Optimal Design of Semirigid Connection Steel Frame with Steel Plate Shear Walls Using Dolphin Echolocation Algorithm

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Abstract: Steel frame with steel plate shear walls (SPSWs) is used to resist lateral loads caused by wind and earthquakes in high-rise buildings. In this load-resisting system, the cost and performance are more efficient than in the moment frame system. Behaviors of beam-to-column connections are assumed to be pinned or fixed to simplify the calculation in the past few decades. However, studies have stated that such a simulation fails to reveal the response of beam-to-column connections. In this paper, a newly developed metaheuristic optimization algorithm—the dolphin echolocation algorithm (DE)—based on foraging prey using echolocation in dolphins is applied as the present study optimizer. Two different two-dimensional semirigid connection steel frames with SPSWs are optimized to obtain the minimum cost of semirigid connection steel frame with steel plate shear walls with constraints to element stresses and story drift ratio according to the American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD). SPSW is modeled as a brace with equivalent lateral stiffness, while the $P−\Delta$ effects are considered in the steel frame. Semirigid connections are used to reveal the actual responses of beam-to-column connections. The results demonstrate the proposed method’s effectiveness for optimizing semirigid connection steel frames with SPSWs and the interaction between semirigid connections and the SPSWs.

Keywords: optimal design; semirigid connection steel frame; steel plate shear wall; dolphin echolocation algorithm; lateral stiffness

1. Introduction

Steel plate shear wall (SPSW) is a relatively new lateral bearing system that has been around since the 1970s. Several researchers have revealed that the strength caused by postbuckling plays a vital role in thin unstiffened SPSW systems, making it a more economical alternative than traditional lateral bearing systems. In addition, high initial stiffness, substantial ductility, stable hysteretic characteristics, and a large capacity for plastic energy absorption are also the advantages of SPSW [1]. During the 1980s and the 1990s, analytical and experimental works were conducted by various researchers, such as Thorburn et al. [2], Timler and Kulak [3], Tromposch and Kulak [4], Elgaaly [5], and Driver et al. [6]. In the last two decades, plastic analysis has been carried out by Berman and Brunneau [7], Kharmale and Ghosh [8], Zadeh et al. [9], and Guo et al. [10].

In 1931, Wagner [11] developed diagonal tension field theory, and it shifted the use of thick SPSW to thin SPSW by considering the postbuckling strength of infilled panels. On the basis of the diagonal tension field theory, Thorburn et al. [2] proposed a simple...
analytical model to represent the shear behavior of thin unstiffened SPSW. In this model, also known as the multistrip model (SM) (Figure 1), a series of pin-ended inclined members that can only be subjected to tension forces are used to model the tension field. The angle of the tension field was calculated using the principle of least work.

Apart from the multistrip model, Thorburn et al. [2] proposed a modified strip model (MSM) (Figure 2), in which the infilled panel is modeled as a single diagonal brace by sharing the same lateral stiffness. Compared with the multistrip model, the advantage of the equivalent brace model is the reduced computation. The downside of this model is the lack of accuracy in the results since it does not represent the distributed forces applied by the SPSW on the vertical boundary element (VBE) for a column and horizontal boundary element (HBE) for a beam.

Figure 1. Multistrip Model.

Figure 2. Modified Strip Model.

Zhou [12] introduced a new analytical model named the unified strip model (USM) (Figure 3), which takes both pure shear and pure tension fields into account. The proportion of pure shear is $\eta$, while that of the tension field is $(1 - \eta)$ in this model, and by changing the thickness of the infilled plate, the $\eta$ consequently differs.
Metaheuristics shows tremendous advantages in the optimization problem. Inspired by biology, researchers developed various aspects of optimum design tools. Kaveh and Farhoudi [13] complied with four algorithms for the layout optimization of steel-braced frames. Toğan [14] introduced teaching–learning-based optimization (TLBO) technique to optimize planar steel moment-resisting frames. Kaveh and Farhoudi [15] proposed a new bio-based dolphin echolocation algorithm (DE). By comparing it with the other four algorithms, they concluded that DE shows higher convergence rates which makes it more efficient than other algorithms.

Studies on the optimization design of steel brace frames can be found, such as Doan and Lee [16], Lee and Shin [17], and Lee et al. [18]. Gholizadeh and Shahrezaei [19] studied the arrangement of steel plate shear walls, and Saedi Daryan et al. [20] presented an optimization design of steel frames with SPSW using a modified dolphin algorithm. Apart from the studies mentioned above, a few studies of steel frames with SPSW have also been conducted.

In this paper, the DE algorithm will be used to perform the optimization design of steel frames with SPSW. In addition, semirigid connections are applied instead of pinned or fixed connections. Moreover, the \( P - \Delta \) effects are also considered. The USM model is used to evaluate the effects of shear and tension fields, and a brace with equivalent lateral stiffness is used to replace the SPSW.

