Addis Ababa University
Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

STRUCTURAL MODELLING AND ANALYSIS OF DOUBLE CIRCUIT LATTICE POWER TRANSMISSION TOWER SUBJECTED TO BLAST LOADS

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A Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

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Adviser: Dr. Shifferaw Taye

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The undersigned have examined the thesis entitled '**Structural Modelling and Analysis of Double Circuit Lattice Power Transmission Tower Subjected to Blast Load**' presented by **Amanuel Tesfaye**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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Advisor	Signature	Date
Internal Examiner	Signature	Date
External Examiner	Signature	Date
Chairperson	Signature	Date

UNDERTAKING

I certify that research work titled "Structural Modelling and Analysis of Double Circuit Lattice Power Transmission Tower Subjected to Blast Load" is the author's work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged.

Amanuel Tesfaye

ABSTRACT

A study on the effects of blast loads on tower-line coupling, single tower structure and tower-line structure system has been conducted and a variety of structural response variables have been compared. Time history analysis has been carried out by considering different charge blast parameters and stand-off distances. Various combinations of charge weights and stand-off distances have been considered both on coupled- and uncoupled tower arrangements along with detailed study of the effects of conductor coupling during the analysis modeling and subsequent design of transmission towers.

The modular and powerful finite element analysis software Dlubal-RFEM has been utilized for the blast-oriented linear implicit dynamic analysis of latticed transmission line structures covered in this study. The transient dynamic equilibrium equations of the tower have been directly solved by Newmark time integration method. The various internal forces, displacements and a variety of dynamic response behaviors have been isolated, recorded and compared for a single tower and tower-line coupled systems under different charge weight and stand-off scenarios. When numerically modeling and analyzing the structure, both coupled and uncoupled tower-line arrangements, three-dimensional natural vibration analysis has been taken into account.

Findings from numerical experiences on several analysis models have indicated that the tower-line coupling system significantly influences the response of tower structures with particular significance on tower supports. Current design codes and guidelines have few, if any, methods or recommendations to deal with power transmission towers subjected to blast loads and, when they do exist, they do not comprehend corresponding tower-line coupled effects. The study has indicated that internal forces in members of uncoupled towers are generally larger than the corresponding values in coupled systems. The main difference arises in the support reactions. Several study models from this work have indicated that there is up to nearly four-fold increase in support forces in uncoupled systems which subsequently greatly influence the planning, selection, design and detailing of support systems for such structures. Thus, the study has revealed the fact that establishing support forces by considering a single uncoupled tower will lead to underestimating critical loads. When designing such structures, coupling effects should be incorporated in the analysis process.

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Finally, I must express my very profound gratitude to my family for providing me with unfailing support and continuous encouragement throughout my years of study and through the process of researching and writing this thesis. This accomplishment would not have been possible without them. Thank you.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1 Introduction

1.1 Background

Terrorist attacks on energy infrastructure are more common than many might think. According to a report by the Electric Power Research Institute of USA, which researches issues related to the electric power industry in the US, from 1996 – 2006 there were approximately 2500 attacks from terrorist groups against transmission lines and towers in various parts of the world and 500 attacks on substations (Heidi V, 2014).

Blast proofing structures is becoming more essential in high-security structures due to safety measures against maliciously intended terrorist attacks. Designing embassies and some important government buildings may be classified as high-security risk structures and may have to be designed against blast loads. Power transmission towers being classified as an important structure should be checked against such loads. Even though the probability of life loss is virtually null, the disruption caused by the failure of power transmission lines is not negligible.

The main objective of designing protective structures is to provide safe structures against loads, to protect structural components, as well as valuable assets on the structure. The analysis and design of structures prone to explosion require a thorough understanding of the effects and behavior of blast loads, blast phenomena, and dynamic response of structures. Members of a steel lattice power transmission tower forming part of the frame are essential structural load-carrying elements, failure of which may lead to significant damage or worse progressive collapse and cascading failure.

Considering the effect of blast load on structures was limited to military purpose structures. Information regarding explosives was classified. But due to the aforementioned reasons, the need to assess' prominent civil structures has become mandatory. In 1990 the US Army published a comprehensive paper describing the effects of blast loads and how to design for it (TM 5-1300, 1990). Vast research has been conducted on explosion behaviors since then. Virtual laboratories (simulation programs) have helped advance studies in the field. Several numerical and experimental studies on steel lattice power transmission tower have been conducted giving focus to the wind, ice, and earthquake loading. Which some are mentioned in the literature review. The response of power transmission lines subjected to blast loading is not parametrically studied, forming a knowledge gap.

Most towers are not designed to withstand severe loading to an explosive. Therefore, there has been a requirement to assess the behavior of structures and their components under explosion loading. An extreme load such as blast and impact applied in a short duration can change the failure mode of steel components. Steel members that are designed for flexural failure mode under static load may experience buckling failure under extreme loads.

Previous studies focused on wind, ice, earthquake, and cable galloping. Because these are the common loads on tower structures. Experimental analysis is required in some countries as safety verification. Transmission towers in overhead high voltage lines are critical elements of the power transmission grid. Despite their crucial function, many of these structures are installed in remote regions which makes them exposed to terrorist attacks. Extensive research is yet needed to understand the vulnerability of lattice towers to manmade disasters.

There have been documented incidents of lattice transmission towers subjected to blasts in North America. And there is no guarantee it won't happen elsewhere. The blast resulted from minor explosions that only caused localized structural damage instead of catastrophic power failure. On March 14, 1994, a Hydro-Quebec 131-foot-high transmission tower carrying a 230-kV line was damaged by an explosion (Hamilton, 1994). It is futile trying to monitor the security of a vast rural transmission system. And rendered impossible for the utilities to protect each of their thousand power pylons spread across regions.

The dynamic properties of transmission towers coupled with conductors when subjected to blast loading should be studied to clarify the effect blast has on power transmission lines.

1.2 Statement of the problem

A lot of detailed studies are conducted regarding steel latticed power transmission tower subjected to wind, ice, and earthquake loading. Design is done for these parameters. Because these are the most common and recurring loads on steel towers. But there is a lack of sufficient study around steel latticed power transmission tower subjected to blast loading. The effect of transient dynamic load and tower line coupling behavior in these structures has a significant role in the response of the tower. Analyzing and designing for blast loading in most design guides are done for building structures. There is no explicit study conducted on blast loaded transmission tower line systems.

1.3 Aim of the Study

1.3.1 General Objective

The objective of this research project is to assess the blast behavior of steel lattice power transmission towers subjected to open explosion threat scenarios. The main focus is placed on studying dynamic characteristics of tower line coupled systems. Time history analysis will be implemented to check the dynamic properties of the structure.

1.3.2 Specific Objective

- Review of literature on blast loads and dynamic response of steel latticed frame structures.
- Finite element time history analysis of a modeled power transmission tower with a conductor. Verifying the numerical model analytically and against published experimental results.
- Studying the effect of explosion charge location and mass. To investigate the dynamic responses of the tower line coupled system.
- The paper will show the effect of modeling techniques on the output of the result.

1.4 Scope of Research

This research focuses on the finite element modeling of a steel lattice power transmission tower-conductor and determining its behavior when subjected to blast loading. The blast load is kept minimum to produce impact based load and not cause member failure. Also, temperature effects are not covered in the study. Different blast parameters and the tower line coupling effect is checked on the study.

1.5 Organization of the Thesis

- Chapter one gives background information about the motivation and objective of the thesis. A general introduction is given in this chapter.
- Chapter two is a collection of literature paraphrased to explain the principles of blast loading, geometric and material modeling of power transmission tower-line systems, and dynamic analysis methods.
- Chapter three discusses the methodology and input parameters used in the study. It discusses in detail about properties of the structures. Verification of the modeling technique is done for both tower and conductor individually. The method of analysis is also described briefly.
- Chapter four is about analysis results and their change concerning different parameters. Mainly the effect of charge mass, standoff distance, and tower-line coupling are discussed.
- Chapter five gives a conclusion statement attained from the dynamic property study of power transmission structures. Finally, it recommends the need for dynamic analysis of blast loaded transmission structures and future study areas to consider.

1.6 Research Significance and Contribution

The vibration response of blast loaded tower structures is studied. This paper gives a baseline for the next researchers who are interested in the field. The significance of tower-line coupling in tower support loads is elaborated. It also recommends modeling and analysis methods. It provides important input for design guidelines and codes.

