International Journal of Advanced and Applied Sciences, 5(1) 2018, Pages: 73-80

Contents lists available at Science-Gate

International Journal of Advanced and Applied Sciences

Journal homepage: http://www.science-gate.com/IJAAS.html

Review on analysis and design of lattice steel structure of overhead transmission tower

Siti Aisyah Kamarudin [1,*], Fathoni Usman [2], Intan Nor Zuliana Baharuddin [2]

[1]*College of Graduate Studies, Universiti Tenaga Nasional, Kajang, Malaysia*
[2]*Institute of Energy Infrastructure, Universiti Tenaga Nasional, Kajang, Malaysia*

ARTICLE INFO

Article history:
Received 14 August 2017
Received in revised form
23 October 2017
Accepted 20 November 2017

Keywords:
P-delta
Linear static
Tower failure
Overhead transmission tower
Lattice steel

ABSTRACT

Demand for electricity is increasing as the population grows. Transmission towers need to be upgraded in order to satisfy these increasing demands for power supply. A study on the stability of the transmission tower is significant to make sure that the tower is stable and capable enough to transmit electricity due to high demand. In this paper, a review on analysis and design of the lattice steel structure of the overhead transmission tower together with its design as in ASCE and Eurocode standard are presented. Two methods are introduced which are linear static analysis and p-delta analysis to compare its maximum internal force, maximum displacement as well as location for critical part of the tower when different loadings applied. In analysis, the tower model used for modelling and simulation is assumed as fully-beam and fully-truss. Standard code MS 1533:2000 is referred for wind loading calculation. It is found that the highest internal force is at the leg of the tower on a normal case with an angle of attack of wind is 45°.

1. Introduction

An increasing demand in power supply means that these towers require an upgrading to carry the resultant heavier loading and electric power (Albermani et al., 2004). A continued community's growth need more demands on development and expansion of power plants also power distribution networks. The reliable and uninterrupted operation of power transmission lines is crucial (Asgarian et al., 2016).

In Malaysia, Tenaga Nasional Berhad (TNB) is the largest electricity utility with the core business of providing electricity to the country's businesses, homes and industries, TNB now is the key contributor to the Nation building. The total length of transmission network recorded in Peninsular Malaysia is 22,478 km, with the total transmission substation of 426 station (TNB, 2016). After all, the highest demand for electricity recorded in Peninsular Malaysia is 17,788 megawatts (MW) during April 2016 as stated in Tenaga Nasional Berhad Annual Report 2016. The peak reading recorded is 37.82% increase compared to demand on January 2016 (12,906 MW) due to an El-Nino phenomenon (Kamarudin et al., 2017). About 2.9% year-on-year increase in revenue to RM44.5 billion primarily due to 4% electricity demand growth in Peninsular Malaysia is recorded (TNB, 2016). In order to maintain stability of the transmission tower, the structure of the tower itself need to be stable. The collapse mechanism depends on many factors such as improper detailing, material defects, fabrication errors, and forces during the erection, a variation in bolt strength detailing, connection failures and more although the design transmission line towers follows the code provision (Rao et al., 2012). So, the structural response of the transmission tower under extreme loading needs a better understanding. It is necessary for improving the safety and security of power lines. The impact of element failure on the response of the entire system and failure modes should be realized (Asgarian et al., 2016). According to (Ellingwood et al., 2007), some of the possible abnormal loads which can cause progressive collapse arise from the following events: design errors, heavy object collision, fire, an explosion, accidental overload, lack of proper connectivity, etc.

The existing National Grid for power distribution is able to serve the Nation and this current demand still can be fulfilled. In order to prevent power outage due to increasing demand in the future, monitoring need to be implemented. To establish a new infrastructure for future development, land acquisition is a challenge to get an area for the

* Corresponding Author.
Email Address: AisyahS@uniten.edu.my (S. A. Kamarudin)
https://doi.org/10.21833/ijaas.2018.01.010
2313-626X/

construction. At this moment, optimization is the best way to fix this problem. In this matter, the objective of optimization is to enhance the capability of the transmission tower for carrying heavier loading as well as for cost saving.

