

Optimal design of eccentric braced frames using IJAYA algorithm

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(Communicated by Mohammad Bagher Ghaemi)

Abstract

Optimum design of structures is one of the important design goals because it improves the structural and architectural features and reduces the cost of building the structure. In most of the research, the weight of the structure has been considered as a function of the cross-sectional area of the members and the location of the braces in the braced frames, which should be according to the rules and regulations of the case. Follow the designer's opinion. In order to optimize steel frames, many researchers use meta-heuristic algorithms; each of these algorithms, with its own mechanisms, has optimized the objective function in steel frames (weight). In this paper eccentric braced frame (EBF) is the chosen system for weight optimization. The most common method of modelling the link beam is almost ineffective for evaluating its performance according to FEMA 356. For this reason, in this study, a method was first presented for the modelling of the link beam, based on which it is possible to evaluate the performance of the link beam. Then the optimization problem was defined based on the acceptance criteria of the performance-based design method. In order to measure the possibility of optimization based on the defined problem, the optimal design of two EBFs was discussed using an improved JAYA (IJAYA) meta-heuristic algorithm and the results of their optimization were presented. Results show better performance of the IJAYA algorithm optimization relative to other algorithms.

Keywords: EBF, optimal design, JAYA algorithm, link beam
2020 MSC: Primary 90C90; Secondary 65K10, 74P10

1 Introduction

Nowadays, the design of structures can be considered as an optimization problem. A problem whose goal is to obtain the lowest possible weight for the structure by meeting all the requirements and limitations of resistance, exploitation and implementation. In order to solve such optimization problems, which have many variables and it is necessary to satisfy many constraints, it is practically impossible to use classical mathematical methods such as derivation, because the amount and complexity of the calculations of classical methods in such problems is very high. This problem has caused the complexity of the design to be reduced in many designs by considering conservative limits, which makes the final design far from an optimal design. Today, various methods are proposed to solve such problems, which are often based on the assumption of the answer to the problem and trial and error. Some of these methods are numerical methods. The main problem with these methods is that the final answer is dependent on the initial

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assumption. This issue makes numerical methods almost ineffective in solving problems that have local optima. One of the most efficient methods suggested today to solve such problems is the use of meta-heuristic algorithms. These algorithms are often population-based and inspired by a natural phenomenon. Meta-heuristic algorithms search the response space in a targeted manner by using the random property and have the ability to go beyond local optima. Other features of these algorithms include their ease of use, optimal flexibility for most optimization problems, and appropriate speed of solution.

Kaveh and Talatahari in [9] used the Charged System Search (CSS) algorithm to optimize three steel bending frames (two two-dimensional frames and one three-dimensional frame). Toğan [20] used the optimization algorithm based on TLBO to optimize two two-dimensional steel bending frames. In the same year, the optimal design of four steel structures (two space trusses and two steel bending frames) was carried out by Talatahari et al., using the Artificial Bee Colony (ABC) algorithm [19]. Murren and Khandelwal proposed a design based on harmony search to solve the optimization problem by applying improvements to the harmony search algorithm. They investigated the performance of this algorithm in two articles: They addressed the optimization of three-dimensional steel bending frames on a large scale in [13], and they addressed the optimal design of three two-dimensional steel bending frames in [14]. Another modification applied to the harmony search algorithm to increase the escape speed of this algorithm from the local optimum was done by Maheri and Narimani, and the modified harmony search was presented. In their study, to prove the effectiveness of the applied improvement, four steel bending frames were optimized [11]. Talatahari et al., by combining the eagle strategy algorithm with the differential evolution algorithm, presented the DE-ES meta-heuristic algorithm. The ability of this algorithm to solve the weight optimization problem of steel bending frames was evaluated on three two-dimensional frames and one three-dimensional frame [18]. Kaveh and Ilchi Ghazaan, by improving the convergence speed and response accuracy of the whale optimization algorithm, presented the advanced whale optimization algorithm. In their research, four weight optimization problems of steel structures (two spatial trusses and two two-dimensional bending frames) have been solved using the presented algorithm [7]. Another meta-heuristic algorithm that was used to solve the problem of optimal design of steel frames is the Ant Lion Optimizer (ALO) algorithm. Talatahari [17] used the algorithm for the optimal design of three steel structures (a three-dimensional truss, a large-scale two-dimensional truss and a large-scale three-dimensional bending frame). Kaveh and Ilchikzan investigated the effectiveness of particle swarm optimization (PSO) algorithms with some other algorithms in solving weight optimization problems. The authors chose two large-scale spatial trusses and two two-dimensional bending frames as investigated problems and compared the results of solving these problems with the results of other algorithms [8].

