# NUMERICAL INVESTIGATION ON THE STRENGTHENING WORK OF TRANSMISSION TOWER

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NUMERICAL INVESTIGATION ON THE STRENGTHENING WORK OF TRANSMISSION TOWER

By

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# ABSTRAK

Kejadian angin ribut di Malaysia adalah suatu perkara yang tidak dijangka terutama sekali dengan keadaan Bumi yang semakin tua. Keutuhan bangunan dalam merekabentuk struktur di bawah angin ribut adalah penting bagi memastikan tiada kesan negatif yang boleh memberi impak yang besar terutamanya terhadap masyarakat. Oleh itu, menaik taraf pencawang elektrik adalah penting untuk mengatasi bencana ini. Secara umumnya, kaedah menguatkan menara ini adalah dengan menambah pendakap mendatar atau dikenali sebagai diafragma atau member sekunder. Dalam kajian ini, member yang kritikal dalam setiap panel telah dikenal pasti dan jenis diafragma yang paling berkesan dikesan melalui pelbagai analisis. Empat jenis diafragma diperkenalkan dalam kajian ini. Keputusan dalam kajian ini menunjukkan bahawa, jenis diafragma A adalah yang paling berkesan kerana daya yang rendah daripada kapasitinya dengan beban berat yang sedikit berbanding diafragma yang lain. Oleh itu, kaedah yang paling ekonomi telah dihasilkan dalam kajian ini dengan pengurangan kepada keupayaannya member tersebut.

# ABSTRACT

Windstorm occurrence in Malaysia is unpredictable as the world becomes old. The susceptibility in designing structures under wind storm is important to ensure no damages occurs which leads many negative impacts mostly towards the society. Therefore, an upgrading the transmission tower is important nowadays to overcome this wind disaster. Basically, the method on retrofitting of existing transmission tower is by adding the horizontal braces or as known as diaphragm or by secondary member. In this study, the critical member in each panelise been identify and the most effective type of diaphragm is being introduce through multiple analysis. Four types of diaphragm are introduced in this study. The results shown that, diaphragm type A is the most efficient as it can carry the same load with the minimize steel weightage. Lattice structure need to be upgrading by adding the diaphragm to eliminate the failure members. Diaphragm type A gave the lowest additional steel weightage with zero number of failed member. Hence, the most economical method was produced with a reduction on its used capacity.

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# LIST OF ABBREVIATIONS

| | |
|---|---|
| BS | **B**ritish **S**tandard |
| IS | **I**ndian **S**tandard |
| MS | **M**alaysian **S**tandard |

# NOMENCLATURES

| | |
|---|---|
| $A_e$ | Effective Projected Area |
| $C_f$ | Force Coefficient |
| $k$ | Exponent for a Short Period Gust |
| $k_1$ | Risk Factor |
| $k_2$ | Terrain Coefficient |
| $k_3$ | Local Topography |
| $L_e$ | Nominal Effective Length |
| $p_d$ | The Pressure on the Surface |
| $P_z$ | Design Wind Pressure |
| $r$ | Radius of Gyration |
| $V_b$ | Regional Basic Wind Velocity |
| $V_{gs}$ | Velocity at Gradient Height |
| $V_Z$ | Gust Velocity at Height Z |
| $V_z$ | Design Wind Velocity |
| $Z$ | Height Above the Ground. |
| $Z_g$ | Gradient Height |
| $\lambda$ | Slenderness |
| $\rho_c$ | Modified Compressive Strength |

# CHAPTER 1

# INTRODUCTION

## 1.1 Background

Windstorm can cause serious destruction to many types of structures such as houses, schools, and other structures. In recent years, numerous transmission tower lines have been developed as the surrounding areas continue growing rapidly. The demand of electric consumption increases equally with the development of a country. The design of transmission tower itself must be adequate to resist any form of failure (Walker, 1992). The wind loading is an important aspect that needs to be considered as it is the primary environmental load. A proper understanding of the windstorm characteristics is very important for safe and serviceable design. The effect of windstorm towards transmission tower is shown in the Figure 1.1 and Figure 1.2.

Figure 1.1: Transmission Line Tower collapsed in China caused by wind damage (Jiang et al. 2011).

Figure 1.2 shows that in some cases, the damage of the tower leg due to buckling occurred at the 2nd to 4th panels above the tower leg and the diagonal members in these panels failed completely. For the two failed transmission tower lines, no diaphragm was provided at the lower tower body section.

Figure 1.2: 500 kV transmission tower collapsed during severe storm (Cai et al., 2014).

In Malaysia, several cases of collapsed transmission towers were recorded such as transmission tower near Serendah, Selangor mainly due to vandalism where there were reports revealing that structural members of the tower that were suspected of being stolen (Kamarudin et al., 2107).

On the other hand, failure of transmission tower due to strong wind loading can be found in several countries in the world. In 2013, many transmission towers collapsed and power facilities destroyed by the typhoon Fitow in Zhejiang province of China (Tian et al., 2014). It was reported that that out of 94 structural failures in Australia that were recorded, more than 90% of the failure events were induced by severe thunderstorms (Hawes and Dempsey, 1993). Similar cases were also reported by Abdallah et al, (2008), Shehata and Damatty (2008) and Tamura (2009). This phenomenon shows that the structure stability contributes to the stability of the power transmission and distribution.

