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M. Yanda
Civil Engineering Department,
Umaru Ali Shinkafi
Polytechnic, Sokoto, Nigeria.

M. A. Matazu
Civil Engineering Department,
Umaru Ali Shinkafi
Polytechnic, Sokoto, Nigeria.

O. S. Abejide
Civil Engineering Department,
Ahmadu Bello University,
Zaria. Nigeria.

Correspondence:
M. Yanda
Civil Engineering Department,
Umaru Ali Shinkafi
Polytechnic, Sokoto, Nigeria.

Structural Response of Steel Towers to Wind Loading

M. Yanda, M. A. Matazu, and O. S. Abejide

Abstract
In this work, effort is made to analyse how steel structures such as steel towers used in telecommunication industries behaves under wind load which is the most predominant and dynamic load. A telecommunication steel tower of 67.12m high is chosen for this work, and the geometry of the tower, as modelled, is 7.6m at the base and 2.0m at the top. It has sloping legs to about 42.72m in height and remain straight up to the top. The structure is divided into four sections in order to allow the wind load to be modelled adequately on the structure for the global analysis. The determination of wind loads was carried out on the basis of BS EN 1993-3-1 (2006), using the Nigerian basic wind speeds from five different zones 1, 2, 3, 4 and 5 with the values 42m/s, 45.8m/s, 50m/s, 55m/s and 56m/s respectively. Bracing members' projections in faces parallel to the direction of wind, and in plan and hip bracing were not considered in the determination of the projected area of the structure. It was seen from the analysis that the background wind loading of 3360.88 N/m^2 from zone 5 is the quasi-static loading produced by fluctuations due to turbulence. The wind load in that zone produced the highest displacement of joints, which can provide enough frequency that is capable of triggering collapse on the tower.

Keywords: Steel Structure, Wind loads, Mast, Tower, Telecommunication.

1.0. Introduction

Masts and towers are used in the telecommunications industry for the purpose of wireless communication or in electrification industry for power transmission. Masts and towers are tall structures exposed to dynamic loadings. These loads could be wind loads, loads as a result of earthquakes, sudden rupture of guys, galloping of guys. In some cases, these loads have led to the loss of signals for unwarranted periods of time as a result of displacement and rotation of the antennas, and in other cases they have resulted in permanent deformation or collapse. Wind load is the main factor affecting the stability in tall and slender structures such as masts and towers, (Sullins, 2006). Masts and towers are in three categories based on their height. They include the monopoles which could rise with its cross sectional area decreasing up to a height of 70m, the self-supporting towers, which is in the form of lattice structure and could be between 120m and 300m high, and the guyed masts, which is the tallest of all and could rise up to a height of 620m (Wahba, 1999). The selection of any of these masts and towers for any purpose is based on four major considerations. These factors include; (i) load, (ii) footprint, (iii) height and (iv) budget (Dinu, 2014). Load on masts and towers depend on their structural capability. The more - exposed the surface area of equipment, such as antennas, coaxial cables, brackets and other equipment mounted on the tower to wind, the more robust they are required. The tower footprint is the amount of free space on the ground that is required. Depending on the types of structure, towers and masts require more or less space for installation. Also, in practical terms, when a mast is supported by means of guy cables, it will allow for the construction of higher structures. Among all towers, monopoles have the smallest footprint, and are hence the most expensive towers. It is followed by self-supported towers and then guyed masts which require the largest footprints (Dinu, 2014). Engineers are faced with great challenges as per these tall and slender structures, and many experts have mentioned that "a guyed mast is one of the most complicated structures an engineer may be faced with". This assertion is a clear testimony to the happenings globally, that quite a number of collapse cases of masts and towers are seen, which is relatively far greater than other types of structures (Andersen, 2009).

