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Comparison of Various Bracing System for Self-Supporting Steel Lattice Structure Towers

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Abstract: This paper deals with the effectiveness of various bracing systems used in lattice towers. Seven types of bracings used in 4-legged square based self-supporting power transmission and telecommunication towers and four types of bracings used in 3-leg triangular based self-supporting telecommunication towers are analyzed. The investigated bracing systems are K, KD, Y, YD, D, XB and X. This study has focused on identifying the economical bracing system for a given range of tower heights. Towers of height 40 to 60 m for telecommunication and 35 m for transmission towers have been analyzed under critical loads such as wind and earthquake loads. The load cases include diagonal wind has been found to be most critical cases for towers. The performance of various bracing system has been identified and reported.

Keywords: Transmission Tower, Telecommunication Tower, Bracing System, Lattice Tower, Nonlinear Analysis, Load Cases, Self-Supporting Tower

1. Introduction

The expansion of the telecommunication systems, as well as the natural requirements for increasing the electrical power transmission systems in the world were the main reasons for the continuous demands for the production of steel transmission and telecommunication towers. Lattice type steel structures have long been largely utilized in World to support cellular and microwave transmission antennas or to enable electrical power transmission lines to be built interconnecting the cities and countries territory [1].

The transmission and telecommunication towers design are not a straightforward process, but an interactive compromise between many factors, which must ultimately satisfy basic strength requirements. The design of transmission and telecommunication towers in this slenderness range is very competitive aiming on lower global costs and higher quality issues [2].

Latticed structures are ideally suited for situations requiring a high load carrying capacity, a low self-weight, an economic use of materials, and fast fabrication and construction. For these reasons self-supporting latticed towers are most commonly used in the field of telecommunication and power line system. Because one latticed tower design may be used for hundreds of towers on a power transmission and communication purposes, it is very important to find an economic and highly efficient design. The arrangement of the tower members should keep the tower geometry simple by using as few members as possible and they should be fully stressed under more than one loading condition. The goal is to produce an economical structure that is well proportioned and attractive [3].

Steel lattice towers are usually fabricated using angles for the main legs and the bracing members. The members are bolted together, either directly or through gusset plates. In order to reduce the unsupported length and thus increase their buckling strength, the main legs and the bracing members are laterally supported at intervals in between their end nodes, using secondary bracings or redundant.

In order to mitigate the extreme loading conditions due to wind load and icing, study on retrofitting of tower structures is of great significance and urgency. Steel angles are commonly used as members in the construction of tower. Due to the asymmetry of member cross sections, the stability of these angle members would be a complex issue [4].

Over the past several decades, considerable studies have been carried out to capture the structural behavior of angle members. Kemp and Behncke [5] performed a series of 13 tests to investigate the property of cross-bracing systems in tower structure. It could be concluded that the end eccentricities caused by bolting one leg of each bracing to the main legs would significantly influence the displacement within the bracing system. The intersection joints of tension and compression bracing system deflected along the out-of-plane direction even at low loads and bending moments were then induced. At global structure level, Albermani and Kitipornchai [6] and Albermani et al. [7] established an analysis model which took the influence of both geometric and material nonlinearities into account. The tower was modeled with beam-column and truss elements. The analysis results were found to agree well the corresponding test results.

Alam and Santhakumar [8] and Moon et al. [9] conducted an experimental study on one towers to examine the failure mode of structures. They found that the buckling of leg and cross-arm bottom members would bring the tower to failure. The test result showed that the main leg buckled under the bending moment caused by eccentric compression and unbalanced deformation.

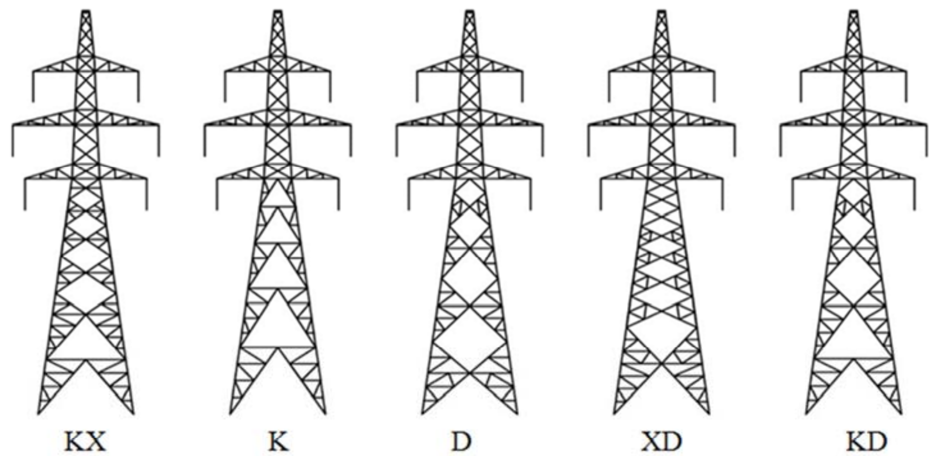
Based on the experimental works, some retrofitting strategies have been proposed by researchers to improve wind-resistant performance of towers. Albermani et al. [10] and Xie et al. [11, 12] considered the feasibility of adding diaphragm to strengthening tower structures. They found that adding diaphragm significantly improved the structural performance and thus increased load-carrying capacity of the structure under strong wind.