Before proceed with the optimization, a brief study on the transmission tower analysis must be conducted by using suitable methods. Like in this study, the chosen methods are linear static analysis and p-delta analysis. Those methods are most likely has the same procedure but the different theory assumptions and parameters lead the methods producing different results. Thus, this paper is aimed to study the comparison of linear static analysis and p-delta analysis. A brief review of analysis and design of a lattice steel structure of the overhead transmission tower is presented in this paper as well.

2. Overview of the lattice overhead transmission tower

This part explains the failure, modeling and design codes of the transmission tower.

2.1. Failure of the transmission tower

There are a lot of failures reported and one of it was due to rain loads and wind. The collapse of transmission towers causes great economic loss and many accidents. The fact of the transmission tower collapsed during hurricanes or typhoons attracts researchers to accomplish their research to this issue. Typhoons or hurricanes are always followed by the strong rainfall, and the influence of a rain load on the tower collapse has not been studied before. Therefore,

considering the rain load, even the action of both wind and rain loads together is very necessary and significant (Fu and Li, 2016).

Transmission tower-conductor coupling systems are more susceptible to natural disasters which have been more frequent and severe over the past few years due to the rapidly increased height and span. In China, during 2008, the Southern area suffered severe ice disasters. Power systems sustained serious damage due to the catastrophic freezing rain. According to released statistics, the gross collapsed number and damaged towers with rating above 35 kV of the State Grid Corporation of China and local electrical companies reached 7263 towers (Xie and Sun, 2012). Meanwhile, in 2010, five transmission towers collapsed due to the strong wind and rain in Guangdong Province of China. During 2013, natural disaster typhoon Fitow in Zhejiang Province of China affected the area and collapse of the tower also charged to power interruption (Tian et al., 2014). Vandalism also contributed to the reason of the failure. In Malaysia, a power blackout occurred on 22 April 2008 and Sabah had the worst power outage since the commissioning of the east west power grid. Suspected vandals were believed to have removed steel pieces of a 132 kV overhead transmission tower led to its collapse, triggering a major power blackout. An emergency temporary tower was built immediately, but it also collapsed during construction killing TNB personnel. While on 1 May 2008, another tower collapsed due to missing structural members of the tower that were suspected of being stolen. After all, reported cases of power supply disruption in 2015 are 293 cases caused by wind. It was 0.42% of total power supply disruption in Peninsular Malaysia (Kamarudin et al., 2017).

However, according to Eslamlou and Asgarian (2016) the main causes of failure in transmission towers may be cable rupture during a storm, improper behavior of a member or a connection and an explosion near the tower.

2.2. Modeling for simulating and analysing of the tower

There are studies that using a nonlinear analytical technique to simulate and assess the ultimate structural response of latticed transmission towers. The technique may be used to verify new tower design and reduce or eliminate the need for full-scale tower testing. This also can be used to determine strength of existing tower or to upgrade old tower. It has been calibrated with results from full-scale tower tests with good accuracy in terms of the failure load and failure mode (Albermani and Kitipornchai, 2003). Using the same concept, study about non-linear analysis of angle compression members and the single panel of angle planar as well as three-dimensional lattice frames, as in typical lattice towers, are carried out using MSC-NASTRAN software. Member of eccentricity, local deformation as well as a rotational rigidity of joints, a beam-column effects and material non-linearity is taken into account. The analytical models are calibrated with test results. Using this calibrated model, parametric studies are carried out to evaluate the forces in the redundant and the results are compared with code provisions (Rao and Kalyanaraman, 2001). Other than that, a systematic methodology to model both the static and dynamic behaviors of a line due to conductor breakage using the ADINA (Automatic Dynamic Incremental Nonlinear Analysis) program is presented as by Haldar et al. (2010). Based on the analysis, the extent of a cascade damage or failure zone is estimated and a mitigation approach for correcting the situation is provided (Haldar et al., 2010). According to Vinay et al. (2014), transmission towers are generally analyzed by linear static analysis methods and second order analysis are usually neglected. Linear static analysis does not reflect the structural characteristic of transmission towers. There is a study in (Vinay et al., 2014) that presented p-delta effects. A tower is modelled using angle and tubular sections using STAAD.Pro V8i software and analyzed for the wind load by using linear static and p-delta analysis to study the importance of p-delta analysis for the transmission tower. After analysis, the comparative study is presented with respective to cost and displacement for both sections. A saving in steel weight up to 20.9% resulted when a tubular section is compared with the angular section. The displacement values increased for both angular and tubular sections when the tower is analyzed for p-delta as compared to static analysis (Vinay et al., 2014).

