Structural Engineering and Mechanics, *Vol. 58, No. 3 (2016) 597-612*
DOI: http://dx.doi.org/10.12989/sem.2016.58.3.597

Optimum design of steel space frames under earthquake effect using harmony search

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(Received January 9, 2016, Revised March 13, 2016, Accepted April 1, 2016)

Abstract. This paper presents an optimization process using Harmony Search Algorithm for minimum weight of steel space frames under earthquake effects according to Turkish Earthquake Code (2007) specifications. The optimum designs are carried out by selecting suitable sections from a specified list including W profiles taken from American Institute of Steel Construction (AISC). The stress constraints obeying AISC- Load and Resistance Factor Design (LRFD) specifications, lateral displacement constraints and geometric constraints are considered in the optimum designs. A computer program is coded in MATLAB for the purpose to incorporate with SAP2000 OAPI (Open Application Programming Interface) to perform structural analysis of the frames under earthquake loads. Three different steel space frames are carried out for four different seismic earthquake zones defined in Turkish Earthquake Code (2007). Results obtained from the examples show the applicability and robustness of the method.

Keywords: harmony search algorithm; optimum design; steel space frames; earthquake effect

1. Introduction

Optimum design of steel structures is very crucial to reduce steel consumption. So, minimum weight design of steel structures using various algorithm methods is an important research area in structural engineering. Several metaheuristic methods such as genetic algorithm, harmony search algorithm, ant colony algorithm, particle swarm optimization, teaching learning optimization and the other techniques that mimic natural events have been widely developed and studied by many researches in recent years.

One of the first basic techniques, genetic algorithm is studied on discrete optimization of structures by Rajeev and Krishnamoorthy (1992). Moreover, several researchers (Daloglu and Armutcu 1998, Kameshki and Saka 2001, Togan and Daloglu 2006, Hayalioglu and Degertekin 2004) used this basic algorithm to carry out optimum designs of steel structures. Kaveh and Talatahari (2007) researched a discrete particle swarm ant colony optimization for design of steel frame structures. Degertekin (2007) studied a comparison of simulated annealing and genetic algorithm for optimum design of nonlinear steel space frames. Hasancebi *et al.* (2011) focused on optimum design of high-rise steel buildings using an evolution strategy integrated parallel algorithm and examined several high-rise structures. Aydogdu and Saka (2012) researched irregular steel frames according to LRFD-AISC using ant colony optimization. Kaveh and

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http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

Talatahari (2012) used a hybrid CSS and PSO algorithm for optimal design of structures. Dede and Ayvaz (2013) researched structural optimization with teaching-learning-based optimization algorithm. Dede (2013) focused on optimum design of grillage structures to LRFD-AISC with teaching-learning based optimization. Rafiee *et al*. (2013) studied optimum design of steel frames with semi-rigid connections using Big Bang-Big Crunch method. Dede (2014) researched application of teaching-learning-based-optimization algorithm for the discrete optimization of truss structures. Hadidi and Rafiee (2014) studied harmony search based, improved particle swarm optimizer for minimum cost design of semi-rigid steel frames. Artar and Daloglu (2015) performed optimum design of steel space frames with composite beams using genetic algorithm.

Harmony search algorithm, one of the other basic techniques, is selected in this study to carry out the optimum designs of steel space frames. Lee and Geem (2004) substantially researched on a basic study of a new structural optimization based on harmony search algorithm. Saka (2009) examined optimum design of steel sway frames according to BS5950 using harmony search algorithm. Degertekin *et al.* (2009) studied optimum design of geometrically non-linear steel frames with semi-rigid connections using a harmony search algorithm. Degertekin and Hayalioglu (2010) focused on harmony search algorithm for minimum cost design of steel frames with semi-rigid connections and column bases. Togan *et al.* (2011) studied optimization of trusses under uncertainties with harmony search. Degertekin *et al*. (2011) focused on optimum design of geometrically nonlinear steel frames with semi-rigid connections using improved harmony search method. Martini (2011) studied harmony search method for multimodal size, shape and topology optimization of structural frame works.

In the literature, there are many studies available for the optimum design of steel space frames. However, it is hard to see enough studies on the optimization of steel space frame under earthquake effects. Therefore, this study focused on the optimum design of the steel space frames under earthquake load. Three different examples are carried out for four different seismic zones as defined in Turkish Earthquake Code (2007) specifications. In order to carry out optimum designs, a program is developed in MATLAB incorporated with SAP2000 OAPI. The results obtained from analyses show that the seismic zones play very active roles in optimum designs of steel frames.

2. Harmony search algorithm

One of the metaheuristic techniques, Harmony Search (HS) Algorithm is developed by using the improvisation process for a better musical harmony. HS algorithm consists of three basic steps as explained below,

Step 1: Harmony memory matrix (HM) is initialized. Harmony Memory Size (HMS) shows a specified number of solution vectors. Each row of harmony memory matrix represents a steel design and includes design variables which are selected from an available section list. HMS is very similar to the total number of individuals in the population of the genetic algorithm. The form of HM matrix is as follows

$$H = \begin{bmatrix} x_1^1 & x_2^1 & .. & x_{n-1}^1 & x_n^1 \\ x_1^2 & x_2^2 & .. & x_{n-1}^2 & x_n^2 \\ .. & .. & .. & .. & .. \\ x_1^{HMS-1} & x_2^{HMS-1} & .. & x_{n-1}^{HMS-1} & x_n^{HMS-1} \\ x_1^{HMS} & x_2^{HMS} & .. & x_{n-1}^{HMS} & x_n^{HMS} \end{bmatrix} \tag{1}$$

where, x_i^j is the i th design variable of j th solution vector, n is the total number of design variables.