The literature review indicate that the best type of communication towers are self-support and mast towers with having large face widths, face width of towers greater than or equal to the diameter of the mounted dishes. On the other hand, the worst towers are self-supporting towers with small face widths causing top of structures yield high twist and sway values. Otherwise, mast towers with small face widths often not built for large dishes may result in large structural failures and may result in large twist and sway values. Monopoles often yield large twist and sway values, expensive to stiffen, often too far out of tolerance to convert for twist and sway to be feasible to fix [13].

The survey study shows that the simplest scheme to increase the compression capacity of the tower leg member is to add additional bracing to the tower to reduce the slenderness ratio of the critical member. The effectiveness of this method is dependent on type of bracing and the slenderness of the original leg member. Additional bracing may also be provided as horizontal diaphragms, which has been shown to be effective. Based on the aforementioned findings, some important design codes have been developed [14, 15].

The aim of this paper is to examine the efficiency and effectiveness of various bracing systems with a view to developing design recommendations for reinforced towers. The square and triangular base self-supporting steel lattice towers are studied. The 132 kV double circuit, twin teal, 33.58 m height power transmission tower and 40, 50 and 60 m height telecommunication towers are examined. Firstly, square based power transmission towers with five types of bracings, square based telecommunication towers with seven types of bracings, and triangular based telecommunication towers with four types of bracings are modeled. The towers are modelled and analyzed based on current design codes TIA 222 and ANSI [16]. All loading condition, load combinations and design constraints (such as allowable stresses in the members and the allowable displacement) are defined based on the requirements of design codes. Then, the linear and nonlinear analyses of towers are carried out using PLS-TOWER [17]. The effect of bracing systems on the structural performance and their physical work mechanism are investigated.

2. Bracing Systems



(a) Power transmission towers

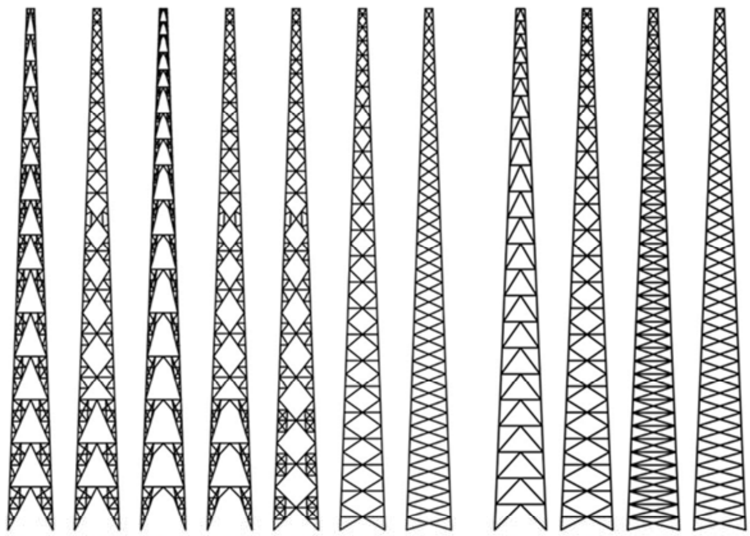


Figure 1. *Towers with different base and bracing systems.*

In the current study, commonly provided with five different types of bracing systems are considered in the power transmission towers such as KX, K, D, XD, and KD as shown in Figure 1(a). These towers are modeled, analyzed and designed accordance to the ASCE 10 code [14]. Seven different types of bracing system consist of K, KD, Y, YD, D, XB and X are considering for rectangular base telecommunication towers with a height of 60, 50 and 40 m. Four different bracing systems consist of K, D, XB and X for triangular base telecommunication towers are also studied. Figure 1(b) illustrates both rectangular and triangular base towers with different bracing patterns. Load, load combination and other design parameters specified in ANSI/TIA-222-G [16] code have been used for analysis and designed of towers.

3. Numerical Analysis

The steel transmission and telecommunication tower design is not a straightforward process, but an interactive compromise between many factors, which must ultimately satisfy basic strength requirements. Generally, in structural analysis, the actual complex structure and loading are modelled numerically, using several simplifying assumptions. On the other hand, the most commonly used tower geometries, when the truss solution is adopted, possess structural mechanisms that compromise the assumed structural behavior. The linear elastic analysis of transmission tower, nonlinear effects at member and system level (geometric) are taken into consideration and the tower is modeled and analyzed using column-beam and truss elements. Thus moments produced by the continuity of members are generally not considered since each leg member is assumed pinned between two joints [18].