## 1.2 Problem Statement

Due to the climate change, change in land use and topography, the wind characteristic may be influenced. Damage caused by strong wind event wind event can be catastrophic. Generally, when a strong wind event hit an area, most of the structures are exposed to high loading regime. In some cases, the excitation of the wind speed exceeds the design capacity of the structural members. Transmission towers are tall structure with high eccentricity between members. Collapsed of this structure can cause major disruption to the power supply and affect the business activity and development of a nation. There are many transmission towers that have been constructed and to change these towers due to new wind loading regime can be very costly. As such, strengthening work by adding additional members to the towers is the best option to counter this type of upcoming problem.

## 1.3 Objectives

1. To identify the critical members in a transmission tower subjected to design load.
2. To determine the most efficient method to enhance the compression capacity of the transmission tower.

## 1.4 Scope of Work

This work is part of the consultation work given by the Utility Provider in Malaysia. Most of the data are confidential and cannot be published in open literature. The wind loading exerted on the tower will be calculated using IS 875: Part 3: 1987 and shall be compared with the design report provided by the Utility Provider. The loading tree that covers all the self-weight and reaction at the cross-arm shall be taken directly from the design report. The wind angle will be applied normal and parallel to tower and transmission lines only. The loading tree only considers normal condition. The modelling will use commercial software SAP2000 version 14.

# CHAPTER 2

# LITERATURE REVIEW

## 2.1 Overview

Transmission towers are susceptible to collapse in varying degrees. The response of a transmission tower during collapse event can be complex due to the high eccentricity between the structural members and there are many ways in which a local failure of a member may propagate from initial damage to its final stage. In this chapter, a concise overview on the finite element modelling, wind induced damaged on towers and past research on the modification and strengthening of transmission towers.

## 2.2 Windstorm

Battista et al. (2003) reported that the structural modelling of the chosen Transmission Line Tower (TLT) was based on observation of the system's behaviour and video images of some recent accidents in Brazil where the storm wind velocities reached values close to 100 km/h. The dynamic characteristics of the towers and the lateral movement of the electric cables have brought up the importance of fluid flow cables structure interaction when evaluating the towers behaviour under the action of wind forces. They also proposed a new analytical-numerical modelling for the structural analysis of TLT's, as originally proposed by Rodrigues (1999) and Rodrigues et al. (2000). Yasui et al. (1999) used this approached to study the differences in the behaviour of power lines supported by tension- or suspension-type transmission line towers. The overall results were used to unveil the mechanism of collapse and envisage a remedial measure to attenuate top horizontal displacements and overall stresses, which is the installation of non-linear pendulum-like dampers (NLPD) on the top of the TLT, like the ones that have been proposed by Pinheiro (1997), Battista et al. (1999) and Battista and Pinheiro (2000) for other slender and tall towers.

### 2.2.1 Windstorm Occurrence

As cited by Majid et al. (2016), EM-DAT reported that the total damage caused by natural disasters from 2012 to 2014 is higher in Asia than in other regions in the globe. The Malaysian Meteorological Department (MET) is committed in delivering early warning sign alerts on upcoming windstorm events through their website and social media. This proactive approach creates and increases awareness among Malaysians and minimizes windstorm impact. Majid et al. (2011) reported that windstorm predominantly damages houses in the northern region of Peninsular Malaysia. Windstorm predominantly occurs during April, May, and October, which cover the inter-monsoon period.

### 2.2.2 Damages from Windstorm Occurrence

An increase in severe windstorm events increases damage, losses, and even mortality. Damage, losses, and social problems are related to this natural disaster. Extensive damage was observed in rural non-engineered buildings in Penang. The highest and lowest numbers of houses damaged were recorded in Northern Seberang Perai (SPU) and Central Seberang Perai (SPT), representing 47% and 11% of the total houses damaged in Penang, respectively (Majid et al., 2016).

Transmission line structures are consistently governed by wind loading, which is a major concern to the design of transmission towers with the characteristics of lightweight, small rigidity and damping. As the global weather becomes changeable and unusual, it is frequently reported that a large majority of overhead transmission lines have failed due to wind disasters in China. Yang and Zhang (2007) reported that ince the year 2000, the wind damage of about one billion Renminbi have been recorded caused by about 30 accidents involving the failure of more than 100 transmission towers. In June 2005, a serious accident happened with the collapse of 10 towers in Renhuai 500 kV HUV transmission line and 5 towers in the 110kV transmission line nearby in Jiangsu province. In August in the same year, the wind attack resulted in the collapse of a transmission tower for a 110kV power line in Fujian province while in April 2006, two transmission towers supporting the 500 kV Gefeng transmission line fell in Hubei province from 2006 to 2008, hundreds of 500 kV, 220 kV and 110 kV transmission lines were out of operation and thousands of 35 kV and 10 kV transmission towers collapsed due to the huge wind loads in Guangdong province (Peng et al., 2010).

## 2.3 Transmission Tower

Transmission towers are a vital component and management needs to assess the reliability and safety of these towers to minimise the risk of disruption to power supply that may result from in-service tower failure. Latticed transmission towers are constructed using angle section members which are eccentrically connected. Factors such as fabrication errors, inadequate joint details and variation of material properties are difficult to be quantified (Albermani and Kitipornchai, 2003).