Kaveh et al. worked on the optimal design of three benchmark 2-D steel bending frames by introducing modified dolphin monitoring (MDM) and using it with the Colliding Bodies Optimization (CBO) algorithm as the combined MDM-CBO algorithm [6]. Fathali and Vaez presented an optimum performance-based design of eccentrically braced frames. The main objective of their work was to examine the possibility of optimizing the EBF weight based on performance-based design (PBD) [3]. Ramonell and Chacon presented a systematic implementation of structural optimization in different EBF configurations. The goal of their work was the topological optimization of horizontal links in eccentrically braced frames [15]. The application of the genetic algorithm in optimization of the Y-shape EBF presented by Liang-dong et al. The results of their work showed good convergence of the proposed genetic algorithm (GA) for the seismic performance optimization problem of the frames [10]. Mazdarani et al. presented an approach for a reliability-based optimal design of a 3-D concentric braced frame (CBF). The objective function in their paper is considered as a reduction in the total weight of the frame [5]. Marzok and Lavan tried to optimal seismic design of multiple-rocking steel concentrically-braced frames in 3D irregular buildings. Their design method was applied for the design of an 8-story building with two setbacks and high torsional irregularities [12]. Alanani and Elshaer developed a structure-wind optimization framework (SWOF) to find the optimal shear wall layout of tall buildings subjected to wind loads. SWOF is considered a GA based framework that uses an Artificial Neural Network (ANN) surrogate model [1].

Based on the above-mentioned literature, in this paper, each of the performance levels according to FEMA 356 has been matched with one of the risk levels of earthquakes. Therefore, by performing non-linear static analysis and obtaining the structural capacity curve (base shear against target node displacement), a target displacement is obtained for each of the performance levels. Then, using meta-heuristic algorithms and MATLAB and ABAQUS software, the optimal design based on the performance of the eccentric braced frame is discussed. An optimal design response is considered to have the least weight by meeting all the constraints and requirements of the performance levels. In other words, the lightest design that behaves in each of the seismic risk levels within the performance range related to that level is the optimal design. Moreover, in order to evaluate the performance of the plastic joint of the link beam, a new method for behavioural modelling of the plastic joint of the link beam has been presented.

2 Presented method of modeling

An eccentric brace is a brace in which at least one of the two ends of the diagonal members cuts the axis of the beam at a short distance from the intersection of the beam with the column or the beam with another diagonal member. Among the advantages of the eccentric bracing frame (EBF), it is possible to reduce the amount of shear force and bending moment in order to reduce the dimensions of the foundation, reduce the lateral displacement and reduce the effect of $P - \Delta$ in the columns compared to the bending frame. The main reason for the high stiffness of the frame and its suitable plasticity is the presence of the link beam. The link beam is like a malleable fuse that prevents the failure of other members. The reduced energy of the earthquake or other lateral loads in the beam system is dissipated by the method of stability. To achieve this goal, it is necessary to design the link beam in a completely correct way so that it has the necessary strength and stability to absorb energy. Also, all other members of the structure should be designed in such a way that they remain elastic when the lateral loads of the earthquake are applied.

If the link beam has a shear behavior, rotation does not occur on both sides of plastic shear hinge, but according to Table 1 and Table 2, it can be seen that in all the behavior states of the link beam, the acceptance criteria and the parameters of the generalized behavior model of this beam are expressed based on the degree of rotation of the beam as shown in Table 2. This issue makes it necessary to adopt a method for modelling the link beam based on which the performance of the link beam can be evaluated.

Table 1: Modeling parameters of the generalized nonlinear behavior of the link beam

| FEMA 356 An American National Standard | | | |
|---|---|--------------|-----------------------|
| Formula | Behavior of link beam | | Modeling parameters |
| | Plastic rotation angle | | Residual stress ratio |
| | a | b | c |
| $e \leq 1.6 \frac{M_{CE}}{V_{CE}}$ | $15\theta_y$ | $17\theta_y$ | 0.8 |
| $e \geq 2.6 \frac{M_{CE}}{V_{CE}}$ | $9\theta_y$ | $11\theta_y$ | 0.6 |
| $1.6 \frac{M_{CE}}{V_{CE}} < e < 2.6 \frac{M_{CE}}{V_{CE}}$ | Linear interpolation between above parameters | | |

where:
 M_{CE} =Expected bending capacity of the link beam, (N.m)
 V_{CE} =Expected shear capacity of the link beam, (N)
 e =length of link beam, (m)
a, b and c=Generalized force-deformation curve parameters
 θ_y =Rotation in yield strength , (Radians)