Step 2: Each row of harmony memory matrix is evaluated and their corresponding objective function values ($\varphi(x^1)$, $\varphi(x^2)$,..., $\varphi(x^{HMS-1})$, $\varphi(x^{HMS})$) are calculated. The steel designs in the harmony memory matrix are sorted according to the objective function values. First row presents the best solution in the matrix.

Step 3: A new harmony $x^{nh} = [x_1^{nh}, x_2^{nh}, ..., x_n^{nh}]$ is improvised by selecting each design variable from either harmony memory matrix or the entire section list with the probability of harmony memory consideration rate (HMCR) which changes between 0 and 1. The value of first design variable x_1^{nh} is selected from any value of the first design variables [x_1^1, x_1^2 ,..., x_1^{HMS-1}, x_1^{HMS}] in the harmony matrix or entire section list, X_{sl}. This rule is also used for the other variables of the new harmony. Harmony memory consideration rate (HMCR) is applied as follows

$$\begin{cases} x_i^{nh} \in \left\{x_i^1, x_i^2, ..., x_i^{HMS}\right\} \textit{with probability of HMCR} \\ \quad x_i^{nh} \in X_{sl} \textit{ with probability of } (1 - HMCR) \end{cases} \tag{2}$$

Moreover, the new value of the design variable selected from harmony memory matrix is checked whether this value should be pitch-adjusted or not depending on pitch adjustment ratio (PAR). This decision is determined as below

$$\begin{cases} \textit{Yes, with probability of PAR} \\ \textit{No, with probability of } 1 - PAR \end{cases} \tag{3}$$

In the examples, HMS, HMCR and PAR are selected as 20, 0.8 and 0.3, respectively. The detailed information about HS algorithm can be obtained from Lee and Geem (2004).

3. MATLAB-SAP2000 OAPI

Each row of harmony matrix randomly prepared in MATLAB programming represents a space frame design. The numbers in each row indicate corresponding profiles that are assigned to space frames members by SAP2000 software. The space frames represented by these rows are analyzed by using SAP2000 software and the required analyses results are sent to MATLAB to determine the value objective function. Harmony Search Algorithm operators such as HMCR and PAR are applied to get a stronger matrix. Iterations including all processes are repeated until the iteration numbers previously defined. The flowchart of all steps used MATLAB-SAP2000 OAPI and the HS algorithm methods are presented in Fig. 1.

4. Optimization of space frames and constraints of the design

4.1 The objective function

The optimum design problem of steel space frames for minimum weight can stated as follows

$$\min W = \sum_{k=1}^{ng} A_k \sum_{i=1}^{nk} \rho_i L_i \tag{4}$$

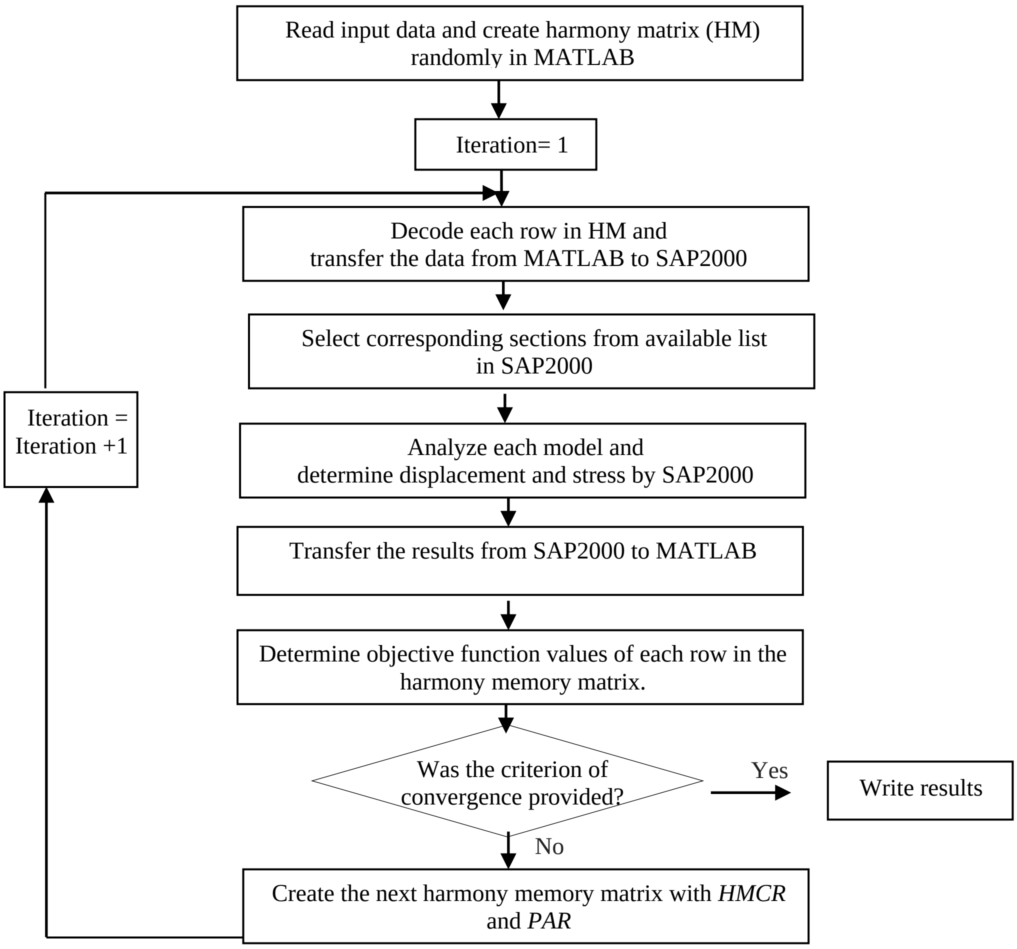


Fig. 1 Flowchart for the optimum design procedure of steel space frames

where W is the weight of the frame, A_k is cross-sectional area of group k, ρ_i and L_i are density and length of member i, ng is total number of groups, nk is the total number of members in group k.