2. Dolphin Echolocation (DE) Algorithm

During the hunting process, a dolphin generates clicks, and when the clicks hit its prey, it reflects toward the dolphin. As soon as the dolphin receives the sound, it generates another click. The time gap between two clicks allows the dolphin to locate the prey. By simulating the way of dolphin hunting, Kaveh and Farhoudi [13] introduced the DE algorithm.

Before conducting the algorithm, the alternatives should be sorted in an ascending or descending order, and the vector \( A_j \) of the length \( L_A_j \) for variable \( j \) is created. Combining \( j \)th variable next to another, the matrix of alternatives with the dimension of \( MA \times NV \) is generated, in which \( MA \) is \( max(LA_j)_{j=1}^{NV} \), and \( NV \) is the number of variables.

A curve with the changing convergence factor during the optimization process is assigned.

\[
PP(\text{Loop}_i) = PP_1 + (1 - PP_1) \left( \frac{\text{Loop}_i^{Power} - 1}{(\text{Loops Number})^{Power} - 1} \right)
\]

where \( PP \) is the predefined probability, \( PP_1 \) is the convergence factor of the first loop, \( \text{Loop}_1 \) is the number of the current loop, \( Power \) is the degree of the curve, and \( \text{Loops Number} \) is the total number of loops in which the algorithm should reach the convergence point. This number is determined by the user.
The main steps for utilizing DE are as follows:

1. Generate NL locations for a dolphin randomly.
   This step creates $MA \times NV$ matrix, in which $MA$ is the number of locations and $NV$ is the number of variables;
2. Calculate the $PP$ of the first loop using Equation (1);
3. Calculate the fitness of each location.
   The fitness of the better answers in one loop should be assigned higher values;
4. Calculate the accumulative fitness
   \[
   F_{(A+k)j} = \frac{1}{Re} \times (Re - |k|) \text{Fitness}(i) + AF_{(A+k)j}
   \] (2)
   where $AF_{(A+k)j}$ is the accumulative fitness of the $(A + k)$th alternative to be chosen for the $j$th variable, and $Re$ is the effective radius in which the accumulative fitness of the alternative $A$’s neighbors is affected by its fitness. It should be noticed that the $AF$ is calculated using a reflective characteristic for alternatives near edges.
5. A small value is added to all the arrays to distribute the possibility evenly with
   \[
   AF = AF + \epsilon
   \] (3)
6. Calculate the probability of each variable based on the following relationship:
   \[
   P_{ij} = \frac{AF_{ij}}{\sum_{i=1}^{LA} AF_{ij}}
   \] (4)
7. Assign a probability equal to $PP$ to all alternatives chosen for all variables of the best location and devote the rest of the probability to the other alternatives according to the following equation:
   \[
   P_{ij} = (1 - PP)P_{ij}
   \] (5)
8. Generate the following location according to the probabilities assigned to each alternative.
   Repeat steps 2–8 $Loops Number$ times. The flowchart is given in Figure 4.
3. Problem Formulation

Finding a set of design variables that satisfy the minimum cost is the main objective of an optimization problem. Certain conditions constrain these variables. In the steel frame, this type of optimization problem can be summarized as the following formulation to find \( X \):

To minimize:

\[
C(X) = \sum_{i=1}^{n_e} \rho_{1i} A_i L_i + \sum_{k=1}^{n_s} \rho_{2k} t_k l_k h_k + \sum_{m=1}^{n_c} \rho_{3m} S_m \tag{6a}
\]

Subject to:

\[
g_j(X) \leq 0, j = 1, 2, \ldots, n_g \tag{6b}
\]

where \( C \) is the structural cost and the total structural weight, \( n_e \) is the number of members, \( \rho_{1i} \), \( A_i \), and \( L_i \) are the cost per unit volume, cross-sectional area, and length of the \( i \)th member, respectively, \( n_s \) is the number of SPSWs, \( \rho_{2k} \), \( t_k \), \( l_k \), and \( h_k \) are the cost per unit volume, thickness, length, and height of the \( k \)th SPSW, respectively, \( \beta_i \) is the density of steel, \( n_c \) is the number of semirigid connections, \( \rho_{3m} \) and \( S_m \) are the cost per initial rotation stiffness and initial rotation stiffness of the \( m \)th connection, respectively, \( g_j \) is the \( j \)th constraint, and \( n_g \) is the number of constraints.

In this paper, the structural member should satisfy the American Institute of Steel Construction Load and Resistance Factor Design (AISC-LRFD) in terms of strength, inertia, and story drift.