CHAPTER 2 Literature Review

Explosions occur when energy is released rapidly. This sudden release of energy is manifested in the form of light, sound, and pressure. Which all are termed collectively as an explosion. The pressure of the explosion creates a shock wave that will spread outwards spherically from the point of detonation. The speed of this process is supersonic ending after a few milliseconds.

2.1 Previous Researches

(TM 5-1300, 1990), a manual titled “Structures to resist the effects of accidental explosions” which guides designers, the step-to-step analysis, and design procedure. Including the information on such items (1) blast, fragment, and shock loading. (2) The principle of dynamic analysis. (3) Reinforced concrete and structural steel design and (4) Several special design considerations. Guides for electrical transmission line structural loading such as (ASCE 10-15, 2015) and (EN 50341-1, 2012) provides loading guidelines for extreme ice and wind as well as security and safety loads. These guidelines use reliability based procedures and allow the design of transmission line structures to incorporate specified levels of reliability depending on the importance of the structure.

The majority of the research presenting information on structural response to overpressures due to blasts is focused on a bridge and building structures subjected to large-scale explosions. In his research Rittenhouse (Rittenhouse T, 2001) discusses the most important spatial blast loading property that affects steel structures is direct loading, distributed along the length of the member, and not an end loading as is the case for gravity or seismic loads. This is particularly troubling for lattice towers as the majority of the structural members (legs, braces, etc.) are single angle members modeled as truss type members that only carry concentric axial compression or tension except for minor climbing load.

Bo Chen et al (Chen B, 2014), presented a state of the art review on the dynamic analysis and control of the transmission tower-line system in the past forty years. Dynamic modeling, analysis, and simulation techniques were reviewed in the paper. Vibration effect and control are discussed briefly. Aravind (Aravind S, 2014), conducted a parametric study on cascading failure properties of electrical transmission lines. The study was performed on power transmission lines, whose supporting towers were modeled with (a) linear elastic truss and beam element, (b) moment-curvature beam elements with elastic-plastic material properties, and (c) towers with load-limiting devices or tower load controllers. Free vibration analysis was made considering the damping parameter. It was concluded from the paper that linear material could not predict the failure or cascading of the transmission line tower.

Dai K (Dai, 2009), In his Ph.D. thesis, studied the behavior of power transmission poles subjected to underground blast loading. The blast-induced ground motion was compared to the seismic load. The behavior of the pole was studied in detail. Among the main issues discussed in the study is the development of site-specific spectra of blast-induced ground vibration based on field measurement data, proposed simplified but relatively accurate finite element analysis models that consider the structure cable coupling, obtained dynamic responses of transmission pole structures under blast caused ground vibration both by spectrum and time history analysis, the establishment of blast limit for transmission pole structures.

In a study conducted in China, Meng Z et al (Meng Z, 2017), checked the coupling effect of an already built steel lattice power transmission towers. A three-dimensional finite element model was developed using ANSYS simulation program. The dynamic responses of the tower-line system under different wind speeds and directions were analyzed and compared with the Chinese design code. In their study, the effect of wind speed was determined to be a significant factor in the tower line coupled system. In their conclusion, the Chinese code method of design using the quasi-static loading was deemed as unsafe and proposed a modification to the tower line coupling effect coefficient.

2.2 Explosion and Blast Wave

2.2.1 Ideal Blast Wave Characteristics

According to researchers (Karlos V, 2013), an explosion is a very fast chemical reaction involving a solid, dust, or gas, during which a rapid release of hot gases and energy takes place. The occurrence lasts only some milliseconds and it results in the production of very high temperatures and pressures. During detonation the hot gases that are produced expand to occupy the available space, leading to wave type propagation through space that is transmitted spherically through an unbounded surrounding medium. Along with the produced gases, the air around the blast (for air blasts) also expands and its molecules pile-up, resulting in what is known as a blast wave and shock front. The blast wave contains a large part of the energy that was released during the detonation and moves faster than the speed of sound (Karlos V, 2013).

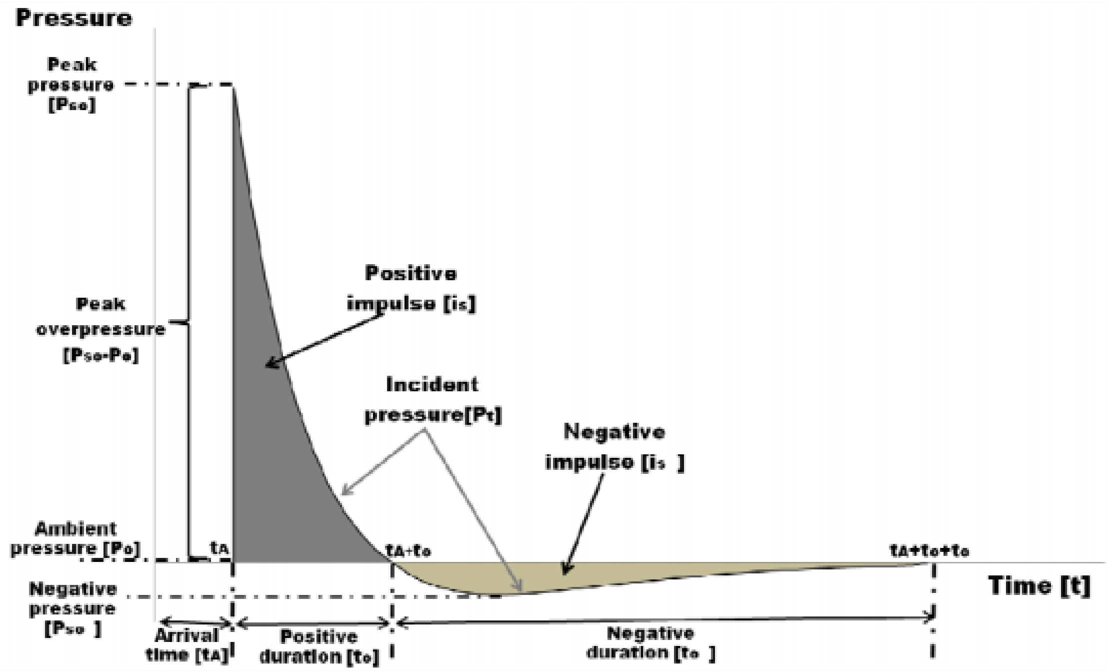


Figure 2-1:Blast wave parameters (Karlos V, 2013)

The above figure shows an idealized profile of the pressure with time for the case of a free air blast wave, which reaches a point at a certain distance from the detonation. The pressure surrounding the element is initially equal to the ambient pressure Po, and it undergoes an instantaneous increase to a peak pressure Pso at the arrival time tA when the shock front reaches that point. The time needed for the pressure to reach its peak value is very small and for design purposes, it is assumed to be equal to zero (Karlos V, 2013). After the blast reaches peak overpressure, the load will decrease exponentially changing to negative pressure. This is called the positive phase duration. Then the pressure will be below the ambient pressure. The duration of the negative phase is relatively longer than the positive duration. The negative pressure magnitude is very small when compared to the positive value thus it’s not considered in design calculation.

Michael (Michael S, 1994), has proposed modified Kingery’s equations that are widely used to describe the properties of blast pressure. The equation is a polynomial function in which the variables are stated in tabular form. Variable parameters are based on the scaled distance of the structure and detonation point.

$$\boldsymbol{Function} = \exp(\boldsymbol{A} + \boldsymbol{B} * (\ln(Z)) + \boldsymbol{C} * (\ln(Z))^2 + \boldsymbol{D} * (\ln(Z))^3 + \boldsymbol{E} * (\ln(Z))^4 + \boldsymbol{F} * (\ln(Z)^5 + \boldsymbol{G} * (\ln(Z))^6$$

Where: Z is the scaled distance and A, B, C, D, E, F, G are coefficients that are determined from a table based on the scaled distance. The function represents; time of arrival, incident pressure, reflected pressure, positive phase duration incident pulse, reflected impulse, and shock front velocity.