4.2 The design constraints

Stress, displacement, inter-storey drift and geometric constraints used in this study for optimum designs of the space frames are presented as below,

• The stress constraints taken from AISC-LRFD (2001) are shown as

$$\text{for } \frac{P_u}{\phi P_n} \geq 0.2 \qquad g_{il}(x) = \left(\frac{P_u}{\phi P_n}\right)_{il} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right)_{il} - 1.0 \leq 0 \qquad \begin{array}{l} i = 1,\ldots,nm \\ l = 1,\ldots,nl \end{array} \tag{5}$$

$$\text{for } \frac{P_u}{\phi P_n} < 0.2 \qquad g_{il}(x) = \left(\frac{P_u}{2\phi P_n}\right)_{il} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right)_{il} - 1.0 \leq 0 \qquad \begin{array}{l} i = 1,\ldots,nm \\ l = 1,\ldots,nl \end{array} \tag{6}$$

where nm is the total number of members, nl is the total number of loading conditions, P_u is the required axial strength, P_n is the nominal strength, M_{ux} is the required flexural strength about major axis, M_{uy} is the required flexural strength about minor axis, M_{nx} is the nominal flexural strength about major axis, M_{ny} is the nominal flexural strength about minor axis, ϕ is resistance factor for compression (0.85) and for tension (0.90), ϕ_b is resistance factor for flexure (0.90).

The nominal compressive strength is determined as

$$P_n = A_g F_{cr} \tag{7}$$

$$\text{for } \lambda_c \le 1.5 \qquad F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y \tag{8}$$

$$\text{for } \lambda_c > 1.5 \qquad F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right) F_y \tag{9}$$

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} \tag{10}$$

where A_g is the cross-sectional area; K is the effective length factor; E is the elastic modulus; r is the governing radius of gyration; L is the member length; F_y is the yield stress of steel, F_{cr} is critical stress, λ_c is slenderness ratio.

• Displacement constraints are expressed as

$$g_{jl}(x) = \frac{\delta_{jl}}{\delta_{ju}} - 1 \le 0 \qquad \begin{array}{l} j = 1,\dots,m \\ l = 1,\dots,nl \end{array} \tag{11}$$

where δ_{jl} is the displacement of jth degree of freedom under load case l, δ_{ju} is the upper bound, m is the number of restricted displacements, nl is the total number of loading cases.

• Inter-storey drift constraints are formulated by

$$g_{jil}(x) = \frac{\Delta_{jil}}{\Delta_{ju}} - 1 \le 0 \qquad \begin{array}{l} j = 1,\dots,ns \\ i = 1,\dots,nsc \\ l = 1,\dots,nl \end{array} \tag{12}$$

where Δ_{jil} is the inter-storey drift of ith column in the jth storey under load case l, Δ_{ju} is the limit value, ns is the number of storey, nsc is the number of columns in a storey.

• Column-to-column geometric constraints (size constraints) can be defined by

$$g_n(x) = \frac{D_{un}}{D_{ln}} - 1 \le 0 \qquad n=2,\dots,ns \tag{13}$$

$$g_{na}(x) = \frac{A_{un}}{A_{ln}} - 1 \le 0 \qquad na=2,\dots,ns \tag{14}$$

where D_{un} is the depth of upper floor column, D_{in} is the depth of lower floor column, A_{un} is the section area of upper floor column and A_{in} is the section area of lower floor column.

• Beam-to-column geometric constraints (size constraints) can be shown by

$$g_n(x) = \frac{b_{fbk,i}}{b_{bck,i}} - 1 \le 0 \qquad i=2,\dots,n_{bf} \tag{15}$$

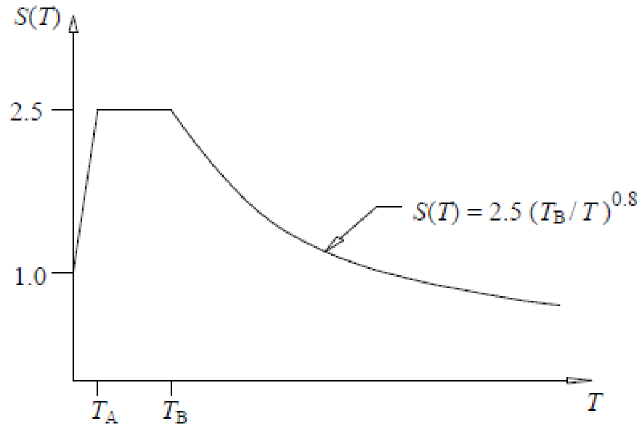


Fig. 2 The spectral coefficient, $S(T)$ (Turkish Earthquake Code 2007)

Table 1 Spectrum characteristic periods (T_A, T_B) (Turkish Earthquake Code 2007)

Local Site Class according to Table 2	T_A (second)	T_B (second)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

where n_{bf} is the number of joints where beams are connected to flange of column, $b_{fbk,i}$ and $b_{fck,i}$ are the flange widths of beam and column, respectively.