### 2.3.1 Structural Model of Transmission Tower

The geometry of a typical tower by Albermani and Kitipornchai (2003) is shown in Figure 2.1. The tower has a rectangular base of 6.5 m in the transverse direction, 3.55 m in the longitudinal direction and a height of about 39.0 m. The self-weight of the tower was 36 kN. Nine new revised loading conditions were used to evaluate the as-built tower response. The tower was modelled using 1100 elements and 730 nodal points. This gave a total of 4380 degrees-of-freedom. The ultimate load factors obtained for the nine revised loading conditions varied from 0.59 to 1.55 for the as-built tower. Results obtained from the nonlinear analysis revealed that the tower collapsed under three of the nine loading conditions. The collapse was due to either spread of plasticity or premature buckling.

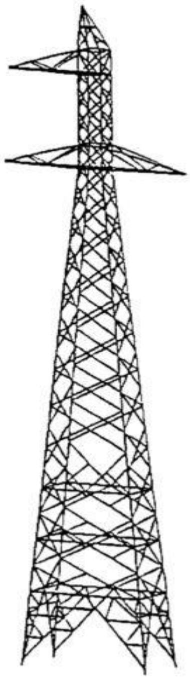

Figure 2.1: An existing 220-kV as built tower (Albermani and Kitipornchai, 2003).

### 2.3.2 Manual in Designing of Transmission Tower

Design practices for transmission towers are different from those for other steel structures in that stresses are permitted to be higher because towers are tested to their ultimate design strength and the designs incorporated modification based on test results. The two most widely used design specifications for the design of axially loaded angle members in self-supporting transmission towers are the ASCE Manual No. 52: 'Guide for Design of Steel Transmission Towers ' and the 'ECCS Recommendations for Angles in Lattice Transmission Towers '. The lattice tower structure is considered to consist of members supported by stress-carrying bracing and redundant members which are nominally unstressed. The design manuals specify limiting slenderness ratios for different member types to account for partial end restraint and joint eccentricity (Albermani and Kitipornchi, 1993).

### 2.3.3 Analysis of Transmission Tower

Stress calculations in a transmission tower structure are generally based on a linear elastic analysis, normally assuming that members are axially loaded and pin-connected, with the stiffer main leg members considered as continuous beams. Forces or stresses in the members are usually determined using a computer-aided method of analysis.

Albermani and Kitipornchai (1993) stated that two basic approaches have been used to develop computer programs for analysing transmission towers. The first approach translated the logic of conventional methods into routines to carry out the analysis of the structure. The second approach used structural analysis methods such as the stiffness method. Two computer programs available are based on a linear 3D elastic truss BPA TOWER and TRANTOWER.

#### 2.3.3.1. OpenSees v.2.4.0

By a study from Asgarian (2006), a three-dimensional nonlinear model of the transmission tower was developed using OpenSees v.2.4.0 platform. All members are modelled by “nonlinear Beam Column” frame element with “Fiber Section” object. The end connections with two or more bolts were treated as semi- rigid rather than ideal pin connection. Therefore, the frame ends were assumed to be moment connection. Steel02 nonlinear material model with a kinematic bilinear stress-strain curve is utilized in fiber section. The conductor loads were applied as point loads, however their masses were not included because of they cannot be lumped in nature. Damping of the structure was modelled using Rayleigh formulation as 2% of critical damping associated with the first and second modes of vibration. The self-weight of elements was also included as uniform loads along the members. In addition to the main model, a linear elastic model is constructed using SAP2000 structural analysis package to validate the OpenSees model in the elastic range (Asgarian, 2016).

#### 2.3.3.2. BPA Tower

The BPA TOWER program is a linear elastic truss analysis program adjusted to handle long, slender, tension-only bracing members. The analysis requires a certain number of iterations to determine which bracing members are loaded beyond their compression capacity and to remove such members from the model, thus forcing the remaining bracing members to carry the tensile load. The member response is determined via the use of a member performance curve obtained from a member performance data base gathered from available test results for single members.

#### 2.3.3.3. TRANTOWER

In the TRANTOWER program, members are assumed to be fully active when in tension and can sustain only a certain compression. The compression members are characterized as having a bilinear force-displacement relationship where the member buckling load is obtained using appropriate design formulae recommended by codes or design manuals.

#### 2.3.3.4. AK Tower

The numerical program AK Tower can confirm the use of a suitable diaphragm bracing system, depending on the tower structure and loading conditions. An upgrade scheme using diaphragm bracings was successfully implemented on an existing 105 m high TV tower. This scheme used less steel than the replacement of the existing diagonal bracings, was easier to implement in practice, and led to improved tower performance. Although no dynamic assessment of a tower retrofitted with diaphragm bracings was conducted, it is expected that such retrofitting will improve the tower's dynamic response since it enhances the stiffness without too much increase in mass.

## 2.4 Past Research on Upgrading, Strengthening Transmission Tower

Albermani and Kitipornchai (2003) proposed a nonlinear analytical technique to simulate and assess the ultimate structural response of latticed transmission towers. The method can also be used to assess the strength of existing towers, or to upgrade old and aging towers. The method has been calibrated with results from full-scale tower tests with good accuracy both in terms of the failure load and the failure mode. Using the AK TOWER program, they managed predict the ultimate structural behaviour of four different electric transmission towers tested in Australia.

Albermani et al. (2004) conducted experimental work and validated the analytical investigation on 105 meter high TV tower. Experimental work was carried using several types of diaphragms as shown in Figure 2.2. The results showed that bracing type 2a was the most efficient strengthening technique. The analytical predicted showed good agreement with the experimental data.