Wind load is considered as a dynamic load and towers and masts are slender in nature, as a result they are always sensitive to dynamic loadings, hence, it becomes necessary to analyse them to determine their response to the wind load. In this study a self-supporting tower, 67.12m high, whose natural frequencies usually are well separated, and the response of the structure to wind gusts is governed by the fundamental mode of vibration is analysed under the influence of dynamic loads such as wind. This is achieved through simplified analysis procedures adopted using appropriate finite element analysis to determine the response of the structure to the wind loads from the different wind zones in Nigeria and in turn obtain the displacement of the joints of the tower as a result of the impact from these loads.

2.0. Review of Literatures

2.1. Loads on Masts and Towers

Wind load is considered the most dominant load on tall structures such as masts and towers, although in some locations, there are also the atmospheric icing of the structures which may have a great effect on their design. Furthermore, combining ice load with wind load may be critical for the design, this is one of the cases obtainable in some countries. Since the natural frequency of self-supporting towers are well separated, its response to wind gusts is determined by its mode of vibration. In this case, a simplified procedure of analysis is employed using the most suitable gust response factor. In view of this, care need to be taken during the design of these structures, most especially, the guyed mast which more complex and cumbersome than other types of towers (Andersen, 2009). The magnitude of wind load on masts and towers is proportional to the exposed area of the structure to the wind direction and as well the height of the structure from the ground. Hence, perforated shapes (grids and trusses) can offer low resistance to the wind load and are therefore preferred for the structure. As for the accessories, solid dishes are more vulnerable to wind and as a result it should avoided in windy environments (Dinu, 2014).

2.2. Descriptions of Telecommunication Masts and Towers

Several authors have classified masts and towers into three classes. This was done considering their height. The monopoles, self-supporting towers and guyed mast are the classification given in this regard. The tallest of all are the guyed masts which can rise up to a height above 600m (Wahba, 1999). The first and ever tallest is the Warsaw Radio Mast in Poland, 646.4m high, which was constructed since 1974 and collapsed in 1991 due to catastrophic failure as a result of renovation by replacing one of the cables (Meier, 2013). The free standing or self-supporting towers are next to the guyed mast in height. They can rise to a height between 120 and 300m. They are not supported by guys or cables as the guyed mast, rather they stand on their own with a lattice work of cross braces bolted to sloping vertical tower legs (three or four legs). Monopoles are shortest of all which are normally used as stands for floodlights and surveillance cameras. Plates 1, 2 and 3 show the three types of towers.

Plate I: Monopole (Dinu, 2014).

Plate II: Self-supporting tower (Sulin, 2006).

Plate III: Guyed Mast (Anderson, 2009).

2.3. Researches Involving Masts and Towers

Wahba et al (1996) used finite element method and modelled tower bars as 3D truss and 3D beam elements. They took into consideration the dynamic nature of loads such as wind, earthquake and cable galloping acting on the guyed mast and were able to obtain the structural models dynamic characteristics. The results obtained were compared to Wahba et al (1998) in which they performed an investigation of the numerical models used in telecommunication guyed steel towers. At the end they stressed the relevance of considering the non-linear effects present even at the service load levels. Albermani and Kitipornchai (2003) simulated the response of telecommunication and transmission towers using finite element method appliying the principles of geometrical and physical non-linear analysis. Amiri (1997) conducted a study to determine the sensitivity of seismic indicators for guyed towers to determine seismic indicators for guyed masts, that is, to see whether seismic effects will be important in the design of tall guyed towers. Eight existing towers varying in height from 150 to 607 m (492 to1991 ft) were subjected to three different seismic excitations to determine if there were any similarities present in the dynamic tower response.

3.0. Materials and Methods

This work takes into consideration a free standing or self-supported tower, which is a typical of those telecommunication towers widely used in Nigerian telecommunication industry. Yanda et al (2020) modelled a typical 4-leged tower using LinPro Computer Software and exposed to simulated wind loading from different wind zones across Nigeria as classified by Onundi et al., (2009). Nigerian wind zones were classified into five different zones with different basic wind speeds, which includes; 42m/s, 45.8m/s, 50m/s, 55m/s and 56m/s respectively for zones 1 to 5. These values were substituted in the wind load parameters to obtain the wind for across these zones in order to determine their influences on the tower and on the other hand the responses of the tower towards them. This work adopted the results of the tower modelling done by Yanda *et al* (2020).