4.3 Turkish Earthquake Code (2007)

According to Turkish Earthquake Code (2007), definition of elastic seismic loads and spectral acceleration coefficient are presented as below

$$A(T) = A_o \; I \; S(T) \tag{16}$$

$$S_{ae}(T) = A(T)\, g \tag{17}$$

where $A(T)$ is the spectral acceleration coefficient, A_o, the effective ground acceleration coefficient, is 0.40, 0.30, 0.20 and 0.1 for seismic zones 1, 2, 3 and 4, respectively, I is the building importance factor and it changes between 1 and 1.5 according to the type of building. In the present work, it is assumed as 1 for residential structures. The spectrum coefficient $S(T)$ shown in Fig. 2, is determined by Eq. (18) and depending on local site conditions and building natural period, T. $S_{ae}(T)$ is spectral acceleration, g is gravity. Table 1, Table 2 and Table 3 present some information about the local site conditions.

$$S(T) = 1+1.5\frac{T}{T_A} \qquad 0 \le T \le T_A$$

$$S(T) = 2.5 \qquad T_A \leq T \leq T_B$$

$$S(T) = 2.5 \left(\frac{T_B}{T}\right)^{0.8} \qquad T_B \leq T \tag{18}$$

In the examples, seismic loads are fully resisted by frames. Therefore, structural system behavior factor (R) is considered as 5 (Turkish Earthquake Code 2007).

5. Design examples

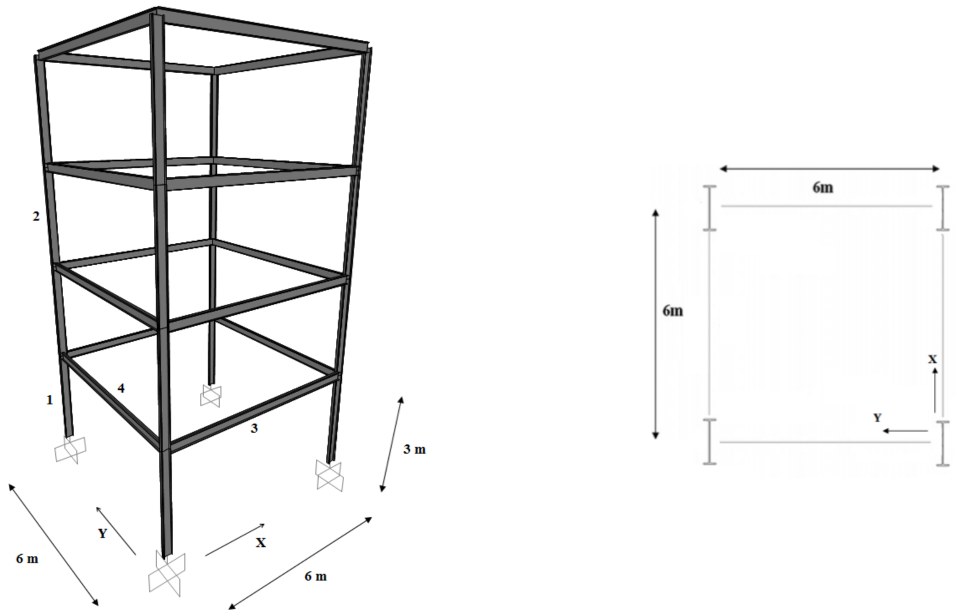
Three different steel space frames are considered on Z2 site class and the space frames separately designed for 4 different seismic zones expressed in Turkish Earthquake Code (2007). The results are evaluated and compared in tabular and graphical formats. Optimum cross sections in the designs are selected from a W-section list of 64 sections (W8×15, W 8×21, W8×24, W 8×28, W 8×31, W 8×35, W 8×40, W 10×15, W 10×22, W 10×26, W 10×33, W 10×39, W 10×54,

Table 2 Local site classes (Turkish Earthquake Code 2007)

Local Site Class	Soil Group according to Table 3 and Topmost Soil Layer Thickness (h_1)
Z1	Group **(A)** soils Group **(B)** soils with $h1 \leq 15$ m
Z2	Group **(B)** soils with $h1 > 15$ m Group **(C)** soils with $h1 \leq 15$ m
Z3	Group **(C)** soils with 15 m$< h1 \leq 50$ m Group **(D)** soils with $h1 \leq 10$ m
Z4	Group **(C)** soils with $h1 > 50$ m Group **(D)** soils with $h1 > 10$ m

Table 3 Soil Groups (Turkish Earthquake Code 2007)

Soil Group	Description of Soil Group
(A)	1. Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks 2. Very dense sand, gravel... 3. Hard clay and silty clay…
(B)	1. Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity…… 2. Dense sand, gravel.......... 3. Very stiff clay, silty clay…
(C)	1. Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity 2. Medium dense sand and gravel......…………………. 3. Stiff clay and silty clay.....
(D)	1. Soft, deep alluvial layers with high ground water level 2. Loose sand.................….. 3. Soft clay and silty clay…..