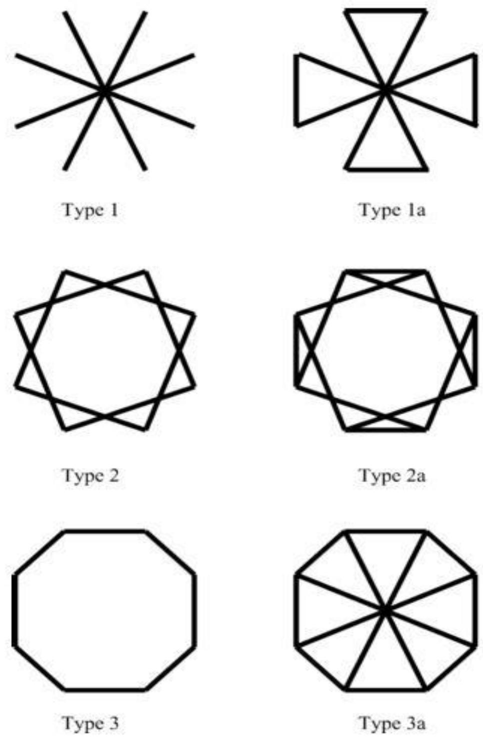


Figure 2.2: The type of diaphragm bracings (Albermani et al., 2004).

## 2.5 Summary

Based on the past research work, the failure of transmission tower due to abnormal wind loading condition is a global issue. The repair work can be very costly. Several strengthening techniques were proposed either experimentally or numerically. However, they are many types of transmission towers based on the rated capacities, height and design consideration. Furthermore, the strengthening method using diaphragms provide very limited information on the percentage of the additional steel used compared to the original model.

# CHAPTER 3

# METHODOLOGY

## 3.1 Overview

This chapter presents the numerical procedures involved in modelling the transmission tower subjected to wind and cable loading. The finite element simulation employed the SAP2000 version 14 software package. On the other hand, the wind calculation was derived using IS 875: Part 3: 1987. The loading tree applied on the cross arms were taken directly from a design report. However, some of the details cannot be revealed due to legal issues.

## 3.2 Typical Arrangement of Transmission Tower

Figure 3.1 shows the front view of the transmission tower. The overall height of the transmission tower is 30.48 meter. The height of each panel is shown in the figure. The tower is divided into 13 panels and 4 sets of cross arms. Generally, the transmission tower is constructed using equal angle sections. The lower section consists of inclined members, commonly known as the ‘tower leg’ while the upper section is commonly known as ‘tower’. The geometry of the transmission tower can be treated as lattice structure with main and secondary (internal) members.

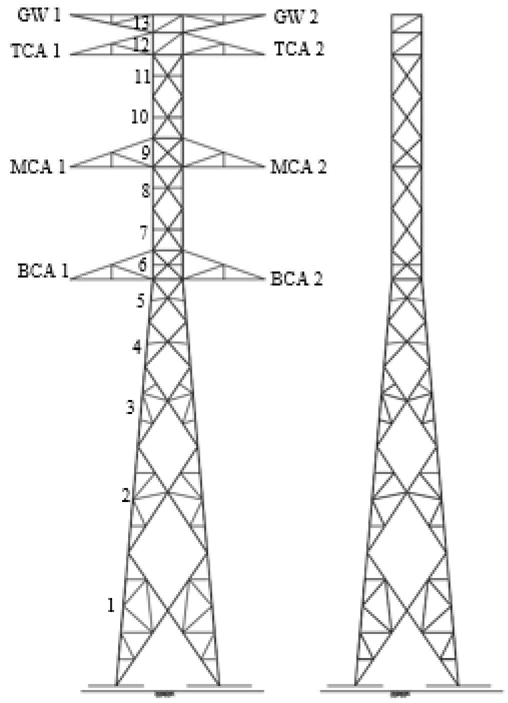


Figure 3.1: Front view of transmission tower

The flowchart of the step involve in overall study is shown in the Figure 3.2.

