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Structural Capacity and Failure Mechanisms of Transmission Towers under High Intensity Wind Loading

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Abstract: *In January 2010, 81 towers had failed at Aswan, Egypt as a result of high intensity wind (HIW). The wind speed reached a value of 200 Km/h., and the pressure induced from this wind was 193 Kg/m2, which is strongly higher than the design values. The economic drawbacks from the interruption of the electrical network are huge. Thus, the study of the modes of failure, the causes of failure of these towers is essential. The lessons drawn from this disaster leads to the necessity to evaluate the design wind speed and the requirement of mitigate the (HIW) effects.*

This paper examines the feasibility of employing Nonlinear Pushover Analysis, to capture the failure mode and the determination of the full capacity of transmission towers under the effect of the wind loads, consequently predict the structures response under HIW.

The adopted techniques facilitate the tracing of the plastic-hinges formulation using the Pushover technique as well as the prediction of actual load factor at failure under the effect of wind loads.

Keywords: *Transmission Tower (T.T) Failure, High Intensity Wind (H.I.W), nonlinear static Pushover (NSP).*

I. Introduction

Transmission towers play a very important role in power distribution networks and are often subject to massive wind loads. Design of lattice tower often based on a linear elastic response to wind loading by methodology derived for atmospheric boundary layer winds. Many failures referred to (HIW) events such as downbursts and tornadoes. The force deformation relation between the base shear and the displacement of the tip of the tower used to show the capacity curve of a structure under high intensity wind loading.

Capacity estimates obtained using nonlinear static pushover (NSP) procedures indicate good agreement with failure mods subjected to winds. The analyses consider both material and geometric nonlinearity. The NSP analysis used to estimate the capacity of the tower under various wind profiles for the transverse wind direction

Specifications for wind loads on transmission structures provided in the design codes as well as recommended in ASCE Manual No. 74: issued in 2010 " Guidelines for Electrical Transmission Line Structural Loading", (ASCE-74) [1]. Many transmission and distribution companies and authorities also have proprietary load and resistance criteria as in the Egyptian Ministry of Electricity Code [2]. These codes and guidelines assume a linear elastic response under wind loading and do not discuss the inelastic behavior of transmission structures. As a result, nonlinear inelastic analysis of transmission towers not carried out frequently in design practice, but becomes necessary for the assessment of ultimate behavior and structural reliability of the tower under wind load.

Sudhan Banik, Hanping Hong, and Gregory A. Kopp [3] studied assessment of structural capacity of an overhead power Transmission Towers (T.T) under Wind Loading using (NSP) and incremental dynamic analysis (IDA). Two nonlinear hinges assigned at the ends of each member of the modeled tower to confine the interaction of axial and flexural stresses. Wind loads from longitudinal and transverse directions considered in the analysis. It observed that the capacity curves obtained using (NSP) and (IDA) procedures showed a bilinear load-deformation relationship.

Strength Assessment of Both telecommunication tower and T.T Steel Towers was studied by (Baskaran, et al) [4] to identify the reasons for failure of towers and proposing methods to evaluate the tower strength capacities. (Shakeel Ahmad et al) [5] performed response of Transmission Tower subjected to tornado loads. A 35m high transmission tower under tornado loads analyzed and the results showed that the response of the T.T was enormously high due to the tornado wind loads. The study of dynamic response indicates that the section of tower at 27 m height affected if the tower meets the resonance conditions in modes greater than sixth mode. Wei Zhang1*, et. al. [6] studied the Probabilistic capacity assessment of lattice transmission towers under strong wind. The lattice transmission towers built with L-shape steel members and truss, beam elements or their combinations used for modeling the structure. The material and geometric non-linearity were included by using bilinear elasto-plastic material properties and by implementing large deformation analysis, respectively. To efficiently the proposed probabilistic capacity assessment methodology, the finite element model for a 550- kV-

68.6 m height of single circuit transmission tower built primary for the capacity analysis of the structure. The modal analysis performed to find the mode shapes and mode frequencies. The study had demonstrated an effective probabilistic capacity assessment approach for transmission towers considering stochastic wind loadings. Aboshosha, H. and El Domatty, A. [7] investigate the progressive failure of two types of transmission lines, namely self-supported and guyed towers under the effect of downbursts. The downburst field based on previously computational fluid dynamics analysis. The outcomes of the study are the determination of the different modes of failure of the two systems of towers affected by the downburst loading. N.Prasd Rao et al. [8] investigate the capacity of five previously tested transmission towers with range from 220KV to 400KV. They drown significant conclusions such as the location of failed members, the necessity of nonlinear analysis, modifying the capacity of bracing members in ASCE and IS Codes, the effect of the shape of bracing, the design of redundant members.

