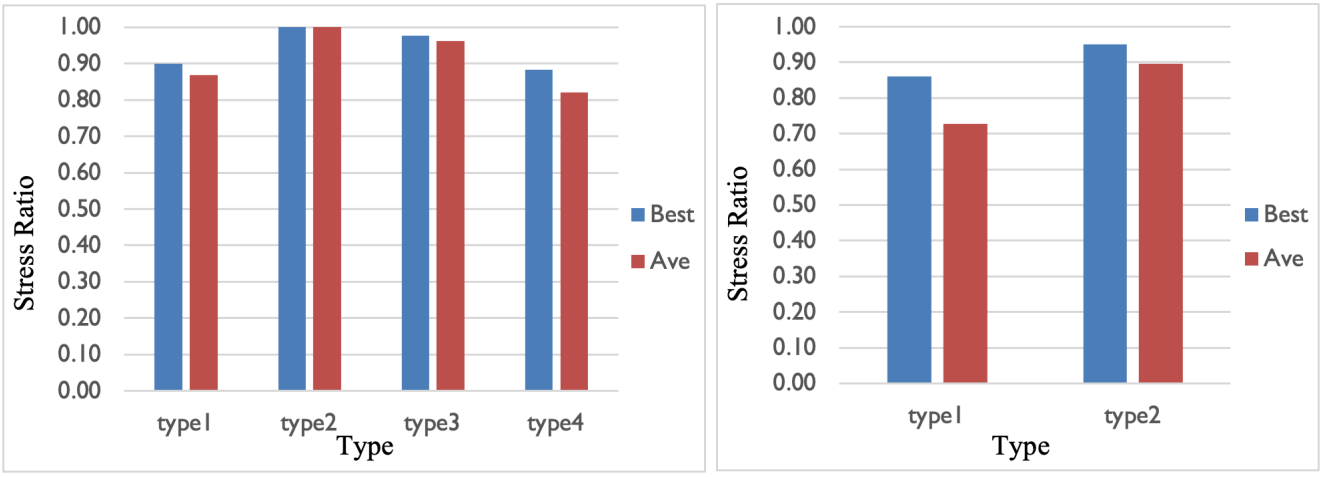


Figure 5: Ratio of stresses created in beams and columns for the first ultimate strength model (I section)

Also, for the better performance of the algorithm, only one of the beam sections, which has a stress ratio of 0.86, is replaced with a point of a smaller number. In this case, the amount of stress ratio of beams and columns increases, but the amount of increase of the stress ratio is such that the beams are in the allowed range and show better performance,

Table 4: Specifications of beams type (3 layers) - final strength

Type	I section	Stress ratio
1	IPE 240	0.86
2	IPE 160	0.95

Table 5: Column type specifications (3 floors) - final strength

Type	I section	Stress ratio
1	IPE 220	0.90
2	IPE 220	1.00
3	IPE 140	0.98
4	IPE 180	0.88

Best Weight	2260.30 kg
Worst Weight	5687.80 kg
Ave Best Weight	2318.90 kg
Displeacment	4,01 cm

but the columns are out of range. They are allowed to leave. Therefore, even by shrinking one of the sections that has a lower stress ratio than the other sections, it is not possible to reduce the weight values of the first model.

Table 6: Specifications of beams type (3 layers) - final strength

Type	I section	Stress ratio
1	IPE 180-70*10	0.77
2	IPE 160	0.96

Table 7: Column type specifications (3 floors) - final strength

Type	I section	Stress ratio
1	IPE 220	1.04
2	IPE 220	1.12
3	IPE 140	1.04
4	IPE 180	0.82