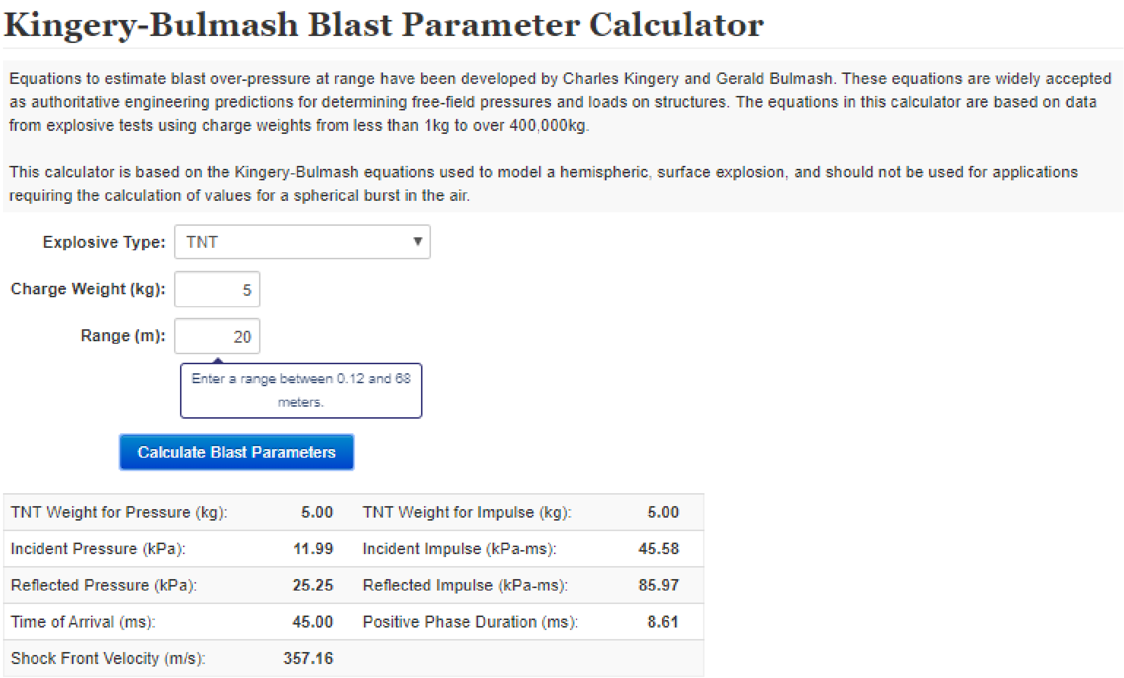


Figure 2-2: Blast load parameter calculation interface (Kingery C, 2016)

UN Safeguard (Kingery C, 2016) has a website platform that calculates blast pressures based on Kingery-Bulmash. Blast load parameters in this paper are calculated utilizing the platform.

2.2.2 Scaling Laws

One of the most critical parameters for blast loading computations is the distance of the detonation point from the structure of interest. The peak pressure value and velocity of the blast wave, which were described earlier, decrease rapidly by increasing the distance between the blast source and the target surface, as shown in Figure 2-3. In the figure only the positive phases of the blast waves are depicted, whose durations are longer whenever the distance from the detonation point increases (Karlos V, 2013).

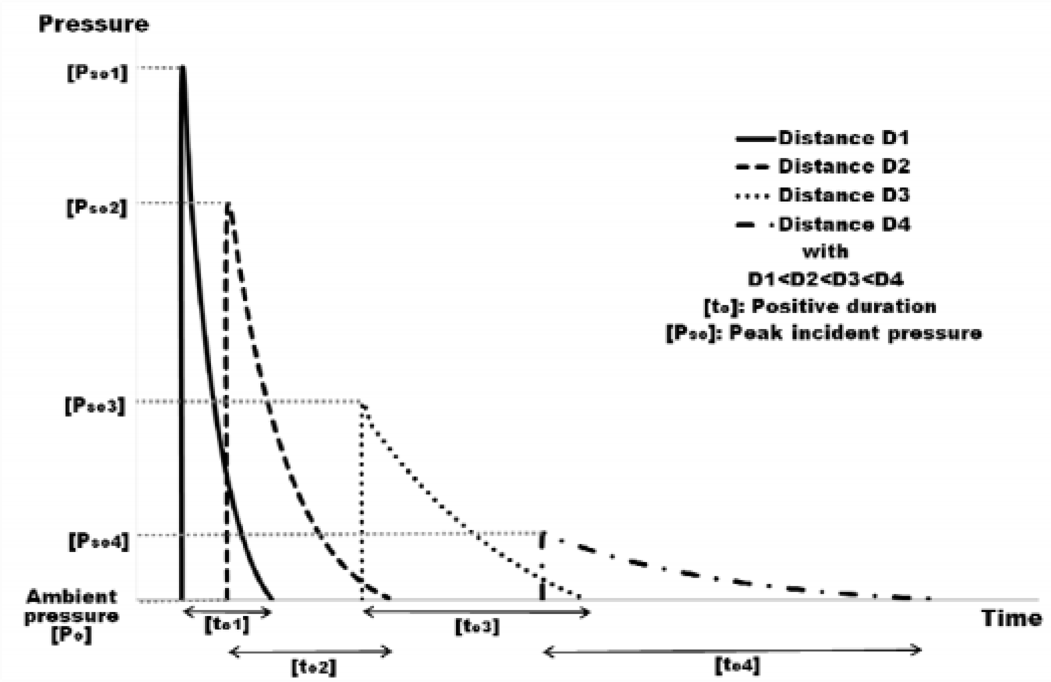


Figure 2-3: Blast wave positive phase variance with time (Karlos V, 2013)

The effect of distance on the blast characteristics can be taken into account by the introduction of scaling laws. These laws can scale parameters, which were defined through experiments, to be used for varying values of distance and charge energy release. The experimental results are, in this way, generalized to include cases that are different from the initial experimental setup. The most common blast scaling laws are the ones introduced by Hopkinson-Cranz and Sachs. According to Hopkinson-Cranz law, a dimensional scaled distance is introduced as described by Equation (2) (Karlos V, 2013),

$$Z = \frac{R}{\sqrt[3]{W}}$$

2.2.3 Blast loaded steel Structures

According to Mendis et.al (Mendis P, 2007). “the essential characteristics of loading and building response for transient loads produced by explosions depend primarily on the relationship between the effective duration of the loading and the fundamental period of the structure on which the loading acts.” When the effective duration is very short, for example, less than one-third of the period, then the impulse due to the transient loading is of major importance, and the response of the structure can be based entirely on a consideration of impulse and momentum. On the other hand, when the duration of the loading is relatively long compared with the fundamental period, then a quasi-static design can be made.

Structure motions are caused by what is normally termed shock loads. These are loads that cause transient or short-duration vibratory motions of the ground surface and the structure. They do not cause significant structural damage but instead induce motion which can damage the structure. There are two distinct types of shock loads: ground shock and air shock. Ground shock results from the energy which is imparted to the ground by an explosion. Some of this energy is transmitted through the ground as a direct-induced ground shock. Both of these forms of ground shock when imparted to a structure will cause the structure to move in both a vertical and horizontal direction. Vertical, horizontal, and overturning motions are imparted to the structure by the air blast. However, since the vertical motion of the structure is restricted by the ground which is already compressed due to the dead load of the structure and its contents, vertical motions must necessarily be small and can be safely neglected. Structural steel shapes are considerably slenderer, both in terms of the overall structure and the components of a typical member cross-section. As a result, the effect of overall and local instability upon the ultimate capacity is an important consideration in the analysis and design of steel structures (UFC 3-340-02, 2008).

2.3 Steel Lattice Power Transmission Tower

2.3.1 Tower Property [(Shu-jin F, 1999)]

A typical Double circuit, horizontal configuration, self-supported lattice tower is shown in Figure 2-4. The design of a steel lattice tower begins with the development of a conceptual design, which establishes the geometry of the structure. In developing the geometry, structure dimensions are established for the tower window, cross arms and bridge, shield wire peak, bracing panels, and the slope of the tower leg below the waist. The most important criteria for determining structure geometry are the minimum phase to phase and phase to steel clearance requirements, which are functions of the line voltage. The spacing of phase conductors may sometimes be dictated by conductor galloping considerations. The height of the tower peak above the cross arm is based on shielding considerations for lightning protection. The width of the tower base depends on the slope of the tower leg below the waist.