Behrouz Asgarian, et al [9], in their study, evaluate the progressive collapse of 400 kV transmission tower. They determined the load increase factors after the failed element removal through static analyses. In addition, the capacity to demand ratio suggested to identifying the most critical members after different removal scenarios. They compare these parameters with overload factor calculated from pushdown analysis.

F.Albermani,. et al [10] present a nonlinear technique for the failure analysis of transmission towers. They use this technique to verify a new tower design. In addition, the authors suggest using this technique to save the costs of full test.

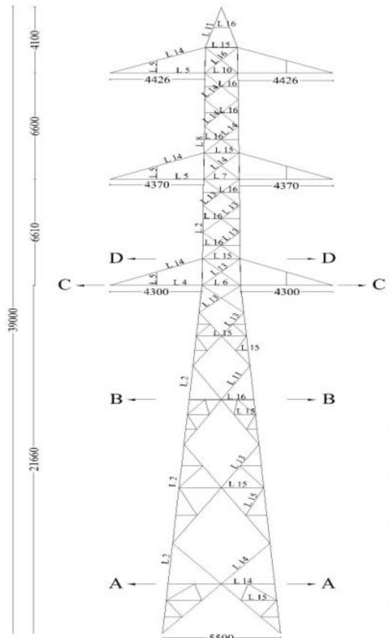
This paper investigates the nonlinear inelastic transmission towers response under HIW loading and provides a comparison of the tower capacities (i.e., yield and maximum) for wind loading. The modeling of actual and real lattice transmission tower presented herein. Commercial software SAP 2000 used in the modeling due to its ease handling of nonlinear material properties and3-D numerical simulation capability. The analysis considers both material and geometric nonlinearity and it shown that an adequate approximation of the capacity curve of the tower obtained using the NSP method. Capacity curve for wind loading condition in transverse direction to the tower obtained.

II. Analytical Models For Transmission towers

1. Configurations of Towers

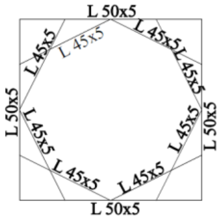
For tracing of failure mechanisms and determination of its capacity under the effect of High Intensity Wind loads, two types of failed towers in Aswan at January 2010 studied. The structural design of the tower based on the wind loads acting on the conductor/tower body as well as self-weight of the conductor /tower

The first studied tower is T.T 220 KV that has a 39 ms height, and 5.5 m × 5.5 m square base distance. It has a total of 885 members and 186 joints and the structural system is self-supported cantilever type. All members of the tower are equal legged angle sections and modeled as three dimensional frame elements. Bracing members, which used to decrease the slenderness ratio of the main members thereby increasing their bucking capacity. The diagonal triangulation systems in this type of transmission towers are X- bracing. The tower configuration, dimensions and members cross sections are shown in Figure (1-a to 1-e).

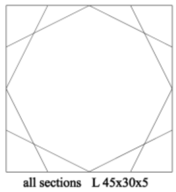


L	Angle Section (mm)
L 1	L 140x140x10
L 2	L 120x120x10
L 3	L 110x110x8
L 4	L 100x100x7
L 5	L 90x90x7
L 6	L 80x80x6
L 7	L 75x75x5
L 8	L 65x65x8
L 9	L 65x65x6
L 10	L 65x65x5
L 11	L 60x60x8
L 12	L 56x56x5
L 13	L 55x55x5
L 14	L 50x50x5
L 15	L 45x45x5
L 16	L 45x30x5
L 17	L 40x40x4

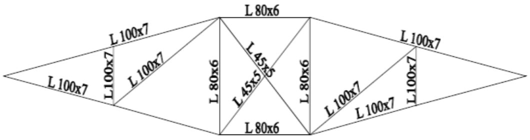
(1-a) 220KV Tower dimensions and members cross sections



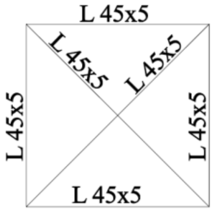
(1-b) Section A-A



(1-c) Section B-B



(1-d) Section C-C at cross arms



(1-e) Section D-D on face tower

Figure (1) 220KV configuration

The second analyzed tower is T.T 500 KV. The tower has a framed configuration with horizontal truss as the frame girder and two vertical trusses as the vertical columns. The connections between the horizontal truss and vertical one through two points in each column using only single bolt for each point as shown in Figure (3-a to 3-e). This tower has a height of 30 m and 3 m × 3 m as a square base dimensions for each vertical truss. The tower has a total of 1618 members and 698 joints. All members of the tower are equal legged angle sections with different sizes as detailed in Figure (4-a to 4-e) and modeled as three dimensional frame elements.

