Optimum Design of Transmission Towers Subjected to Wind and Earthquake Loading

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ABSTRACT

Transmission towers (1S2 and 2S2-132 kV types) subjected to multiple combinations of wind, seismic and dead loads are optimally designed for least weight. The member areas and joint coordinates are treated as design variables. Members are designed to satisfy stress limits. Joint coordinate variables are linked to reduce the number of independent design variables. The optimization problem is divided into two design spaces. While changing the coordinate variables, using Hooke and Jeeves method, the member areas are treated as dependent design variables. The sections used are: angle and pipe sections which represent the commonly used sections in lattice transmission towers, in addition to tube sections. The structural analysis and the fully stressed design are performed using STAAD pro v.2006. It is found that the 1S2 tower with angle section and X-bracing, under anti-cascade loading condition, has a reduction in weight of about 14% of the weight before optimization, while the reduction for 2S2 was 24% for the same conditions. For seismic loading conditions, the reductions were about 24% for 1S2 tower and 22% for 2S2 tower with angle section and X-bracing. The results showed that the 1S2 tower with pipe section has a weight of 85% of that with angle section, under the same loading condition, while for 2S2 tower, the weight with pipe section was about 88% of that with angle section.

KEYWORDS: Transmission towers, Optimum design, Wind load, Earthquake.

INTRODUCTION

Towers or masts are structures that are built in order to fulfill the need for placing objects or persons at a certain level above the ground. Transmission towers support the phase conductors and earth wires of a transmission line (Shu-Jin Fang et al., 1999). Fig.(1) shows typical details of transmission towers.

The optimum design of towers and steel structures has been studied by many researchers. Sheppard (1972) used dynamic programming method to achieve the minimum cost of transmission towers. It was concluded that the dynamic programming method is a useful technique for the synthesis of optimal layout for structures having a simple interaction between their parts. Green (1985) studied the minimum weight sizing of guyed antenna towers. Numerical studies were performed on a typical VLF antenna tower. It was concluded that there are several difficulties of automated design when using non-linear structural analysis. Jalkanen (2007) studied tubular truss optimization using heuristic multipurpose algorithms. The multicriteria topology, shape and sizing optimization problem has been formulated based on practical real life needs. The tubular truss optimization problem has been solved using four multipurpose heuristic algorithms. Two of them are local search algorithms; simulated annealing (SA) and tabu search

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(TS), and two of them are population based methods; genetic algorithm (GA) and particle swarm optimization. Also, the conflict of mass and cost and effect of the number of different design variables on the final solution have been studied.

In this paper, the optimum design of 1S2 and 2S2-132kV transmission towers subjected to wind, seismic and self-weight will be investigated using the optimality criteria and Hooke and Jeeves method.

132 kV Transmission Tower Types

Transmission towers of 132kV shall be of the self-supporting type in single or double circuit configuration as specified in Tables (1) and (2) (MOE, 2006), depending on the deviation angle of the conductor. The common types are 1S2 or 2S2 tower with suspension insulator sets normally used subject to the sum of adjacent span limitation.

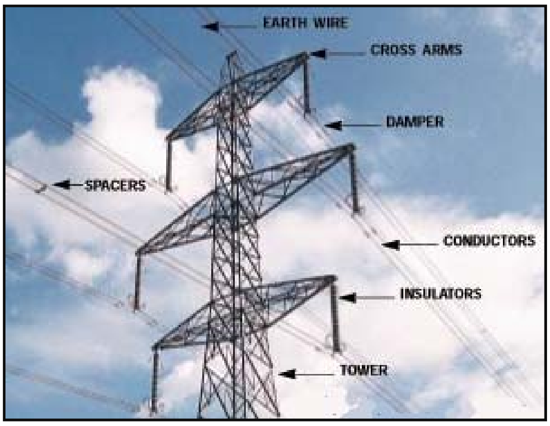

Figure 1: Typical Details of Transmission Towers

Table 1. 132kV Single Circuit Transmission Towers

Type	Position of use	Angle of deviation or entry	Type of insulator
1S2	Tangent	0^o-2^o	Suspension
1R2	Long span tangent	0^o	Suspension
1M2	Medium angle 30^o	0^o-30^o	Tension
1T2	Medium angle 60^o	30^o-60^o	Tension
1E2	Large angle	60^o-90^o	Tension
	Full dead end	0^o-30^o	Tension
1K2	Under-crossing tower	0^o-30^o	Tension
1SP2	Tee-off/special purpose	90^o	Tension

Table 2. 132kV Double Circuit Transmission Towers

Type	Position of use	Angle of deviation or entry	Type of insulator
2S2	Tangent	0^o -2^o	Suspension
2R2	Long span tangent	0^o	Suspension
2M2	Medium angle 30º	0^o-30^o	Tension
2T2	Medium angle 60º	30^o-60^o	Tension
2E2	Large angle	60^o-90^o	Tension
	Full dead end	0^o-30^o	Tension
2K2	Under-crossing tower	0^o-30^o	Tension
2SP2	Tee-off/special purpose	90^o	Tension

Type of Load

The loads acting on a transmission tower are (ASCE 10-97(2000):

a. dead load of tower (self-weight).

b. dead load from conductors and other equipment.

c. load from ice, rime or wet snow on conductors and equipment.

d. ice load on the tower itself.

e. erection and maintenance loads.

f. wind load on the tower.

g. wind load on conductors and equipment.

h. loads from conductor tensile forces.

i. damage forces.

j. earthquake forces.

Wind Load

Wind possesses kinetic energy by virtue of its velocity and mass, which is transformed into potential energy of pressure when a structure obstructs the path of wind. Natural wind itself is neither steady nor uniform; it varies along the dimensions of the structures as well as with time. When the complete assembly of the lattice structures is considered, wind forces on different members of the structure are only partially correlated and time varying.

Wind Characteristics

Due to variation of wind speeds with height, terrain and averaging time, wind load codes describe a reference wind speed. The mean wind speed is usually represented by power law as:

$$U_{(Z)} = U_{ref}\left(\frac{z}{Z_{ref}}\right)^{\alpha} \quad \text{... (1)}$$

where:

$U_{(Z)}$: velocity of wind that varies with height;

U_{ref} : mean velocity of wind;

z : height above ground in terrain;

Z_{ref} : reference height =10 m;

α : power law exponent = 0.16 for exposed and windy areas.

Wind loads on each element can be determined according to Eq. (2). If $U_{(Z)}$ is assumed to be much larger than along wind fluctuation *u(z, t)*, the second-order term involving *u(z, t)* can be ignored. Thus the magnitude of drag force, $F_{(z)}$, acting on the element along a specific direction is:

$$F_{(Z)} = 0.5\,\rho\, C_d\, A(U_{(Z)} + u_{(z,t)})^2 \quad \text{….(2)}$$

By ignoring the second order fluctuation component:

$$F_{(Z)} = 0.5\,\rho\, C_d\, A U_{(Z)}^{\ 2}\left(1 + \frac{2u_{(z,t)}}{U_{(Z)}}\right) \quad \text{....(3)}$$

where:

$F_{(Z)}$: drag force;

$u_{(z,t)}$: along wind fluctuation velocity;

ρ : air density;

C_d : drag coefficient which is empirically calculated and depends on various factors such as solidity ratio;

A: surface area of member.