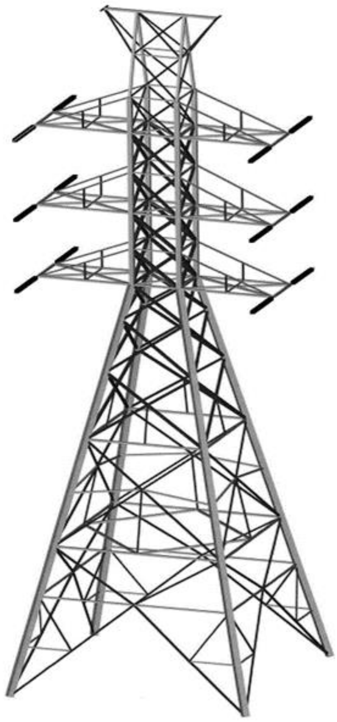


Figure 2-4: Model of a transmission tower (Sriram K, 2017)

The overall structure height is governed by the span length of the conductors between structures. The primary members of a tower are the leg and the bracing members which carry the vertical and shear loads on the tower and transfer them to the foundation. Secondary or redundant bracing members are used to provide intermediate support to the primary members to reduce their unbraced length and increase their load-carrying capacity. The slope of the tower leg from the waist down has a significant influence on the tower weight and should be optimized to achieve an economical tower (Shu-jin F, 1999).

2.3.1.1 Tower classification

Transmission line towers are generally specified by voltage, number of circuits, and type. Thus, these parameters become the basic parameters, which govern the structural design of the tower. The voltage classification of transmission line towers is according to the voltage of the line it carries (Satish S, 2019).

Generally, configurations adopted are square types. The square type of broad-based towers is the most commonly used. The number of circuits the tower can carry is either single, double, or multi-circuit. The number of earth wires, right of way, etc. also affect the configuration of the tower. Along the transmission line route, depending upon the profile along the centerline of the transmission line, towers are classified into three categories such as tangent tower, angle tower, and dead-end tower. Further, transmission line towers are also classified according to their shape as Barrel, Corset, and Guyed towers (Satish S, 2019).

The Barrel type towers are considered in this study for dynamic analysis. Which are depicted in Figure 2-5. The functional requirements such as minimum ground clearance, and clearance between conductor and tower body, are governed by the electrical regulations and they mainly depend on the voltage carried by the conductor. The number of circuits decides the number of cross arms on the tower. Parameters such as the number of cross arms, the vertical spacing between cross arms, the height of the ground-wire peak, minimum ground clearance, maximum sag, and other clearances decide the overall height of the tower. The staging of the transmission line tower should be high enough to provide minimum ground clearance under maximum sag condition.

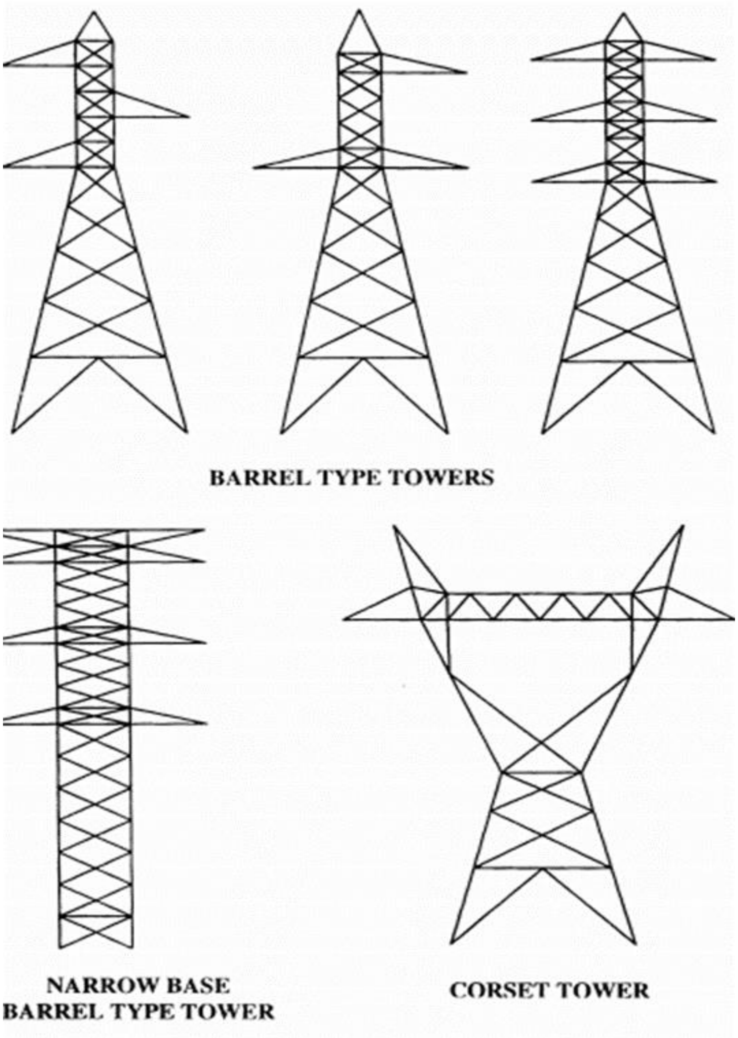


Figure 2-5: Typical Barrel and corset tower configurations (Satish S, 2019)

2.3.1.2 Transmission Tower-Line Tower Configuration

The geometric parameters of transmission line tower configuration are height of the tower, base width of the tower, top-hamper width, length, and depth of cross arm. Some of the parameters governing the geometry of a tower are shown in Figure 2-6. Approximate structural behavior of the tower or conventional practice is taken as the basis for fixing these parameters of the tower. Sag tension and clearances also play an important role in deciding the configuration.

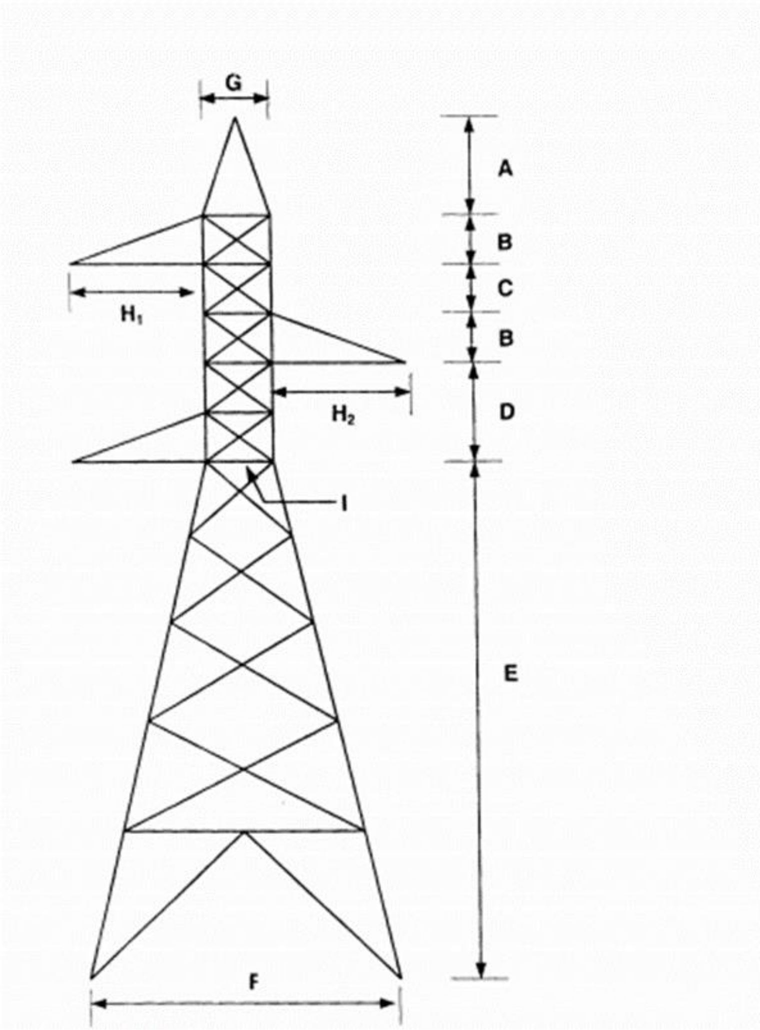


Figure 2-6: Geometric configuration of tower structure (Satish S, 2019)

A- Ground wire peak

B- Cross arm height

C- Panel height

D- Vertical sag between conductors

E- Staging

F- Base width

G- (H1 & H2) Cross arm length

H- (I) Top hamper width

2.3.1.3 Conductors

Power transmission lines span between towers. They are used to conduct electricity efficiently and sufficiently. The general trend in the design of overhead tower power transmission lines is to analyze the transmission dead load, wind load, and ice load in the vertical and traverse direction.