(a) Beam and column strength

\[
\frac{P_u}{\phi_c P_n} < 0.2; \left( \frac{P_u}{2\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} \right) \leq 1 \tag{7a}
\]

\[
\frac{P_u}{\phi_c P_n} \geq 0.2; \left( \frac{P_u}{\phi_c P_n} + \frac{8}{9} \frac{M_{ux}}{\phi_b M_{nx}} \right) \leq 1 \tag{7b}
\]

where \( P_u \) is the required axial strength, \( P_n \) is the nominal axial strength, \( \phi_c \) is the resistance factor, \( M_{ux} \) is the required flexural strength, \( M_{nx} \) is the nominal flexural strength, and \( \phi_b \) is the flexural resistance reduction factor.

(b) For VBE and HBE, the stiffness requirement given by Sabelli and Bruneau [21] is as follows:

\[
l_{VBE} \geq 0.00307 \frac{t_wh^4}{L} \tag{8a}
\]

\[
l_{HBE} \geq 0.003 \frac{\Delta t_w L^4}{h} \tag{8b}
\]

where \( \Delta t_w \) is the difference between the web plate thickness above and below the HBE.

(c) The design story drift at level \( i \) is as follows:

\[
\Delta_i = (\delta_{ei} - \delta_e(i - 1)) \frac{C_d}{L_e h_{si}} \leq 0.02 \tag{9}
\]

where \( h_{si} \) is the storey height below level \( i \), \( \delta_{ei} \) is the deflection at level \( i \) determined by an elastic analysis, \( C_d \) is the deflection amplification factor, \( L_e \) is the important factor.

(d) In the preliminary design stage, the shear is assumed to be carried by the SPSW only, and the thickness under the shear load is given as follows:
where $V_u$ is the required shear strength, $\phi$ is the resistance factor given in AISC 341, $F_y$ is the infilled panel yield stress, $L_{cf}$ is the clear distance between VBE flanges, and $\alpha$ is assumed to be 45°.

4. Semirigid Connection Steel Frame with Steel Plate Shear Wall

4.1. Semirigid Connection

The present practical design process assumes the beam-to-column connections behave as pinned or fixed connections. This approach could lead to a less accurate prediction of the system’s response and exclude the effect of the internal force distribution because of the flexibility. Therefore, it is suggested that the complete nonlinear moment rotation characteristics of the connection should be considered during the design process. Additionally, since the wall plate and beam are not meant to slip, the bolt slip does not affect stiffness in the strip model. As a design method, the base shear and the force exerted by the horizontal seismic force are assumed to be the same in the equivalent system with one degree of freedom.

The initial rotational stiffness, which is used for the elastic analysis of semirigid connection, is given as follows [22]:

$$S_{j_{ini}} = \frac{M_j}{\phi_j} = \frac{Fz}{\sum \frac{1}{k}} = \frac{z^2}{\sum \frac{1}{k}}$$

where $M_j$ is the bending moment of connection, $\phi_j$ is the rotation of connection, $z$ is the distance between the bottom edge of the beam and the top edge of the slab, and $k$ is the stiffness coefficient of the components.

According to Moghaddam and Sadrara [23,24], Qin et al. [25], Zhang et al. [26], Azizinamini et al. [27], and Brown and Anderson [28], the initial rotational stiffness varies between 1000 kNm/rad and 300,000 kNm/rad. It is thus the stiffness of joints, particularly their initial rotational stiffness, that is of importance in this regard. Consideration of the stiffness of semirigid joints at their initial rotation, both in the sway and rotation directions, has been shown to result in significant cost savings frames [23–25] and non-sway frames [26]. Members’ buckling behavior is also affected by rotational stiffness. For this reason, many design standards [23–25] and handbooks [26] provide chord stress functions for measuring the effects of these stresses on the joints’ resistances. A joint’s surface may be subject to normal stresses at the point where the connected member is mounted. Literature results reported that the axial stress in the main member of square hollow section joints is found to significantly affect the initial rotational stiffness of these joints based on 3D FEM analysis. With compressed loads, stiffness decreases by 50%, while tensile loads increase stiffness by 30%. Based on the brace-to-chord width ratio as well as the chord width-to-thickness ratio, it has been found that the observed effect is dependent on these variables [25,26].

According to Xu and Grierson [29], the cost of a semirigid connection is calculated as follows:

$$C = R_t \cdot \frac{0.225w_i a_i}{R_{ij}}$$

where $R_t$ is the rotation stiffness of the joint $i$, $w_i$ and $a_i$ are the weight and cross-sectional area of a beam, and $R_{ij}$ is the average rotation stiffness of all joints.
4.2. P—Δ Effect

The flexibility of semirigid connections increases the deformation of a structure [30–32]. Therefore, the P—Δ effect should be considered during the analysis process.