In present study, structural analysis based on a less conservative solution, for the steel tower design considering all the actual structural forces and moments. A modelling strategy combining three-dimensional beam and truss finite elements is proposed. In tower models the main members such as legs use beam elements while the bracing system utilizes truss elements [19].

The linear and nonlinear analyses of tower are carried out for obtaining the performance of bracing systems. The TOWER [17] used in this study to evaluate the structural performance of bracing system. The towers have been modelled in 3D using TOWER program [17]. This program capable to carry out linear and nonlinear analysis and also provide a chance for checking design such structures under user specified loads and can also calculate maximum allowable wind and weight spans.

3.1. Design Loads

Calculation of tower loading which is most important part of tower design is the first step towards tower design. Any mistake or error in the load assessment will make the tower design erroneous. Various types of loads are to be calculated accurately depending on the design parameters.

The gravity loads are almost fixed, since these are dependent on the structural design. In the load calculation the wind plays a vital role. The correct assessment of wind will lead to proper load assessment and reliable design of tower structure. Maximum wind pressure is the chief criterion for the design of lattice towers. Simultaneous concurrence of earth quake and maximum wind pressures are unlikely to take place. However, in particular regions where earthquakes are experienced frequently, as in the North regions of Turkey and other parts of the country, seismic load is also critical should be considered in the design of towers in accordance with ANSI/TIA 222-G code [16].

The failure containment loads are taken in account in the power transmission tower design. These are unbalanced longitudinal loads and torsional loads due to broken wires (All towers should have inherent strength for resisting the longitudinal and torsional loads resulting from breakage of specified number of conductors and/or earthwire.) and anti-cascading loads (Failure of items such as insulators, as well as failure of major components such as conductors may result in cascading condition. In order to prevent the cascading failures angle towers is checked for anti - cascading loads for all conductors and earth wires broken in the same span) [20].

3.2. Load Combinations

Differing external loads acting simultaneously on the supports of towers are combined to load cases in an adequate manner. These combinations of actions need to comply with the requirements concerning reliability, security and safety. The load cases should take care of all loading conditions to be expected during construction and during the whole life period of towers such that damage will be unlikely. In many standards for power transmission and telecommunication towers are distinguished between normal and exceptional loads, whereby differing stability requirements or differing permissible stresses apply. The classification of those specific loads may occur to be chosen arbitrarily in a specific case. The load cases according to EN 50341-1 for power transmission and TIA/EIA-222-G [16] for telecommunication are used in present study [21].

4. Design Examples Power Transmission Towers

The geometry of tower which is analyzed and design shown in Figure 2 has the base width of tower 7 × 7 m and the height of tower 33.50 m. Tower consist of eight sections, these are peak of tower, cross arms of tower, cage of transmission tower, common body 0 to 3, and legs of tower. Basic span (distance between two towers) is 305 m, the minimum horizontal and vertical phase to phase is 7.50 m, and 3.85 m respectively, minimum clearance from conductor to ground 7.50 m, cross arm lengths are 7.62 m top, 10.52 m middle and 8.82 m bottom, as shown in Figure 2. The support of tower is rigidly fixed to the foundation of tower.

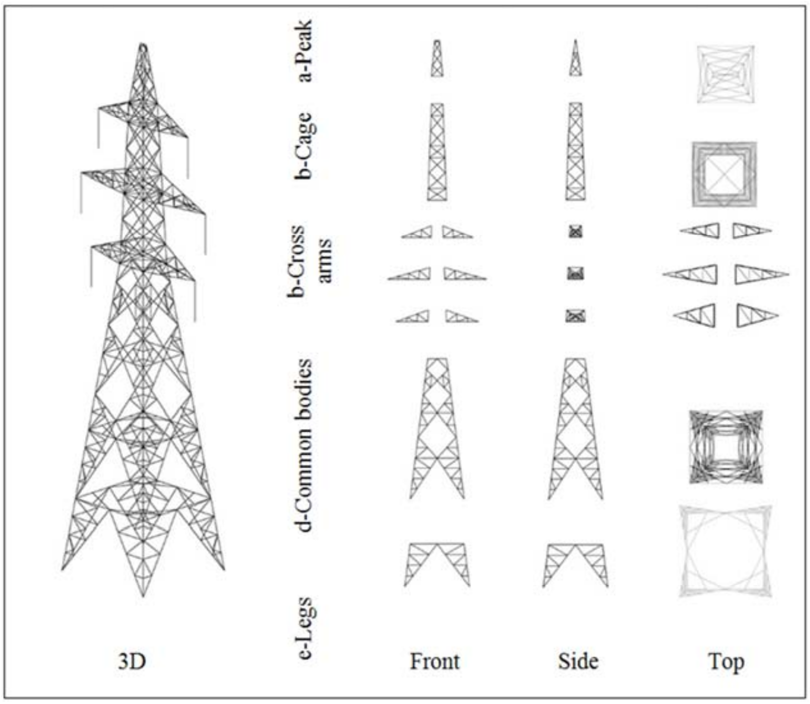


Figure 2. *Geometry of 132 kV lattice tower.*

The linear and nonlinear simulation are carried out and the analysis results evident that the tower 132 kV double circuit, compared the output data between the linear and nonlinear simulation, it was found that the tower members less prone to the effects during the comparison between them. The difference percentage of maximum element usage between linear and nonlinear analysis type are 0.14 %, 0.46 %, 0.09 %, 0.09 %, and 0.03 %, for KX, K, D, XD, KD, respectively.