Table 2: Acceptance criteria of link beam in nonlinear methods

| FEMA 356 An American National Standard | | | |
|---|---|--------------|------------|
| Formula | Acceptance criteria | | |
| | CP | LS | IO |
| $e \leq 1.6 \frac{M_{CE}}{V_{CE}}$ | $14\theta_y$ | $11\theta_y$ | 0.5 |
| $e \geq 2.6 \frac{M_{CE}}{V_{CE}}$ | $8\theta_y$ | $6\theta_y$ | θ_y |
| $1.6 \frac{M_{CE}}{V_{CE}} < e < 2.6 \frac{M_{CE}}{V_{CE}}$ | Linear interpolation between above parameters | | |

where:
 M_{CE} =Expected bending capacity of the link beam, (N.m)
 V_{CE} =Expected shear capacity of the link beam, (N)
 e =length of link beam, (m)
CP, LS and IO=Collapse Prevention, Life safety and Immediate Occupancy Performance
 θ_y =Rotation in yield strength , (Radians)

In this paper, to model the plastic hinge behavior of link beam instead of two plastic shear and bending hinge, only one plastic bending hinge is modeled at both ends of the link beam and to model the behavior of the link beam, the behavior of the bending hinge is defined based on equation (2.1). With this modeling method due to the creation of pure rotation, it is possible to evaluate the performance of the plastic hinge of the link beam.

$$Q_{CE,M} = \begin{cases} \frac{eV_{CE}}{2}, & e \leq \frac{1.6M_{CE}}{V_{CE}} \\ \left[1.37 - 0.23 \frac{eV_{CE}}{M_{CE}} \right] \frac{eV_{CE}}{2}, & \frac{1.6M_{CE}}{V_{CE}} < e \leq \frac{2.6M_{CE}}{V_{CE}} \\ M_{CE}, & e > \frac{2.6M_{CE}}{V_{CE}} \end{cases} \quad (2.1)$$

in which: $Q_{CE,M}$ is expected strength of link beam.

In this way, to model the behavior of the link beam, only the values of the yield rotation of the link beam, the expected strength of the link beam, and the slope of the strain hardening section are needed. The reason for not considering the loss of strength in the behavior of the link beam, in addition to the ease of modeling, is the inability to accurately model the loss of strength in some finite element software, which is used to validate the modeling results. The stages of modeling the link beam and its plastic hinges have been shown in Table 3.

Table 3: The stages of modeling the link beam and its plastic hinges

| | |
|----------|--|
| Step 1: | Calculation of V_{CE} and M_{CE} |
| Step 2: | Determination of limits of $1.6M_{CE}/V_{CE}$ and $2.6M_{CE}/V_{CE}$ |
| Step 3: | Determination of link beam behavior (bending, shear or combined) |
| Step 4: | Calculation of $Q_{CE,M}$ |
| Step 5: | Calculation of θ_y (for short link beam: $\theta_y = 0.01$ Radians) |
| Step 6: | Calculation of F_y (Yield strength) |
| Step 7: | Calculation of E_0 (The initial slope of the elastic part) |
| Step 8: | Calculation of b (The ratio of the slope after yielding to the initial slope of the elastic part) |
| Step 9: | Calculation of E (Corrective elastic stiffness of the elastic beam-column element) |
| Step 10: | Definition of material properties based on Steps 6 to 8 |
| Step 11: | Definition of two co-ordinate nodes in the places of hinge formation (two ends of the link beam) |
| Step 12: | Dependency of translational degrees of freedom of co-ordinate nodes |
| Step 13: | Definition of zero length element between co-ordinate nodes |
| Step 14: | Definition of elastic beam column element between the nodes of two ends of the link beam with E parameter (from step 9) and section properties |

3 Optimization of eccentric braced frame problems

In this section, the definition of the optimization problem, which includes the definition of the objective function and constraints is discussed first. Then the meta-heuristic algorithms used in this paper will be introduced and explained. At the end, the investigated problem is described and its optimization results are presented.

The cost of implementing a structure is one of the most important design features, and the lower it is, the more optimal the design will be. One of the main factors affecting the final cost of a structure is its weight. Assuming that the overall geometry of the structure is constant in steel structures, the selected cross-section for the members determines an important part of the structure's weight. So the goal of the current problem is to choose an arrangement of sections that meets the design requirements while having the least weight. In other words, the task of the optimization algorithm is to minimize the Z function according to equation (3.1).

$$Z(x) = F(x) + P(x) \quad (3.1)$$

where X is a vector that specifies the sections of the structure and is determined by the algorithm; F is the objective function, which is a measure of the weight of the structure; P is the function of the penalty for violation of design restrictions and requirements.

In the following, the objective function of the optimization problem will be explained, and then the hypothetical constraints will be presented. Also, at the end, the general process of calculating the objective function is stated. Objective function in addition to the weight of the structure is also considered as a measure of the damage of the structure under the lateral load of the earthquake, as presented by Grierson et al. in [4] and shown in equation (3.2):

$$F(x) = \omega_1 F_1 + \omega_2 F_2 \quad (3.2)$$

where F_1 is the normalized weight of the structure relative to its maximum weight, and F_2 is a measure of structural failure. ω_1 and ω_2 are considered as 0.95 and 0.05, respectively, based on Grierson et al. [4] Recommendations. Details of calculations of F_1 and F_2 have been mentioned in [4].