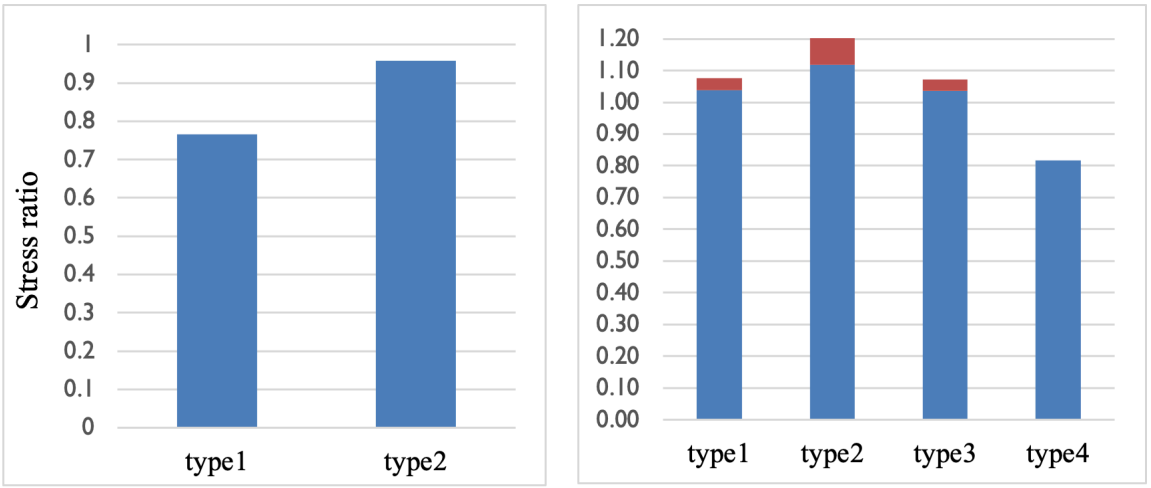


Figure 6: The ratio of stresses created for the first model in case of shrinking one of the sections (ultimate strength)

The weight comparison of sections I and W in LRFD mode is as follows:

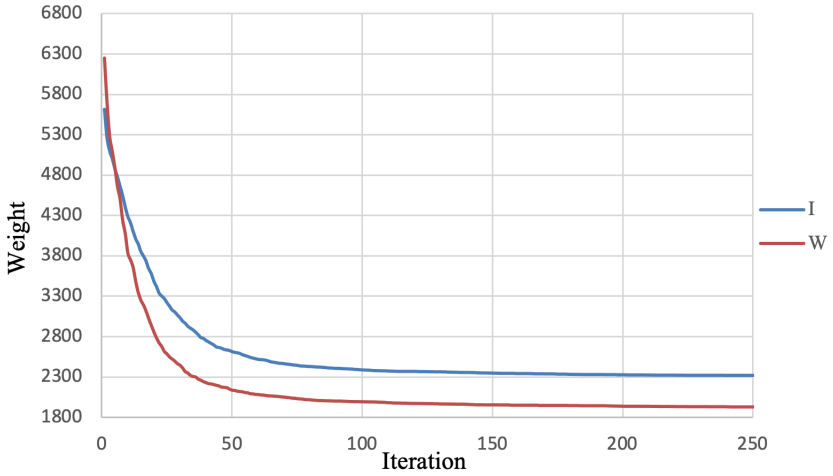


Figure 7: Showing the comparison of the optimization process of the first model for sections I and W (ultimate strength)

Sample 2

As in the first example, the length of the openings is 4 meters and the height of the floors is 3 meters for this example as well. According to the number of beams and columns, four types for beams and eight types for columns have been considered. Gravity loads including live load and dead load are the same as the previous example and there is no difference. The values of lateral loads and the type of beams and columns are given in figure 8. In order to clarify the issue better, the values of the stress on the beams and columns are also given separately and in different forms. Allowable displacement values are calculated as follows.

$$\begin{cases} \Delta m = 0.7R\Delta w \\ \Delta m < 0.02h \end{cases} \longrightarrow \Delta w < \frac{0.02 \times 900}{0.7 \times 5} = 20.57\ cm$$