Other than that, numerical analysis of steel lattice towers has recently been performed by other researchers worldwide (Fu and Li, 2016; Xie and Sun, 2012;Tian et al., 2014). There is a study about a bolted splice connection used in the main legs of steel lattice transmission towers. This method conducted using finite element modelling of test specimens prepared in the commercially available computer program ANSYS (Baran et al., 2016). Meanwhile, there is another study that illustrated with a case study of the line section having suffered two tower failures due to conductor breakages during an ice storm. This study successfully applied cable dynamic model to several examples by using ADINA software (Mcclure and Lapointe, 2003).

There is a study for transmission towers where advanced non-linear analysis applied. They emphasized that the problem is more complicated by the spatial nature of the configuration and by the fact that individual components are a symmetric angle shapes that are eccentrically connected. Then, the elements undergo uniaxial loading and biaxial bending effects, which are impossible to model using conventional 3D elastic truss-type methods (Eslamlou and Asgarian, 2016). According to Eslamlou and Asgarian (2016), non-linear analysis results are compared between a truss model and a frame model where the tower legs are represented with frame elements while the secondary bracing members (redundant members ignored in linear analysis) were also taken into consideration. Both models yielded similar results, but the frame method is more preferable. Eslamlou and Asgarian (2016) discussed the dynamic effects of progressive member failure of a truss structures. Their method is to replace the damaged member by the adequate external force functions at its end joints. They observed that sudden buckling failures can cause significant stress redistributions near adjacent members and might cause a second member failure, and possibly trigger progressive collapse.

2.3. Design codes

According to the American Society of Civil Engineers as in ASCE 10-97, latticed steel structures shall be designed with geometric configurations based on electrical, economic, and safety requirements (ASCE, 1997). The compression capacity, *fa* is calculated referring clause 3.6 and clause 3.7 of ASCE 10-97 (ASCE, 1997). It is started by comparing the ratio value of flat width to thickness of the leg, $\frac{w}{t}$ against the $\left(\frac{W}{t}\right)_{lim}$. If $\left(\frac{w}{t}\right) \leq \left(\frac{W}{t}\right)_{lim}$, use (1) or (2), and if $\left(\frac{w}{t}\right) \geq \left(\frac{W}{t}\right)_{lim}$

$$Fa = 1 - \left[\frac{1}{2} - \left(\frac{\frac{KL}{r_v}}{C_c}\right)^2\right] Fy \tag{1}$$

$$Fa = \frac{\pi^2 E}{\frac{KL^2}{r}} \tag{2}$$

$$F_{cr} = \left[1.677 - 0.677 \frac{\left(\frac{w}{t}\right)}{\left(\frac{W}{t}\right)_{lim}}\right] \tag{3}$$

The compression capacity,

$$fa = Fa \times As \tag{4}$$

According to ASCE 10-97, all structure components (e.g., members, connections, guys) are selected to resist design-factored loads on stresses approaching failure in yielding, buckling, fracture, or any other specified limiting condition. Design-factored load is an unfactored load multiplied by a specified load factor to establish the design load n a structure. Other than that, all applied loads shall be measured at the point of attachment to the prototype. Loads shall be measured through a verifiable arrangement of strain devices or by predetermined dead weights. Load-measuring devices shall be used in accordance with a manufacturer's recommendations and calibrated prior (ASCE, 1997).

The actual yield point for tension and compression members of effective length with a radius of gyration ratio, *KL/r* values that less than 120 are critical in determining the member capacity. Thus, the design members must conform to the standard material specifications, but their actual yield points are not as critical to their load-carrying capabilities. Meanwhile, in EN 1993-3-1:2006, as far as overhead transmission towers are concerned all matters related to wind and ice loading, loading combinations, safety matters and special requirements (i.e., such as for conductors, insulators, clearance, etc.) are covered by the European Committee for Electrotechnical Standardization Code (CENELEC) EN 50341, that can be referred for the design of such structures. Loads along the member length including wind, or dead loading on other members framing into the member should be considered as well as the loading to be used to calculate bracing member forces should be based on the configuration of the tower. The structure should generally be divided into a sufficient number of sections to enable the wind loading to be adequately modelled for global analysis (ECS, 2011).