(a) 3D view (b) plan view

Fig. 3 32-member steel space frame

W 10×77, W 12×19, W 12×26, W 12×30, W 12×35, W 12×40, W 12×45, W 12×50, W 12×53, W 12×58, W 12×72, W 12×96, W 14×26, W 14×30, W 14×34, W 14×38, W 14×43, W 14×48, W 14×53, W 14×61, W 14×68, W 14×74, W 14×82, W 14×90, W 14×120, W 16×26, W 16×31, W 16×36, W 16×40, W 18×35, W 18×40, W 18×50, W 18×76, W 21×50, W 21×62, W 21×132, W 24×68, W 24×103, W 27×94, W 27×161, W 30×108, W 30×148, W 30×191, W 33×221, W 36×150, W 36×170 W, 36×182, W 36×194, W 40×149, W 40×167, W 40×183). Modulus of elasticity is E=200 GPa, yield stress is fy=250 MPa, material density is ρ=7.85 ton/m^3.

Example 1: Four-storey, 32-member steel space frame

The three dimensional and plan views of the 32-member steel space frame is shown in Fig. 3. All members of the space frame are collected into four groups, two for the beams and two for the columns, as seen in Fig. 3. Vertical (gravity) loads on each beam are defined as 15 kN/m. The maximum lateral displacement and inter-storey drift are restricted to 3 cm and 0.75 cm, respectively. The analyses are carried out for Dead load (D) + Earthquake Load (E).

The steel space frame is separately designed five times for all cases. The best, worst and the average minimum weights are presented in Table 4. The optimum results of the best solution reached in the present study are shown in Table 5 and the design histories of all cases are shown in Fig. 4.

It is observed from Table 5 that the optimum design for seismic zone 1 is much higher than the other solutions. Inter-storey drift for this zone is 0.73 cm that is very close to the upper limit 0.75 cm. It shows that the inter-storey drift constraints play very active role in optimum design of the steel space frame. However, this value significantly decreases for the seismic zone 2, 3 and 4. This

Table 4 The minimum weights (kN) obtained for the space frame

Designs of five different runs	Zone 1	Zone 2	Zone 3	Zone 4
Best run	90.84	72.47	65.49	61.93
Worst run	96.18	90.76	68.04	64.22
Average run	92.99	83.65	66.34	62.44

Table 5 Optimum designs for 32-member steel space frame

Group no	Seismic zone 1	Seismic zone 2	Seismic zone 3	Seismic zone 4
1:Columns (1. and 2. storeys)	W27×94	W21×62	W14×43	W14×43
2: Columns (3. and 4. storeys)	W14×61	W18×40	W12×40	W12×40
3: Beams (*X* drictions)	W16×26	W14×26	W12×26	W8×21
4: Beams (*Y* drictions)	W10×26	W12×26	W10×26	W10×26
Total Weight (kN)	90.84	72.47	65.49	61.93
Top drift (cm)	2.13	2.04	1.84	1.33
Inter storey drift (cm)	0.73	0.65	0.56	0.42

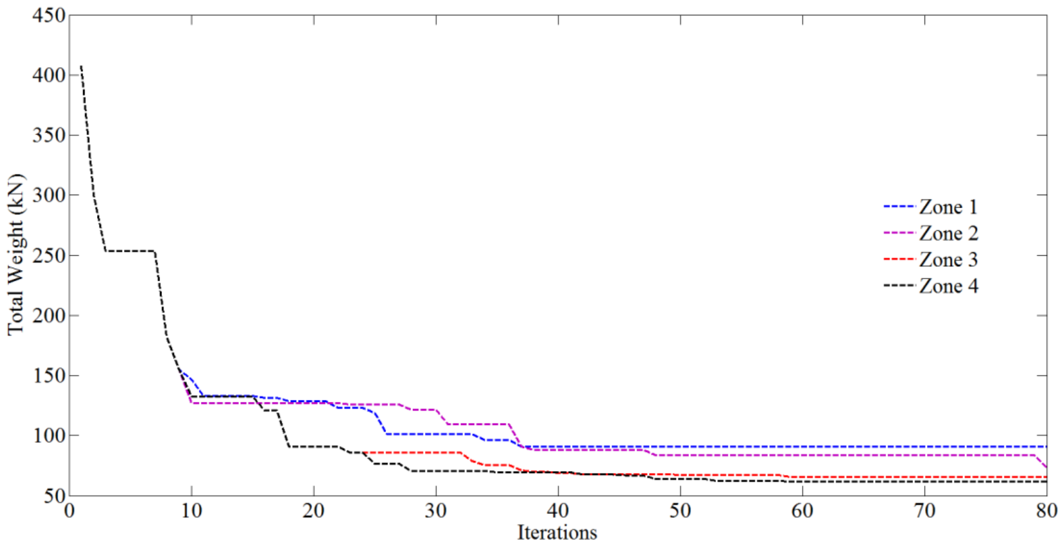
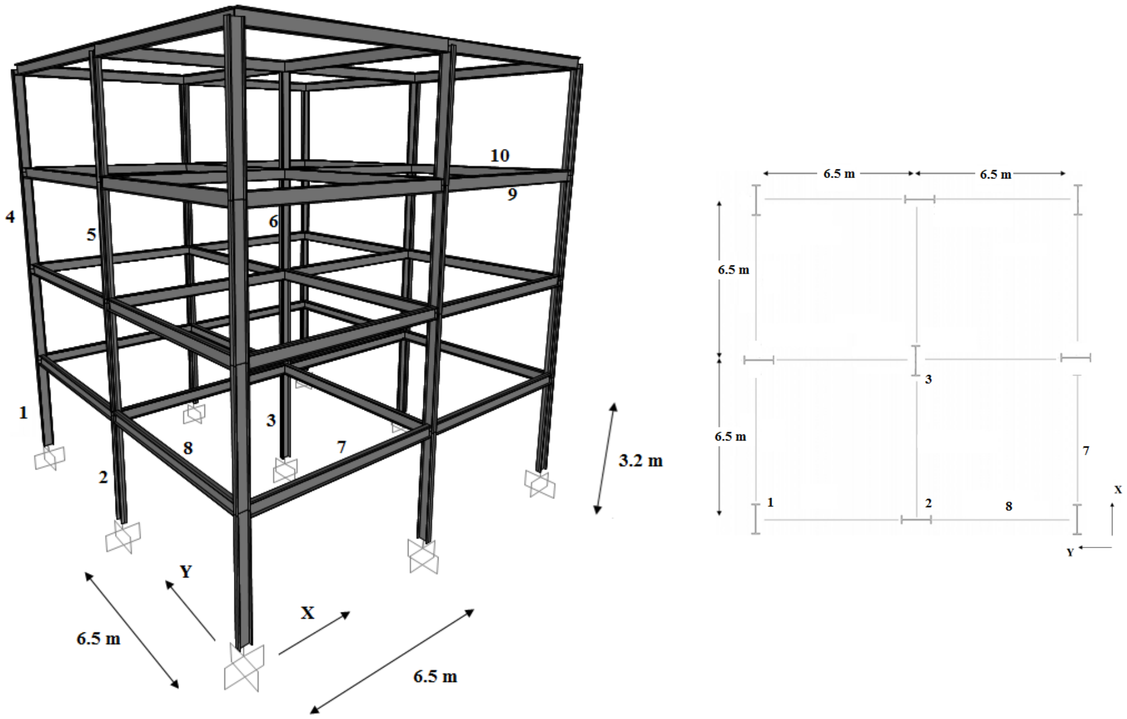