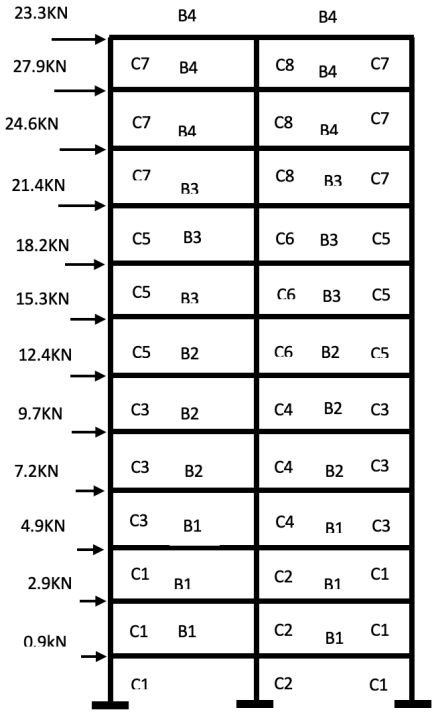


Figure 8: Show the second model under review

The second model (two openings, twelve floors)

Earthquake load calculations:

$H = 36m$: $T = 0.08H^{0.75} = 0.08 \times 36^{0.75} = 1.176\ s$

$A = 0.3$, $I = 1$, section II

$T_0 = 0.1$, $T_S = 0.5$, $S = 1.5$, $S_0 = 1$

$B_1 = (S + 1)\left(\frac{T_S}{T}\right) = 1.06$, $N = 1$, $B = B.N = 1.06R_U = 5$

$C = \frac{ABI}{R} = \frac{0.3 \times 1.06 \times 1}{5} = 0.0636$

$W = (0.2 \times 200 + 550) \times 5 \times 8 \times 8 + (0.2 \times 150 + 500) \times 8 \times 8 + (3 \times (6 - 1)) \times 4 \times 8 \times 250 = 713.28$

$K = 0.5T + 0.75 = 1.388$, $V = C.W = 0.107 \times 342.72 = 36.67$

$f_i = \frac{w_i.h_i}{\sum w_i \cdot h_i}$, $F = f_i \cdot V$

Table 8: Lateral forces acting on the structure

Load applied to the node	Load applied to the frame	Load applied to the story	Story
1	0.27	0.09	0.03
2	0.86	0.29	0.1
3	1.47	0.49	0.16
4	2.17	0.72	0.24
5	2.92	0.97	0.32
6	3.73	1.24	0.41
7	4.58	1.53	0.51
8	5.48	1.82	0.61
9	6.41	2.14	0.72
10	7.38	2.46	0.82
11	8.38	2.79	0.93
12	7.2	2.33	0.78