The analysis focused on their behavior especially regarding the influence of environmental action, such as wind action which is the only environmental influence, with respect to loading, that is peculiar to Nigeria, and as well the combined effect of the structural capacity.

3.2. Description of the Tower

A 4-leged self-supporting steel telecommunication towers is selected for the evaluation in this work, which is designed for height of 67.12m. The reason for this choice is inherent to the wide usage of 4-legged self-supporting towers globally for the purpose of telecommunication (Rajasekkhara & Vijaya, 2014). The tower was modelled in the form of square cross section and provided with K-bracing at its down part and X-Bracing at its upper part of the tower. The screenshot of the modelled tower is shown in section four.

3.3. Loads on Towers

The basic loads considered in the work were the dead loads of all the elements, the imposed live loads, and the environmental load (wind load). The permanent loads on the steel tower, includes the dead weight of the structure (self-weight of structure), the ladders, the antennas and the platforms. Grey (2006) asserted that most of the researches undertaken are into the modelling of a spatial lattice tower as an equivalent beam element, but the self-weight of a lattice truss is substantially different to that of a beam element with similar behavioural properties. In order to control the dead load of the tower, the self-weight and mass of the tower (calculated from section properties of the equivalent beam) is set to zero. The correct values, calculated from the original lattice tower is then applied as additional loadings to the tower. All ancillary loadings are applied in a similar manner.

3.3.1. Dead / Imposed loads

The dead loads include the combination of tower self-weight, which is taken as 564.8kN, the antenna load is mounted on the tower at a height of 46m and it is taken as up to 10% of the dead load. Imposed loads on the platform is given as 2.5kN/m^2.

3.3.2. Wind loads

The wind load acting on towers is normally determined by applying the total horizontal wind force on each tower section at the centre of each of the sections (Travantry, 2001). In this work the structure is divided into eleven panels to enable the wind loading to be simulated on the tower accurately in order to undertake the global analysis. The wind load was determined in accordance with BS EN 1993-3-1 (2006), which provides the methodology for all the relevant calculations and states that in calculating the wind forces on towers, it should be divided into a number of sections, where a section can comprise several identical panels. Projections of bracing members in faces that are parallel to the wind directions, and in plan and hip bracing should not be included while determining the projected area of the tower. The following nomenclatures were used in determining the wind loads on the tower;

Wind load was determined using:

$$\mathrm{F} = \mathrm{Cf} \times \mathrm{qz} \times \mathrm{G} \times \mathrm{Af} \qquad (1)$$

Where, F, is the magnitude of the wind load, C_f, is the shape factor of the structure (tower), q_z, is the wind pressure, G, is the gust coefficient, which considers the resonance and lack of correlation of loads, and A_f, is the exposed area of the tower, perpendicular to the plane normal to the loads.

Equation (2) used to determine the wind pressure, q_z:

$$qz = 0.613V^2 \qquad (2)$$

Where, V^2, is the design wind speed

$$\mathrm{V}^2 = \mathrm{V} \times \mathrm{S1} \times \mathrm{S2} \times \mathrm{S3} \qquad (3)$$

Where, V, is the basic wind speed (42m/s, 50m/s and 55m/s were used for zone 1, zone 3 and zone 5 respectively), S_1, is topographic factor which equals 1(Except very exposed hill and valley shaped to produce a tunnelling of wind), S_2, is ground roughness factor for class C and value change as $S_2(z)$= if Z > 10m, $S_2 = (V_{et}/V_{e3})\ (V_{e3}/V_{10,3})\ ((H_Y)/10)^a$, S_3, is Statistic factor which equals 1, for 50 years return period. Here z is the height of the point considered. Table 1 shows the coefficients for calculating the wind load.