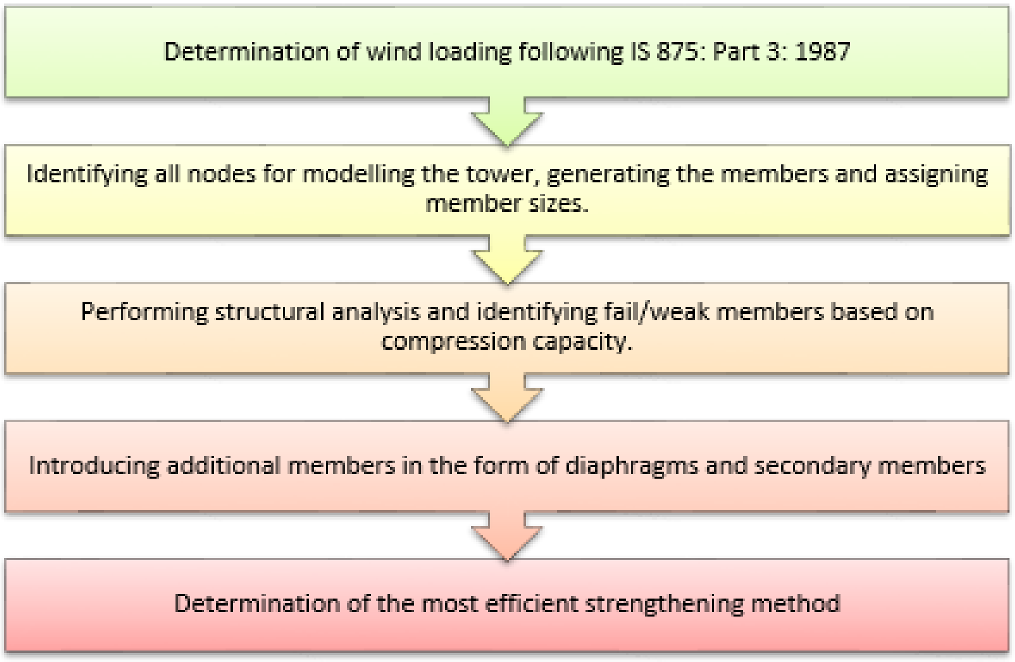


Figure 3.2: Flowchart of the study.

### 3.3 Wind Load

The wind load on the tower was calculated using the Indian standards IS 875: Part 3: 1987 and the analysis was carried out using BS 8100: Part 1: 1996. The maximum basic wind speed 33.5 m/s is chosen as stated in Malaysia Zone I MS 1552 2002. The zonation is shown in Appendix A1. The design wind speed, $\boldsymbol{V_z}$ was modified to induce the effects of:

i. risk factor ($k_1$),

ii. terrain coefficient ($k_2$),

iii. local topography ($k_3$).

Hence, the design wind speed, $V_z$ is

$$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3 \tag{3.1}$$

The parameter $k_1$, $k_2$, and $k_3$ represents multiplying factor to account for chosen probability of exceedance of extreme wind, terrain category and height, local topography and size of gust, respectively. The wind loading calculated on each was done using tributary area as shown in Appendix A2 while Appendix A3 shows the wind loading at each member by panels. Several assumptions have been made in this analysis such as;

i. Each panel have an addition load 15% was introduced due to the presence of gusset, bolt and nut.

ii. For leg members, an additional 10% was introduced due to the presence of secondary members.

iii. The analysis only covers normal condition for loading tree at the cross-arm.

### 3.3.1 Risk Probability Factor ($k_1$)

The life period and the corresponding $k_1$ factors for different classes of structures for design are included in Table 3.1. The factor $k_1$ was based on the statistical concepts which take into account of the degree of reliability required within the return period. In other words, whatever wind speed is adopted for design purposes, there is always a probability that may be exceeded in a storm of exceptional violence.

Table 3.1: Risk coefficients.for different classes of structures in different wind speed zones (IS 875: Part 3: 1987).

| Classes of Structure | Mean probable design life of structure in years | $k_1$ for each basic wind speed | | | | | |
|---|---|---|---|---|---|---|---|
| | | 33 | 39 | 44 | 47 | 50 | 55 |
| 1.All general buildings and structures | 50 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| 2.Temporary sheds, structures such as those used during construction operation (for example, form-work and falsework), structures in construction stages and boundary walls | 5 | 0.82 | 0.76 | 0.73 | 0.71 | 0.7 | 0.67 |
| 3.Buildings and structures presenting a low degree of hazard to life and property in event of failure, such as isolated towers in wooded areas, farm buildings except residential building | 25 | 0.94 | 0.92 | 0.91 | 0.9 | 0.9 | 0.89 |
| 4.Important buildings & structures like hospitals, communications buildings or towers, power plant structures. | 100 | 1.05 | 1.06 | 1.07 | 1.07 | 1.08 | 1.08 |

### 3.3.2 Terrain Categories Factor ($k_2$)

Selection of terrain categories was made with due regards to the effect of obstructions which constitute the ground surface roughness. Four categories were recognised as shown in Table 3.2.

Table 3.2: Terrain categories (IS 875: Part 3:1987)

| Category | Description |
|---|---|
| 1 | Exposed open terrain with few or no obstructions and in which the average height of any object surrounding the structure is less than 1.5m.<br>- Open sea coasts and flat treeless plains |
| 2 | Open terrain with well scattered obstructions having heights generally ranging from 1.5 to 10m<br>-Air fields, open parklands and undeveloped sparsely built-up outskirts of towns and suburbs. |
| 3 | Terrain with numerous closely spaced obstructions having the size of buildings or structures up to 10m in height with or without a few isolated tall structures<br>-Well-wooded areas and suburbs, towns and industrial areas fully or partially developed. |
| 4 | Terrain with numerous large high closely spaced obstructions<br>- Large city centres and well-developed industrial complexes. |

The variation of wind speed with height of varied sizes of structures depends on the terrain category as well as the type of structure. The buildings/structures are classified into the following three different classes depending upon their size:

i. Class A - Structures and/or their components such as cladding, glazing, roofing, etc., having maximum dimension (greatest horizontal or vertical dimension) less than 20 m.
ii. Class B - Structures and/or their components such as cladding, glazing, roofing, etc., having maximum dimension' (greatest horizontal or vertical dimension) between 20 and 50 m.
iii. Class C - Structures and/or their components such as cladding, glazing, roofing, etc., having maximum dimension (greatest horizontal or vertical dimension) greater than 50 m.

Table 3.3 shows the multiplying factor by which the reference wind speed should be multiplied to obtain the wind speed at different heights, in each terrain category for different classes of structures.