These loads on the transmission line will cause cable vibration. When designing transmission lines two issues must be addressed. The first issue is that the internal forces in tower members and cable must not pass their strength capacity. The second issue is that during cable vibration the two cables must not come into contact. This will create a short circuit (Mohammed M, 1997). Besides, conductor oscillations may lead to flashover as well as wear and fatigue damage to the towers.

The conductor is flexible with a constant mass per unit length and therefore once strung out, will conform approximately to the shape of a catenary. This means that the sag of cable can be calculated and is often a critical design factor in transmission line design (Alasdair B, 2017).

Figure 2-7: Typical arrangement of wires in conductor bundle

Wire bundles in a conductor are twisted around the center for strength purposes and erosion protection. In a conductor line, bare wires are used. Aluminum alloy in most cases. The material is chosen based on cost and efficiency.

2.3.2 Scenario Study

Blasts can have different sources. Blasts involving chemical reaction can be classified by their reaction rates. They are deflagrations and detonations. The main difference between the two is the speed in which they propagate. Deflagration is an oxidation reaction that propagates at a rate less than the speed of sound. Detonation propagates in a supersonic speed. Generally, reaction rate influences pressure of blasts. Thus, detonation pressures are usually many times higher than deflagration pressures.

Usually power transmission towers are located far from industrial areas where accidental explosion is a possibility. Even if it occurs they will be situated at a distance where the shock has a minimal effect. Thus, main possibility of blast origin will be from terrorist threats.

Terrorist threats principally involve solid materials such as plastic explosives or improvised explosives such as ammonium nitrate and fuel oil (ANFO). These materials can readily produce detonations, but the blast strength may be low due to inefficient configuration or the presence of contaminants. Flammables can potentially be used, but they are more difficult to transport and successively initiate (Donald O, 2010). Thus, blast pressure capacity (or weight in terms of TNT) will be in the lower two digits. When it comes to the standoff distance, transmission towers don't have any protective structures for the most parts. Explosives could be placed in close proximities. Taking all the above scenarios into consideration parameters of blast characteristics were selected for the study.

2.3.3 Dynamic Response

In structural engineering, the natural frequency, mode shape, and damping ratio are used to describe the structural dynamic properties. These properties relate to intrinsic structural features such as structural type, geometry, mass distribution, structural stiffness distribution, and joint construction.

Individual transmission towers and line systems are subjected to dynamic actions, such as various types of wind actions. These towers have several unique features compared with other engineering structures including 1) relatively tall but with low mass; 2) truss-frame structural type; 3) material and geometric nonlinearity, and 4) sway components. It is important to study the dynamic properties of lattice transmission towers to predict their behavior and design them effectively (Lu C, 2016).

The fluctuating nature of dynamic load can potentially excite resonant vibrations on structures. This leads to complex response analysis. Structures with a low natural frequency (below 1 Hz) are particularly vulnerable to resonance phenomena, which in turn can significantly affect structural dynamic responses (Holmes J, 1996). Full-scale measurements have shown that lattice transmission towers usually have a natural frequency which is below or just over 1Hz (Denoon R, 1996). Although this frequency value can be varied by aerodynamic damping and structural damping, it is still unpredictable. Once wind-induced resonant responses occur on a structure, counteracting structural forces including (1) inertial forces; (2) elastic and stiffness forces; (3) damping and energy-absorbing forces start to balance the wind forces (Glass T, 1990). Hence, the behavior of a structure changes and needs to be studied carefully to determine the dynamic response of lateral transmission towers and line systems.

As mentioned previously, dynamic responses are dependent on time histories. Hence, the time-domain analysis method has been widely used to understand the dynamic responses of transmission towers and line systems. The data from such analysis is presented with the amplitude (e.g. displacement, stress, acceleration) as the vertical axis and the elapsed time as the horizontal axis.

To understand the dynamic properties of lattice transmission towers, three full-scale dynamic experiments based on the forced vibration test were conducted in Japan (Maeno Y, 1996) and the results claimed that owing to the rotational inertia of the cross arms, the natural frequency of the tested tower in the transverse direction was smaller than that in the longitudinal direction. Also, a coupled lattice transmission tower line system further increased the resonance effects by expressing an even lower natural frequency at the same condition in both examined directions. It should be noted that the tower with conductors has more Eigen frequencies, due to the complex nature of the structure. Hence, the need for determining both longitudinal and transverse dynamic properties was suggested by these authors.

Meng Z. et al (Meng Z, 2017) conducted a study of a three-dimensional finite element model of a 500kV high voltage transmission tower line system numerically using ANSYS software. The model was verified with field measured data. The dynamic response of the tower line system was checked under different wind speed scenarios. The results indicate that wind speed plays an important role in the tower line coupling effect. Under the low wind speed, the coupling effect is less obvious and can be neglected. With increased wind speed, the coupling effect on the response of the tower gradually becomes prominent, possibly resulting in the risk of premature failure of the tower line system. And concluded that a design based on a quasi-static method is inappropriate.

2.3.3.1 Free Vibration

In a transmission tower-line coupling system, the tower and the line work together. The basic dynamic characteristics of the tower-line system can be obtained based on modal analysis. The dynamic characteristics include vibration mode and frequency. Through modal analysis, the vibration response of the system when subjected to external excitation can be predicted.

Modal analysis is an effective technique of system identification by determining the inherent dynamic characteristics of a system. Itis based upon the philosophy that the vibration response of a linear time-invariant system can be expressed as the linear combination of a set of vibration modes. Modal testing is an experimental technique to obtain modal data representative of a physical model in reality. Based on the concept that error-free measurements truly represent a structure, the mathematical model can be verified or updated (He J, 2001).

2.3.3.2 Forced Vibration

Forced vibration of electric power transmission structures is one of the main concerns for structural design considering the large span characteristics of the transmission lines. These power grids, mainly composed of conductors and towers/poles, are vulnerable to wind, ice, earthquake, and accidental loads. The complexity caused by the coupling between cables and supported structures increases the difficulty in studying dynamic responses of transmission systems (Dai, 2009).

2.3.3.3 Cable Galloping

Galloping is a phenomenon where transmission conductors vibrate with large amplitudes and usually occur when the steady, moderate wind blows over a conductor covered with ice. Ice build-up makes the conductor slightly out-of-round irregular in shape leading to aerodynamic lift and conductor movement.

In gross terms, galloping is a vertical oscillation of the conductor span in one or a few loops. However, it usually incorporates other, less visible, twisting, lateral, and longitudinal motions. The most common design of overhead transmission lines consists of many spans of similar length supported on steel lattice towers with suspension insulators. The galloping of these lines is usually in one or two loops with generally similar amplitudes in each span. Aside from the visible vertical motions, there are usually noticeable longitudinal motions of the suspension insulator strings. The galloping motions can be large enough to cause flashovers between adjacent phases, especially when the phases are above each other. Occasionally, the vertical motion may result from a combination of modes, including one-, two-, three-, and four-loop motions. The single loop mode can occur even on relatively long spans (Lilien J, 2005).

To answer the question that how cable oscillation initiates, Venkatasubramananian (Venkatasubramananian S, 1992) investigated the galloping of electric power transmission lines. The coupling between axial modes and torsional modes and its influence on natural frequencies of cable vibrations were studied through theoretical analysis as well as numerical modeling. It was found that the sag-to-span ratio did not have a significant effect on torsional frequencies although it caused great changes of vertical oscillation frequencies. The author pointed out that galloping might be initiated in a purely vertical mode, purely torsional mode or a combination of these two. But the torsional mode was more likely to be the initiating mode for transmission line galloping (Dai, 2009).

Chapter 3 Modelling and Analysis of Power Transmission Structure

The electric transmission lines are constructed using different types of supporting structures. In the present research work, to determine the effect of blast shock loading on the towers, Barrel type steel lattice tower was chosen as supporting structures to model a transmission line. It was then designed to resist dead and live loads according to Eurocode. The same transmission line model was also used to study the uncoupled effect on dynamic behavior. The following sub-section describes the modeling of steel lattice supporting structure and transmission line model.