(3-a)

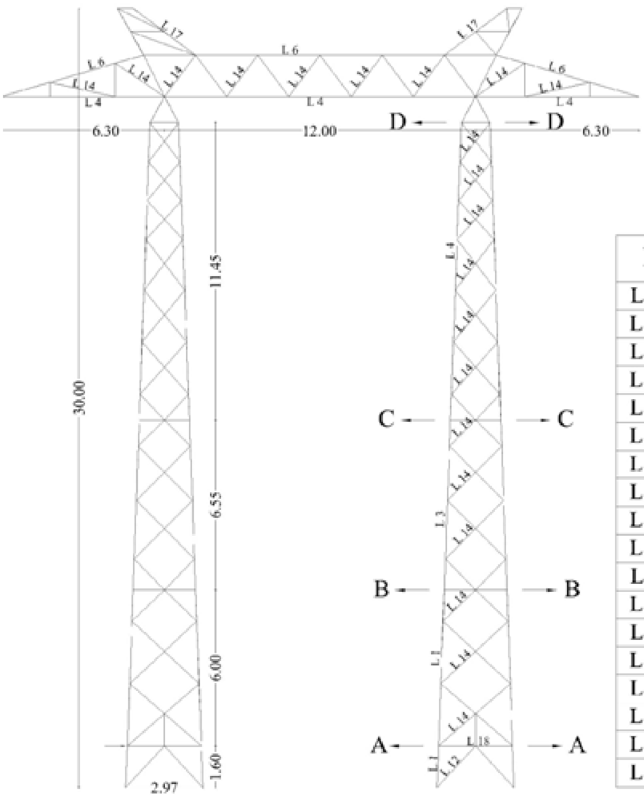
(3-b)

(3-c)

(3-d)

(3-e)

Figure (3) 500 KV Framed connection



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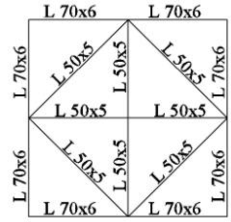
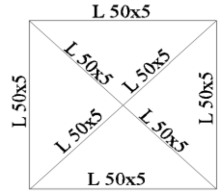
L 8	L 65x65x8
L 9	L 65x65x6
L 10	L 65x65x5
L 11	L 60x60x8
L 12	L 56x56x5
L 13	L 55x55x5
L 14	L 50x50x5
L 15	L 45x45x5
L 16	L 45x30x5
L 17	L 40x40x4
L 18	L 70x70x6

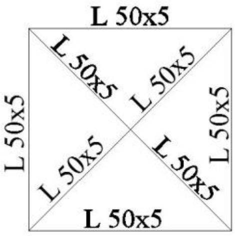
(4-a) Tower Elevation and sections



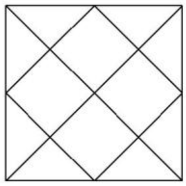
(4-b) Section A-A



(4-c) Section B-B



(4-d) Section A-A



(4-e) Section B-B
All L 50x50

Figure(4) 500KV Tower dimensions and members cross sections

2. Finite elements models

Three-dimensional nonlinear models developed using SAP 2000-V17.3 (8). The static pushover analysis procedure, which is well known technique in performance-based design for seismic and wind loading is adopted. All members modeled by introducing the material nonlinearity at discrete, user defined plastic hinges at terminals. The hinge properties created with pushover analysis and are provided based on FEMA-356 criteria. The used nonlinear static analysis procedures capable to capture the sharp drop-off in load carrying capacity through push over analysis. The used technique that allow displacement control, so that unstable structures can be pushed to desired displacement targets. The used modulus of Elasticity equals to 2100 t/cm2, basic wind speed value of 35m/s, the main legs formed from high tensile steel with proof strength 3.60 t/cm2 while the web members made of mild steel of yield strength 2.4 t/cm2. The applied loads considered as per Egyptian Ministry of Electricity Code (2), the design loads in normal operating conditions are shown in Figures (5) and (6).