The one of basic parameters in the design of tower is weight of tower. The weight of the towers with various bracing system is given in Table 1. The weight of towers designed based on linear and nonlinear analyses are the same. From output data obtained, the heaviest bracing system is appearing that KX type of bracing system is achieved with a value of 78004.80 N, while the lightest tower is achieved in the D type bracing system with a value of 68272.90 N.

Table 1. *Summary of linear and nonlinear analysis results for transmission towers.*

Types	Weight (N)	%Max. Usage Linear Anal.	%Max. Usage Nonlinear Anal.	Critical Member
KX	78004.80	93.36	93.23	Top cross-arm bottom
K	72405.00	99.08	99.54	Middle cross-arm bottom
D	68272.90	99.17	99.08	Bracing
XD	71431.00	93.36	93.27	Top cross-arm bottom
KD	68680.4	99.18	99.15	Bracing

Maximum usage of all members considering the individual members are mainly loaded by axial compression and tension forces, considering all load cases. The member forces are calculated based on three-dimensional linear and nonlinear analysis. The maximum elements usage is listed in Table 1. In both linear and nonlinear analyses, the critical loading for KX, XD structure type, failure containment loads case (security broken conductor wire at top left cross arm: transverse full wind at +15°C) is critical, for K, D, and KD the failure containment load case (security broken conductor wire at middle left cross arm: transverse full wind at +15°C) is critical. The critical members which is given in Table 1, are the same in linear and nonlinear analyses. The all critical members are in compression.

Table 2 shows the results of maximum overturning moments which occur at reliability normal condition loading case (transverse full wind at +15°C, maximum weight span) for linear and nonlinear analyses. In Table 2, the maximum resultant moment occurred at the bracing type KX, which was 7990.115 kN.m and 8035.038 kN.m for linear, and nonlinear analysis respectively.

Table 2. *Overturning moment results for linear and nonlinear analysis.*

Types	Linear			Nonlinear		
	Transverse Moment (kN.m)	Longitudinal Moment (kN.m)	Resultant Moment (kN.m)	Transverse Moment (kN.m)	Longitudinal Moment (kN.m)	Resultant Moment (kN.m)
KX	7990.1	0	7990.1	8035.0	- 0.001	8035.0
K	7968.3	0	7968.3	8010.3	- 0.001	8010.3
D	7937.5	0.1	7937.5	7983.4	0.087	7983.4
XD	7977.2	0.0	7977.2	8018.7	- 0.001	8018.7
KD	7945.6	0.1	7945.6	7990.6	0.075	7990.6