Table 3.3: Factors to obtain design wind speed variation with height in different terrains for different classes of buildings structures. (IS 875: Part 3: 1987)

| Height | Terrain Category Class 1 | | | Terrain Category Class 2 | | | Terrain Category Class 3 | | | Terrain Category Class 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| (m) | A | B | C | A | B | C | A | B | C | A | B | C |
| 10 | 1.05 | 1.03 | 0.99 | 1 | 0.98 | 0.93 | 0.91 | 0.88 | 0.82 | 0.8 | 0.76 | 0.67 |
| 15 | 1.09 | 1.07 | 1.03 | 1.05 | 1.02 | 0.97 | 0.97 | 0.94 | 0.87 | 0.8 | 0.76 | 0.67 |
| 20 | 1.12 | 1.1 | 1.06 | 1.07 | 1.05 | 1 | 0.98 | 0.98 | 0.91 | 0.8 | 0.76 | 0.67 |
| 30 | 1.15 | 1.13 | 1.09 | 1.12 | 1.1 | 1.04 | 1.03 | 1.03 | 0.96 | 0.97 | 0.93 | 0.83 |
| 50 | 1.2 | 1.18 | 1.14 | 1.17 | 1.15 | 1.1 | 1.09 | 1.09 | 1.02 | 1.1 | 1.05 | 0.95 |
| 100 | 1.26 | 1.24 | 1.2 | 1.24 | 1.22 | 1.17 | 1.17 | 1.17 | 1.1 | 1.2 | 1.15 | 1.05 |
| 150 | 1.3 | 1.28 | 1.24 | 1.28 | 1.25 | 1.21 | 1.21 | 1.21 | 1.15 | 1.24 | 1.2 | 1.1 |
| 200 | 1.32 | 1.3 | 1.26 | 1.3 | 1.28 | 1.24 | 1.24 | 1.24 | 1.18 | 1.3 | 1.22 | 1.13 |
| 250 | 1.34 | 1.32 | 1.28 | 1.32 | 1.31 | 1.26 | 1.26 | 1.26 | 1.2 | 1.3 | 1.26 | 1.17 |
| 300 | 1.35 | 1.34 | 1.3 | 1.34 | 1.32 | 1.28 | 1.31 | 1.28 | 1.22 | 1.3 | 1.26 | 1.17 |
| 350 | 1.37 | 1.35 | 1.31 | 1.36 | 1.34 | 1.29 | 1.32 | 1.3 | 1.24 | 1.31 | 1.27 | 1.19 |
| 400 | 1.38 | 1.36 | 1.32 | 1.37 | 1.35 | 1.3 | 1.34 | 1.31 | 1.25 | 1.32 | 1.28 | 1.2 |
| 450 | 1.39 | 1.37 | 1.33 | 1.39 | 1.37 | 1.32 | 1.36 | 1.33 | 1.28 | 1.34 | 1.3 | 1.21 |
| 500 | 1.4 | 1.38 | 1.34 | 1.39 | 1.37 | 1.32 | 1.36 | 1.33 | 1.28 | 1.34 | 1.3 | 1.22 |

The multiplying factors in Table 3.3 for heights well above the heights of the obstructions producing the surface roughness, but less than the gradient height, are based on the variation of gust velocities with height determined by the following formula based on the well-known power formula explained earlier:

$$V_z = V_{gs}\left(\frac{Z}{Z_g}\right)^k = 1.35 V_b \left(\frac{Z}{Z_g}\right)^k k_3 \qquad (3.2)$$

$V_Z$ = Gust velocity at height Z

$V_{gs}$ = Velocity at gradient height

$k$ = Exponent for a short period gust

$Z_g$ = Gradient height

$V_b$ = Regional basic wind velocity

$Z$ = Height above the ground

### 3.3.3 Topography Factor ($k_3$)

The effect of topography will be significant at a site when the upwind slope ($\theta$) is greater than $3^o$, and below that, the value of $k_3$ may be taken to be equal to 1.0. The value of $k_3$ varies between 1.0 and 1.36 for slopes greater than $3^o$.

The influence of topographic feature is considered to extend 1.5 $L_e$ of upwind and 2.5 $L_e$ of summit or crest of the feature, where $L_e$ is the effective horizontal length of the hill depending on the slope. The values of $L_e$ for various slopes are given in Table 3.4.

Table 3.4: Variation of effective horizontal length of hill and upwind slope $\theta$

| Slope, $\theta$ | $L_e$ |
|---|---|
| $3^0 < \theta < 17^0$ | L |
| $> 17^0$ | Z/0.3 |

Note: L is the actual length of upwind slope in the wind direction, and Z is the effective height of the feature.

If the zone downwind from the crest of the feature is relatively flat ($\theta < 3^o$) for a distance exceeding $L_e$, then the feature should be treated as an escarpment. Otherwise, the feature should be treated as a hill or ridge. The topography factor, $k_3$ is given by the equation: -

$$k_3 = 1 + Cs \tag{3.3}$$

The parameter C is the value appropriate to the height H above mean ground level and the distance 'x' from the summit or crest relative to effective length $L_e$ as given in the Table 3.5. The factor 's' was determined from Figure 3.3 for cliffs and escarpments and Figure 3.4 for ridges and hills.