When the fluctuation velocity is much less than the mean velocity, the magnitude of drag force acting normal or in the across wind direction to a surface of area, is defined as:

$$F = 0.5\,\rho\, C_d\, A U_{(Z)}^{\ 2} \quad \text{.............(4)}$$

Eq. (4) is used to determine the wind loads on the mast. For all cables, drag coefficient is assumed to be equal to 0.6 **(**Naser, 2010).

In this study, the direction of wind is taken in two directions.

(1) Maximum wind velocity at 90° to line direction

The maximum design wind velocity considered in this study is 40 (m/s) acting at an elevation of 10 m above the ground level. The pressure on members of tower structure is calculated with the following formula:

$$P_t = 0.5\rho\, C_d\, U_t^{\ 2} \quad \text{…………} \quad (5)$$

where:

P_t : pressure at tower elevations;
U_t : maximum mean velocity at tower elevations.

(2) Maximum wind velocity at 45° to line direction

When the wind direction is at 45° to tower line, this case can be stated as:

a) for longitudinal face of tower:

$$P = U^2 (1 + 0.2 \ \sin^2 2\phi) \cos\phi \quad \ldots (6)$$

b) for transverse face of tower

$$P = U^2 (1 + 0.2 \ \sin^2 2\phi) \sin\phi \quad \ldots (7)$$

In calculating wind loads, the effects of terrain, structure height, wind gust and structure shape are included. The effective height of conductor and shield wires can be explained as follows:

1) The effective height of the conductors is calculated by:

$$h_c = \bar{h}_c - l_S - \frac{1}{3} S_c \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

where

h_c : effective height of the conductor;
$\bar{h}_c$: average height of the conductor (height above the ground of wire attachment points);
l_S : length of insulator;
S_c : sag of conductor.

2) The effective height of the shield wires is calculated by:

$$h_{sh} = \bar{h}_{sh} - \frac{1}{3} S_{sh} \quad \ldots\ldots\ldots\ldots \quad (9)$$

where:

h_{sh}: effective height of the shield wire;
$\bar{h}_{sh}$: average height of the shield wire;
S_{sh}: sag of shield.

Swinging of Isolator

Swinging of isolator is dimensioning the head of tower; i.e., vertical distance between cross-arms and their length. In modeling of a tower with isolator, the isolator swinging with one of three angle cases depends on the wind loading case. These cases are explained in Table (3). In this study, isolator swinging is considered to be 60° for maximum speed.

Span Length Design

In transmission line calculations, there are different span lengths that can be considered. These are weight span and wind span, as well as the angle of tower line deviation which is related with span, where a decreased angle can be accommodated with an increased span or *vice versa*. In this study, a light angle of (2°) is used.

Table 3. Swinging Angle of Isolator

Loading case	Swinging angle of isolator
Without wind	0°
Reduced wind speed	±15°
Maximum wind speed	±30°

Earthquake Tower Design

Earthquakes are natural phenomena which cause the ground to shake violently; thereby triggering landslides, creating floods, causing the ground to heave and crack and causing large-scale destruction to life and property. In particular, the effect of earthquakes on structures and the design of structures to withstand earthquakes with no or minimum damage form the subject of earthquake resistant structural design. The important factors which influence earthquake resistant design are: the geographical location of the structure, the site's soil and foundation conditions, the importance of the structure as well as the dynamic characteristics of the structure such as the natural periods and the properties of the structure, like: strength, stiffness, ductility and energy dissipation capacity.

Seismic Coefficient Method

This is the simplest of the available methods and is applicable to structures which are simple, symmetric and regular**.** In this method, the seismic load is idealized as a system of equivalent static loads, which is applied to the structure, and an elastic analysis is performed to ensure that the stresses are within allowable limits. The sum of the equivalent static loads is proportional to the total weight of the structure and the constant of proportionality, known as the seismic coefficient, is taken as the product of various factors which influence the design and are specified in the codes.

Base Shear-International Building Code (IBC)

The IBC addresses the probability of significant seismic ground motion by using maps of spectral response accelerations (S_s and S_1) for various geographic locations. These mapped spectral response accelerations are combined with soil conditions and building occupancy classifications to determine Seismic Design Categories A through F for various structures. Seismic Design Category A indicates a structure that is expected to experience very minor (if any) seismic activity. Seismic Design Category F indicates a structure with very high probability of experiencing significant seismic activity. The equivalent static force procedure in the International Building Code (IBC) specifies the following formula for calculating base shear (V):

$$V = C_s W \qquad \text{…(10)}$$

where the seismic response coefficient, C_s, is defined as:

$$C_s = (2/3)\, F_v\, S_1 I_E /(R_s\, T_f) \qquad \text{…(11)}$$

The IBC specifies the following upper and lower bounds for C_s:

Upper bound:

$$C_s < (2/3)\, F_a\, S_s\, I_E / R_s \qquad \text{...(12)}$$

Lower bound:

$$C_s > (0.044)\,(2/3)\, F_a\, S_s\, I_E \qquad \text{...(13)}$$

For structures located where, $S_1 > 0.6g$, Cs shall not be less than:

$$C_s > 0.5\, S_1\, I_E / R_s \qquad \text{…(14)}$$

W = effective seismic weight of the structure (dead loads plus applicable portions of some storage loads and snow loads).

I_E = seismic importance factor.

The IBC provides the following simplified method for estimating T_f based on the height of the structure (h_n):

$$T_f = C_t\,(h_n)^{3/4} \qquad \text{……..…..……} \qquad (15)$$

where:

T_f = fundamental (natural) period of vibration for a structure.

C_t=0.0853 for steel frames;

C_t=0.0731 for other structures;

h_n = height of the top level of a structure (ft).

For structures with flat roofs, h_n is the distance from the ground to the roof/ceiling system. For structures with sloped (pitched) roofs, h_n may be taken as either the height of the ceiling system above the ground or the mean roof height.

R_s= structural response modification factor.

S_s and S_1 are maximum spectral response accelerations for short (0.2 second) periods of vibration and for longer (1.0 second) periods of vibration, respectively. Values for S_s and S_1 are provided as contour lines superimposed on maps, in units of percent acceleration due to gravity (%g).

F_v and F_a are seismic coefficients associated with structural sensitivity to the velocity and acceleration (respectively) of seismic ground motion. F_v and F_a are based on the spectral response accelerations (S_s and S_1) associated with the geographic location of the structure and soil conditions at the site. Values for F_v and F_a are specified in IBC. For this study, it is assumed that S_1=0.39 and S_s=0.98

Optimization of Tower Geometry

There is a need for reducing the number of independent design variables. Accordingly, the optimization problem is completely stated by only three independent design variables; these are:

1-B: base width of tower.

2-NP: number of panels.

3-R: panel height ratio.

R can be defined as:

H(i+1)=R*H(i) i=1,2,3,… (NP-1)

where:

H(i): height of the i_{th} panel (m);

NP: no. of panels;

R: panel height ratio.