Table 1: Coefficients for calculating wind loads.

Coefficient	(G)	A_f	C_f	S_3	S_1	S_2
Value	0.85	0.57 m^2	2.42	1.00	0.85	1

Applying the above coefficients in Equation (2), wind pressure on the tower was determined using the basic wind speed from Nigerian wind zones shown in Table 2.

Table 2: Basic wind speeds in the zones (Onundi et al. 2009).

Basic Wind Speed	**Zone 1**	**Zone 2**	**Zone 3**	**Zone 4**	**Zone 5**
Minimum. (M/S)	35	42	45.8	50	55
MAXIMUM (M/S)	42	45.8	50	55	56

4.0. Results and Discussion

4.1. Results

The result of the modelled tower is as shown in Plate 1V, using the appropriate coordinates of joints and members. It is more of a free-standing cantilever. It has a 67.12m high with 31 joint and 54 members. The profile of the tower as modeled show its elevation sloping to a height of 42.72m, there by rising straight for the remaining height. The slope was produced, during the modelling with a difference of 0.4m between the width of a lower panel and a higher panel, while the height, with a difference 6.1m from one panel to another. Plate V shows the reactions at the support and the displacement at the joints of the tower during loading.

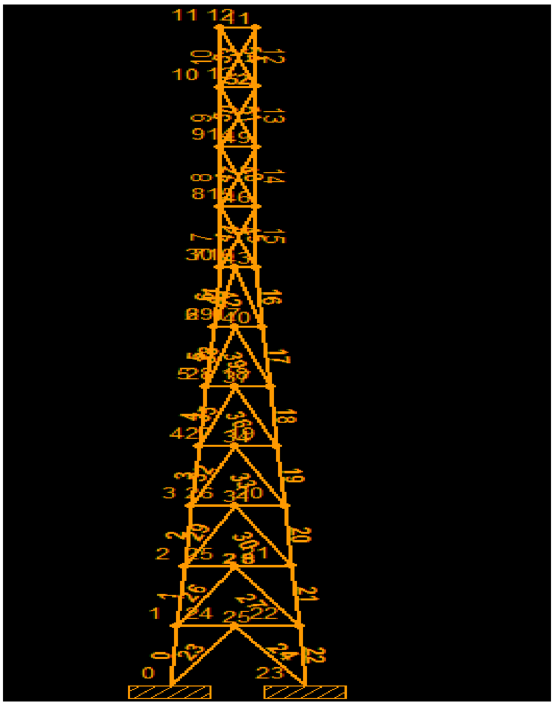

Plate IV: Modelled Tower

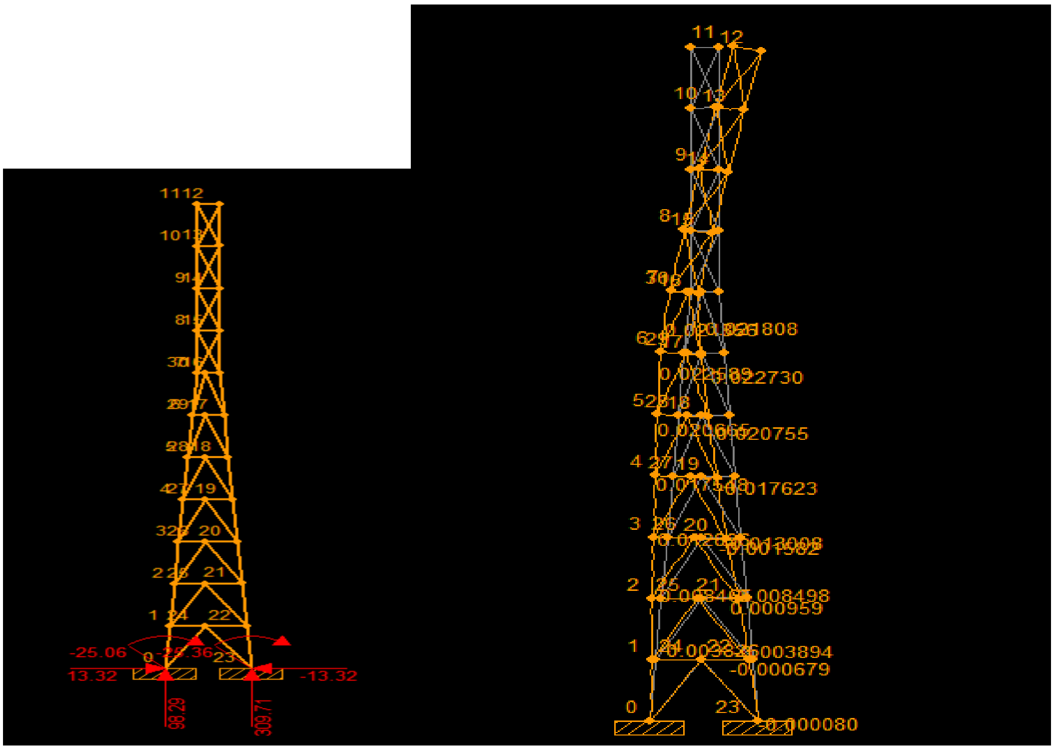

Plate V: Supports Reactions and Joints Displacement for wind load in Zone 5

Table 7: Effective Wind Pressure for 67.12m Tower using Nigerian Basic Wind Speed.

Panel no.	Bottom width (m)	Top width (m)	Height of panel (m)	Height of panel from bottom (m)	S_2	Effective wind pressure (KN/m^2)				
						Z1	Z2	Z3	Z4	Z5
1	10.3	9.72	6.1	6.12	0.98	1.23	1.34	1.41	1.55	1.63
2	9.72	9.14	6.1	12.2	0.98	1.24	1.34	1.45	1.55	1.68
3	9.14	8.56	6.1	18.3	1.05	1.26	1.36	1.46	1.65	1.71
4	8.56	8.13	6.1	24.4	1.08	1.36	1.39	1.46	1.69	1.77
5	8.13	7.87	6.1	30.5	1.10	1.39	1.44	1.55	1.73	1.80
6	7.87	7.45	6.1	36.6	1.11	1.41	1.58	1.57	1.77	1.85
7	7.45	7.14	6.1	42.7	1.13	1.45	1.59	1.60	1.78	1.88
8	7.14	6.78	6.1	48.8	1.14	1.49	1.59	1.61	1.79	1.90
9	6.78	5.62	6.1	54.0	1.14	1.49	1.62	1.62	1.80	1.94
10	5.62	4.50	6.1	61.0	1.16	1.56	1.66	1.68	1.83	1.97
11	4.5	2.0	6.1	67.12	1.17	1.56	1.69	1.72	1.84	1.99