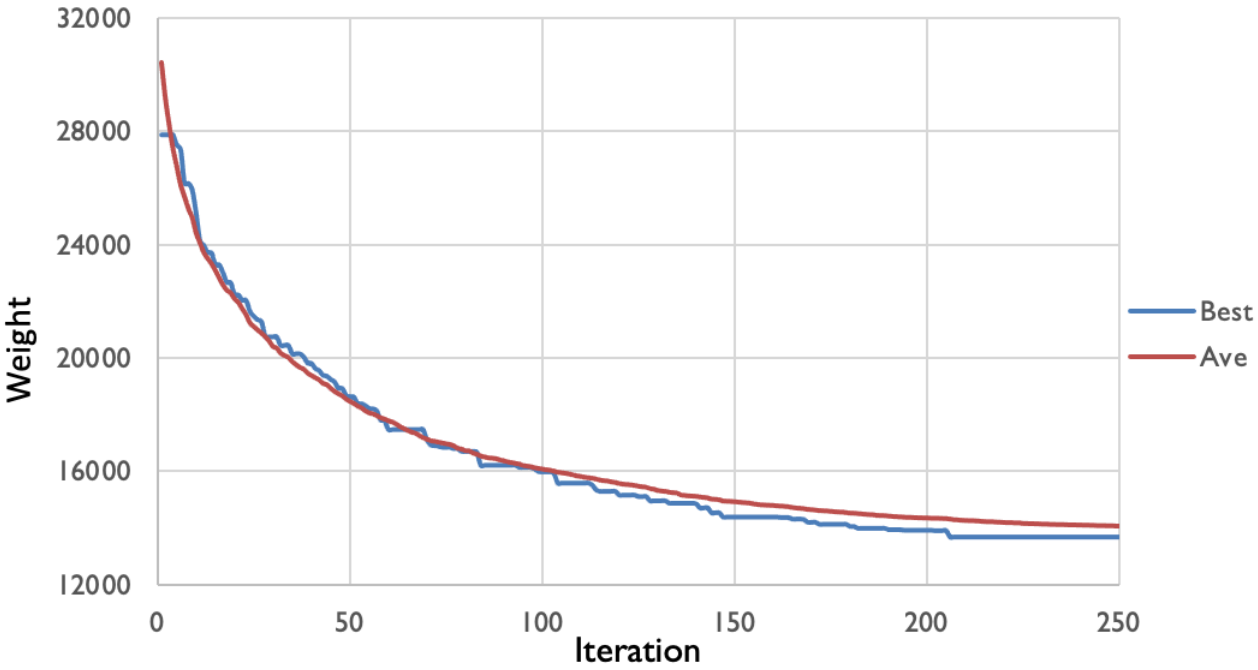


Figure 9: Showing the process of optimization of the second ultimate resistance sample (I section)

In the state of final strength with the I section in the 12-story structure, the weight has been reduced from 28,000 kg to 14,000 kg in the best case.

Figure 10 shows the average stress values of beams and columns for the first example of the LRFD method. The ratio of stresses obtained in both cases were favorable values in such a way that almost all the capacity of columns and beams was used.

The presented figure shows the highest stress in the beams of the seventh, eighth and ninth floors, for which the calculated stress is 0.93 Meanwhile, the lowest stress is related to the beams of the first and second floors, which have a stress equal to 0.80. For the columns, the maximum and minimum stress values are 1.00 and 0.89, respectively.

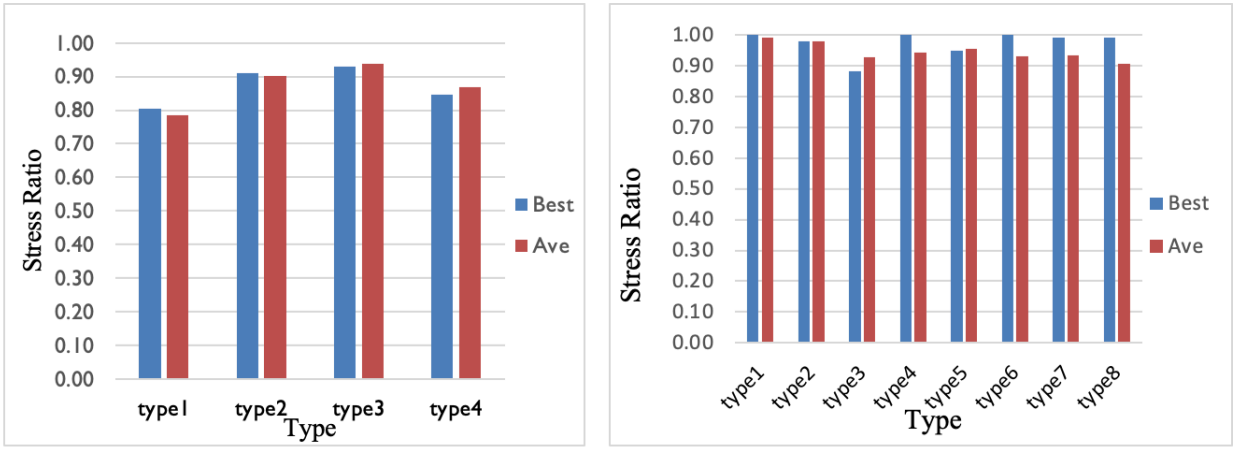


Figure 10: The ratio of stresses created in beams and columns of the third model (ultimate strength)