The design buckling resistance on a compression member in a lattice tower or mast should be determined according to ECS (2005). In addition, all the basic assumptions for the calculations shall reflect the structural behaviour at the relevant limit state with appropriate accuracy and reflect with the anticipated type of behaviour of the cross sections, members, joints and bearings (ECS, 2011; ECS, 2005).

3. Methodology

This part discussed the analysis of a transmission tower using p-delta analysis and to be compared with linear static analysis. In this part also interpreted on the details of assumption and procedure of conducting this study.

In the p-delta analysis, a single element may be subjected to arbitrarily large displacements and rotations as a rigid body, but the true deformation (i.e., this one which generates strains, and in consequence stresses), remain small within one element. It means that for the element interior geometrical relationships stay linear. Geometrical non-linearity is thus treated on the level of the structure.

3.1. Structure model

A 275 kV lattice steel of the overhead transmission tower (OTT) and 24SL type is used in this study. Studied structures are representative of typical lines in Malaysia. There are 11 different steel sections used in this tower model (i.e. EQA 100 x 100 x 8, EQA 90 x 90 x 7, EQA 70 x 70 x 5, EQA 60 x 60 x 5, EQA 50 x 50 x 5, etc.). The placement of the different steel section depends on compression capacity of the steel section for main members and bracing accordingly. The support condition for this tower model is restrained condition. The members are assumed as a beam and a fixed connection for each joint. The main load cases are divided into four conditions which are Normal Condition (NC), Ground Wire Broken (GWB), Middle Conductor Broken (MCB) and Top Conductor Broken (TCB). The factor of safety (FOS) of 2.0 for normal condition and FOS of 1.25 for broken wire condition. Fig. 1 shows loading trees for the Normal Condition case. While the Table 1 presents the loading calculation for Normal Condition. The model is generated in Autodesk Robot Structural Analysis Professional 2017 (ARSA).

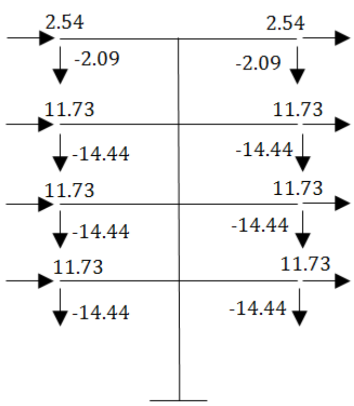

Fig. 1: The loading tree for normal condition

The outlined drawing of the tower as shown in Fig. 2 is provided by the manufacturer. Three different directions of the load assigned to the structure of the tower which are vertical, transverse and longitudinal. The longitudinal loads act parallel to the line and the transverse load is perpendicular to the line. For the vertical load, it is basically from the self-weight of the Overhead Transmission Tower (OTT) (Usman and Megat Asyraf, 2011).

Table 1: Loading calculation for normal condition (conductor Zebra)

No.	Description of Loads	Calculation	Force (N)
1	Wind on insulator string	1 × 0.5 × 3.28 × 0.254 × 430	179
2	Wind on conductor	2 × 365 × 0.02862 × 430	8984
3	Transverse load due to tension in conductor	2 × 2 × 3754 × sin 1 × 9.81	2571
4	Weight of insulator string	1 × 1690	1690
5	Weight of conductor (downward load)	2 × 600 × 1.635 × 9.81	19247
6	Minimum vertical load		6502

4. Result and discussion

This part discusses a result taken from the p-delta analysis and linear static analysis to determine the effect of an angle of attack of wind to the tower and internal force in every member. It is fully concerned to a description of a p-delta analysis on a 275 kV lattice steel of the overhead transmission tower and to present the obtained results. A simplified analytical model of the transmission tower based on standard regulations has been presented in this paper. Also, the comparison between both analyses is focused on the maximum internal force terms of a bar section, the main part of the tower, loading condition, displacement, the angle of attack and deflection ratio.