Hypothetical constraints in this research can be divided into three parts: member acceptance criteria controlled by deformation, acceptance criteria of members controlled by force and geometrical (executive) constraints. For each of the restrictions, a violation estimation function is defined. If the condition is met, the value of the function will be

zero, and if the condition is not met, the value of the function will be a positive value and equal to the violation of the condition. In order to move the optimization algorithm away from the illegal limits, for each constraint, the penalty function for violating that constraint is defined according to the following equation (3.3):

$$P = \alpha.V \quad (3.3)$$

in which: α is the penalty coefficient, the value of which is stated in the explanation of each of the constraints. The penalty coefficient is a measure of the value of fulfilling a constraint. In other words, the more valuable the fulfillment of a constraint is the higher the penalty factor will be. The constraint considered for members controlled by deformation is defined according to equation (3.4). The explanation is that in the eccentric braced frame, the only member that has this condition is the link beam.

$$|\theta_{P,j}^i| \leq \theta_{A.C.j}^i \quad (i = IO, LS, CP; j = 1, 2, \dots, 2n_{L.B}) \quad (3.4)$$

in which: $\theta_{P,j}^i$ and $\theta_{A.C.j}^i$ are the plastic rotation and its acceptance criteria in the j th plastic hinge at the i th performance level respectively. $n_{L.B}$ is the number of link beams, which due to the presence of two plastic hinges in each link beam, the number of plastic hinges will be twice the number of link beams.

The violation of the plastic rotation of the plastic hinges of the link beams can be expressed based on the $V_{L.B}$ function according to equation (3.5).

$$V_{L.B} = \sum_i \left(\sum_{j=1}^{2n_{L.B}} \max \{ (|\theta_{P,j}^i| - \theta_{A.C.j}^i) . 0 \} \right) \quad (i = IO, LS, CP) \quad (3.5)$$

The penalty function for exceeding the acceptance criteria is defined according to relation (3.6).

$$P_{L.B} = \alpha_{L.B} . V_{L.B} \quad (3.6)$$

in which: $\alpha_{L.B}$ is the penalty factor for violation of the plastic rotation of the plastic hinges of the link beams which is considered 100 in this paper.

Except for the link beam, all members of the eccentric braced frame are force-controlled. Therefore, at all levels of performance, the capacity of these members must respond to the force created in them. For beams in unbraced spans, since the connection of the beam to column has been considered as hinge connection, the bending moment created in these beams is only due to gravity load. Therefore, the lateral load does not affect it. As a result, the violation function is not written for these beams. Beams of braced spans are subjected to the interaction of axial force and bending moment. The constraint related to these beams according to equation (3.7) is defined as follows:

$$\frac{|(P_{UB})_j^i|}{(P_{BL})_j} + \frac{|(M_{UB})_j^i|}{(M_{BL})_j} \leq 1 \quad (i = IO, LS, CP; j = 1, 2, \dots, n_B) \quad (3.7)$$

in the above relationship, $(M_{UB})_j^i$ is the maximum bending moment created in the j th beam at the i th performance level. $(M_{BL})_j$ is the lower bound is the bending strength of the beam. $(P_{UB})_j^i$ and $(P_{BL})_j$ are the maximum axial force of beam j at performance level i and the lower limit of axial resistance of beam j respectively. The lower bound of the axial resistance of the beam is calculated like a column. n_B is also the number of beams in the braced spans of the structure.

Equations (3.8) and (3.9) respectively express the violation function and the penalty function of the combination of the bending moment and the axial force of the beams.

$$V_{B,P-M} = \sum_i \left(\sum_{j=1}^{n_B} \max \left\{ \left(\frac{|(P_{UB})_j^i|}{(P_{BL})_j} + \frac{|(M_{UB})_j^i|}{(M_{BL})_j} - 1 \right) . 0 \right\} \right) \quad (i = IO, LS, CP) \quad (3.8)$$

$$P_{B,P-M} = \alpha_{B,P-M} . V_{B,P-M} \quad (3.9)$$

in which: penalty factor coefficient $\alpha_{B,P-M}$ is considered 10 in this paper. Beam sections are selected from the list of American AISC W-Shapes.