Table 9: Specifications of columns type (12-Story) – LRFD

Type	I section	Stress ratio
1	IPE 300	1.00
2	IPE IPB 260-230*18	0.98
3	IPE 260	0.88
4	IPE 300	1.00
5	IPE 220	0.95
6	IPE 280	1.00
7	IPE 180	0.99
8	IPE 220	0.99

Table 10: Specifications of the beam type (12 floors) – LRFD

Type	I section	Stress ratio
1	IPE 330	0.80
2	IPE 300	0.91
3	IPE 270	0.93
4	IPE 220	0.85

Best Weight	13682 kg
Worst Weight	27881 kg
Ave Best Weight	14078.90 kg
Displeacment	19.29 cm

Figure 3-5 The second sample with W sections:

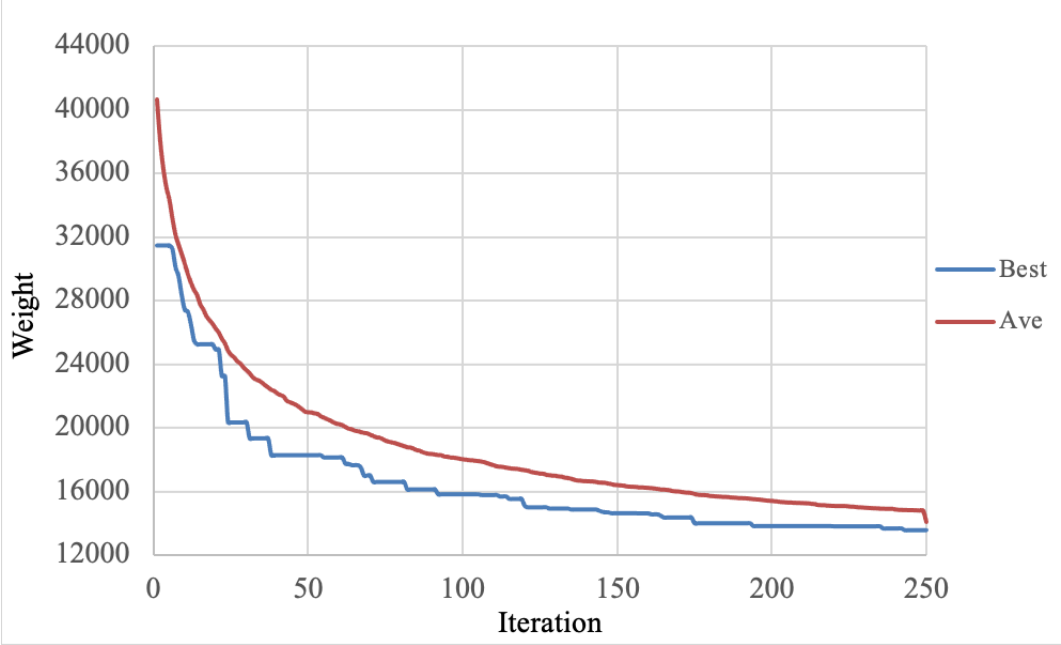


Figure 11: Showing the optimization process of the second sample in the state of ultimate strength (W section)

In the state of ultimate strength with W sections in a 12-story structure, the weight was found to be from 31483 kg to 13567 kg in the best condition, and also on average, the weight of the structure decreased from 40636 kg to 5648 kg