Fig. 4 Design histories of 32-member steel space frame

situation indicates that the lateral effect on the space frame in seismic zone 1 is greater than the lateral effects in the other zones. The minimum weight of the frame obtained for seismic zone 1 is 90.84 kN which is about 20%, 26% and 30% heavier than the minimum weights of optimum design for seismic zones 2, 3 and 4, respectively. The variation of total weights with the iterations is presented in Fig. 4 for all cases.

Example 2: Four-storey, 84-member steel space frame

The three dimensional and plan views of the 84-member steel space frame is shown in Fig. 5.



(a) 3D view (b) plan view

Fig. 5 84-member steel space frame

Table 6 Gravity loading on the beams

Floor Number	Dead Load (kN/m)		Live Load (kN/m)		Snow Load (kN/m)	
	Outer Beam	Inner Beam	Outer Beam	Inner Beam	Outer Beam	Inner Beam
1-3	6.24	12.48	5.18	10.36	-	-
4	6.24	12.48	-	-	1.64	3.28

Table 7 Load combinations

Load case1:1.4D
Load case2:1.2D+1.6L+0.5S
Load case3:1.2D+0.5L+1.6S
Load case4:1.2D+1.0E+0.5L+0.2S

Note*D : dead load, L : live load, S : snow load and E : eartquake load

All members of the space frame are divided into ten groups as shown in Fig. 5. The loading information and load combinations are given in Table 6 and Table 7, respectively. The maximum lateral displacement and inter-storey drift are restricted to 4.26 cm (H/300) and 1.06 cm (h/300), respectively. The 84-member steel space frame is separately designed five times for all cases. The best, worst and the average minimum weights are given in Table 8. The optimum results of the

Table 8 The minimum weights (kN) obtained for the space frame

Designs of five different runs	Zone 1	Zone 2	Zone 3	Zone 4
Best run	214.94	212.51	188.27	191.01
Worst run	225.66	214.90	193.50	191.60
Average run	218.88	217.35	192.09	191.21

Table 9 Optimum designs for 32-member steel space frame

Group no	S. Zone 1	S. Zone 2	S. Zone 3	S. Zone 4
1	W14×30	W14×34	W14×30	W14×38
2	W14×61	W14×43	W10×54	W14×53
3	W30×148	W27×161	W12×53	W14×53
4	W14×30	W8×31	W8×24	W10×22
5	W12×45	W14×38	W10×39	W14×34
6	W24×68	W16×40	W10×39	W10×22
7	W14×30	W18×35	W14×26	W14×30
8	W16×26	W16×26	W16×26	W12×26
9	W14×30	W16×31	W16×31	W16×31
10	W14×30	W12×30	W16×26	W12×26
Total Weight (kN)	214.94	212.51	188.27	191.01
Top Drift (cm)	3.55	2.76	2.73	1.47
Inter-Storey Drift (cm)	1.011	0.96	0.84	0.53