Autodesk Robot Structural Analysis Professional 2017 (ARSA) has been used to construct the model and carry out the analysis as mentioned earlier in the methodology part. The load cases and a combination of loads have been applied according to the specific manual and load specification. The p-delta analysis together with the iterative method, a Newton-Raphson method which is a solver in ARSA is used.

Based on the result as stated in Table 2, in linear static analysis, the highest value for internal force is 488.43 kN. Unlike p-delta analysis, the highest value is 495.21 kN. There is 1.5% difference in the internal force between both methods. Both value is in compression and located at the bar number 4 which is at the leg of the tower. From the analysis, the velocity for the wind is 33.5 m/s as it is based on the MS 1533:2002. After all, this tower can be considered as safe because the highest internal force still not exceeding the compression capacity value calculated by referring to the clause 3.6 and clause 3.7 of ASCE 10-97 as in (4) which is 553.70 kN. Basically, analysis shows that the highest value for internal loading in compression ranging from 100 – 495 kN.

In both linear static and p-delta analysis, the maximum value of internal loading is found to be at member size 100 × 100 × 8. Table 3 below shows, the larger value found in other bar members for both analysis. Here, it can be seen that the difference in sizing would produce a different value in internal loading. Lattice of Overhead Transmission Tower (OTT) can be divided into three main parts which are arm, body and leg.

Table 2: Maximum internal force

No.	Maximum Internal Force		
	Details	Static Analysis	P-Delta Analysis
1	Compression	488.43 kN	495.21 kN
2	Bar	4	4
3	Case	15 (C)	15 (C)

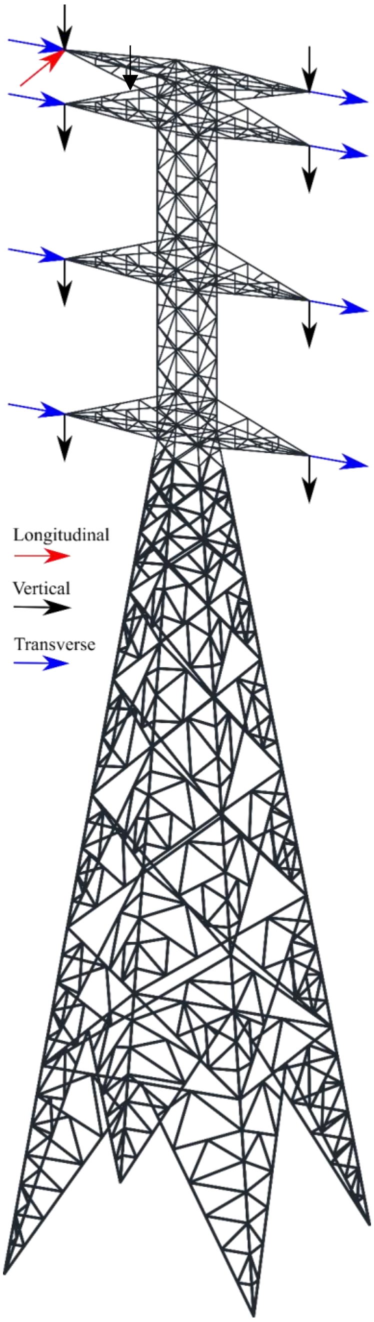

Fig. 2: The 3D model for 275kV OTT

After observation, in linear static and p-delta analysis, the leg areas have the larger value of internal. Besides, 480.49 kN of force is found to be at the body part which contributed to the second largest loading for linear static analysis while for p-delta analysis is 420.81 kN. Next, the internal loading at the arm part is 93.96 kN in linear static analysis and 94.96 kN in p-delta analysis. Furthermore, according to the study in (Albermani et al., 2009), if the failure occurred in the lower part of the tower, it is actually triggered by the elastic buckling of a hip bracing member, leading to buckling of the main diagonal bracing member, which initiated the compression leg to buckle as well, resulting in the full collapse of the tower.