3.1 Tower Geometry and Material

The tower is of the commonly constructed lattice type of tower which utilizes a truss system of members to transfer self-induced, variable, and conductor loads from the structure to the foundations. The tower conforms to the definition of a "high tower" with a maximum height of 42 m above ground.

The transmission-line system consists of structural truss elements and conductor cables. The conductor cables are attached to the towers at the top cross arm of the tower. The conductors are all pre-stressed and take the form of a parabolic curve when installed. The beam-column elements of the tower consist of steel angles that are connected with structural bolts.

Span length between towers is correlated with conductor spacing. When distance in between towers increases, the sag and pretension in cables increases as well. In his research Holland (Holland H, 1980), implies span is determined comparing different structure height, conductor sag, clearance above ground and height of tower. The following equation has been proposed for optimal design.

$$C = \sqrt{\frac{P - L}{D}}$$

- P is height of the structure,
- L is conductor clearance above ground level
- C Ruling Span

- Conductor sag for ruling span

European code (EN 50341-1, 2012), gives a table of factors that correlates reference height for conductor and terrain category with different structural loading factors. To select the optimum span distance between two towers several factors will be taken into consideration. Such as terrain categories, height of plantation, nearby population and etc. Based on trial and error iterations optimum tower height and span will be selected. For this thesis height and span length between towers is selected based on built power transmission towers from previous literature (Meng Z, 2017).

The structure is first modeled in SAP2000 software because it has an easy and familiar interface. Next, an (iges) file is created and transferred to Dlubal RFEM5. Then cross-section, material properties, and loading conditions were defined from the imported file.

3.1.1 Tower Geometry

The tower is 42 meters high with an 8 m^2 base. The top width of the tower is 2 meters. Spacing between cross-arms is 4 meters. This is done based on the requirement for cable spacing to avoid flashovers. In the tower line coupled system as depicted in Figure 3-2, three towers and two spans of conductors were modeled. The first and second spans are 300 and 250 meters long respectively. The height of the tower, its silhouette, and overall dimensions depend on required electrical clearances in detail presented in the electrical part of the Euro code (EN 50341-1, 2012).

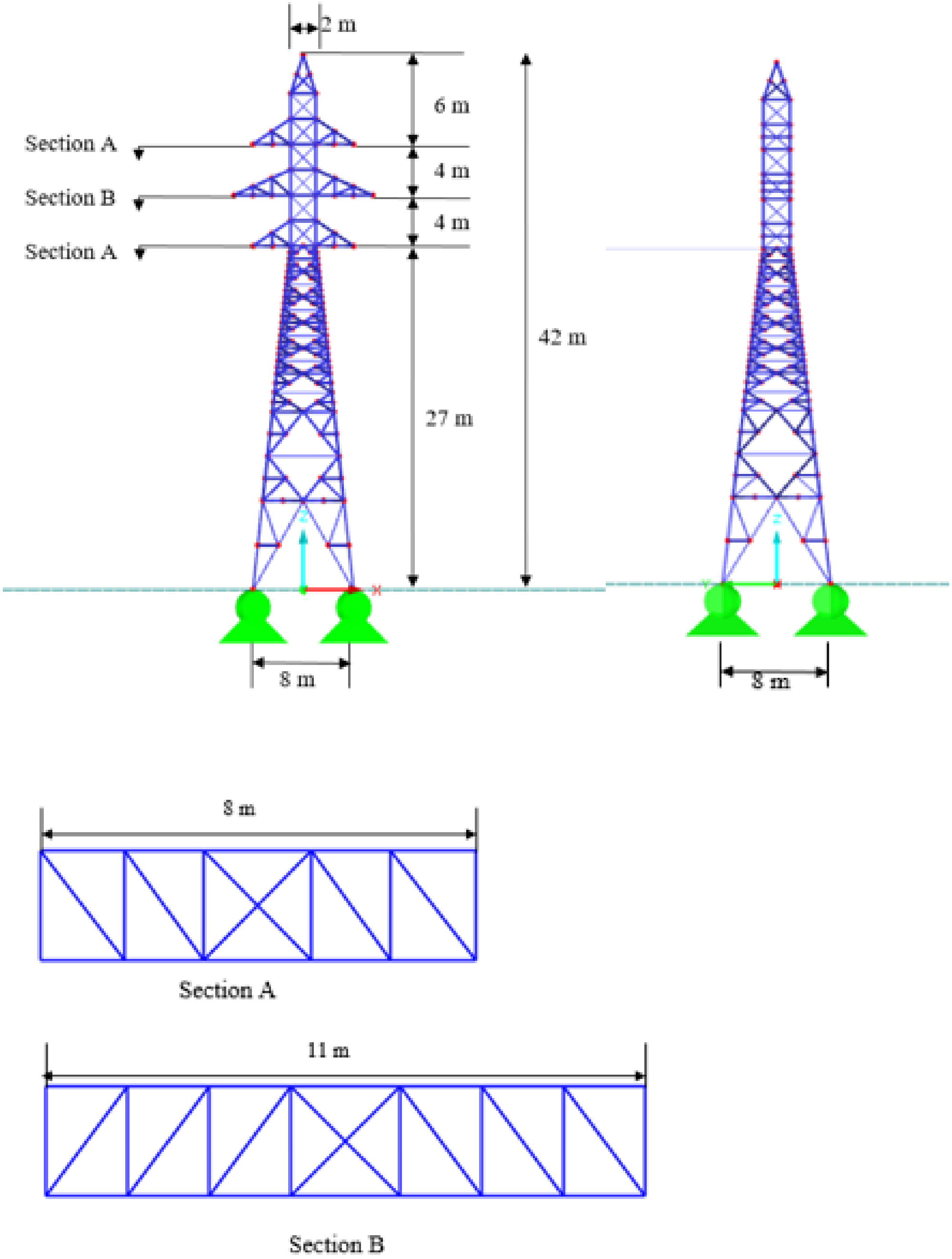


Figure 3-1: Geometric properties of the towers on Dlubal RFEM5

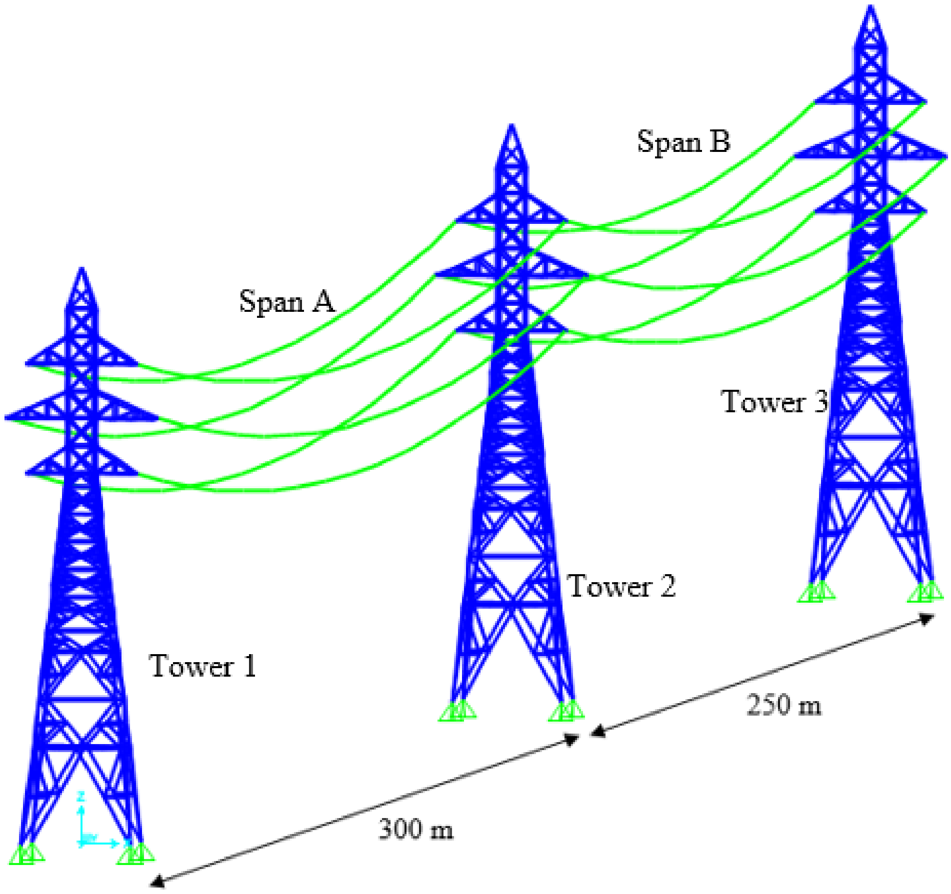


Figure 3-2: Tower-Line coupled system configuration on SAP2000

3.1.2 Member Cross-Section

For this thesis, all members were modeled accurately for profile sizes and steel grade to ensure realistic dead load contributions in the models. However, it is not within the remit of this study to analyze the steel connections, therefore items such as bolts and bolt holes were omitted.