The constraint related to the combination of axial force and bending moment in columns can be defined based on equation (3.10) as follows:

$$\frac{|(P_{UC})_j^i|}{(P_{CL})_j} + \frac{|(M_{UC})_j^i|}{(M_{CL})_j} \leq 1 \quad (i = IO, LS, CP; j = 1, 2, \dots, n_c) \quad (3.10)$$

In the above relationship, $(M_{UC})_j^i$ is the maximum bending moment created in the j th column at the i th performance level. $(M_{CL})_j$ is the lower bound is the bending strength of the column. $(P_{UC})_j^i$ and $(P_{CL})_j$ are the maximum axial force of column j at performance level i and the lower limit of axial resistance of column j respectively. The lower bound of the axial resistance of the column is calculated like a beam. n_c is also the number of columns of the structure. Equations (3.11) and (3.12) express the violation function and the penalty function of the combination of the bending moment and the axial force of the columns respectively.

$$V_{C,P-M} = \sum_i \left(\sum_{j=1}^{n_c} \max \left\{ \left(\frac{|(P_{UC})_j^i|}{(P_{CL})_j} + \frac{|(M_{UC})_j^i|}{(M_{CL})_j} - 1 \right) .0 \right\} \right) \quad (i = IO, LS, CP) \quad (3.11)$$

$$P_{C,P-M} = \alpha_{C,P-M} \cdot V_{C,P-M} \quad (3.12)$$

in which: penalty factor coefficient $\alpha_{C,P-M}$ is considered 10 in this paper. Column sections are selected from the list of American AISC W-Shapes too.

According to paragraph E-2 of AISC 360-05 regulations, the maximum slenderness coefficient of members under pressure should not exceed 200. Therefore from the beginning, all sections whose slenderness coefficient for a certain column is more than 200 are removed from the list of selected sections of that column. The axial force constraint in braces is defined according to equation (3.13):

$$|(P_{UBrace})_j^i| \leq (P_{BraceL})_j \quad (i = IO, LS, CP; j = 1, 2, \dots, 2n_{Brace}) \quad (3.13)$$

Equations (3.14) and (3.15) express the violation function and the penalty function of the axial force of braces respectively.

$$V_{Brace} = \sum_i \left(\sum_{j=1}^{n_{Brace}} \max \left\{ \left(\frac{|(P_{BU})_j^i|}{(P_{BL})_j} - 1 \right) .0 \right\} \right) \quad (i = IO, LS, CP) \quad (3.14)$$

$$P_{Brace} = \alpha_{Brace} \cdot V_{Brace} \quad (3.15)$$

in which: penalty factor coefficient α_{Brace} is considered 10 in this paper. Brace sections are selected from the list of American AISC HSS-Shapes.

According to paragraph 13-2-a of AISC 341-05 regulations, the maximum slenderness coefficient of braces should not exceed $4\sqrt{\frac{E}{F_y}}$. So from the beginning, all sections whose slenderness coefficient for the braces is more than this, are removed from the list of selected sections of that braces.

In this research, two geometric constraints are considered according to equations (3.16) and (3.17).

$$(b_B)_i \leq (b_c)_i \quad (i = 1, 2, \dots, n_{BC}) \quad (3.16)$$

$$(d_{S+1})_j \leq (d_s)_j \quad (j = 1, 2, \dots, n_{CC}; S = 1, 2, \dots, n_s - 1) \quad (3.17)$$

in which: $(b_B)_i$ and $(b_c)_i$ are the width of beam and column in the i th connection of beam to column and $(d_s)_j$ and $(d_{S+1})_j$ are the depth of column in S and $S + 1$ stories in the j th connection of column to column.

Equations (3.18), (3.19), (3.20), (3.21) express the violation function and the penalty function of the geometrical constraints.

$$V_{G,B-C} = \sum_{i=1}^{n_{BC}} \max \left\{ \left(\frac{(b_P)_i}{(b_C)_i} - 1 \right), 0 \right\} \quad (3.18)$$

$$V_{G,C-C} = \sum_{j=1}^{n_{CC}} \max \left\{ \left(\frac{(d_{S+1})_j}{(d_S)_j} - 1 \right), 0 \right\} \quad (3.19)$$

$$P_{G,B-C} = \alpha_{G,B-C} \cdot V_{G,B-C} \quad (3.20)$$

$$P_{G,C-C} = \alpha_{G,C-C} \cdot V_{G,C-C} \quad (3.21)$$

in which: penalty factor coefficient $\alpha_{G,B-C}$ and $\alpha_{G,C-C}$ is considered 10 in this paper.

The overall penalty function is obtained from the sum of all penalty functions according to equation (3.22):

$$P = P_{L,B} + P_{B,P-M} + P_{C,P-M} + P_{Brace} + P_{G,B-C} + P_{G,C-C} \quad (3.22)$$

The algorithms used in this research are: improved JAYA algorithm (IJAYA), accelerated water evaporation optimization algorithm (AWEO), particle swarm optimization algorithm (PSO), and collision optimization algorithm (CBO). All these algorithms are based on population and are included in the set of crowd intelligence algorithms. Improved JAYA Algorithm (IJAYA) is a recently presented algorithm by Rao and Saroj in [16], and its effectiveness in solving optimization problems has not been measured.