Table 3.5: Variation of factor C with slope θ

| Slope, θ | Factor C |
|---|---|
| $3^0 < \theta < 17^0$ | 1.2(Z/L) |
| $> 17^0$ | 0.36 |

Note: L is the actual length of upwind slope in the wind direction, and Z is the effective height of the feature.

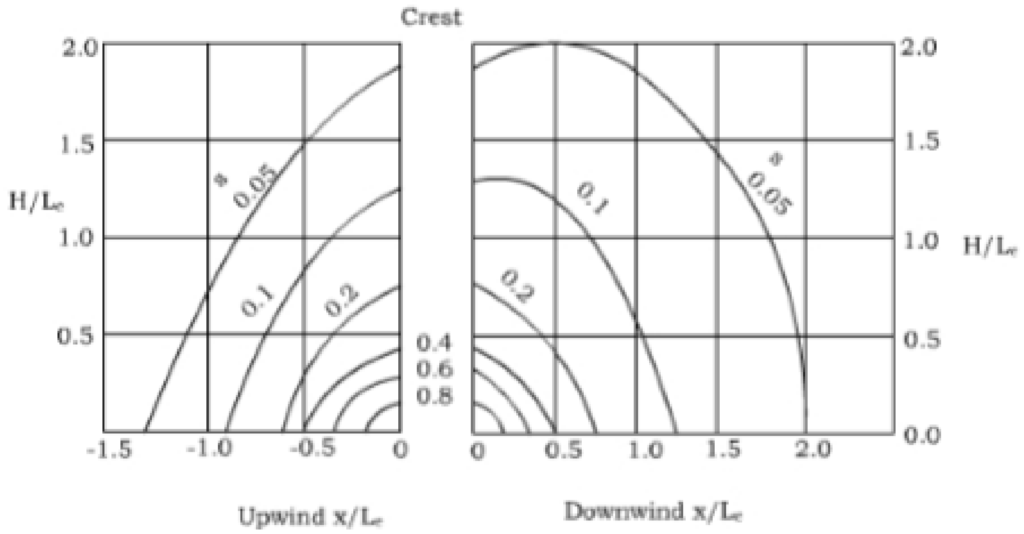


Figure 3.3: Factors for cliff and escarpment (IS 875: Part 3: 1987).

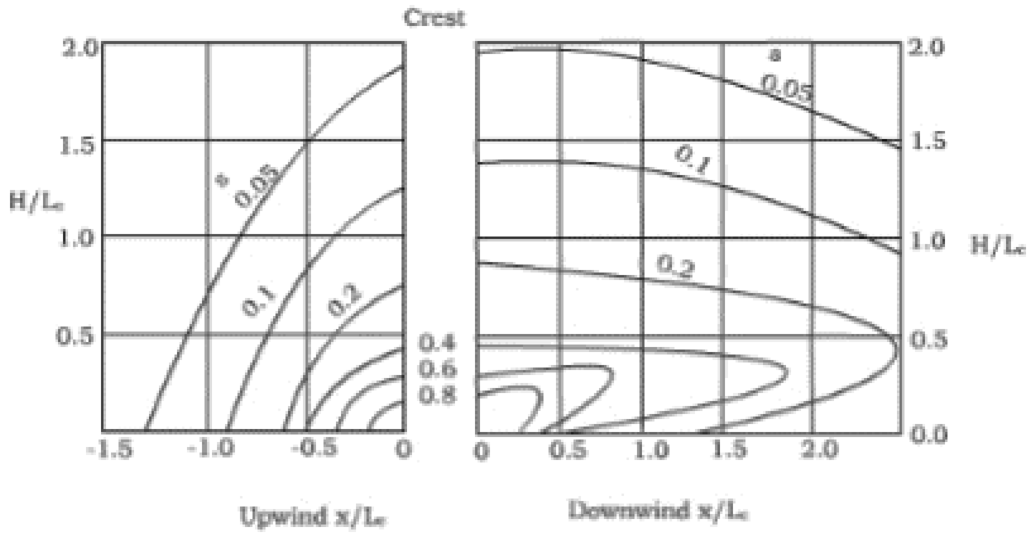


Figure 3.4: Factors for ridge and hill (IS 875: Part 3:1987).

## 3.4 Design Wind Pressure

The design wind pressure $P_z$ at any height above mean ground level is obtained by the following relationship between wind pressure and wind velocity:

$$P_z = 0.6\, V_z^2 \tag{3.4}$$

where,

$P_z$ = Design wind pressure in N/m$^2$

$V_z$ = Design wind velocity in m/s

The coefficient 0.6 in the above formula depends on several factors and mainly on the atmospheric pressure and air temperature. The pressure of each point is affected by its solidity ratio, height, and overall coefficient by panel. Therefore, each of panel and point give different values. The calculated values are shown in Appendix A2.

## 3.5 Wind Force on The Structure

The major portion of the wind force on the tower is due to the wind acting on the frames, the conductors and ground wires. The wind pressure acting on the tower is depending on its $k_2$ value and solidity ratio. The force on a structure is given by;

$$F = C_f A_e p_d \tag{3.5}$$

where,

$C_f$ = Force coefficient

$A_e$ = Effective projected area

$p_d$ = Pressure on the surface

### 3.6 Wind Force on Lattice Tower

The wind load on a square tower can either be calculated using the overall force coefficient for the tower given in Tables 3.6, using the equation 3.5 or calculated using the cumulative effect of windward and leeward trusses from the equation: -

$$F = C_f\,(1 + \psi)\,A_e p_d \tag{3.6}$$

In this research, the wind load on a square tower was calculated by using the overall force coefficient and equation 3.5. Force coefficients for lattice towers of square or equilateral triangle sections with flat-sided members for wind direction against any face are given in Table 3.6.