Element second-order stiffness matrix of semirigid connections in local coordinates considering connection flexibility is given as follows [33]:

\[
\begin{bmatrix}
\frac{EA}{L} & 0 & \frac{E_i L}{L^2} & \frac{\bar{p}}{L} \\
0 & \frac{E_i L}{L^2} & \frac{\alpha_{ii} + \alpha_{ij}}{E_i L} & \frac{\alpha_{ij}}{E_i L} \\
E_i & 0 & \frac{E_i L}{L^2} & \frac{\alpha_{ii} + \alpha_{ij}}{E_i L} \\
0 & \frac{\bar{p}}{L} & -\frac{(\alpha_{ii} + \alpha_{ij}) E_i L^3}{L} & -\frac{(\alpha_{ii} + \alpha_{ij}) E_i L^3}{L} \\
0 & \frac{\alpha_{ij} E_i L}{L^2} & \frac{E_i L}{L} \alpha_{ij} & 0 \\
0 & \frac{\alpha_{ij} E_i L}{L^2} & \frac{E_i L}{L} \alpha_{ij} & \frac{E_i L}{L} \alpha_{ij}
\end{bmatrix}
\]  

(13a)

\[\alpha_{ii} = \frac{1}{R} \left[ C + \frac{K(C^2 - S^2)}{R_{KB}} \right] \]  

(13b)

\[\alpha_{jj} = \frac{1}{R} \left[ C + \frac{K(C^2 - S^2)}{R_{KA}} \right] \]  

(13c)

\[\alpha_{ij} = \frac{S}{R} \]  

(13d)

\[R = \left(1 + \frac{KC}{R_{KA}}\right) \left(1 + \frac{KC}{R_{KB}}\right) - \frac{K^2 S^2}{R_{KA} R_{KB}} \]  

(13e)

\[K = \frac{EA}{L} \]  

(13f)

\[C = \frac{u(tg u - u)}{tg u \left(2t g \left(\frac{u}{2}\right) - u\right)} \]  

(13g)

\[S = \frac{u(u - sin u)}{sin u \left[2t g \left(\frac{u}{2}\right) - u\right]} \]  

(13h)

\[u = L \frac{\bar{p}}{E_i L} \]  

(13i)

where \(A\) is the area of the cross section, \(E\) is Young’s modulus, \(I\) is the moment of inertia of the cross section, \(L\) is the length of the element, \(\bar{p}\) is the axial force of the element, and \(R_{KA}\) and \(R_{KB}\) are the initial rotation stiffness of the end \(A\) and \(B\) of the element.
4.3. Unified Strip Model (USM)

Zhou [12] introduced a new model named USM. This model considers both tension field and shear by using a coefficient $\eta$. The proportion of the complete shear mechanism is $\eta$, and that of the entire tension field mechanism is $1 - \eta$.

Coefficient $\eta$ is calculated as follows:

$$
\eta = \begin{cases} 
1.0, & \lambda_n \leq 0.8 \\
1 - 0.88(\lambda_n - 0.8), & 0.8 < \lambda_n \leq 1.2 \\
\frac{0.94}{\lambda_n}, & \lambda_n > 1.2
\end{cases}
$$

(14a)

$$
\lambda_n = \sqrt{\frac{\tau_y}{\tau_{cr}}}
$$

(14b)

$$
\lambda_n \in \begin{cases} 
(-\infty,0.8], & \text{thick plate} \\
(0.8,1.2], & \text{moderate-thick plate} \\
(1.2, +\infty), & \text{thin plate}
\end{cases}
$$

(14c)

where $\lambda_n$ is the common height/thickness ratio, $\tau_y$ is the embedded steel plate’s shear yield stress, and $\tau_{cr}$ is the infilled plate’s elastic critical buckling shear stress.

4.4. Simplified Model

In this paper, a brace with cross-sectional area $A$ is used to replace the SPSW with equivalent lateral stiffness as the simplified model, as shown in Figure 5a and Figure 5b, respectively. The lateral stiffness of the infilled plate caused by the tension field is given by Kharrazi [34]:

$$
K_w = \frac{\tau_{cr} + \frac{1}{2} \times \sigma_{ty} \times \sin 2\theta}{\frac{\tau_{cr}}{G} + \frac{2}{E} \times \sin 2\theta} \times \frac{b \times t}{d}
$$

(15)

Typically, the columns are assumed to be rigid under the tension force caused by the developed tension field. Consequently, the angle $\theta$ equals $45^\circ$ to the horizontal, and Equation (15) is simplified as:

$$
K_w = \frac{Eb t}{4d}
$$

(16)

where $E$ is young’s modulus, $b, t,$ and $d$ are the width, thickness, and height of the infilled plate, respectively.