The tower is modeled using varying sizes of either single L-profile angles. Figure 3-1 and 3-2 show an example of the profile arrangement. Table 3-1 lists the cross-section specifications used on the numerical modeling of the towers.

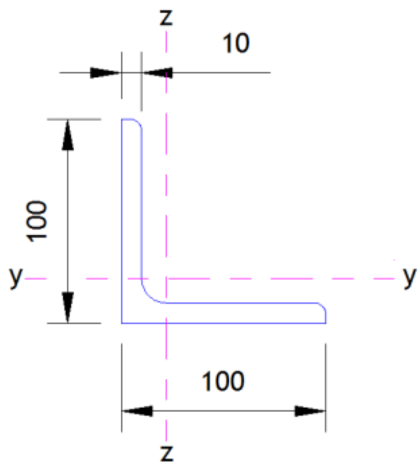


Figure 3-3: Single angle L100X100X10 mm Profile

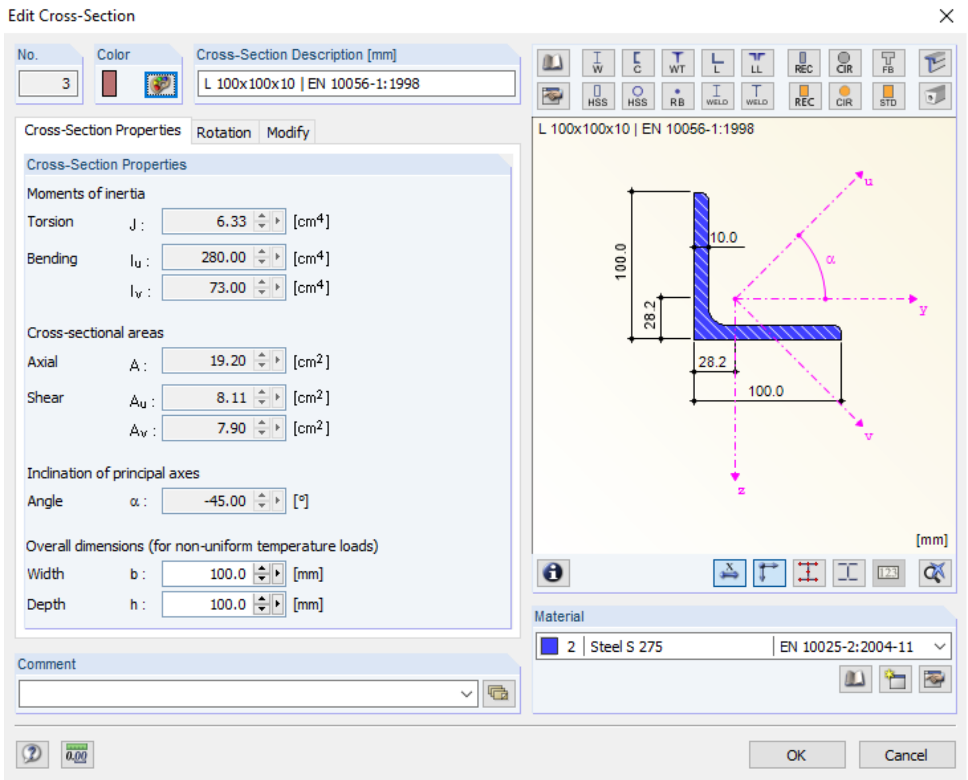


Figure 3-4: Cross-section properties as depicted in the program

Table 3-1: List of member profiles used in the study

Profile Size	Mass per meter [kg/m]	Area of section [cm 2]	Distance to center of gravity C [cm]	Second moment of area Axis x-x,y-y [cm4]	Radius of gyration Axis x-x,y-y [cm]	Elastic modulus Axis x-x, y-y [cm3]
L100X100X10	15	19.2	2.82	177	3.04	24.6
L100X100X8	12.2	15.5	2.74	145	3.06	19.9
L90X90X8	10.9	13.9	2.5	104	2.74	16.1
L80X80X8	9.63	12.3	2.26	72.2	2.43	12.6

3.1.3 Material

The tower is constructed from standard S355 steel for all main members of the superstructure. Table 3-2 shows the material properties of S355 steel used in the study.

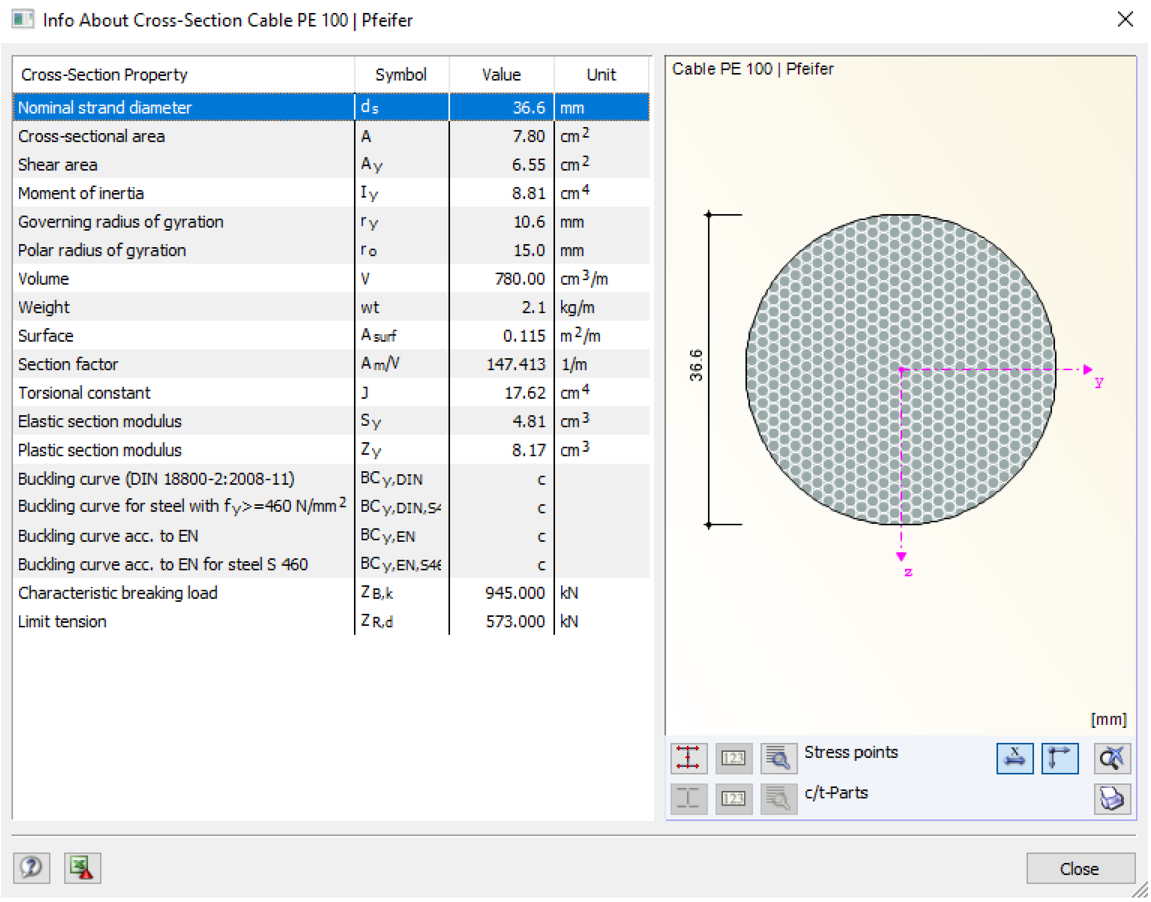
Table 3-2: Material property

Modulus of Elasticity	E	21000	kN/cm^2
Shear Modulus	G	8076.92	kN/cm^2
Poisson´s Ratio	ν	0.3	
Specific Weight	ɣ	78.5	kN/m^3
Coefficient of Thermal Expansion	α	1.20E-05	1/°C
Partial Safety Factor	ɣM	1	

3.2 Conductors

Six conductors on two spans were modeled using three-dimensional nonlinear elastic cable elements, with initial conductor tension of twenty-five percent of the rated tensile strength of the material. These conductors were strung between the towers, as shown in Figure 3-2. ACSR (Aluminum Conductor Steel Reinforced) PE 100 Conductor (trade name-Pfeifer) was chosen for this investigation. The various characteristics considered in modeling the conductors for this analogy can be seen in figure 3-4.