Table 3.6: Overall force coefficients, $C_f$ for towers composed of flat-sided members.

| Solidity ratio(∅) | Force coefficient for | |
|---|---|---|
| | Square towers | Equilateral triangular towers |
| 0.1 | 3.8 | 3.1 |
| 0.2 | 3.3 | 2.7 |
| 0.3 | 2.8 | 2.3 |
| 0.4 | 2.3 | 1.9 |
| 0.5 | 2.1 | 1.5 |

By considering overall coefficient for lattice tower, Table 3 was used for determining $C_f$ based on the Solidity Ratio. Using a linear interpolation, $C_f$ was derived as shown below: -

$$\frac{0.4\text{- }0.305}{0.305\text{-}0.3} = \frac{2.3\text{- }x}{x\text{-}2.8}$$

$$x = 2.78$$

$$C_f = 2.78$$

Each panel have different ∅ due to its projected area of its individual elements and area enclosed by the boundary of the frame normal to the wind direction. Therefore, the solidity ratio of the transmission tower by each panel was calculated and summarized in Table 3.7. The example of calculation for ∅ in Panel 13 is shown below:

$$\textbf{Solidity Ratio } (\emptyset) \tag{3.7}$$

$$= \frac{\text{Projected area of all the individual elements}}{\text{Area enclosed by the boundary of the frame normal to the wind direction}}$$

$$= \frac{(2 \text{ x } 0.065 \text{ x } 0.8)+(2 \text{ x } 0.05 \text{ x } 1.561)+(0.05 \text{ x } 1.34)}{1.34 \text{ x } 0.8}$$

$$= 0.305$$

Table 3.7: Solidity Ratio for Each Panel

| Panel | H | (∅) |
|---|---|---|
| 13 | 30.48 | 0.305 |
| 12 | 29.68 | 0.272 |
| 11 | 28.68 | 0.215 |
| 10 | 26.8 | 0.239 |
| 9 | 24.92 | 0.228 |
| 8 | 23.62 | 0.253 |
| 7 | 21.74 | 0.277 |
| 6 | 19.86 | 0.265 |
| 5 | 18.56 | 0.214 |
| 4 | 16.65 | 0.178 |
| 3 | 14.65 | 0.134 |
| 2 | 11.05 | 0.101 |
| 1 | 6.25 | 0.078 |

## 3.7 Modelling and Analysing

As transmission tower is a complex structure, the modelling was done firstly using AutoCAD. This approach helped in reducing time and generated precise detailing before analysing using software SAP2000. The steps are shown in Figure 3.5.

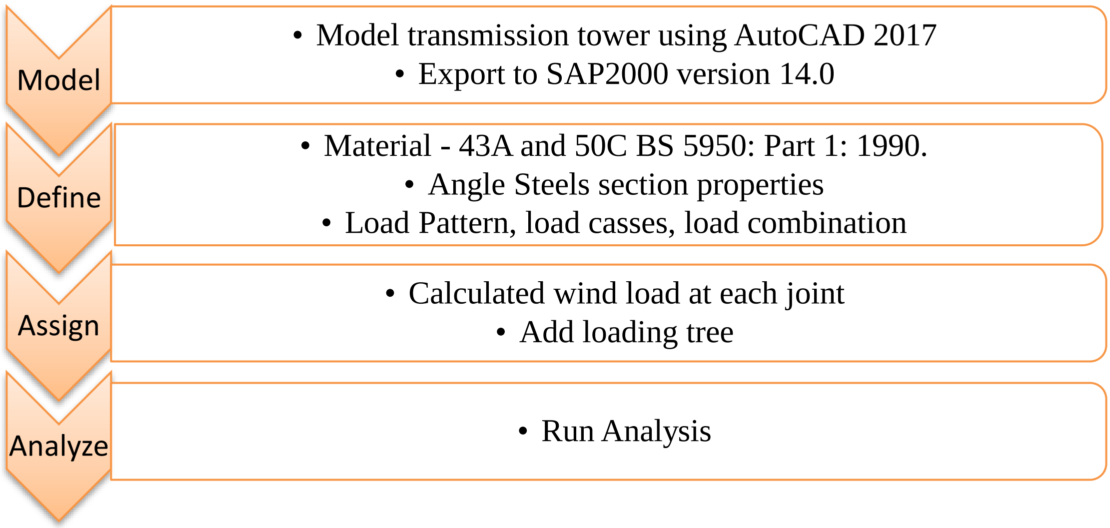


Figure 3.5: Modelling and analysis process

There are 2 types of steel grade been defined in this study which is 43A and 50C or also known as S275 and S355. Table 3.8 shows the section properties and the respective yield strength ($f_y$). The details of the arrangement were kept confidential.

Table 3.8: Section properties and yield strength, $f_y$.

| Section Properties | $f_y$ (N/mm$^2$) |
| --- | --- |
| 45×45×5 | 275 |
| 50×50×5 | 275 |
| 50×50×6 | 275 |
| 60×60×6 | 275 |
| 65×65×6 | 275 |
| 90×90×6 | 355 |
| 100×100×7 | 355 |
| 100×100×8 | 355 |