The lateral stiffness of the embedded wall caused by the shear is given by Guo et al. [35]:

$$
K_c = \frac{1}{\Delta} = \frac{Gt}{1.2 \left(\frac{b}{d}\right)}
$$

(17)

where $G$ is the modulus of rigidity.

Therefore, the lateral stiffness of an infilled panel is calculated according to

$$
K = (1 - \eta)K_w + \eta K_c
$$

(18)

Additionally, the columns are affected by the tension field developed on the infilled plate, and the vertical component of the tensile force would cause an eccentric bending moment in the columns. The practical lateral stiffness of the VBE is:

$$
K_t = \frac{12EI}{r^3} \pm \frac{Etsin^2\theta cos^2\theta}{2} + \frac{Eth_c sin^2\theta cos^2\theta}{2h}
$$

(19)

where $\theta = 45^\circ$ is the angle when the columns are relatively rigid, and $h_c$ is the height of the VBE cross section.
Likewise, the practical lateral stiffness of the VBE under the shear force is:

\[ K_s = \frac{12EI}{h^3} + \frac{5Gth_c}{12h} \]  

(20)

Therefore, the lateral stiffness of the infilled wall is calculated as follows:

\[ K_{overall} = \frac{Eth_c \sin^2 \theta \cos^2 \theta}{h} + \frac{5Gth_c}{6h} + K \]  

(21)

The elastic analysis can be performed by the following procedures:

(a) calculate the lateral stiffness of the infilled panel ‘K’ from Equation (21);
(b) the cross-sectional area ‘A’ is calculated by:

\[ K = \frac{EA}{h} \cos^2 \theta \sin \theta \]  

(22)

(c) The steel plate shear wall is replaced by a brace with cross-sectional area ‘A’.

To verify the simplified model, Matrix Laboratory (MATLAB) software is used for coding to calculate, and the results were compared with the experimental one obtained by Berman [36]. The analysis results in the elastic phase of the MATLAB coding and the testing model are presented in Figure 6. The testing model is around 15% higher than the MATLAB coding. The ABAQUS software is used to simulate the deformation of the structure.

The numerical example presented by Gholizadeh [19] in Section 5.1 is conducted for validation, and W40×372 is used for all the columns and beams, and the thickness of the infilled panels is 5.5mm. Several outcomes of point displacements are listed in Table 1.

| MATLAB | ABAQUS |
|--------|--------|
| X (mm) | Y (mm) | Z (mm) | X (mm) | Y (mm) | Z (mm) |
| Node 2 | 0.43 | 0.067 | -0.18 | Node 2 | 0.40 | 0.062 | -0.14 |
| Node 6 | 2.8 | 0.13 | -0.11 | Node 6 | 2.3 | 0.10 | -0.09 |
| Node 11 | 2.3 | -0.13 | -0.12 | Node 11 | 1.9 | -0.11 | -0.10 |
| Node 12 | 2.7 | -0.15 | -0.082 | Node 12 | 2.5 | -0.14 | -0.080 |
| Node 17 | 2.3 | -0.25 | -0.12 | Node 17 | 2.0 | -0.23 | -0.10 |
| Node 22 | 1.7 | -0.42 | -0.16 | Node 22 | 1.4 | -0.39 | -0.14 |

Average error: 13.15%
Figure 5. (a) A simplified model with a depth of 40.6 inches, flange width of 16.1 inches, flange thickness of 2.05 inches, web thickness of 1.16 inches, and area of 109 inches$^2$, respectively. (b) Model structure beam and column with cross-sectional dimensions.

As shown in Figures 7–9, the nodal displacements had been compared using MATLAB and ABAQUS for the respective x, y, and z geometric positions.

Figure 6. Comparison of numerical (MATLAB) and testing model results.
Figure 7. Comparison of the nodal displacements for the respective geometric position of x-coordinate using MATLAB and ABAQUS.

Figure 8. Comparison of the nodal displacements for the respective geometric position of y-coordinate using MATLAB and ABAQUS.
5. Examples

In this study, two numerical examples are presented to verify the efficiency of the methodology. The first is a three-bay, five-story structure, and the second is a three-bay, 10-story structure. The dimensions of these two structures are identical, with 3 m for the length of each span and 5 m for the height of each floor. The modulus of elasticity and the weight density are 200 GPa and 7.68 t/m$^3$, respectively. The yield stress for beam-column elements and web plates are 248.2 MPa and 220 MPa.

The earthquake-concentrated loads are calculated based on the ASCE/SEI 7-05 (2009), applying the following parameters: $R = 7$; $I_e = 1$; $C_d = 6$; $S_s = 1.7035g$; seismic design category $= D$. The loading condition is shown in Table 2.