The height of tower body, H, is given by:

$$\begin{aligned} H &= H(1) + H(2) + \cdots + H(NP) \\ &= H(1) + RH(1) + R^2H(1) + \cdots + R^{NP-1}H(1) \\ &= H(1)[1 + R + R^2 + \cdots + R^{NP-1}] \end{aligned}$$

$$H(1) = \frac{H}{[1+R+R^2+\cdots\ldots\ldots\ldots+R^{NP-1}]}$$

The objective function is also expressed in terms of these three independent design variables as:

W=f (B, NP, R).

The method of Hooke and Jeeves consists of sequence of exploration steps about a base point, which if successful, are followed by pattern moves.

Optimization of Transmission Tower Subjected to Wind Loading

As previously mentioned, anti-cascade loading condition (load case no. 9) is considered as the critical loading case for the two types (1S2 and 2S2-132 kV) of transmission towers. Tables (4) to (15) summarize the results of the optimization process for towers under wind loads.

Table 4. History of Design Variables for 1S2-Tower with Angle Section and X-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	43.79	40.55	*38.47	38.56	39.18	39.1	40.75	50.2

*** Optimum weight with $\frac{B}{HT} = 0.1558$.**

Table 5. History of Design Variables for 1S2-Tower with Angle Section and K-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	46.92	44.82	43.37	41.8	41.46	*41.45	43.86	46.67

*** Optimum weight with $\frac{B}{HT} = 0.1265$.**

Table 6. History of Design Variables for 1S2-Tower with Pipe Section and X-Bracing under Wind Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	43.69	32.84	32.47	*32.42	33.93	34.3	36.5	44.65

*** Optimum weight with** $\frac{B}{HT} = 0.147$.

Table 7. History of Design Variables for 1S2-Tower with Pipe Section and K-Bracing under Anti-Cascade Loading Condition

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	35.22	34.93	*34.6	35.41	35.73	36.22	37.88	39.49

*** Optimum weight with** $\frac{B}{HT} = 0.1558$.

Table 8. History of Design Variables for 1S2-Tower with Tube Section and X-Bracing under Wind Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	60.87	*59.7	59.86	60.82	62.12	63.02	65.66	73.59

*** Optimum weight with** $\frac{B}{HT} = 0.1708$.

Table 9. History of Design Variables for1S2-Tower with Tube Section and K-Bracing under Wind Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	63	*62.265	62.68	63.46	64	64.74	66.06	67.95

*** Optimum weight with** $\frac{B}{HT} = 0.1708$.

Table 10. History of Design Variables for 2S2-Tower with Angle Section and X-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	52.46	50.24	*48.17	48.9	49.03	50.51	53.21	60

*** Optimum weight with** $\frac{B}{HT} = \mathbf{0.1558}$.

Table 11. History of Design Variables for 2S2-Tower with Angle Section and K-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	56.11	54.67	54.43	53.93	*53.72	55.27	58.88	62.62

*** Optimum weight with** $\frac{B}{HT} = \mathbf{0.1411}$**.**

Table 12. History of Design Variables for 2S2-Tower with Pipe Section and X-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	44.54	*42.5	43.58	43.68	44.77	44.87	48.55	60.96

*** Optimum weight with** $\frac{B}{HT} = \mathbf{0.1708}$.

Table 13. History of Design Variables for 2S2-Tower with Pipe Section and K-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	46	*44.54	46.36	47.6	47.7	48.67	48.92	50.31

*** Optimum weight with** $\frac{B}{HT} = \mathbf{0.1708}$**.**

Table 14. History of Design Variables for 2S2-Tower with Tube Section and X-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	71.27	70.1	*69.9	71.65	73.63	75.78	81.67	90.14

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1558}$.

Table 15. History of Design Variables for 2S2-Tower with Tube Section and K-Bracing under Anti-Cascade Loading Condition

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	73.79	*72.63	74.41	75.42	76.37	78.16	82.773	85.32

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1708}$.

Optimization of Transmission Tower Subjected to Seismic Loads

Tables (16) to (27) represent the results of optimization of towers under seismic loads.

Table 16. History of Design Variables for 1S2-Tower with Angle Section and X-Bracing under Seismic Load

Iteration No. / Design Variables	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	34.4	30.47	28.53	28.15	27.92	*26.848	28.59	30.807

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1265}$.

Table 17. History of Design Variables for 1S2-Tower with Angle Section and K-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	37.912	35.67	31.8	30.13	29.17	*27.926	28.787	31

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1265}$.

Table 18. History of Design Variables for 1S2-Tower with Pipe Section and X-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight(kN)	24.745	23.5	22.58	22.33	22	*21.1	25.2	30.33

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1265}$.

Table 19. History of Design Variables for 1S2-Tower with Pipe Section and K-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	26.48	25.618	24.83	23.93	23.7	*23.6	24.13	25.1

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1265}$.

Table 20. History of Design Variables for 1S2-Tower with Tube Section and X-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	50.8	50.58	*50.018	50.35	50.46	50.28	50.76	52.28

*** Optimum weight with** $\frac{\mathbf{B}}{\mathbf{HT}} = \mathbf{0.1558}$.

Table 21. History of Design Variables for 1S2-Tower with Tube Section and K-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	53.7	*53.19	53.6	53.72	53.77	53.85	53.97	54.28

*** Optimum weight with $\frac{B}{HT} = 0.1708$.**

Table 22. History of Design Variables for 2S2-Tower with Angle Section and X-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	37	33.755	31.039	30.913	30.7	*30.106	31.63	37.379

*** Optimum weight with $\frac{B}{HT} = 0.1265$.**

Table 23. History of Design Variables for 2S2-Tower with Angle Section and K-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	39.911	37.8	35.45	33.714	33.393	*32.878	34.848	36.426

*** Optimum weight with $\frac{B}{HT} = 0.1265$.**

Table 24. History of Design Variables for 2S2-Tower with Pipe Section and X-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	29.461	28.029	27.159	26.925	26.6	*25.514	26.68	31.111

*** Optimum weight with $\frac{B}{HT} = 0.1265$.**

Table 25. History of Design Variables for 2S2-Tower with Pipe Section and K-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	31	29.928	28.932	28.6	*28.395	28.1	29.089	30.905

*** Optimum weight with** $\frac{B}{HT} = 0.1411$.

Table 26. History of Design Variables for 2S2-Tower with Tube Section and X-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	54.8	53.93	53.8	*53.5	53.63	54.32	54.9	57

*** Optimum weight with** $\frac{B}{HT} = 0.147$.

Table 27. History of Design Variables for 2S2-Tower with Tube Sections and K-Bracing under Seismic Load

Design Variables \ Iteration No.	1	2	3	4	5	6	7	8
B (m)	6	5.8	5.3	5	4.8	4.3	4	3.8
NP	11	12	13	14	15	16	17	18
R	1.05	1.02	1	0.97	0.95	0.93	0.9	0.88
Weight (kN)	56.5	*56	57	57.34	57.65	57.9	58	59