5. Telecommunication Towers

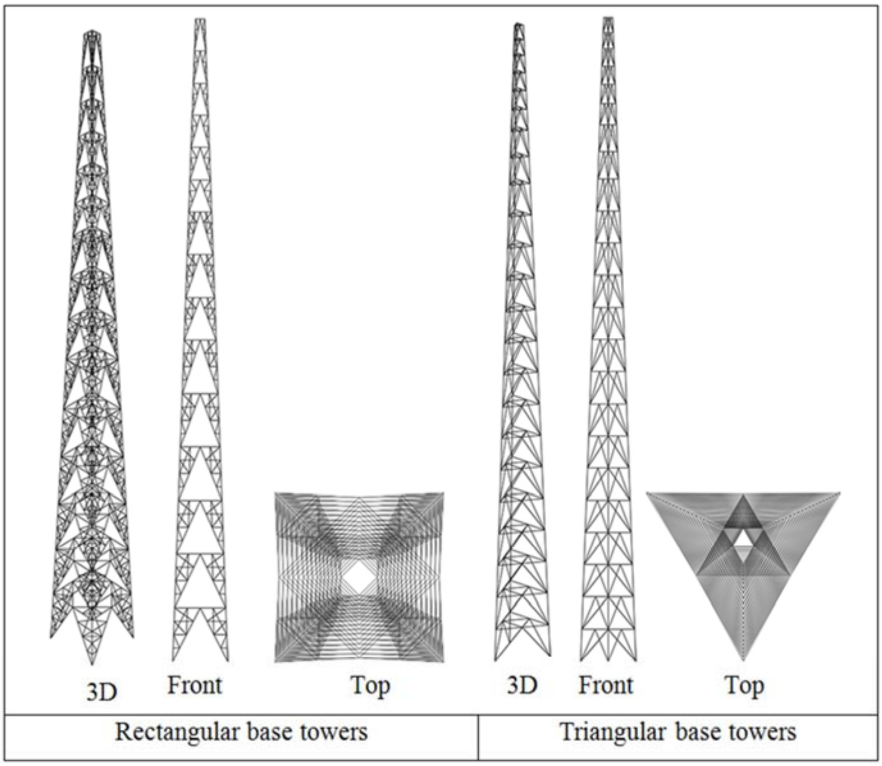


Figure 3. *Geometry of telecommunication tower.*

The steel communication tower is designed for heights of 40 m, 50 m and 60 m. The base width of 60 m and 50 m height rectangular base towers is 5.14 ×5.14 m. The base width of 40 m height rectangular base towers is 4.15 × 4.15 m. Furthermore, triangular base towers is equal triangular and each side of base triangle is 5.94 m and 4.26 m for 60 - 50 m height and 40 m height towers respectively. The top width of all type of rectangular and triangular base towers is 0.85m. Towers are supported by reinforced concrete foundation using fix type of supporting. Figure 3 shows 3D, front and top views of rectangular and triangular base towers.

5.1. Results and Discussion for Rectangular Based Towers

The summary of the linear and nonlinear analyses results are presented in Table 3. In the present study, linear and nonlinear simulation is used, compared the output data between the linear and nonlinear simulation, it was detected that the tower members less prone to the effects during the comparison between them. The difference percentage of maximum element usage between linear and nonlinear analysis type are 0.43, 1.07, 0.06, 0.33, 0.41, 0.69, NG, 0.17, 0.165, 0.22, 0.0, 0.73, 0.35, 0.3, 1.16, 0.49, 0.36, 0.32, 1. 69, 0.68 and 0.58 percentage for K, KD, Y, YD, D, XB and X, respectively, for 60 m, 50 m and 40 m heights.

Maximum usage of all members considering the individual members are mainly loaded by axial compression and tension forces, considering all load cases. The member forces are calculated based on three-dimensional linear and nonlinear analysis. The maximum elements usage is listed in Table 3. In both linear and nonlinear analyses, the critical loading is obtained for X and XB structure type for 60,50 and 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°) is critical, for XB and YD the failure containment load case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°) is critical. The critical members which is given in Table 3, are the same in linear and nonlinear analyses. The all critical members are in compression.

The weight of towers designed based on linear and nonlinear analyses are the same. According to Table 3, the minimum weight of tower for 60 m, 50 m and 40 m heights obtained for the KD and YD bracing system is 60868.1, 46352.5 and 30876 N, respectively, On the other hand, the heaviest tower for 60 m, 50 m and 40 m heights obtained for the XB and Y bracing system is 145852.6, 131852.3 and 43606.4 N as present in Table 3.

Results of structure fundamental frequency of all towers are nears to each other's. The best achievement fundamental frequency for 60m, 50 m and 40 m heights is concerned for KD, Y and D bracing system, respectively, is 1.1339, 1.6064 and 2.0626 Hz. The worst achievement is for 60m, 50 m and 40 m heights is concerned for K, D and KD is 1.2262, 1.7847 and 2.2804 Hz, respectively.

In all bracing systems, the critical combination of load for maximum and minimum value of out of plumb and sway values for both linear and nonlinear analysis is obtained for LC2 which is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°) expect K bracing system in the height of 50 m which is LC1 is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°). As described in Table 3, the minimum value of out of plumb obtained for X and XB bracings is 51.12cm, 28.34 cm and 23.52 cm was obtained for 60 m, 50 m, and 40 m height. The maximum value of out of plumb for 60 m, 50 m, and 40 m heights is 67.37, 41.87 and 29.41 cm, respectively, obtained for KD, K and Y bracings. Difference between minimum and maximum values of linear and nonlinear analysis for 60, 50 and 40 m heights is 0.33, 0.09 – 0.09 and 0.08 cm, respectively, for X, X – XB and XB and 0.48, 0.39 and 0.34 cm for KD, K and Y bracings, respectively.

The best performance for linear analysis of sway values in degree is obtained for X and XB bracing for 60 m, 50 m, and 40 m heights are 0.84, 0.52 and 0.56 degrees. The worst performance is return to KD and D bracing with value 1.2 degrees for 60 m and for 50 m height is obtained for Y bracing system is 1.09 degree. YD and D bracings have a value of 0.73 degrees for 40 m height. The maximum difference between linear and nonlinear analysis results is 0.01 degree for all type of bracings.