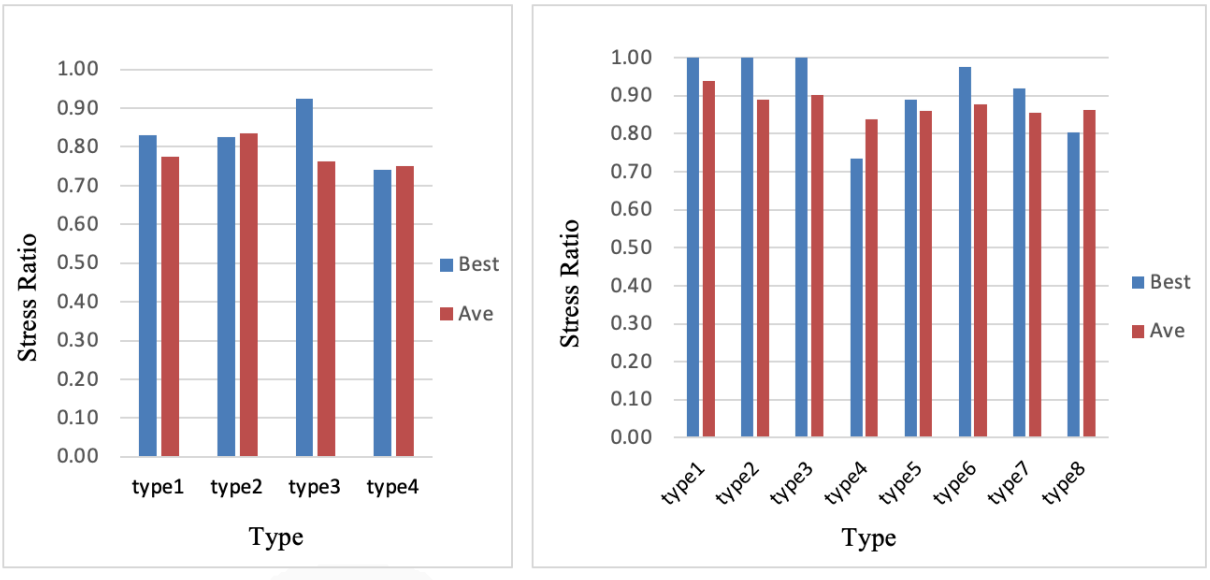


Figure 12: The ratio of stresses created in beams and columns (W section)

Table 11: Specifications of the type of beams (12 floors) - final strength

Type	I section	Stress ratio
1	W16X26	0.83
2	W16X26	0.83
3	W10X22	0.92
4	W8X21	0.74

Table 12: Specifications of columns type (12 floors) – LRFD

Type	I section	Stress ratio
1	W10X68	1.00
2	W14X109	1.00
3	W12X58	1.00
4	W10X100	0.73
5	W10X54	0.89
6	W12X79	0.98
7	W8X31	0.92
8	W10X49	0.80

Best Weight	13567 kg
Worst Weight	31483 kg
Ave Best Weight	14078 kg
Displeacment	17.49 cm

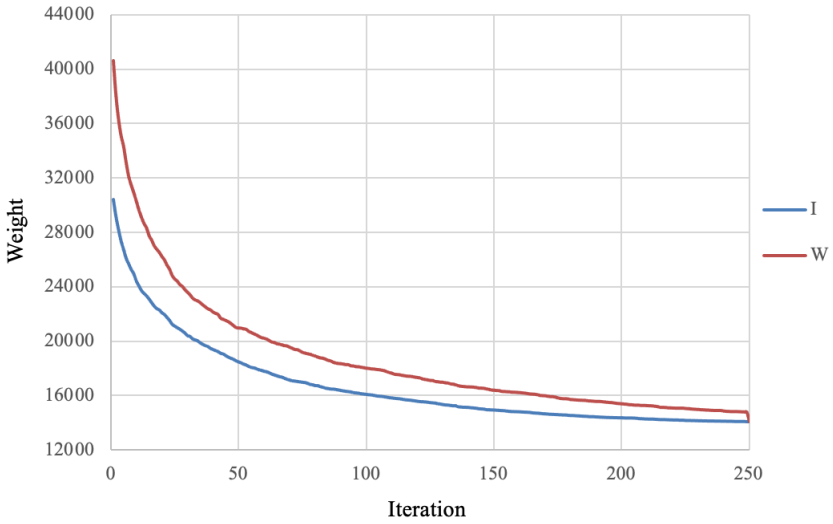


Figure 13: Showing the comparison of the optimization process of the second model for sections I and W (ultimate strength)