*** Optimum weight with** $\frac{B}{HT} = 0.1708$.

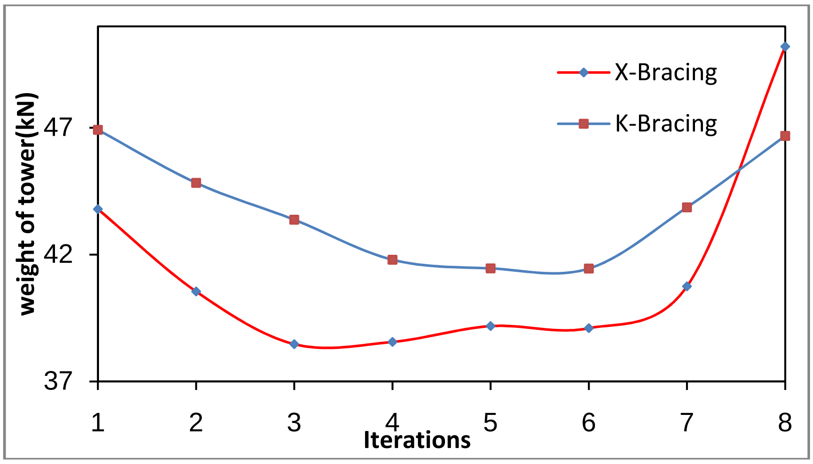

Figure 2: Comparison between X and K-Bracing for 1S2 Tower with Angle Section under Anti-Cascade Loading Condition

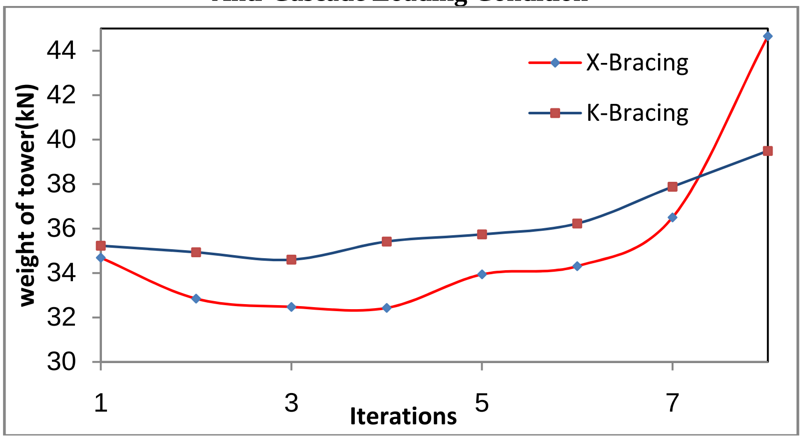

Figure 3: Comparison between X and K-Bracing for 1S2 Tower with Pipe Section under Anti-Cascade Loading Condition

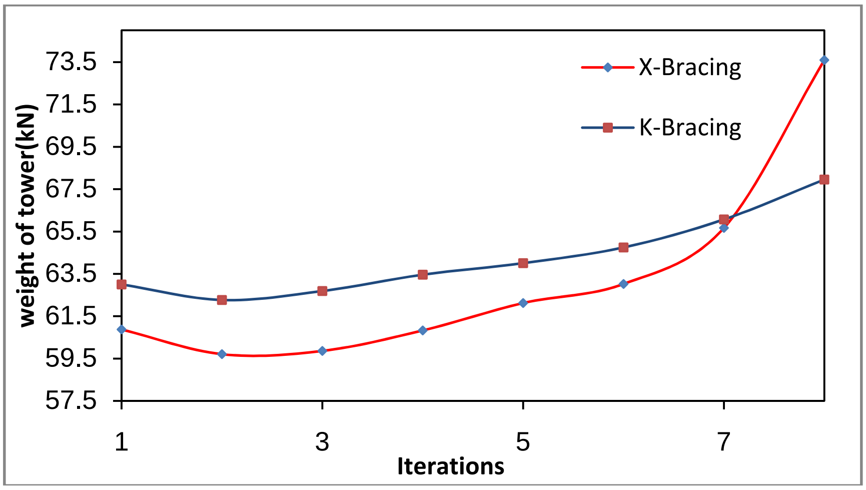

Figure 4: Comparison between X and K-Bracing for 1S2 Tower with Tube Section under Anti-Cascade Loading Condition

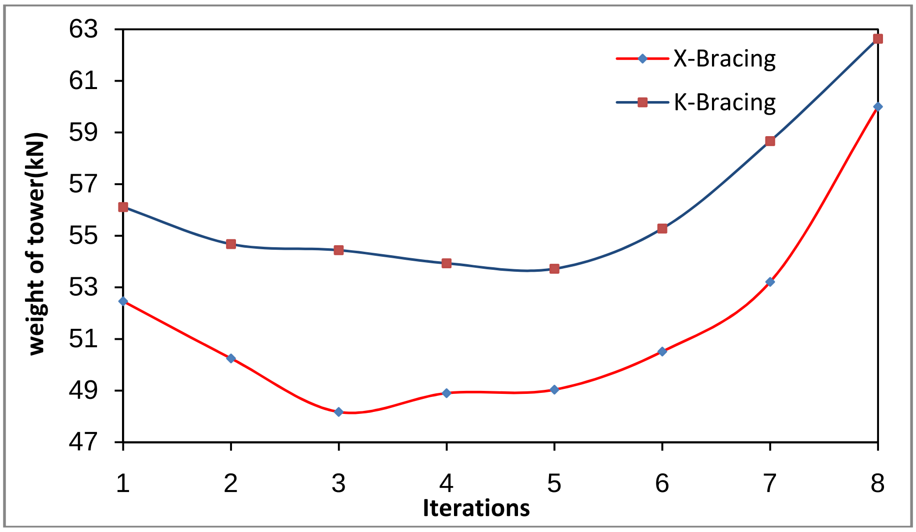

Figure 5: Comparison between X and K-Bracing for 2S2 Tower with Angle Section under Anti-Cascade Loading Condition

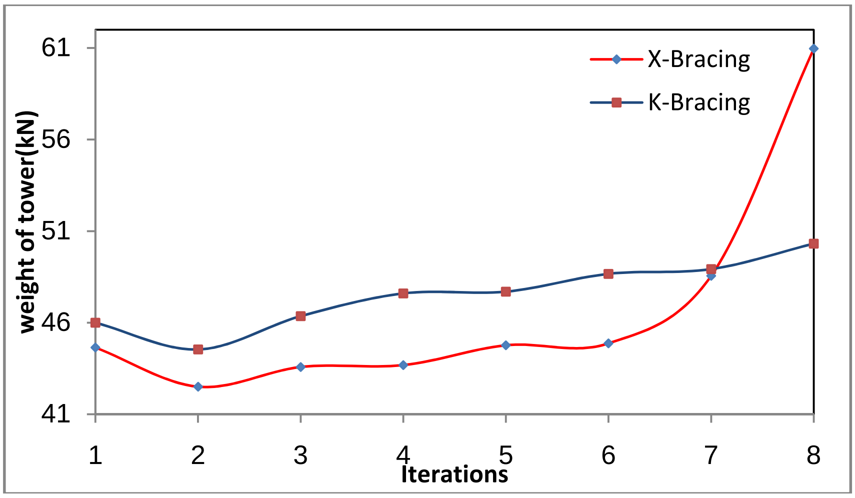

Figure 6: Comparison between X and K-Bracing for 2S2 Tower with Pipe Section under Anti-Cascade Loading Condition