In the following, the optimal design of a two-dimensional steel frame with eccentric bracing is investigated based on the defined objective function. The optimization problem in this example has been solved using the four mentioned algorithms. In this example, first the specifications are explained, and then the optimization results are presented. First, gravity loading is applied, and analysis is done, and then lateral load is applied to the structure. The amount of gravity load introduced is the maximum combination of gravity load, and the amount of live and dead load is not mentioned. The amount of effective seismic weight due to gravity loading is obtained from the combination of dead load and 20% of live load. The specifications of the materials, Geometry, member grouping and gravity loading are exactly defined according to the research of Chaos and Goel in [2]. They designed this frame based on seismic performance, but they did not optimize it. Connection of the beams in the unbraced spans to the columns is hinged. The lateral loading pattern is according to the shape of the first vibration mode of each frame. The cycle time of the first vibration mode of each frame is considered as the main cycle time of the structure. The number of problem variables is equal to the number of groups of members of each frame. To eliminate the effect of the randomness of the algorithm on the results, each example was solved 11 times by each of the algorithms, and their results were presented. Fig. 1 shows the geometry and grouping, and Fig. 2 shows the numbering of frame members of the numerical example.

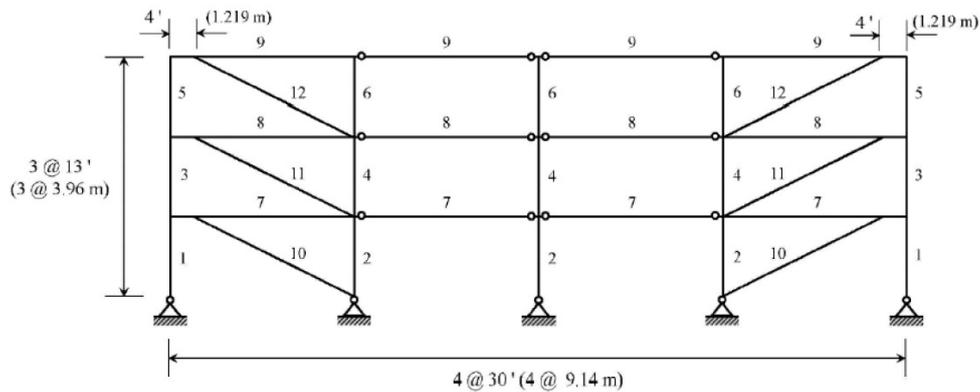


Figure 1: Geometry and grouping of numerical example

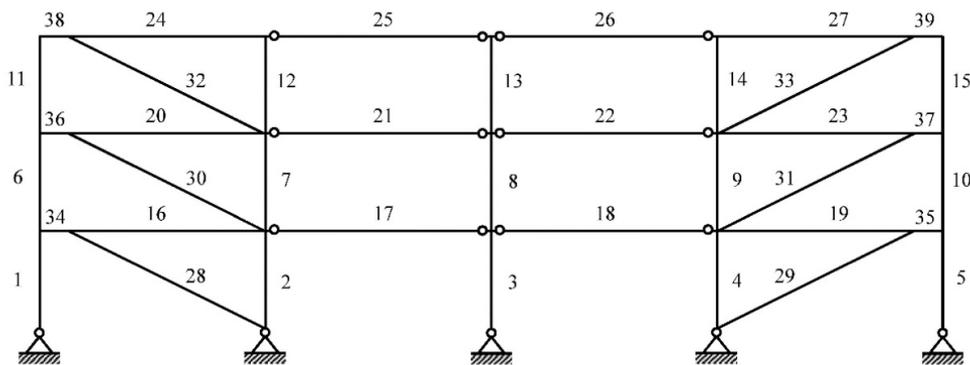


Figure 2: Numbering of frame members

The effective weight of vibrations caused by gravity loading on the first and second floors is 993.6kips (4412kN) and on the roof is 756kips (3363kN). To see the amount of dead and live loads and how to calculate these values, refer to [2]. Number of iterations in all algorithms and their population size considered 400 and 80 respectively. In the following, the results will be discussed. Number of sections in every element group has been shown in Table 4. Table 5 shows the summary of results of optimization. It can be concluded from this table that IJAYA algorithm shows better performance between other algorithms. Also sections for best responses has been shown in Table 6.

Table 4: Number of sections in every element group

| | | Element Group | | | | | | | | | | | |
|--------------------|--|---------------|-----|-----|-----|-----|-----|-----|-----|-----|----|----|----|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Number of sections | | 256 | 256 | 256 | 256 | 256 | 256 | 251 | 251 | 251 | 58 | 58 | 58 |