Comparison of the stress trend with SAP software:

Table 13: Specifications of beams type (3 layers) - final strength

Type	I section	Stress ratio
1	IPE 240	0.86
2	IPE 1608	0.95

Table 14: Column type specifications (3 floors) - final strength

Type	I section	Stress ratio
1	IPE 220	0.90
2	IPE 220	1.00
3	IPE 140	0.98
4	IPE 180	0.88

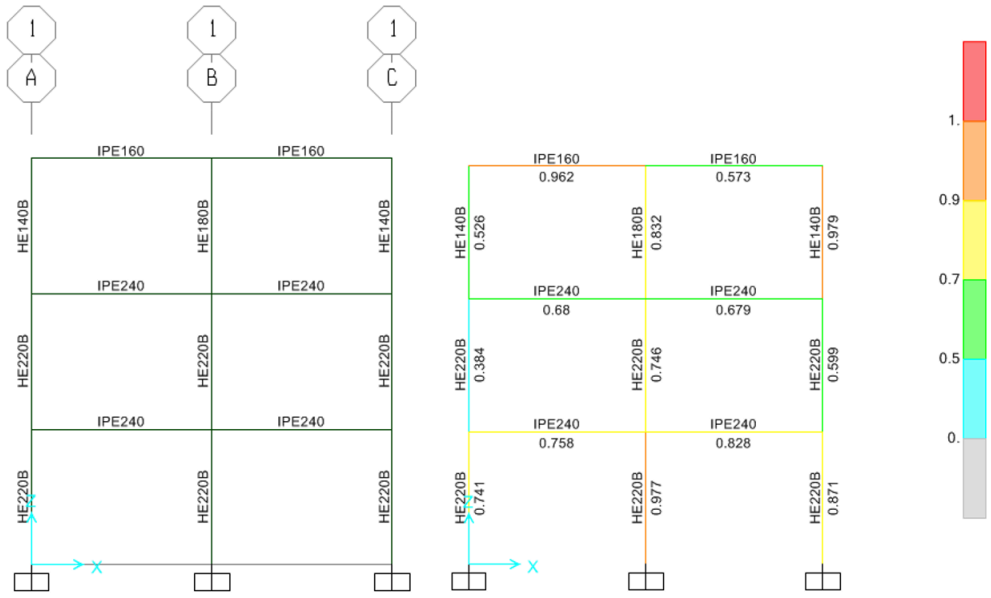


Figure 14: The process of obtaining tension in SAP 2000 software- I section

Comparison of tensions in Opensees and Sap software:

Table 15: Comparison of stresses in beam sections of a 3-story structure in Opensees and Sap software

Type	I section	Opensees	Sap
1	IPE 240	0.86	0,83
2	IPE 160	0.95	0.96

Table 16: Comparison of stresses in beam sections of a 3-story structure in Opensees and Sap software

Type	I section	Opensees	Sap
1	IPB 220	0.90	0,87
2	IPB 220	1.00	0.98
3	IPB 140	0.98	0.98
4	IPB 180	0.88	0.96

Comparison of the stress trend with Etabs software:

Table 17: Specifications of the beams (3-story) – ultimate strength

Type	W section	Stress ratio
1	W14×22	0.72
2	W12×14	0.72

Table 18: Specifications of the columns (3-story) – ultimate strength

Type	W section	Stress ratio
1	W16×26	1.00
2	W18×50	1.00
3	W12×16	1.00
4	W16×26	0.59