Table 3: Bar section and maximum internal force

Bar Section	Internal Force (kN)		Location
	Linear Static Analysis	P-Delta Analysis	
90 × 90 × 7	418.55	426.41	Body
70 × 70 × 5	38.14	14.12	Leg
75 × 75 × 5	64.84	93.77	Bottom Arm
60 × 60 × 5	480.39	490.21	Leg, Body
50 × 50 × 5	44.34	57.79	Upper Arm, Body
45 × 45 × 5	32.08	18.25	Arm Bracing

According to the Table 4, it presents the maximum internal force with different loading condition. It is noticed that the highest force is in Normal Condition case. The percentage difference for all four conditions is ranging from 0.6% - 4.4 %. Those values are different due to the particular arrangement and configuration of each loading condition.

Table 4: Loading condition and maximum internal force

Loading Condition	Internal Force (kN)	
	Linear Static Analysis	P-Delta Analysis
Normal Condition	418.55	426.41
Ground Wire Broken	38.14	14.12
Middle Conductor Broken	64.84	93.77
Top Conductor Broken	480.39	490.21

Other than that, in the term of displacement there is a study in (Fu, 2009) declares that under the same general conditions, a member removal at an upper level will induce larger vertical displacement than a member removal at ground level. From the Table 5 shows that the maximum displacement occur at the top most point of arm position.

Table 5: Node position and maximum displacement

No.	Node Position	Location		
		Linear Static Analysis	P-Delta Analysis	Maximum Permissible Displacement
1	Base of Leg	0	0	0
2	Bottom Hamper Point	184	188	360
3	Middle Cross Arm Tip	316	323	434
4	Top Most Point of Arm	534	547	526

While Fig. 3 presents the relationship between the different angle of attack with internal loading. Maximum loading for linear static and p-delta analysis is detected coming from 45° direction. Both analysis have the maximum internal loading which more than 400 kN. Other than that, the ratio of internal force with compression capacity indicates the allowable loading that can subject to the tower instability. It is found that the highest ratio is 0.93 in linear static analyses likewise 0.89 ratios is for p-delta which both is located at bar 4, the leg member of the tower.

Table 6 represents the value of internal forces compared to the maximum values for the respective load case. The highlighted cell on the ratio's column represents the maximum ratio. It is shown that most of the member subject to less than 0.5 of the ratio of its internal force to its capacity. An optimization method can be applied by reducing the members with the lowest ratio to have more reliable tower design without affecting its stability.

Table 6: Internal forces on some truss members

CASE 14			CASE 15		
MEMBER NO.	INTERNAL LOADING (Fx)	RATIO	MEMBER NO.	INTERNAL LOADING (Fx)	RATIO
2	-214.94	-0.39	2	-120.82	-0.22
3	-215.71	-0.39	3	-298.45	-0.54
4	407.28	0.74	4	488.43	0.88
5	406.63	0.73	5	308.68	0.56
6	-13.16	-0.08	6	-13.24	-0.08
7	33.39	0.21	7	29.87	0.18
8	20.98	0.13	8	11.05	0.07
9	21.17	0.13	9	31.01	0.19
10	33.37	0.21	10	33.17	0.20
11	-13.11	-0.08	11	-8.20	-0.05
12	0.55	0.00	12	16.48	0.10
13	0.51	0.00	13	-15.50	-0.10
14	2.60	0.01	14	0.99	0.00
15	-0.11	0.00	15	5.18	0.02
16	2.63	0.01	16	2.85	0.01
17	-0.25	0.00	17	0.54	0.00
18	4.45	0.03	18	1.50	0.01
19	-2.02	-0.01	19	1.64	0.01
20	1.05	0.01	20	-0.94	-0.01
21	-1.67	-0.01	21	1.72	0.01
22	-1.69	-0.01	22	-0.77	-0.01
23	1.07	0.01	23	-0.89	-0.01
24	-1.99	-0.01	24	-0.89	-0.01
25	4.45	0.03	25	1.57	0.01
27	7.42	0.03	27	4.77	0.02
28	7.41	0.03	28	10.31	0.05
29	-0.43	0.00	29	-1.67	-0.01
30	11.6	0.05	30	9.36	0.04
31	11.6	0.05	31	10.63	0.05
32	-0.44	0.00	32	1.67	0.01
33	-3.15	-0.01	33	5.65	0.03
34	-3.16	-0.01	34	-9.39	-0.04
35	-0.18	0.00	35	-0.2	0.00
36	0.42	0.00	36	0.78	0.01
37	-0.36	0.00	37	-0.5	0.00
38	3.88	0.03	38	2.31	0.02

According to the study by (Preeti and Mohan, 2013), a square tower is found to have the maximum node deflection throughout the tower height, followed by the triangular tower and then the guyed mast. Other than that, this tower is having the maximum factor of safety for the upper cross arm members. This behavior might be because of the minimum length of the members. Upper cross arm member sections are found to be the same for all the towers. This might be because of these members are designed as the tension members and steel already has a good margin of safety in tension.