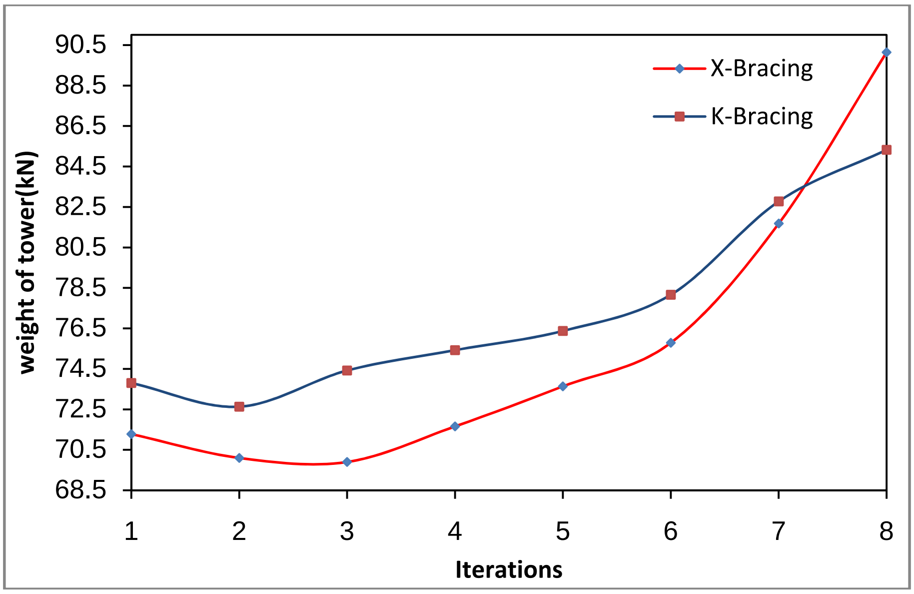

Figure 7: Comparison between X, K-Bracing for 2S2 Tower with Tube Section under Anti-Cascade Loading Condition

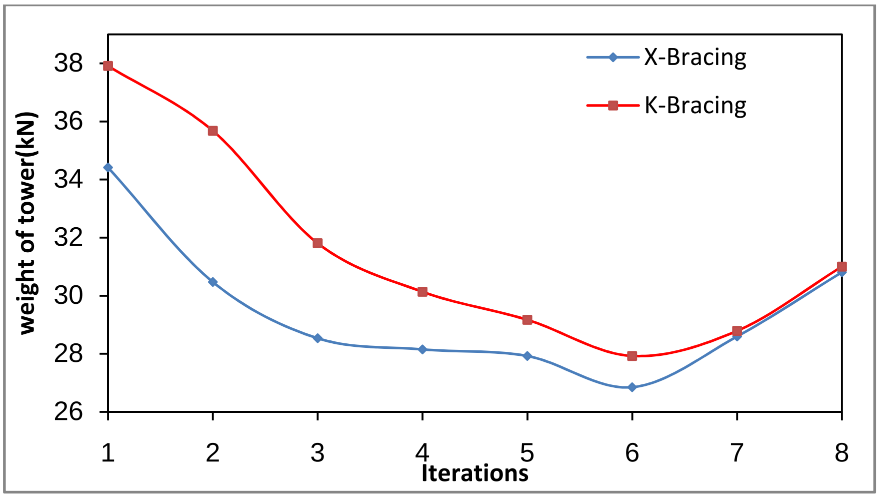

Figure 8*:* Comparison between X and K-Bracing for 1S2 Tower with Angle Section under Seismic Loading Conditions

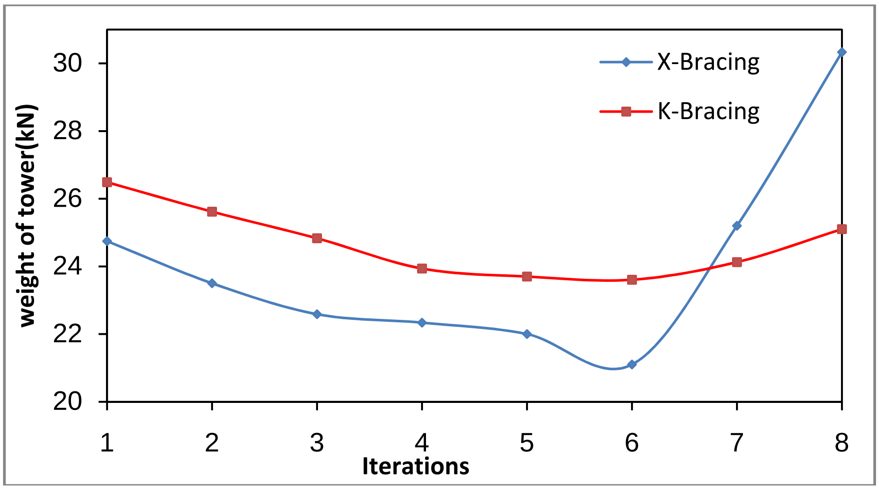

Figure 9*:* Comparison between X and K-Bracing for 1S2 Tower with Pipe Section under Seismic Loading Conditions

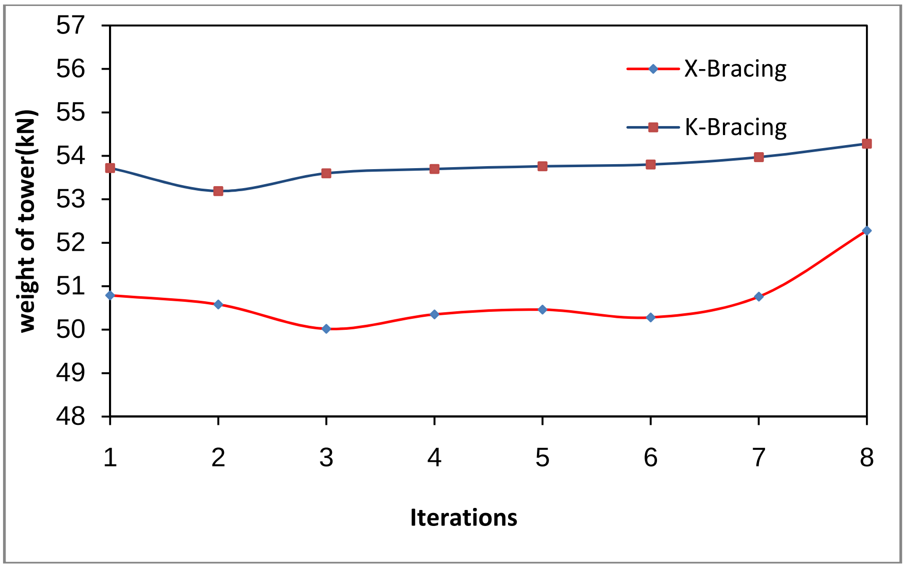

Figure 10*:* Comparison between X and K-Bracing for 1S2 Tower with Tube Section under Seismic Loading Conditions