Table 3. *Linear and nonlinear analysis results of rectangular base towers.*

Type	H (m)	SFF (Hz)	Weight (N)	Linear Analysis				Non-linear Analysis			
				% MEU	El. Type	OOP (cm)	Sway (deg)	% MEU	El. Type	OOP (cm)	Sway (deg)
	60	1.2262	75032.6	97.58	Leg	64.6	1.17	98.01	Leg	65.26	1.18
K	50	1.7565	56188.8	93.93	Brac.	41.87	0.93	95	Leg	42.26	0.94
	40	2.2696	39495.9	94.71	Leg	28.64	0.71	94.65	Leg	28.88	0.71
	60	1.1339	60868.1	99.01	Brac.	67.37	1.2	99.34	Leg	67.85	1.21
KD	50	1.784	46352.5	96.08	Brac.	38.82	0.82	95.67	Brac.	39.02	0.82
	40	2.2804	31003.7	96.19	Brac.	27.35	0.69	95.5	Brac.	27.47	0.7
	60	1.1974	90008.9	97.12	Brac.	65.83	1.16	N. G.	N. G.	N. G.	N. G.
Y	50	1.6064	59951.7	97.15	Leg	41.73	1.09	96.98	Leg	42.17	1.11
	40	2.0876	43606.4	95.58	Leg	29.41	0.73	95.42	Leg	29.75	0.74
	60	1.1349	62683.1	97.4	Brac.	64.81	1.17	97.62	Leg	65.3	1.18
YD	50	1.7833	47033.9	98.9	Leg	38.87	0.8	98.9	Leg	39.1	0.81
	40	2.2802	30876	98.06	Brac.	26.71	0.68	97.33	Brac.	26.83	0.68
	60	1.1359	63642.3	98.79	Brac.	66.4	1.2	98.44	Brac.	66.83	1.2
D	50	1.7847	47585.4	97.05	Leg	40.05	0.83	97.35	Leg	40.23	0.84
	40	2.0626	32702.9	94.22	Leg	28.25	0.73	95.38	Leg	28.39	0.74

Type	H (m)	SFF (Hz)	Weight (N)	Linear Analysis				Non-linear Analysis			
				% MEU	El. Type	OOP (cm)	Sway (deg)	% MEU	El. Type	OOP (cm)	Sway (deg)
	60	1.2214	145853	99.06	Leg	53	0.98	99.55	Leg	53.29	0.99
XB	50	1.756	131852	93.86	Leg	28.34	0.55	94.22	Leg	28.43	0.55
	40	2.2693	33057.7	93.42	Leg	23.52	0.56	93.74	Leg	23.6	0.56
	60	1.1503	122436	93.6	Leg	51.12	0.84	91.91	Leg	51.45	0.84
X	50	1.7441	109958	89.26	Leg	28.34	0.52	88.58	Leg	28.43	0.52
	40	2.2448	38507.9	96.74	Leg	24.79	0.61	97.32	Leg	24.87	0.62

In all bracing systems, the critical load combination for the maximum over turning moments is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°). for linear and nonlinear analyses. In the Table 4, the maximum resultant moment is 5401.62 kN.m, 3565.06 kN.m and 1507.96 kN.m occurred at the bracings Y, XB and YD for 60 m, 50 m, and 40 m heights, respectively, for linear analysis. The minimum resultant for linear analysis of the same towers is 2721.10 kN.m, 2555.72 kN.m and 1507.96 kN.m occurred, respectively, at the bracings KD, D and YD.

Maximum non linearity analysis result returns to the same mentioned bracings. Failure was occurred in Y bracing system for 60 m height by running it in linear analysis and there is 10.42 kN.m and 3.33 kN.m difference between linear and nonlinear analysis results for each 50 m and 40 m height of tower.

Table 4. *Overturning moment results for linear and nonlinear analysis.*

Type	H (m)	Linear Analysis			Non-linear Analysis		
		Trans. Moment (kN.m)	Longitud. Moment (kN.m)	Resul. Moment (kN.m)	Trans. Moment (kN.m)	Longitud. Moment (kN.m)	Resul. Moment (kN.m)
	60	3306.65	3306.67	4676.32	3321.92	3321.94	4697.92
K	50	2275.46	2275.48	3218.00	2283.07	2283.08	3228.75
	40	1370.87	1370.88	1938.71	1374.79	1374.79	1944.24
	60	2721.08	2721.10	3848.20	2731.77	2731.79	3863.32
KD	50	1807.65	1807.66	2556.42	1812.28	1812.28	2562.95
	40	1068.59	1068.60	1511.22	1070.93	1070.94	1514.53
	60	3819.49	3819.56	5401.62	N. G.	N. G.	N. G.
Y	50	2389.88	2389.92	3379.83	2399.39	2399.42	3393.27
	40	1511.26	1511.28	2137.25	1516.58	1516.59	2144.78
	60	2722.85	2722.91	3850.74	2733.28	2733.33	3865.48
YD	50	1831.44	1831.46	2590.06	1836.27	1836.29	2596.89
	40	1066.28	1066.30	1507.96	1068.63	1068.65	1511.29
	60	2740.12	2740.11	3875.11	2750.54	2750.54	3889.85
D	50	1807.16	1807.16	2555.72	1811.91	1811.91	2562.42
	40	1117.82	1117.82	1580.83	1120.50	1120.50	1584.62
	60	3440.89	3440.89	4866.16	3454.00	3454.00	4884.70
XB	50	2520.88	2520.88	3565.06	2528.25	2528.25	3575.48
	40	1137.95	1137.95	1609.30	1140.34	1140.34	1612.69
	60	3241.66	3241.66	4584.40	3251.69	3251.69	4598.59
X	50	2399.53	2399.53	3393.45	2405.03	2405.03	3401.23
	40	1300.57	1300.57	1839.29	1303.23	1303.23	1843.05

5.2. Results and Discussion for Triangular Based Towers

The summary of the linear and nonlinear analyses results are presented in Table 5. The difference percentage of maximum element usage between linear and nonlinear analysis type are for 0.64%, 0.47%, 0.3%, 0.71%, 1.7%, 0.42%, 4.05%, 0.44%, 4.38%, 0.13%, 0.22% and 0.35%, for K, D, XB and X respectively, for 60 m, 50 m and 40 m heights.