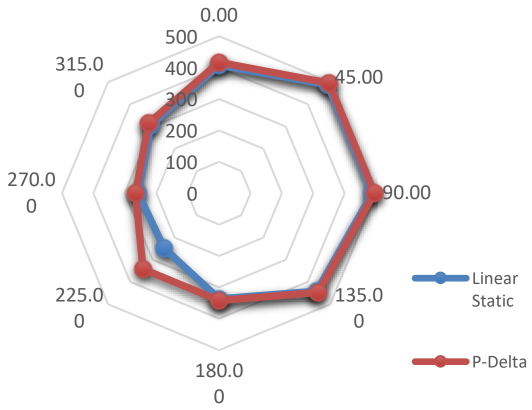

Fig. 3: Chart of an angle of attack with internal loading

There is a marginal difference between linear static analysis and p-delta analysis. Thus, in fact the tower model reveals that the effect of p-delta analysis significantly influence the axial, moment and displacement of the structural component (Bondre and gaikwad, 2016). In the same way, get higher value than linear static analysis. However, a specific modelling methods for OTT need to be developed. Advanced FEM technology with the capability of considering bolted connection behaviors and detecting failure mode needs further investigation. The tower structure is then assumed as a fully truss structure. The difference between beam and truss for both methods are represents in the Table 7 below. The maximum internal force occurs at the bar 4 which is at the leg of the tower with case number 15 (Normal Condition).

Table 7: Maximum internal force for beam and truss

Methods	Maximum Internal Force (kN)	
	Beam	Truss
Linear Static Analysis	418.55	426.41
P-Delta Analysis	495.21	503.21

The highest internal loading occurs at bar number 4. Fig. 4 shows the location of bar 4 which is at the leg part of the tower. After all, latticed steel structures shall be designed with geometric configurations based on electrical, economic, and safety requirements according to the American Society of Civil Engineers 10 - England 1997 (ASCE 10-97) (ASCE, 1997).

5. Conclusion

This paper explains the comparison between p-delta analysis and linear static analysis to determine the maximum internal forces also a displacement and a review on analysis and design of a lattice steel structure of the overhead transmission tower. In addition, both methods showed a minor different values as for the internal forces and the displacement after that will give ideas for further tower optimization.

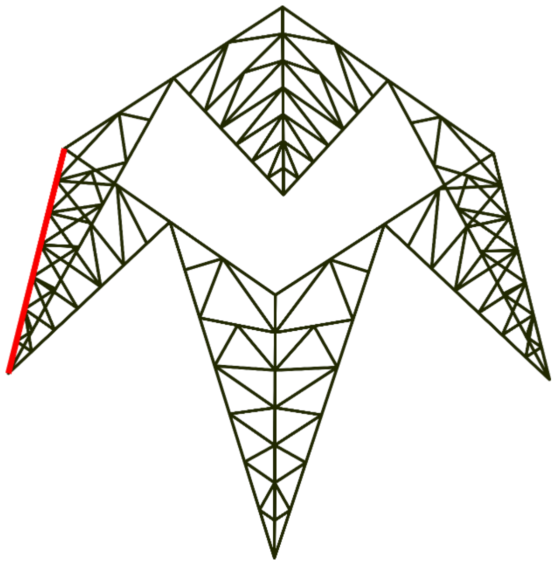

Fig. 4: Location for the critical part of the tower

Acknowledgement

The authors would like to thank UNITEN R and D Sdn. Bhd. for the financial support trough Project Grant No. U-TS-CR-17-01 and for the opportunity to publish this paper.