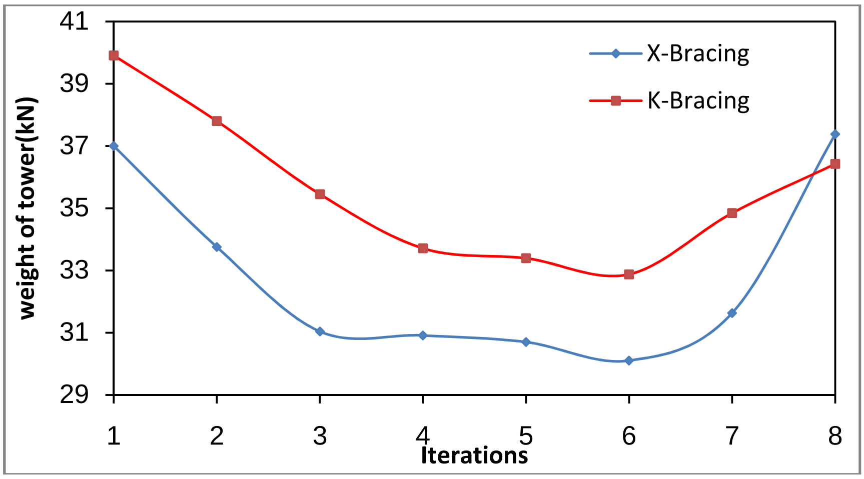

Figure 11: Comparison between X and K-Bracing for 2S2 Tower with Pipe Section under Seismic Loading Conditions

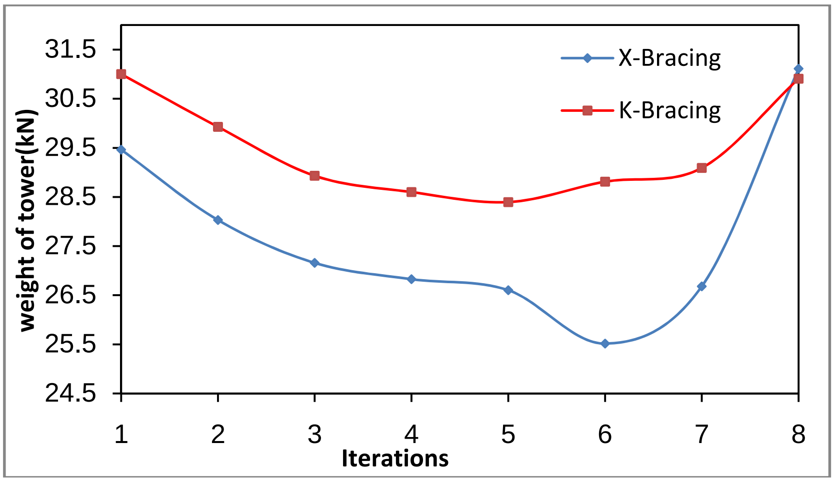

Figure 12: Comparison between X and K-Bracing for 2S2 Tower with Pipe Section under Seismic Loading Conditions

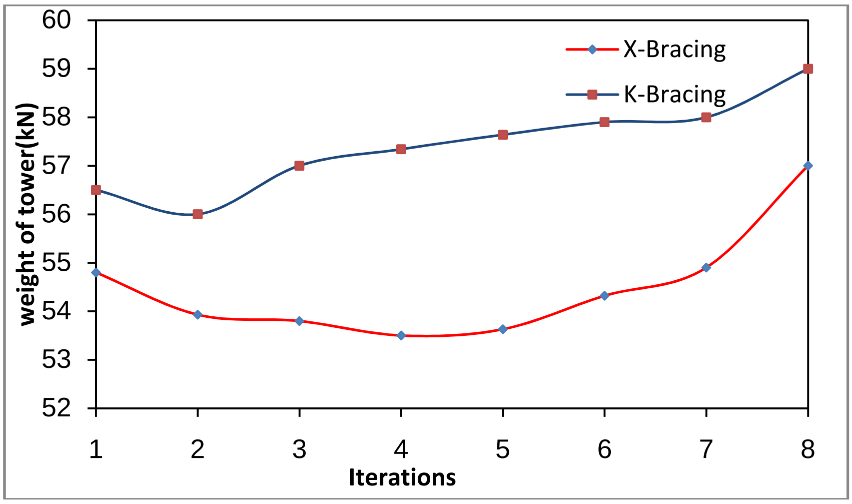

Figure 13: Comparison between X and K-Bracing for 2S2 Tower with Tube Section under Seismic Loading Conditions

Effect of Cross-Sectional Shapes

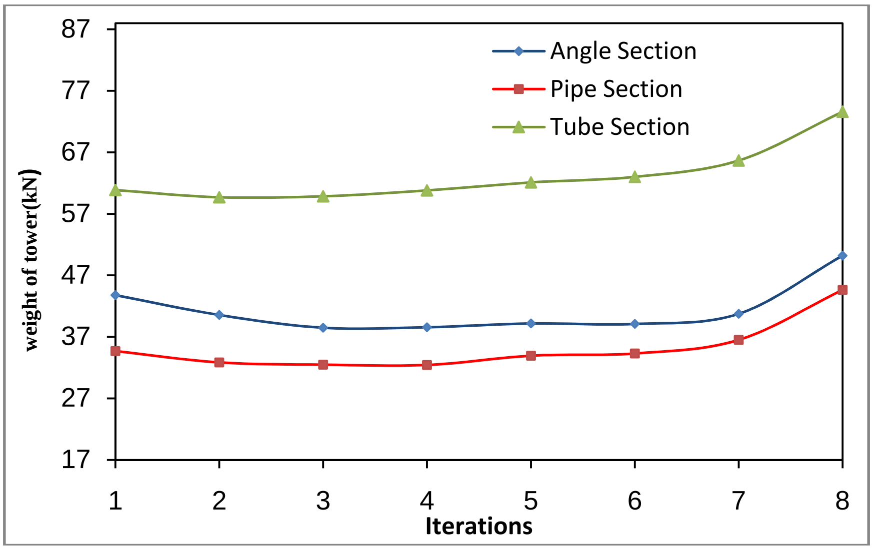

Figure 14: Comparison between Angle, Pipe and Tube Sections with X -Bracing under Anti-Cascade Loading Condition for 1S2 Tower