Table 2. Loading condition.

| Floor | Uniform Distributed Gravity Load (kN/m) | Earthquake Loads (kN) |
|-------|----------------------------------------|-----------------------|
|       | Five-Story Structure | Ten-Story Structure | Five-Story Structure | Ten-Story Structure |
| 1     | 24.9 | 24.9 | 69.11 | 22.97 |
| 2     | 24.9 | 24.9 | 139.19 | 52.31 |
| 3     | 24.9 | 24.9 | 209.64 | 84.66 |
| 4     | 24.9 | 24.9 | 280.33 | 119.13 |
| 5     | 24.9 | 24.9 | 351.21 | 155.27 |
| 6     | --- | 24.9 | --- | 192.81 |
| 7     | --- | 24.9 | --- | 231.54 |
| 8     | --- | 24.9 | --- | 271.32 |
| 9     | --- | 24.9 | --- | 312.06 |
| 10    | --- | 24.9 | --- | 353.64 |
| Base shear | --- | --- | 1049.48 | 1795.71 |

The sections of beams and columns are selected from the 283-W-section list from the AISC database, the infilled plates are chosen between 1 mm and 10 mm, and the initial rotation stiffness varies between 1000 kNm/rad and 300,000 kNm/rad.

The costs of columns, beams, and walls are USD 2500, USD 2500, and USD 2400 per ton, respectively. According to Xu et al. [29], the cost of a semirigid connection is calculated by Equation (23).
Cost = \( \frac{0.225C_i}{R} \times R_i \)  \( \text{(23)} \)

where \( C_i \) is the cost of the \( i \)th beam member, \( R \) is the average initial rotation stiffness, and \( R_i \) is the initial rotation stiffness of the \( i \)th semirigid connection.

5.1. Example 1: Three-Bay, Five-Storey Structure

The dimension of the structure and the infilled plate placement are given in Figure 10a,b, respectively. Figure 10c is the numbering of semirigid connections.

![Figure 10](image_url)

Figure 10. (a–c). Dimensions of the structure and the infilled plates placement with the numbering of semirigid connections.

A semirigid steel frame structure (SF) with the dimension shown in Figure 10a–c is performed as a cost optimization to compare with SF-SPSW. The results are given in Table 3.
Table 3. Results of SF and SF-SPSW.

| Design Variables | Optimum Results | SF | SF-SPSW | F-SPSW |
|------------------|-----------------|----|---------|--------|
| 1                | W36 × 160       | W6 × 12 | W6 × 8.5 |
| 2                | W40 × 183       | W6 × 8.5 | W8 × 10 |
| 3                | W24 × 76        | W6 × 8.5 | W6 × 8.5 |
| 4                | W21 × 55        | W6 × 8.5 | W8 × 10 |
| 5                | W21 × 55        | W6 × 8.5 | W6 × 8.5 |
| 6                | W8 × 18         | W14 × 43 | W16 × 26 |
| 7                | W6 × 8.5        | W24 × 55 | W14 × 30 |
| 8                | W24 × 76        | W18 × 65 | W14 × 26 |
| 9                | W36 × 135       | W16 × 26 | W14 × 22 |
| 10               | W24 × 76        | W14 × 22 | W16 × 31 |
| 11               | W16 × 31        | W21 × 83 | W16 × 26 |
| 12               | W12 × 16        | W16 × 45 | W16 × 26 |
| 13               | W40 × 149       | W16 × 26 | W14 × 22 |
| 14               | W21 × 44        | W14 × 22 | W14 × 26 |
| 15               | W12 × 16        | W12 × 19 | W12 × 14 |
| 16               | W40 × 372       | W6 × 8.5 | W6 × 8.5 |
| 17               | W40 × 183       | W6 × 8.5 | W6 × 8.5 |
| 18               | W21 × 68        | W6 × 8.5 | W6 × 9  |
| 19               | W21 × 44        | W6 × 8.5 | W6 × 8.5 |
| 20               | W12 × 14        | W6 × 8.5 | W6 × 8.5 |
| 21               | W8 × 13         | W6 × 8.5 | W6 × 8.5 |
| 22               | W16 × 31        | W10 × 15 | W10 × 15 |
| 23               | W24 × 68        | W12 × 16 | W6 × 8.5 |
| 24               | W21 × 62        | W6 × 12  | W6 × 9  |
| 25               | W30 × 99        | W10 × 15 | W6 × 9  |
| 26               | W6 × 8.5        | W10 × 12 | W16 × 26 |
| 27               | W30 × 99        | W14 × 22 | W18 × 35 |
| 28               | W30 × 116       | W14 × 38 | W16 × 26 |
| 29               | W27 × 102       | W10 × 12 | W12 × 14 |
| 30               | W18 × 35        | W24 × 55 | W24 × 55 |
| 31               | W6 × 8.5        | W6 × 8.5 | W6 × 8.5 |
| 32               | W36 × 135       | W6 × 8.5 | W6 × 9  |
| 33               | W18 × 40        | W6 × 8.5 | W6 × 8.5 |
| 34               | W18 × 40        | W6 × 8.5 | W6 × 8.5 |
| 35               | W6 × 8.5        | W6 × 8.5 | W6 × 9  |
| P1(mm)           | -               | 3    | 3       |
| P2(mm)           | -               | 3    | 3       |
| P3(mm)           | -               | 3    | 2.5     |
| P4(mm)           | -               | 2    | 2       |
| P5(mm)           | -               | 1    | 1       |
| Steel frame weight (t) | 14.4 | 4.1 | 3.4 |
| Steel plate weight (t) | -   | 1.2 | 1.1 |
| Total weight (t)  | 14.4 | 5.3 | 4.5 |
| Total cost ($)    | 38,095 | 13,179 | 11,221 |
The story drift ratio and stress ratio for frame elements in SF-SPSW and SF are shown in Figures 11 and 12, respectively. Other constraint conditions listed in Equation (7) to Equation (10) are also satisfied. The results demonstrated in Table 4 indicate that the methodology is practical.