The maximum elements usage is listed in Table 5. In both linear and nonlinear analyses, the critical loading is obtained for D, D and XB structure type for 60, 50 and 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of -90°) is critical for 60 m height D bracing system. For 50 m height D bracing system, Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 90°) is critical and for 40 m height Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) for XB bracing system. X and K the failure containment load case for 60 and 50 – 40 m height. Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 90°) is critical for 60 m height X bracing system and Failure containment loads case 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) are critical for both 50 and 40 m height of K bracing system. The critical members which is given in Table 5, are the same in linear and nonlinear analyses. The all critical members are in compression.

The weight of towers designed based on linear and nonlinear analyses are the same. According to Table 5, the minimum weight of tower for 60 m and 40 m heights are obtained for X bracings are 73716.2 N and 34669 N, respectively, and for 50 m height is obtained for XB bracing system is 56489.1 N. On the other hand, the heaviest tower for 60 m, 50 m and 40 m heights obtained for the K bracing system is 242048.3 N, 207310.9 N and 78979.2 N as present in Table 5.

Table 5 also existing on the results of structure fundamental frequency of all towers and it is near to each other's. The best achievement fundamental frequency for 60m, 50 m and 40 m heights is concerned for X and K bracing system, respectively, was 1.1255 Hz, 1.6582 Hz and 1.8397 Hz. The worst achievement for 60m, 50 m and 40 m heights is concerned for D, XB and X is 1.1786 Hz, 1.7006 Hz and 2.0448 Hz, respectively.

In all bracing systems, the critical combination of load for best and worst values of out of plumb and sway for linear and nonlinear analysis is obtained for LC1 which is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 0°) expect X bracing system in the height of 60 m which is LC2 is 1.2D (dead load) + 1.6Wo (wind load applied to tower with an angle of 45°). As described in Table 5, the best performance value of out of plumb is 18.22 cm, 10.51 cm and 10.66 cm obtains for 60 m, 50 and 40 m height, all attained for K bracing system. The worst value of out of plumb for 60 m and 40 m is 35.45 cm and 18.68 cm is return to X bracing system. For 50 m heights is 19.79 cm obtained for XB bracing. Difference between linear and nonlinear analysis results is 0.18 cm, 0.08 cm, 0.00 cm, 0.27 cm, 0.07 cm, 0.08 cm, 0.09 cm, 0.07 cm, 0.25 cm and 0.08 cm, respectively for K, D, XB and X bracings for 60 m, 50 m and 40 m heights.

The best performance for linear analysis of sway values in degree is all obtained for K bracing for 60 m, 50 m and 40 m heights is 0.31, 0.2 and 0.24 deg. The worst performance is return to X bracing with value 0.52 and 0.31 degree for 60 m and 40 m heights. XB bracing has a value of 0.4 degree for 50 m height. 0.01 degree is the maximum different value between linear and nonlinear analysis for K, D, XB and X bracing.

Table 5. *Linear and nonlinear analysis results of triangular base towers.*

Type	H (m)	SFF (Hz)	Weight (N)	Linear Analysis				Non-linear Analysis			
				MEU %	El. Type	OOP (cm)	Sway (deg)	MEU %	El. Type	OOP (cm)	Sway (deg)
K	60	1.1785	242048.3	95.99	Leg	18.22	0.31	96.63	Leg	18.40	0.32
	50	1.6582	207310.9	96.37	Leg	10.51	0.20	96.84	Leg	10.59	0.20
	40	1.8397	78979.2	96.27	Leg	10.66	0.24	96.57	Leg	10.66	0.25
D	60	1.1786	97460.8	85.85	Brac.	31.39	0.49	85.14	Brac.	31.66	0.49
	50	1.6947	59835.2	87.69	Brac.	14.15	0.28	85.99	Brac.	14.22	0.28
	40	1.9285	41423.7	79.21	Leg	15.80	0.40	79.63	Leg	15.88	0.40
XB	60	1.1391	82423.8	94.13	Brac.	28.05	0.47	90.08	Brac.	28.25	0.47
	50	1.7006	56489.1	82.43	Brac.	19.79	0.40	82.87	Brac.	19.88	0.41
	40	1.9123	44851.6	77.32	Leg	14.73	0.35	72.94	Leg	14.80	0.35
X	60	1.1255	73716.2	97.64	Brac.	35.45	0.52	97.77	Brac.	35.70	0.53
	50	1.6780	61513.5	83.00	Brac.	18.01	0.31	82.78	Brac.	18.09	0.31
	40	2.0448	34669.0	94.43	Leg	18.68	0.42	94.78	Leg	18.75	0.42