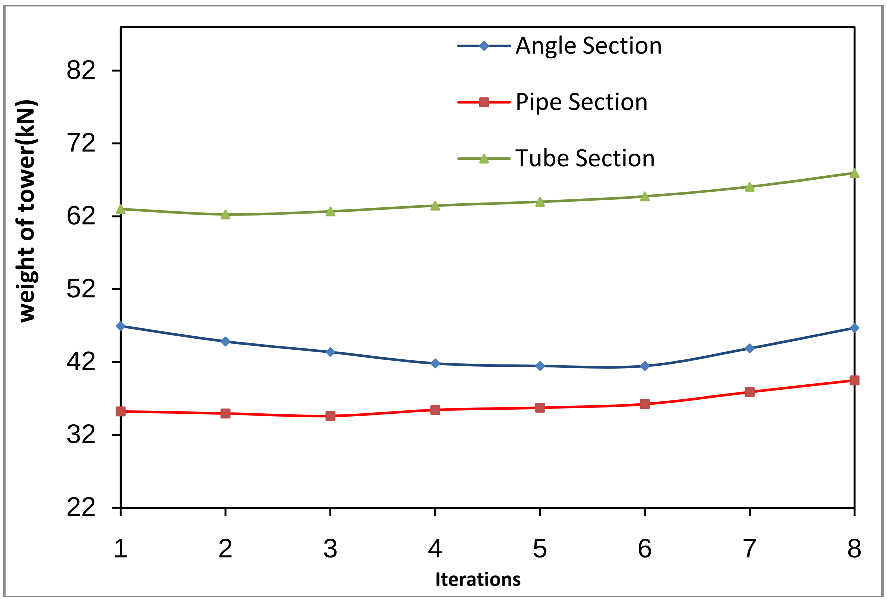

Figure 15: Comparison between Angle, Pipe and Tube Sections with K -Bracing under Anti-Cascade Loading Condition for 1S2 Tower

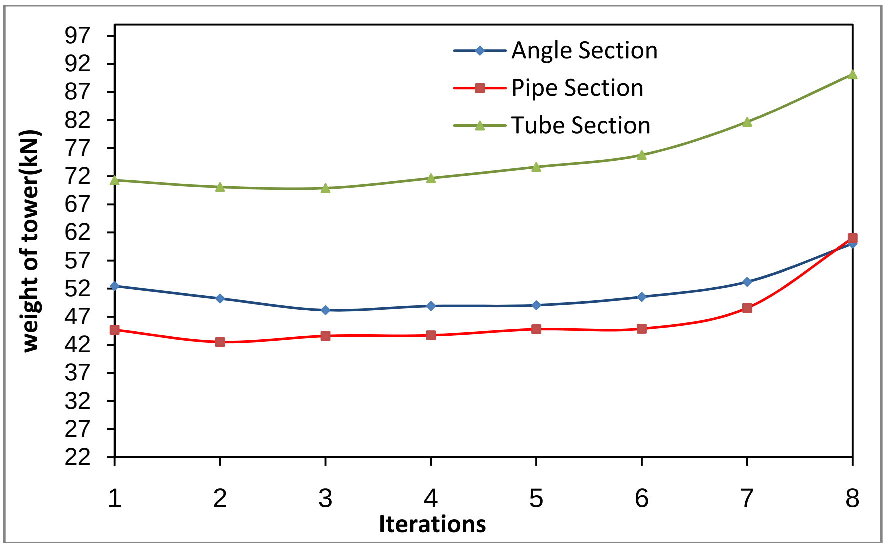

Figure 16: Comparison between Angle, Pipe and Tube Sections with X -Bracing under Anti-Cascade Loading Condition for 2S2 Tower

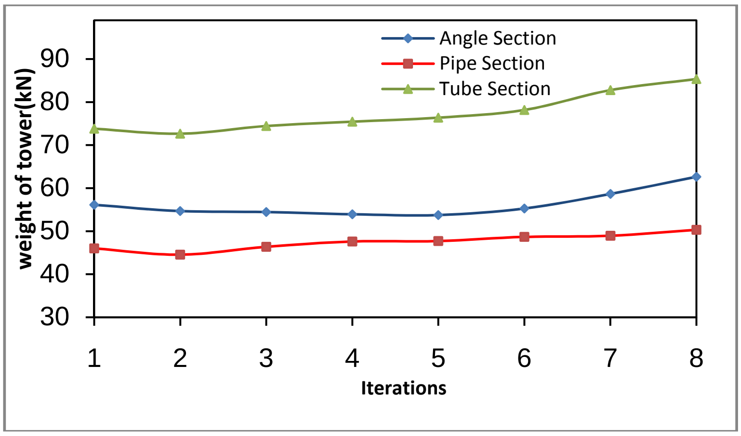

Figure 17: Comparison between Angle, Pipe and Tube Sections with K -Bracing under Anti-Cascade Loading Condition for 2S2 Tower

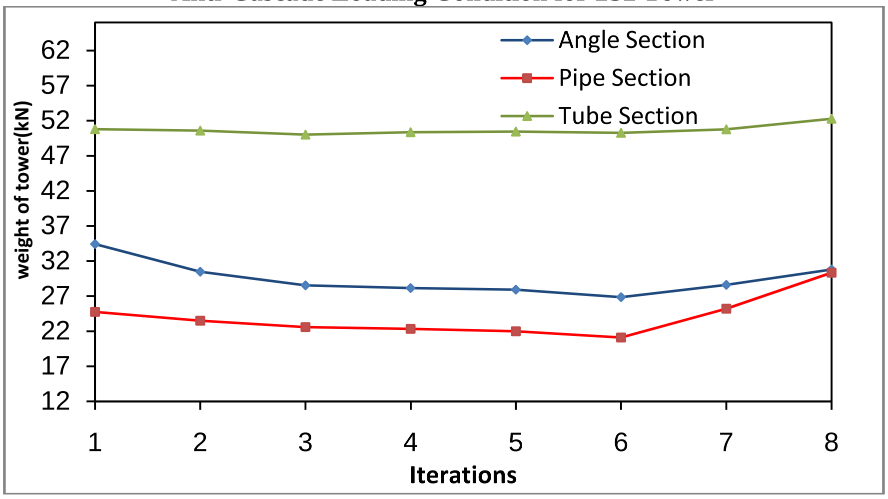

Figure 18: Comparison between Angle, Pipe and Tube Sections with X -Bracing under Seismic Load for 1S2 Tower

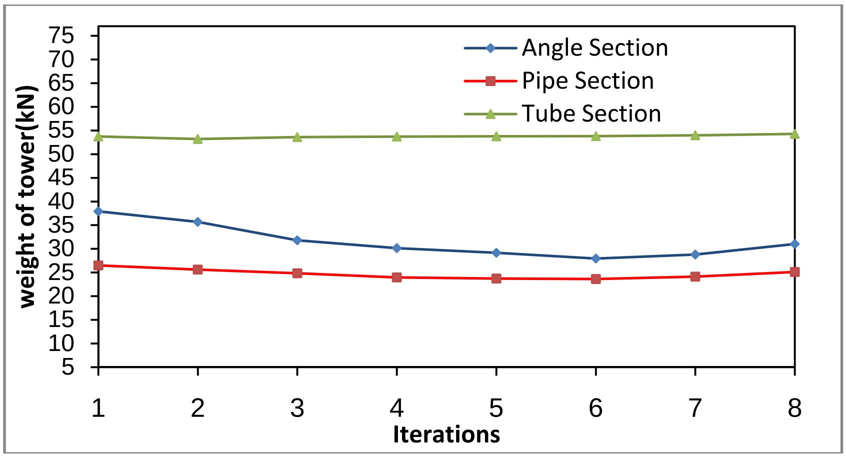

Figure 19: Comparison between Angle, Pipe and Tube Sections with K -Bracing under Seismic Load for 1S2 Tower