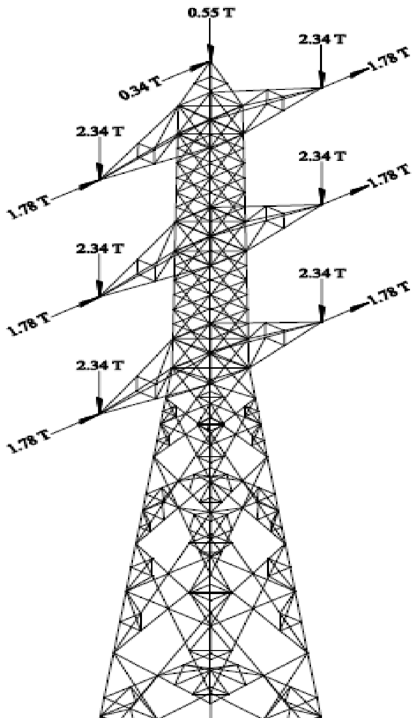


Figure (5) 220 KV tower normal operating loads scheme

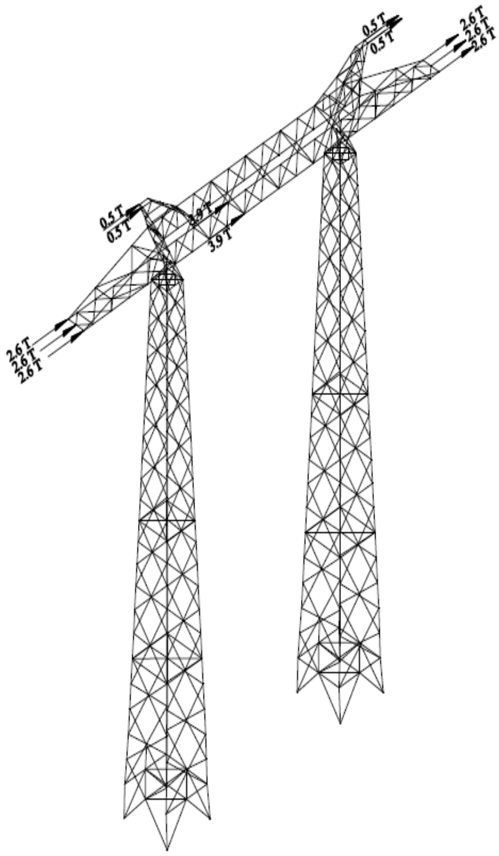


Figure (6) 500 KV tower normal operating loads scheme

III. Analysis And Results

The NSP analysis done for each tower under the normal operating load condition (self-weight and wind pressure). Under this condition, the dead load case was applied at first and then the incremental application of transverse wind load until reaching the ultimate capacity. The results of the analysis are the capacity curve that correlate the relation between the base shear (summation of the horizontal reaction in wind direction) and the horizontal displacement at the top of the tower in dominate direction. Figures (7) and (8) shows the capacity curves for the 220KV and 500 KV towers respectively. From the analysis

The maximum horizontal displacement at failure for the 220KV tower is 969 mm at maximum base shear of the value 25.4 tons. For 500 KV tower, the maximum horizontal displacement is 613 at maximum base shear equals to 33.4 tons.

Referring to the design loads at normal operating condition, the maximum horizontal displacement in the dominant direction is 270 mm at base shear equals to 15.22 tons for 220 KV tower while the corresponding values for the 500 KV tower are 383 mm and 25.70 tons respectively. Table (1) summarize the obtained results. From that table, the actual wind load factors for the normal operating condition (λ = Maximum Base shear at Failure/Design Base shear) for both studied towers are determined. It is important to compare the calculated base shear from the wind induced at the time of windstorm if the body of the towers can sustain this pressure. For the 220 KV tower, the resultant of pressure at wind speed 200 Km/h is 24.05 ton while the resultant of pressure at the same wind speed is 40.73 tons as illustrated in table (2). It is also worthy to highlight that the structural system of 500 KV tower is not stiff enough as it works like a framed structure due to the detail of connection between the horizontal truss (Girder) and the vertical truss (Column) as this connection based on four bolts which perform as intermediate hinges. The numerical results endorses this conclusion as the λ value for this tower 1.29 compared with the same value (1.67) for 220 KV tower.