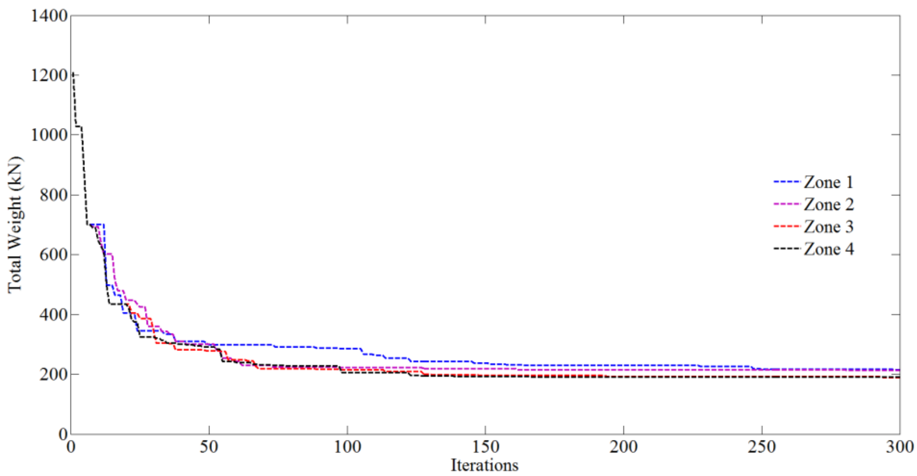


Fig. 6 Design histories of 84-member steel space frame

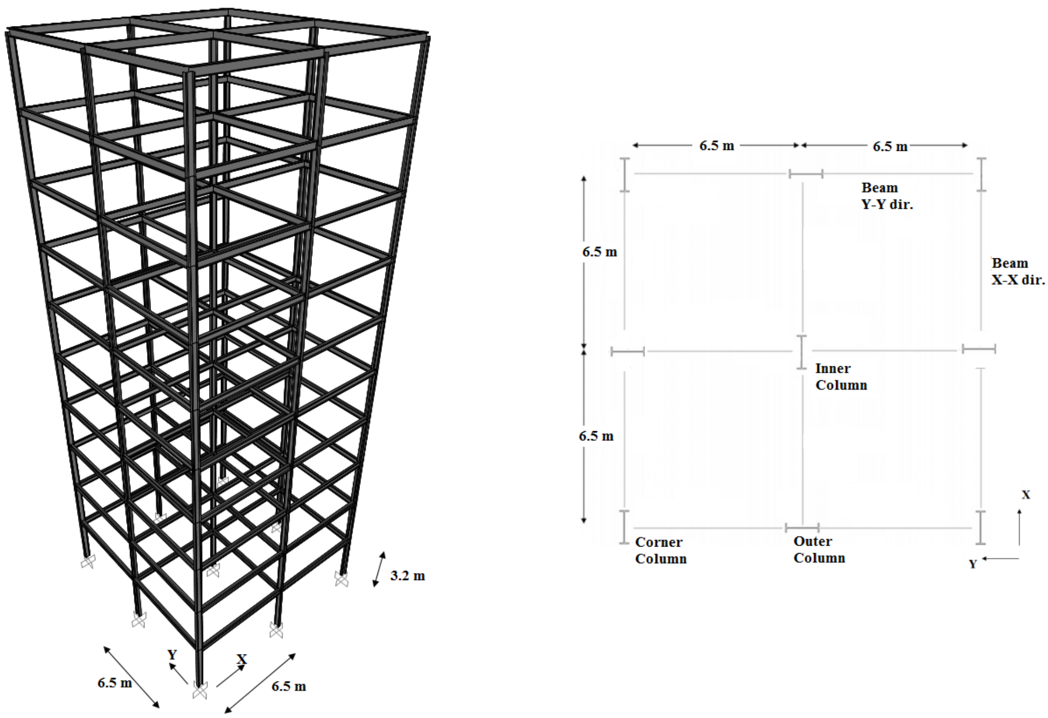
best solution reached in the present study are shown in Table 9 and the design histories of all cases are shown in Fig. 6.

As shown in Table 9, the heaviest value of the minimum weight of the space frame is found as 214.94 kN for seismic zone 1. However, the inter-storey drift value for this zone, 1.011 cm, is higher than the values of the other optimum solutions carried out for seismic zone 2, 3, and 4. This value is also very close to the limit value 1.06 cm. So, the inter-storey drift constraints play very active roles in determining the optimum design of the frame for seismic zone 1. The space frames in seismic zone 1 are subjected to greater lateral effect, so the larger column profiles are selected in the optimum design. The minimum weight 214.94 kN is about 1%, 12% and 11% heavier than the other minimum weights, respectively. Although the space frame in this example is four storey as in the first example, the differences of the minimum weights of optimum solutions according to seismic zones compared with the ones of the first example are significantly reduced by depending on geometry of the structure. The variation of total weights with the iterations is also seen in Fig. 6 for all cases.

Example 3: Ten-storey, 210-member steel space frame

The three dimensional and plan views of the 210-member steel space frame is shown in Fig. 7.

All members of the space frame are divided into 20 groups as seen Table 10. The loading information and applied combinations are given in the second example. The maximum lateral displacement and inter-storey drift are restricted to 10.66 cm (H/300) and 1.06 cm (h/300), respectively. The 210-member steel space frame is separately designed five times for all cases. The best, worst and the average minimum weights are presented in Table 11. The optimum results of the best solution reached in the present study are shown in Table 12 and the design histories of all cases are shown in Fig. 8.