Table 5: Summary of results of optimization

| | | Type of response | AWEO | PSO | CBO | IJAYA |
|------------------|--------------|------------------|-----------------------|----------------------|----------------------|----------------------|
| Best response | Z value | | 0.73 | 0.31 | 0.58 | 0.24 |
| | F1 value | | 0.62 | 0.22 | 0.4 | 0.25 |
| | Weight value | | 291 kips (1294 kN) | 104 kips (462 kN) | 187 kips (833 kN) | 114 kips (506 kN) |
| Average response | Z value | | 1.16 | 0.99 | 0.98 | 0.43 |
| | F1 value | | 0.47 | 0.23 | 0.36 | 0.26 |
| | Weight value | | 221 kips (985 kN) | 108 kips (480 kN) | 168 kips (750 kN) | 122 kips (546 kN) |
| Worst response | Z value | | 2.35 | 2.09 | 1.27 | 0.52 |
| | F1 value | | 0.27 | 0.2 | 0.35 | 0.24 |
| | Weight value | | 126 kips (562 kN) | 93 kips (417 kN) | 164 kips (731 kN) | 113 kips (502 kN) |

Table 6: Sections for best responses

| Element group | AWEO | PSO | CBO | IJAYA |
|---------------|-------------------|-------------------|-------------------|-------------------|
| 1 | W 36 × 800 | W 14 × 398 | W 36 × 800 | W 12 × 336 |
| 2 | W 36 × 247 | W 36 × 135 | W 40 × 324 | W 40 × 235 |
| 3 | W 36 × 800 | W 14 × 193 | W 36 × 800 | W 12 × 230 |
| 4 | W 36 × 800 | W 12 × 136 | W 40 × 397 | W 40 × 230 |
| 5 | W 36 × 800 | W 14 × 176 | W 14 × 342 | W 12 × 230 |
| 6 | W 36 × 800 | W 27 × 235 | W 40 × 431 | W 40 × 215 |
| 7 | W 36 × 191 | W 33 × 141 | W 30 × 191 | W 33 × 152 |
| 8 | W 36 × 652 | W 33 × 118 | W 30 × 191 | W 30 × 124 |
| 9 | W 36 × 282 | W 18 × 130 | W 40 × 215 | W 24 × 104 |
| 10 | HSS 20 × 12 × 5.8 | HSS 20 × 12 × 5.8 | HSS 16 × 16 × 5.8 | HSS 20 × 12 × 5.8 |
| 11 | HSS 16 × 16 × 5.8 | HSS 16 × 16 × 5.8 | HSS 20 × 12 × 5.8 | HSS 16 × 16 × 5.8 |
| 12 | HSS 20 × 12 × 5.8 | HSS 12.75 × 0.5 | HSS 14 × 14 × 5.8 | HSS 16 × 16 × 5.8 |
| Weight (kips) | 290.95 | 103.89 | 187.45 | 113.75 |
| Weight (kN) | 1294.19 | 462.12 | 833.84 | 506.00 |

The combination value of the ratio of axial force and bending moment in the columns for the best response has been achieved from the results as shown in Fig. 3. Also, the value of the slenderness coefficient of the columns for the best response has been shown in Fig. 4.

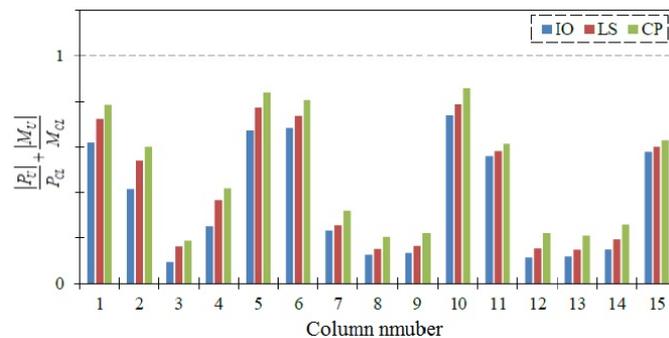


Figure 3: The combination value of the ratio of axial force and bending moment in the columns for the best response

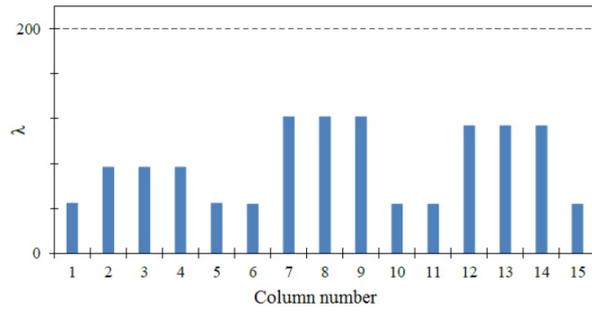


Figure 4: The value of the slenderness coefficient of the columns for the best response

Plastic rotation ratio of hinges to acceptance criteria for best answer has been shown in Fig. 5. Moreover, Fig. 6 shows the ratio of the axial force of the braces to the lower limit of their axial resistance for the best response.