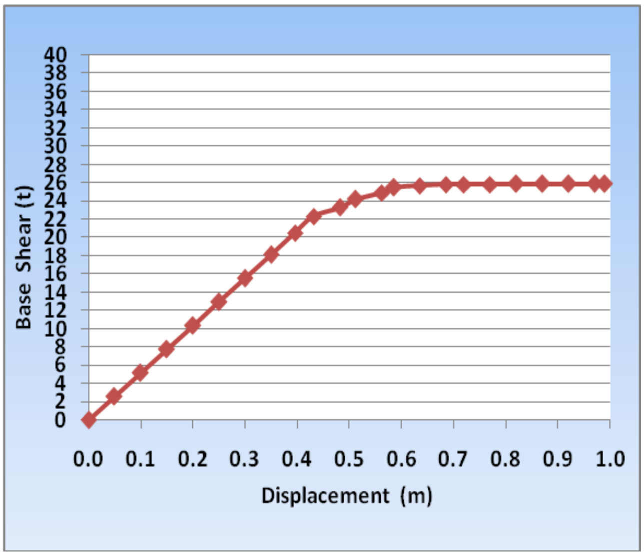


Figure (7) 220 KV tower capacity curve

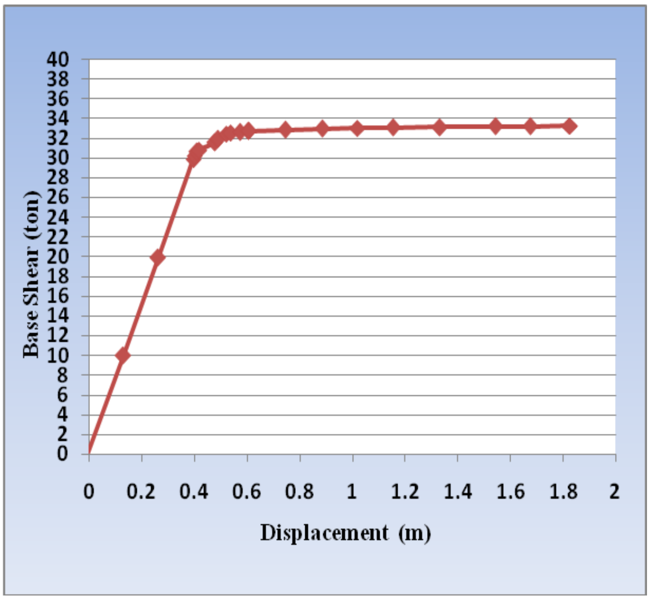


Figure (8) 500 KV tower capacity curve

Table (1) Actual wind load factor

T.T. Type	Design Base Shear (Ton)	Design Max. Horizontal Displacement (mm)	Failure Base Shear (Ton)	Failure Max. Horizontal Displacement (mm)	Wind Load Factor λ
220 KV	15.22	270	25.4	969	1.67
500 KV	25.78	383	33.4	613	1.29

Table (2) Comparison between pressure resultant at windstorm (speed 200 Km/h) and Towers failure base shear

T.T. Type	Failure Base Shear (Ton)	Calculated wind pressure resultant (ton)
220 KV	25.4	24.0
500 KV	33.4	40.7

IV. Failure Mechanisms Of Transmission Towers

The failed towers due to the H.I.W. at Aswan, Egypt in January 2010 demonstrate real loading tests without control for the applied forces or records for the internal forces or deformations and displacements. However, studying the modes of failure of towers and the analysis results out coming from NSP analysis could provide the design engineer with very useful data for the understanding of T.T behaviour.

The mode of failure of the 220 KV tower resulted from the theoretical analysis indicates that the first two plastic hinges formed at the upper part of first diagonal member at the first panel and the failure occurred in the second panel started 5.6 meters from the base of the tower. Figures (9) and (10) illustrate the both obtained theoretical mode of failure and the collapsed real 220 KV tower.

For the 500 KV tower, the mode of failure obtained from the NSP analysis demonstrate that the first two plastic hinges occurred at the lower part of the first diagonal and the failure take place at a height of 8 meters from the base. The comparison between the theoretical mode of failure and the real collapsed tower shown in the figures (11) and (12) respectively.