(a) 3D view (b) plan view

Fig. 7 210-member steel space frame

Table 10 Member groups

Storey number	Group numbers				
	Column			Beam	
	Corner	Outer	Inner	*X-X* dir.	*Y-Y* dir.
1,2,3	1	2	3	13	14
4,5,6	4	5	6	15	16
7,8	7	8	9	17	18
9,10	10	11	12	19	20

Table 11 The minimum weights (kN) obtained for the space frame

Designs of five different runs	Zone 1	Zone 2	Zone 3	Zone 4
Best run	807.31	703.30	628.88	612.08
Worst run	845.17	743.43	658.30	638.75
Average run	819.85	722.26	639.80	626.21

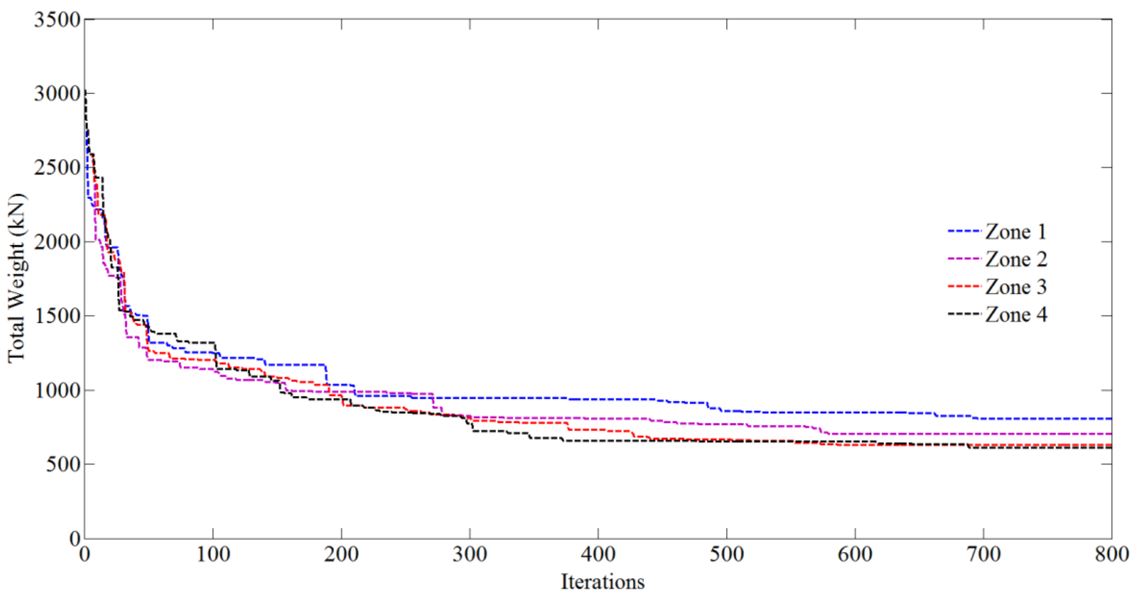


Fig. 8 Design histories of 210-member steel space frame

It is observed from Table 12 that the optimum design results for seismic zone 1 are very larger than the other optimum solutions. The minimum weight for seismic zone 1, 807.31 kN, is about 13%, 22% and 24% heavier than the other minimum weights obtained for seismic zones 2, 3 and 4, respectively. Although the storey plan of the space frame in this example is same with the second example, the differences of the minimum weights according to seismic zones increased by depending on the total number of storeys. The inter-storey drift value for the optimum designs for seismic zone 1 and 2 is 1.01 cm that is very close to 1.06 cm. As seen in Table 12 that the larger profiles are selected for columns in the solution for seismic zone 1 due to greater lateral effect. The variation of steel weights with the iterations is also seen in Fig. 8 for all cases.

Table 12 Optimum designs for 210-member steel space frame

Group no	S. Zone 1	S. Zone 2	S. Zone 3	S. Zone 4
1	W40×167	W27×94	W14×90	W12×50
2	W27×161	W40×167	W14×90	W30×148
3	W36×194	W36×170	W40×149	W36×150
4	W14×68	W16×40	W12×50	W12×50
5	W14×90	W14×120	W14×90	W14×74
6	W36×194	W21×132	W24×103	W24×103
7	W12×58	W16×40	W12×40	W12×40
8	W10×54	W14×90	W14×90	W14×53
9	W36×150	W18×50	W18×50	W10×54
10	W8×28	W8×31	W12×40	W10×39
11	W10×54	W12×45	W14×30	W14×53
12	W12×40	W10×26	W14×34	W8×21
13	W16×26	W16×26	W16×26	W16×26
14	W16×26	W10×33	W14×26	W12×26
15	W18×40	W16×26	W16×26	W14×30
16	W10×39	W12×30	W12×30	W12×30
17	W14×30	W12×26	W10×33	W14×34
18	W12×45	W16×36	W12×30	W16×26
19	W18×35	W12×30	W18×35	W12×26
20	W12×35	W14×34	W14×30	W16×26
Toplam Weight (kN)	807.31	703.30	628.88	612.08
Top Drift (cm)	8.34	7.92	5.42	3.04
Inter Storey Drift (cm)	1.01	1.01	0.68	0.42

6. Conclusions

In the present study, Harmony Search Algorithm is used for optimum designs of steel space frames under earthquake effect obeying Turkish Earthquake Code (2007) specifications. Stress constraints of AISC-LRFD, maximum lateral displacement, inter-storey drift constraints and geometric constraints are subjected to the space frames. A program is developed by MATLAB programming incorporated SAP2000 OAPI for all procedures. Three different steel space frames on Z2 site class are separately designed for four different seismic zones. The analyses results obtained from the solutions are presented in tabular and graphical formats. Three different steel space frames are separately designed for four different seismic zones. In the all examples, the inter-storey drift value for seismic zone 1 is very close to the drift limit value although the larger sections are selected for columns. So, these constraints play a crucial role in optimum designs of space frames in seismic zone 1. Moreover, in the third example, the minimum weight for seismic zone 1 is about 13%, 22% and 24% heavier than the other minimum weights obtained for seismic zones 2, 3 and 4, respectively.

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