![Figure 11. Story drift ratio of SF-SPSW and SF.](image1)

![Figure 12. Stress ratio of SF-SPSW and SF.](image2)

**Table 4.** The rotation stiffness after optimization.

| Connection   | SF (kNm/rad) | SF-SPSW (kNm/rad) |
|--------------|--------------|-------------------|
| Connection 1 | 1000         | 1000              |
| Connection 3 | 1000         | 1000              |
| Connection 5 | 5000         | 1000              |
| Connection 7 | 1000         | 1000              |
| Connection 9 | 70,000       | 1000              |
| Connection 12| 210,000      | 1000              |
| Connection 16| 120,000      | 1000              |
| Connection 19| 70,000       | 50,000            |
| Connection 22| 90,000       | 1000              |
| Connection 25| 210,000      | 10,000            |
| Connection 27| 1000         | 1000              |
Figure 13 depicts the iteration process for SF and SF-SPSW. The outcomes vary at the beginning and converge at around 750th iteration.

![Convergence histories](image)

**Figure 13.** Convergence histories.

The best costs of SF, SF-SPSW, and F-SPSW are USD 38,095, USD 13,179, and USD 11,221, respectively, and the resulting total weights are 14.4 tons, 5.3 tons, and 4.5 tons. The results show that for a structure with 15 m height, the total weight of SF is significantly more than that of SF-SPSW and F-SPSW, which is about 2.7 times larger than that of SF-SPSW, 3.2 times larger than that of F-SPSW, and the cost of SF is about 2.9 times higher than that of SF-SPSW and 3.4 times higher than that of F-SPSW.

Rearrange the rotation stiffness in ascending order, and the composition of the total stiffness is given in Figure 14. The overall rotation stiffness in SF is significantly larger than in SF-SPSW, as indicated in Table 4 and Figure 14. In an SF-SPSW, the lateral forces are resisted by the SPSW. With SPSW, the structure can employ a semirigid connection with minor rotation stiffness. The story ratio and the stress ratio results reveal that, in the case of SF, SF-SPSW, and F-SPSW, the maximum story ratio is 0.0199 for both systems, which is less than the threshold value of 0.02. The maximum stress ratios for SF, SF-SPSW, and F-SPSW are 0.89, 0.63, and 0.82, respectively, which are less than 1.00. Therefore, these three systems are dominated by a story ratio. The total weight of F-SPSW is less than that of SF-SPSW because the semirigid connections give more deformation. To satisfy the story ratio, the cross sections enlarge and then increase the weight of the steel frame.

In this optimization problem, the total number of variables is 75, and the result and the iteration histories make the DE algorithm an acceptable methodology.
5.2. Example 2: Three-Bay, Ten-Storey Structure

The dimension of the structure and the infilled plates’ placement are given in Figure 15a, b, respectively. Figure 15c is the numbering of semirigid connections. The structure contains ten floors and three bays with 70 beam–column elements, ten infilled panels, and 60 beam-to-column semirigid connections in total. The height is 30 m, and the span of each beam is 5 m. Cost optimization is conducted for the 10-story semirigid connection steel frame (SF) and the semirigid connection steel frame with steel plate shear wall (SF-SPSW).
Figure 15. (a–c). Dimensions of the three-bay, ten-story structure and the infilled plates’ placement.