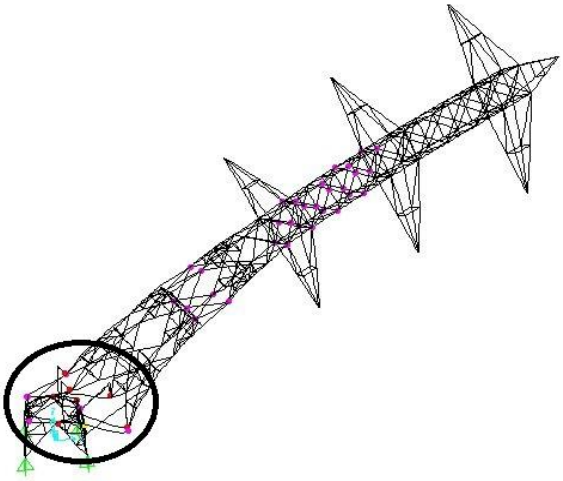


Figure (9) 220 KV tower finite elements failure mode

Figure (10) 220 KV real collapsed tower

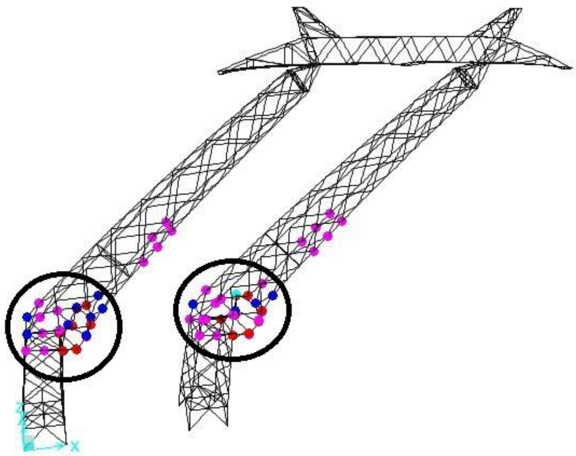


Figure (11) 500 KV tower finite elements failure mode

Figure (12) 500 KV real collapsed tower

V. Conclusion

The application of nonlinear static pushover analysis technique for transmission tower structures presented in this paper. The used technique can be used to predict the T.T capacity with acceptable accuracy. There is reasonable matching between the finite elements based modes of failure and the actual collapse for both studied towers types. In addition, it is important to declare that the structural system of 500 KV tower exhibit un desired flexibility conversely to the 220 KV tower due to the details of connection.

References

[1]. ASCE Manual 74, American Society of Civil Engineering, USA, 2001. "Guidelines for electrical transmission line structural loading," June 2001 Draft revision, ASCE manuals and reports on engineering practice, ASCE, Reston, VA

[2]. Design standard for H.V overhead transmission lines, Arab Republic of Egypt, Ministry of energy and electricity, 1988.

[3]. Sudhan Banik, Hanping Hong, and Gregory A. Kopp "Assessment of Structural Capacity Of An Overhead Power Transmission Line Tower Under Wind Loading, " Milano, Italy, July, 20-24 2008.

[4]. Baskaran, K., Jeneevan, Jayasinghe and Hettiarachchi "Strength Assessment Of Steel Towers, " The University of Moratuwa, Civil Engineering Research for Industry – 2011.

[5]. Shakeel Ahmad and Md. Ehtesham Ansari, "Response Of Transmission Tower Subjected To Tornado Loads, " The Seventh Asia-Pacific Conference on Wind Engineering, November 8-12, 2009, Taipei, Taiwan.

[6]. Wei Zhang1* , Jin Zhu , Huijuan Liu and Huawei Niu "Probabilistic capacity assessment of lattice transmission towers under strong wind, " Wind Engineering Research Center, Hunan University, Changsha, China, Article 20. vol1., 2015.

[7]. Haitham Aboshosha and Ashraf El Damatty "Capacity of Electrical Transmission Towers under Downburst Loading, " Research, Development, and practice in structural Engineering and construction., 2012.

[8]. N.Prasd Rao, G.M Samuel Knight, N.Laksh Manan, Nagaesh R. Lyer "Investigation of Transmission Towers Failure" Engineering Failure Analysis 17 (2010).

[9]. Behrouz Asgarian, Soheil Dadras Eslamlou, Arash E. Zaghi, Masoud Mehr "Progressive collapse analysis of power transmission towers", Journal of Constructional Steel Research 123 (2016) 31–40

[10]. F.Albermani, S.Kitipornchai, R.W.K.Chan "Failure Analysis of Transmission Towers"Engineering Failure Analysis 17 (2010).