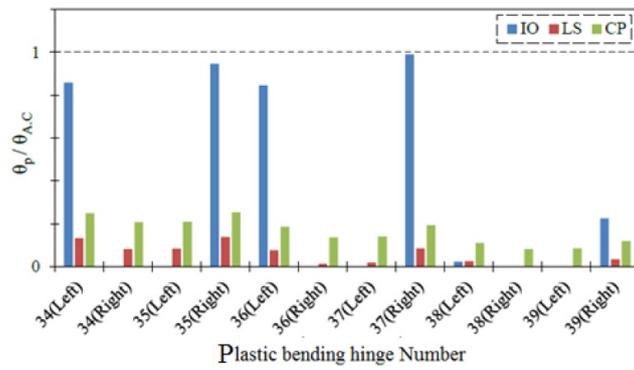


Figure 5: Plastic rotation ratio of hinges to acceptance criteria for best answer

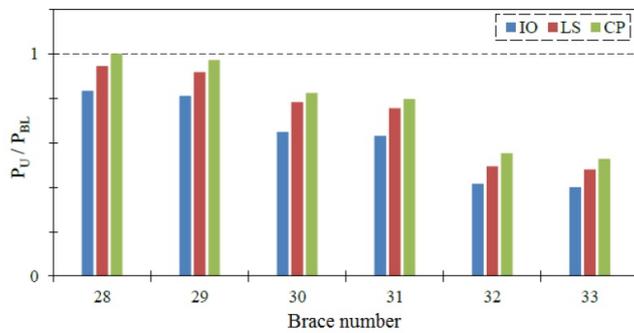


Figure 6: The ratio of the axial force of the braces to the lower limit of their axial resistance for the best response

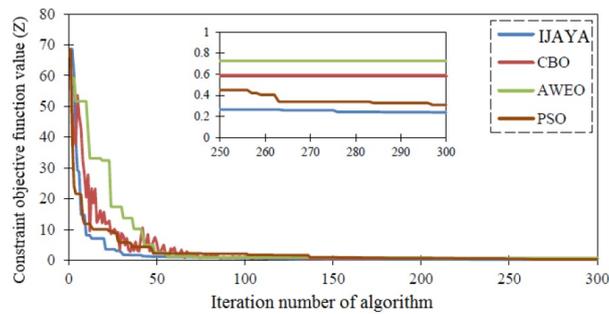


Figure 7: Constraint objective function against iteration for the best response of each algorithm

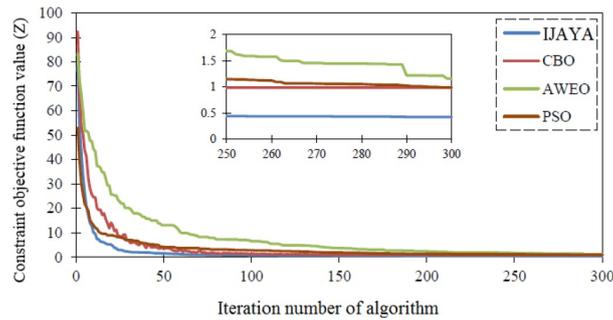


Figure 8: Constraint objective function against iteration for the average response of each algorithm

Constraint objective function against iteration for the best response and the average response of each algorithm have been shown in Fig 7 and Fig 8, respectively.

Examining the results of solving this example shows that an answer with less weight is not necessarily a better answer. In fact, by defining the constraints and the penalty function, the algorithm must first search the acceptable space, i.e. the space in which all constraints are satisfied and the value of the penalty functions is equal to zero. After finding this space, achieving the lowest weight of the structure becomes the goal of the algorithm. Therefore, the answer with less weight will be the better answer among the answers whose penalty functions are the same.

The results of this example show that meta-heuristic algorithms, while finding an acceptable space, have searched for the lowest weight of the structure. Because in the acceptable space, the values of the penalty functions are equal to zero, the answer that has less weight in this space is the best answer.

If the criterion of the algorithm's ability is the time to solve the problem, the population size and the repetition of the algorithms should be selected in such a way that their solution time is equal. Since the problem-solving time is dependent on variables other than the algorithm, time is not considered a suitable criterion for measuring the algorithm's ability. For this reason, to compare the ability of algorithms, they often measure the number of times the objective function is evaluated. In other words, if the answer to an optimization problem is known, an algorithm that evaluates the objective function less often is more capable. In problems such as the present optimization problem, whose answer is not known, an algorithm that finds a more optimal answer in a certain number of evaluations of the objective function is more successful. For this reason, in this research, the number of population and the number of iterations of the algorithms are considered the same, so that the number of times each algorithm evaluates the objective function is the same. By comparing the results of the algorithms with each other, it can be seen that the ability of the IJAYA algorithm to solve this optimization problem is more favorable than the other investigated algorithms. However, the solving speed of the PSO algorithm is higher than that of other algorithms.

4 Conclusion

Since the optimization of the weight of eccentric bracing frames based on the performance-based design method has not been done so far, the results of this research cannot be compared with other results. For this reason, this study only aims to measure the possibility of the optimal design of eccentric bracing frames. The results of the review of the considered frame show that the goal of this paper has been met, and it is possible to optimize the weight of the eccentric bracing frames based on the performance-based design method.

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