The portion of the final results is given in Table 5.
Table 5. Results of SF¹ and SF-SPSW¹.

| Design Variables | Optimum Results       |
|------------------|-----------------------|
|                  | SF¹                  | SF-SPSW¹               | F-SPSW¹               |
| 1                | W44 × 335            | W6 × 12               | W21 × 44              |
| 5                | W40 × 183            | W12 × 22              | W14 × 34              |
| 10               | W16 × 40             | W6 × 8.5              | W18 × 60              |
| 15               | W44 × 262            | W30 × 173             | W21 × 48              |
| 20               | W30 × 90             | W10 × 19              | W14 × 22              |
| 25               | W44 × 230            | W14 × 38              | W14 × 22              |
| 30               | W30 × 90             | W16 × 26              | W14 × 22              |
| 35               | W40 × 149            | W8 × 13               | W5 × 16               |
| 40               | W21 × 48             | W18 × 35              | W16 × 40              |
| 41               | W30 × 90             | W14 × 26              | W12 × 19              |
| 45               | W40 × 183            | W8 × 13               | W24 × 55              |
| 50               | W16 × 40             | W10 × 12              | W10 × 12              |
| 55               | W44 × 262            | W27 × 84              | W21 × 73              |
| 60               | W30 × 90             | W36 × 160             | W30 × 90              |
| 65               | W44 × 230            | W6 × 8.5              | W16 × 31              |
| 70               | W6 × 9               | W8 × 10               | W18 × 35              |
| P1(mm)           | -                    | 5                     | 5                     |
| P3(mm)           | -                    | 5                     | 5                     |
| P5(mm)           | -                    | 4.5                   | 4                     |
| P7(mm)           | -                    | 3.5                   | 3.5                   |
| P9(mm)           | -                    | 2                     | 2.5                   |

| Steel frame weight (t) | 67.4 | 14.6 | 16 |
|------------------------|------|------|----|
| Steel plate weight (t) | -    | 3.3  | 3.2 |
| Total weight (t)       | 67.4 | 17.9 | 19.2 |
| Total cost ($)         | 196,837 | 44,909 | 47,982 |

SF¹ implies the 10-story semirigid connection steel frame; SF-SPSW¹ implies the semirigid connection steel frame with steel plate shear wall. F-SPSW¹ implies the frame with steel plate shear wall.

The iteration histories, story drift ratio, and stress ratio of the two systems are presented in Figures 16–18, respectively. Sorting the rotation stiffness of semirigid connections in ascending order, the result is given in Figure 19. Comparing Figure 14 and Figure 19, which have shown a dramatic difference between SF¹ and SF-SPSW¹, the proportion of 300,000 kNm/rad semirigid joint in SF¹ is much larger than that in SF. The cost and weight increase with the growing height, and the rotation stiffness of the beam-to-column joints enlarge as well. The total weight of SF¹ is 67.4 tons, 17.9 tons for SF-SPSW¹, and 19.2 tons for F-SPSW¹, and the total cost of SF¹, SF-SPSW¹ and F-SPSW¹ is USD 196,837, USD 44,909, and USD 47,982, respectively.

The stress ratio and story drift ratio are 0.99 and 0.0199, respectively. In other words, the stress ratio and story drift ratio dominate both systems simultaneously, as evident from similar studies [37,38]. This is the main reason why the weight of SF-SPSW¹ is less than that of F-SPSW¹ in this case.
Figure 16. Convergence histories.

Figure 17. Story drift ratio of SF-SPSW1 and SF1.

Figure 18. Stress ratio of SF-SPSW1 and SF1.
Figure 19. The proportion of the semirigid connections.

6. Conclusions

This study presents a metaheuristics-based algorithm to optimize the cost of a semirigid connection steel frame with steel plate steel walls. The proposed solution—the Dolphin echolocation algorithm—provides a high-speed, highly efficient way of locating the lowest cost of the structure while satisfying all requirements. Using the simplified model, the steel plate shear wall is able to calculate the tension field and shear force simultaneously, as well as the additional lateral stiffness caused by the tension force acting on the HBE and VBE. Pinned or fixed joints hypothesis in the practical design procedures causes an underestimation of the deformation and a mistake in the bar internal forces. A semirigid connection with actual initial rotation stiffness reveals the precise response of a structure. Considering the different costs between W-sections, walls, and semirigid joints makes the optimum more realistic.

Two numerical examples are presented using a three-bay, five-story, and a three-bay, 10-story steel frame. For each example, three cases are applied to investigate the effects of semirigid connection and steel plate shear walls. The optimum results indicate that, in a low-rise building, the structure is dominated by the story drift ratio only, which makes the steel frame with SPSWs considering the semirigid joints heavier than the one without considering the semirigid joints. The case reverses in the high-rise building scenario because the stress ratio and story drift ratio are dominant in these cases. Moreover, the results also reveal that the semirigid connections reduce the internal force compared with the rigid connections. It is significant to consider the effect of semirigid connections during the analysis process. Finally, the results show that the application of SPSWs is a cost-efficient and weight-efficient alternative, especially in high-rise building schemes.

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