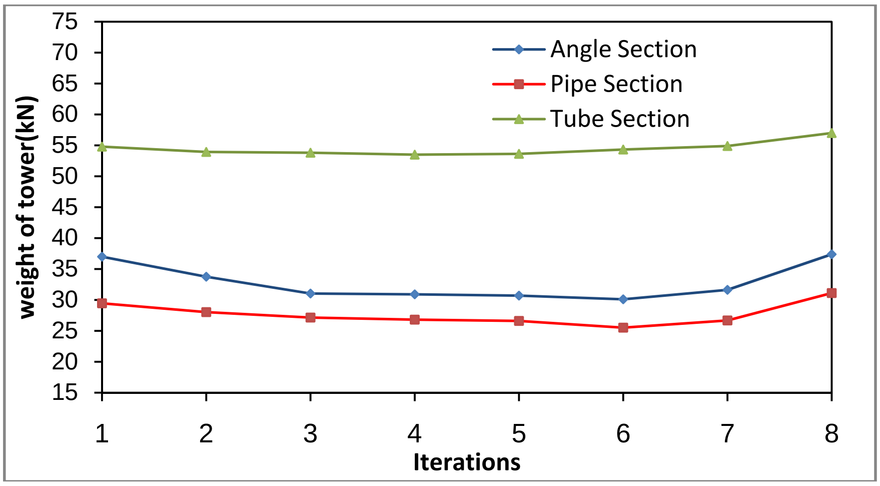

Figure 20: Comparison between Angle, Pipe and Tube Sections with X -Bracing under Seismic Load for 2S2 Tower

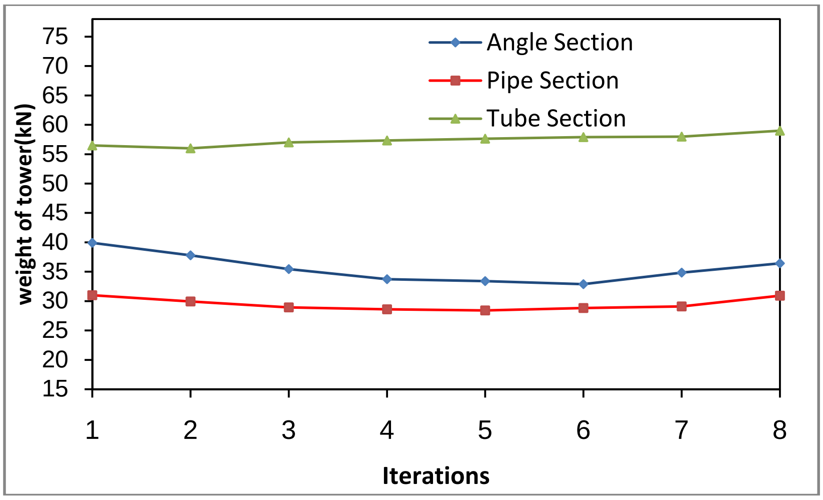

Figure 21: Comparison between Angle, Pipe and Tube Sections with K -Bracing under Seismic Load for 2S2 Tower

Table 28. Weight Reduction for 1S2 Tower under Anti-Cascade Loading

Section Type	Bracing Type	Weight before Optimization	Weight after Optimization	% of Weight Reduction
Angle Section	X-Bracing	44.688	38.477	13.9
	K-Bracing	45.337	41.455	8.5
Pipe Section	X-Bracing	36.31	32.428	10.7
	K-Bracing	37.834	34.601	8.5
Tube Section	X-Bracing	62.154	59.703	4
	K-Bracing	64.119	62.265	2.89

Table 29. Weight Reduction for 2S2 Tower under Anti-Cascade Loading

Section Type	Bracing Type	Weight before Optimization	Weight after Optimization	% of Weight Reduction
Angle Section	X-Bracing	61.045	48.175	21
	K-Bracing	61.087	53.722	12
Pipe Section	X-Bracing	48.924	42.504	13
	K-Bracing	48.908	44.544	9
Tube Section	X-Bracing	71.748	69.9	2.5
	K-Bracing	75.642	72.63	4

Table 30. Weight Reduction for 1S2 Tower under Seismic Loading Condition

Section Type	Bracing Type	Weight before Optimization	Weight after Optimization	% of Weight Reduction
Angle Section	X-Bracing	35.188	26.848	23.7
	K-Bracing	34.046	27.926	18
Pipe Section	X-Bracing	25.344	21.1	16.7
	K-Bracing	25.979	23.605	9.1
Tube Section	X-Bracing	50. 77	50.018	1.5
	K-Bracing	54.113	53.19	1.7

Table 31. Weight Reduction for 2S2 Tower under Seismic Loading Condition

Section Type	Bracing Type	Weight before Optimization	Weight after Optimization	% of Weight Reduction
Angle Section	X-Bracing	38.491	30.106	21.7
	K-Bracing	38.588	32.878	14.8
Pipe Section	X-Bracing	29.067	25.514	12.2
	K-Bracing	30.789	28.395	7.8
Tube Section	X-Bracing	54.127	53.321	1.5
	K-Bracing	56.551	56.075	1

Table 32. Comparison between Optimum Angle and Pipe Sections for 1S2 Tower

Section Type	Bracing Type	Weight of Optimum Tower(kN)		$\frac{\text{Weight (Pipe)}}{\text{Weight (Angle)}}\%$
		Angle Section	Pipe Section	
Cascade Condition	X-Bracing	38.477	32.428	84.27
	K-Bracing	41.455	34.6	83.46
Seismic Load	X-Bracing	26.848	21.1	78.59
	K-Bracing	27.926	23.605	84.52

Table 33. Comparison between Optimum Angle and Pipe Sections for 2S2 Tower

Section Type	Bracing Type	Weight of Optimum Tower (kN)		$\frac{\textbf{Weight (Pipe)}}{\textbf{Weight (Angle)}}\%$
		Angle Section	Pipe Section	
Cascade Condition	X-Bracing	48.175	42.504	88.23
	K-Bracing	53.722	44.544	82.91
Seismic Load	X-Bracing	30.106	25.514	84.74
	K-Bracing	32.878	28.395	86.36

CONCLUSIONS

The following conclusions can be drawn from the present work:

1) The study of different loading conditions on structures is very important to recognize the case that will cause the larger deflection in tower model and exceed the yield stress to decide which case will be optimized.
2) The geometry parameters of the tower can efficiently be treated as design variables, and considerable weight reduction can often be achieved as a result of geometric changes.
3) The tower with angle section and X-bracing has the greater reduction in weight after optimization (reaching 21%).
4) The tower with pipe section and X-bracing has an optimum weight smaller than the other section shapes (about 78% of that of angle section).
5) Tube section is not economic to use in this type of transmission tower.
6) The transmission tower with X-bracing is lighter than that with K-bracing with angle, pipe and tube sections under wind and seismic load conditions.

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