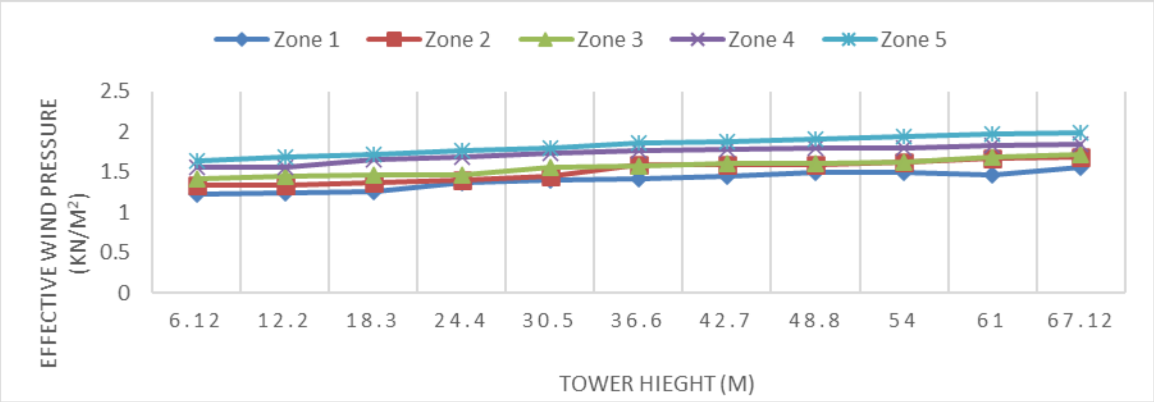

Fig.2: Relationship of effective wind pressure to tower height.

Figure 2 shows that the wind pressure for all the zones kept increasing along the height of the tower. Also, that the wind pressures on the tower from zone 5 has the highest values followed by zones 4, 3, 2 and 1. The pressure also began to exhibit significant increase at almost the mid-height of the tower where wind action is always considered to be more effective and may, to some extent, be capable of triggering collapse of the structure. The behaviour of wind load in this manner is a fluctuation due to turbulence and has frequency that is enough to excite resonant response on the structure (Dinu, 2014).

4.2. Joints Displacement

The joints displacements of the tower were determined when it was subjected to wind loads from the different zones within the country and a sudden removal of structural members was done at successive interval.

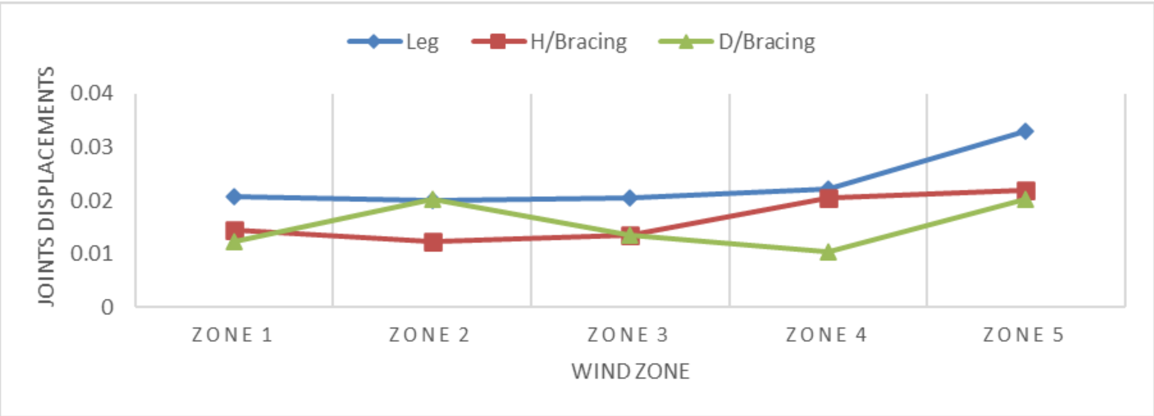

Fig. 3: Joints Displacement / Structural Members.

From figure 3 It can be observed that displacements of the tower under wind load in zone 5 are higher with respect to members' removal. Also, it would be concluded that the displacements are lateral displacement for all member removal except for the diagonal bracings. The displacement of joints for all the wind zone is seen to an extent that as capable of producing certain influence on the tower which is enough to trigger collapse of the structure.

5.0. Conclusions

On the basis of this study, the following conclusions can be made;

1. Wind load on towers in different wind zones in Nigeria behaves in a fluctuation manner due to turbulence and has frequency that is enough to excite resonant response on the towers.
2. Towers within zones 5, 4 and 3 of Nigerian wind zones are more vulnerable to collapse due to wind load owing to the fact that the wind speed is higher in this zone compared to other two zones.
3. The wind load in all the zones produces lateral displacement in the joints due to member removal with that of zone 5 producing the highest displacement of joints, hence the load can provide more frequency that is capable of triggering collapse on the towers within this zone as compared to other zones.

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