6. Conclusions

The result of the linear and nonlinear analysis show that bracing systems used in tower show different structural behavior. Such as, if the height of the tower is less than 50 m the difference between linear and nonlinear analysis negligible. The best structural performance and smallest weight is obtained at D type bracing systems for power transmission towers. The critical member in KD and D type bracing system is brace element where as in all other bracing system one of the main members became critical

For telecommunication towers, the smallest weight is obtained at KD and YD type bracing systems for rectangular base towers. In case of triangular base towers X and XB type bracing systems give the smallest weight design. The best performance according to sway and out of plumb values are obtained at X and XB type bracing system for rectangular triangular base towers. It is worth to mention that Y type bracing system is failed in nonlinear analysis at 60 m height tower.

References

[1] Oliveira, M., Silva, J., Vellasco, P., Andrade, S., & Lima, L. (2007). Structural analysis of guyed steel telecommunication towers for radio antennas. *Journal Of The Brazilian Society Of Mechanical Sciences And Engineering*, 29, 185-195.

[2] Monopole Towers Manufacturer - Monopoles Tower Manufacturer from Delhi. 2016. Available: http://www.galvanisedstore.com/monopole-towers manufacturer.html. Accessed 28.12.2016.

[3] Kroeker, D. (2000). Structural analysis of transmission towers with connection slip modeling. Ph.D. University of Manitoba.

[4] Siddesha, H. (2010). Wind Analysis of Microwave Antenna Towers. *International Journal of Applied Engineering Research*, 1, p.574-584.

[5] Kemp, A. R., & Behncke, R. H. (1998). Behavior of Cross-Bracing in Latticed Towers, *Journal of Structural Engineering*, 124, 360-367.

[6] Albermani, F., Mahendran, M., & Kitipornchai, S. (2004). Upgrading of transmission towers using a diaphragm bracing system. *Engineering Structures*, 26, 735-744.

[7] Albermani, F., S. Kitipornchai, and R. W. K. Chan. (2009). Failure analysis of transmission towers. *Engineering Failure Analysis*, 16, 1922-1928.

[8] Alam, M. J., & Santhakumar, A. R. (1996). Reliability analysis and full-scale testing of transmission tower. Journal of Structural Engineering, 122 (3), 338-344.

[9] Moon, B. W., Park, J. H., Lee, S. K., Kim, J., Kim, T., & Min, K. W. (2009). Performance evaluation of a transmission tower by substructure test. *Journal of Constructional Steel Research,* 65, 1-11.

[10] Albermani, F., Mahendran, M., & Kitipornchai, S. (2004). Upgrading of transmission towers using a diaphragm bracing system. Engineering Structures, 26 (6), 735-744.

[11] Xie, Q., Yan, Q., & Li, J. (2006). Effects of Diaphragm on Wind Resistant Design of Power Transmission Towers [J]. High Voltage Engineering, 4, 000.

[12] Qiang, X., Zhao-dong, D. I. N. G., Gui-feng, Z. H. A. O., & Jie, L. (2009). Wind resistant analysis of power transmission tower with different diaphragm arrangements. High Voltage Engineering, 35 (3), 683-687.

[13] American tower. Collocation Considerations & Best Practices for Financial Services Organizations. A structural engineering perspective. Available at: http://www.americantower.com/Assets/uploads/files/PDFs/American_Tower_Financial_Services_Collocation_Best_Practices.pdf. Accessed 2016.

[14] ASCE. Design of latticed steel transmission structures. Virginia: ASCE 10–97; 2000.

[15] European Committee for Standardization. Eurocode 3 design of steel structures part 3–1: towers, masts and chimneys—towers and masts; 2005. Brussels.

[16] TIA/EIA-222-G, (Draft). Structural Standards for Antenna Supporting Structures and Antennas.

[17] TOWER (2013). Power Line System, Inc ©.

[18] Silva, J. D., Vellasco, P. D. S., Andrade, S. D., & Oliveira, M. D. (2002). An evaluation of structural steel design systems for transmission and telecommunication towers. In *Proceedings of International IASS Symposium «Lightweight Structures in Civil E.*

[19] Da Silva, J. G. S., Vellasco, P. D. S., De Andrade, S. A. L., & De Oliveira, M. I. R. (2005). Structural assessment of current steel design models for transmission and telecommunication towers. Journal of Constructional Steel Research, 61, 1108-1134.

[20] Varma, C. & Lal, P. (1998). Transmission line manual. 1st edition. New Delhi: Central Board of Irrigation and Power.

[21] Nefzger, P., Kaintzyk, U. and Nolasco, J. F. (2003). Overhead power lines: planning, design, construction. 1st edition. Berlin: Springer.