References

Albermani F, Kitipornchai S, and Chan RWK (2009). Failure analysis of transmission towers. Engineering Failure Analysis, 16(6): 1922-1928.

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

Albermani FGA and Kitipornchai S (2003). Numerical simulation of structural behaviour of transmission towers. Thin-Walled Structures, 41(2): 167-177.

ASCE (1997). ASCE 10-97 Manual: Design of latticed steel transmission structures. American Society of Civil Engineers, Virginia, USA. Available online at: www.pubs.asce.org

Asgarian B, Eslamlou SD, Zaghi AE, and Mehr M (2016). Progressive collapse analysis of power transmission towers. Journal of Constructional Steel Research, 123: 31-40.

Baran E, Akis T, Sen G, and Draisawi A (2016). Experimental and numerical analysis of a bolted connection in steel transmission towers. Journal of Constructional Steel Research, 121: 253-260.

Bondre R and gaikwad S (2016). Analysis of structures with respect to linear static analysis using p-delta effect. International Journal of Advance Research and Innovative Ideas in Education, 2(4): 838–845.

ECS (2005). Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings (EN 1993-1-1). European Committee for Standardization, Europe.

ECS (2011). Eurocode 3 - Design of steel structures - Part 3-1: Towers, masts and chimneys - Towers and masts. The European Union, European Committee for Standardization, Europe.

Ellingwood BR, Smilowitz R, Dusenberry DO, Duthinh D, Lew HS, and Carino NJ (2007). Best practices for reducing the potential for progressive collapse in buildings. National Institute of Standards and Technology, Gaithersburg, USA.

Eslamlou SD and Asgarian B (2016). Determining critical areas of transmission towers due to sudden removal of members. Case Studies in Engineering Failure Analysis. http://dx.doi.org/10.1016/j.csefa.2015.09.005

Fu F (2009). Progressive collapse analysis of high-rise building with 3-D finite element modeling method. Journal of Constructional Steel Research, 65(6): 1269-1278.

Fu X and Li HN (2016). Dynamic analysis of transmission tower-line system subjected to wind and rain loads. Journal of Wind Engineering and Industrial Aerodynamics, 157: 95–103.

Haldar A, Veitch M, and Andrews T (2010). Estimation of containment loads on a 230kV steel transmission line using finite element model. In the Electrical Transmission and Substation Structures Conference, American Society of Civil Engineers, Fort Worth, Texas, USA: 1-13. https://doi.org/10.1061/41077(363)3

Kamarudin SA, Usman F, and Omar RC (2017). Evaluation of overhead transmission tower subjected to predominantly wind loading. International Journal of Civil Engineering and Geo-Environment. In the 1st National Colloquium on Wind and Earthquake Engineering, Pahang, Malaysia.

McClure G and Lapointe M (2003). Modeling the structural dynamic response of overhead transmission lines. Computers and Structures, 81(8): 825-834.

Preeti C and Mohan JK (2013). Analysis of transmission towers with different configurations. Jordan Journal of Civil Engineering, 7(4): 450–460.

Rao NP and Kalyanaraman V (2001). Non-linear behaviour of lattice panel of angle towers. Journal of Constructional Steel Research, 57(12): 1337-1357.

Rao NP, Knight GS, Mohan SJ, and Lakshmanan N (2012). Studies on failure of transmission line towers in testing. Engineering Structures, 35: 55-70.

Tian L, Yu Q, Ma R, and Wang C (2014). The collapse analysis of a transmission tower under wind excitation. Open Civil Engineering Journal, 8: 136-142.

TNB (2016). Integrated Annual Report. Tenaga Nasional Berhad, BASIS Publication House Sdn. Bhd., Malaysia.

Usman F and Megat Asyraf MIR (2011). Simulation of progressive collapse for transmission tower. In the 1st TNB ICT Technical Conference, College of Information Technology, Universiti Tenaga Nasional, Kajang, Malaysia.

Vinay RB, Ranjith A, and Bharath A (2014). Optimization of transmission line towers : P-Delta analysis. International Journal of Innovative Research in Science, Engineering and Technology, 3(7): 14563–14569.

Xie Q and Sun L (2012). Failure mechanism and retrofitting strategy of transmission tower structures under ice load. Journal of Constructional Steel Research, 74: 26-36.