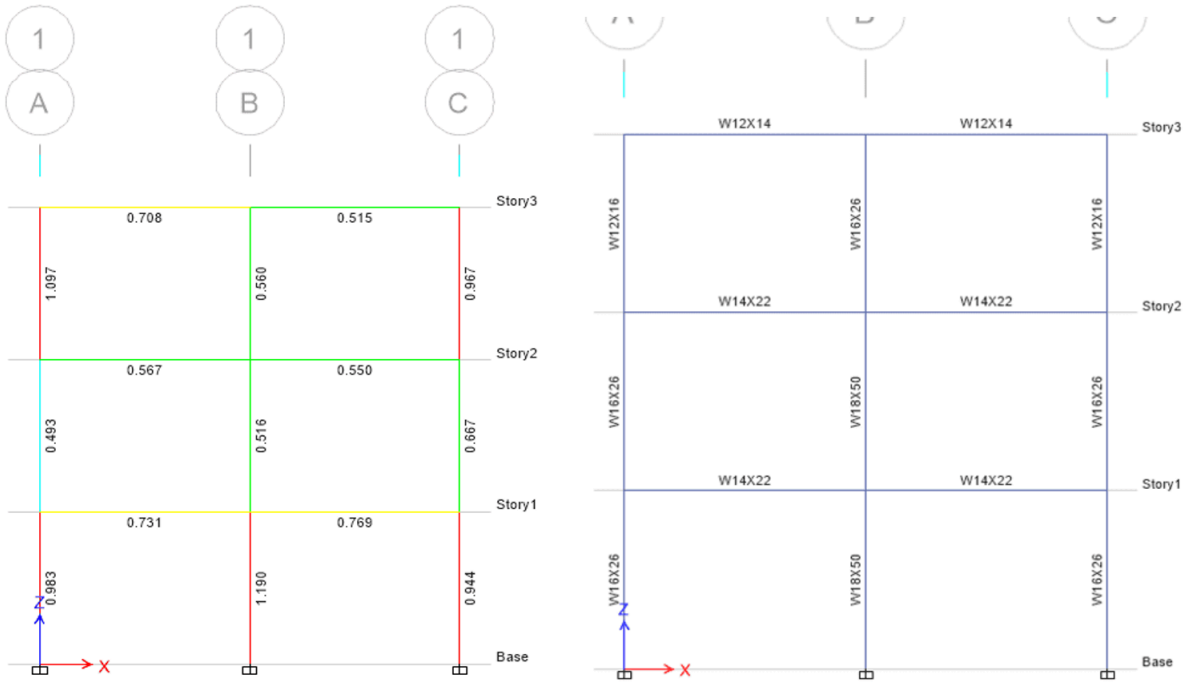


Figure 15: The process of obtaining tension in Etabs software -W section

Comparison of tensions in Opensees and Etabs software:

Table 19: Comparison of stresses in beam sections of a 3-story structure in Opensees and Etabs software

Type	W section	Opensees	Etabs
1	W14×22	0.72	0,77
2	W12×14	0.72	0.71

Table 20: Comparison of stresses in beam sections of a 3-story structure in Opensees and Etabs software

Type	W section	Opensees	Etabs
1	W16×26	1.00	0.98
2	W18×50	1.00	1.00
3	W12×16	1.00	1.00
4	W16×26	0.59	0.56

6 Conclusion

In order to determine how the algorithm of each model works, graphs of the weight of the structure in relation to the number of repetitions and the number of calls of the objective function have been collected. In addition, the stress ratio of each type of beam and column is given separately for each model.

Finally, the cost of the structure, the ratio of stresses and the change of location show the optimal performance of the algorithm. It can also be found by performing various analyzes that the algorithm shows very little dependence on the initial population, which can be considered one of the advantages of this algorithm.

In general, the results obtained during optimization can be presented as follows:

1. The values of the stresses in the beams and columns confirm that in the LRFD method, all the capacity of the section has been used, which is considered the final goal of the algorithm.

2. According to the graphs presented in relation to how the algorithm works, it can be concluded that the algorithm did not stop at the local optimum and achieved the overall optimum.

3. In the comparison of optimization using I section and W section by LRFD method, we can see that in the first example including the structure with low height, W section showed better performance than I section and provided a lighter structure. But in the high-height structure, I section sections showed better performance than W section sections.

4. It can be concluded that W section for low height structure and I section for high height structure have better performance.

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