The conductors used in transmission lines are made of several layers of individual round wires packed tightly together in concentric counter-rotating helices (Figure 2-7). The most common conductor used in transmitting power is the aluminum conductor steel reinforced (ACSR) because of its high tensile-strength-to-weight ratio. Most of the power is transmitted through the aluminum outer layers. The inner layer of the ACSR is made of steel to increase the strength of the conductor. ACSR conductor is available for a wide range of steel alloys (Barry, 2008).



Cross-Section Property	Symbol	Value	Unit
Nominal strand diameter	d_s	36.6	mm
Cross-sectional area	A	7.80	cm^2
Shear area	A_y	6.55	cm^2
Moment of inertia	I_y	8.81	cm^4
Governing radius of gyration	r_y	10.6	mm
Polar radius of gyration	r_o	15.0	mm
Volume	V	780.00	cm^3/m
Weight	wt	2.1	kg/m
Surface	A_{surf}	0.115	m^2/m
Section factor	A_m/V	147.413	1/m
Torsional constant	J	17.62	cm^4
Elastic section modulus	S_y	4.81	cm^3
Plastic section modulus	Z_y	8.17	cm^3
Buckling curve (DIN 18800-2:2008-11)	$BC_{y,DIN}$	c	
Buckling curve for steel with f_y>=460 N/mm^2	$BC_{y,DIN,S4}$	c	
Buckling curve acc. to EN	$BC_{y,EN}$	c	
Buckling curve acc. to EN for steel S 460	$BC_{y,EN,S46}$	c	
Characteristic breaking load	$Z_{B,k}$	945.000	kN
Limit tension	$Z_{R,d}$	573.000	kN

Figure 3-5: Conductor characteristics of the transmission line

Aside from the conductors, a longitudinal steel wire moves along the towers. It is called earth wire. The purpose of the earth wire sometimes referred to as “ground wire” or “shield wire” is to earth the structure to the ground by providing a direct connection from the top of the tower to the ground, insulating the structure in the case of a direct lightning strike. The tower modeled in this thesis has two earth wires peak due to the large voltage nature of the line. The earth wires and conductors used for the calculations were of the standard type with 12 mm diameter steel aluminum wires.

Table 3-3: Conductor and earth wire properties

Wire type	Name	Material	Cross-section Area [cm2]	Radius of gyration [mm]	Weight [kg/m]	Limit tension [kN]
Conductor	Pfeifer Cable PE 100	Aluminum	7.8	10.6	2.1	573
Earth wire	Pfeifer Cable PE 20	Aluminum	1.61	4.8	0.4	118

The cable element strung between the towers is divided into three main elements. Endpoints of conductors are connected to the tower structure as a pin-pin connection. This is done to have a simplified representation of the tower line system. The insulators are not modeled. Dlubal RFEM5 has a built-in program for calculation of actions on cable elements. This module is utilized to capture the full dynamic response of the transmission lines.

3.2.1 Sag and Tension Calculation

A conductor suspended between towers will not have a pure horizontal shape. It will sag under its own weight at the mid-section. This will in turn create pre-tension stress in the cable.

The conductor sag D can be calculated as a function of ΔL, according to the reference conductor length Lref. The overhead transmission line conductor's sag-tension calculations are typically based on the catenary equation, which describes an entirely flexible rope rigidly fixed at both ends. The catenary equation is defined using hyperbolic sine or cosine functions. However, it can be reliably approximated by a parabola. The main difference between a catenary equation and the parabolic approximation is that the catenary assumes a constant weight per unit length through the conductor while the parabolic equation assumes an invariable weight per unit horizontal length. This simplification causes the sag calculation with the parabolic approximation to be smaller than when it is estimated with the catenary equation (Michal W, 2018).

The shape of a catenary is a function of the conductor weight per unit length-weight w, the horizontal component of tension H, span length S, and the maximum sag of the conductor D. The exact catenary equation uses hyperbolic functions, as shown in Equation (3-1). The right side of the Equation (3-1) is an approximation of the hyperbolic cosine using the Maclaurin series expansion (Michal W, 2018):

$$y(x) = \frac{H}{w}\cosh\left(\left(\frac{w}{H}x\right) - 1\right) = \frac{w(x^2)}{2H} \text{.............. (3-1)}$$

For a flat span, the low point is at the center and the wire sag D is found by substituting x = S/2. Exact and approximate formulas for the sag calculations is shown in Equation (3-2):

$$D = \frac{H}{w}\left(\cosh\left(\frac{wS}{2H}\right) - 1\right) = \frac{wS^2}{8H} \text{.................. (3-2)}$$

The parabolic approximation is sufficiently accurate as long as the sag in the span does not exceed 5% of its length (Grigsby L, 2012). The power line sag D varies with the conductor temperature, ice, wind loading, and time as the conductor creeps.

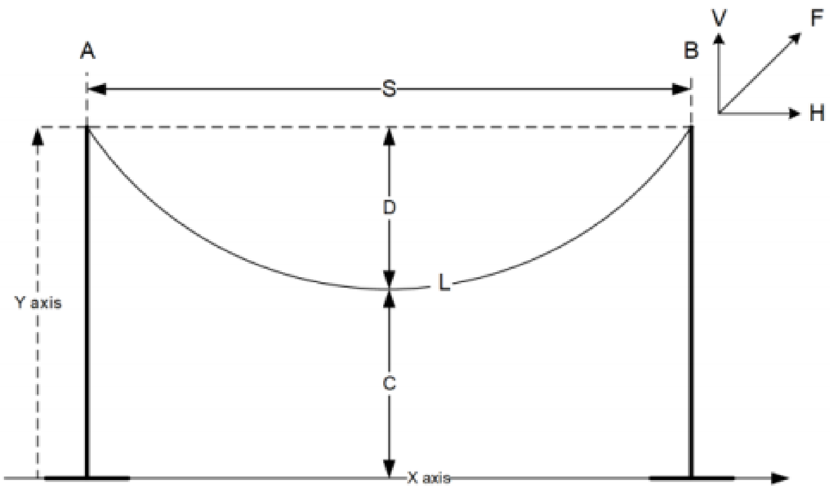


Figure 3-6: Conductor length, sag, clearance, and tension in a transmission line span.

Figure 3-6 shows a simplified drawing of the assumption for sag and tension calculation done on conductors. Taking the above equations, the sag and pre-tension stresses can be calculated as follows:

$$Span\ length\ (l)\ =\ 300\ meters$$

$$Weight\ per\ length\ (w)\ =\ 2.1\ kg/m$$

$$Tension\ capacity\ of\ cable\ =\ 367.3\ kN$$

$$Tension\ in\ conductor\ (H)\ =\ 0.25 * 367.3\ kN\ =\ 91.8\ kN$$

$$D\ =\ \frac{2.1 * (300)^2}{8 * 91.8} = 2.52\ m$$

$D = 1.75\ m\ for\ 250\ meter\ span$

$D = 2.5\ and\ 1.72\ for\ the\ earth\ wire$

Table 3-4: Initial condition of conductors and earth wires

Parameter	Unit
300 m Span conductor sag	2.52 m
300 m Span earth wire sag	2.5 m
250 m Span conductor sag	1.75 m
250 m Span earth wire sag	1.72 m
Initial Tension in Conductors	91.8 kN
Initial Tension in Earth wire	17.62 kN

To consider the load of conductor cables in uncoupled transmission towers, the dissolved forces will be calculated as depicted in figure 3-7. Load transferred to transmission tower comes from the self-weight or pretension in cables.

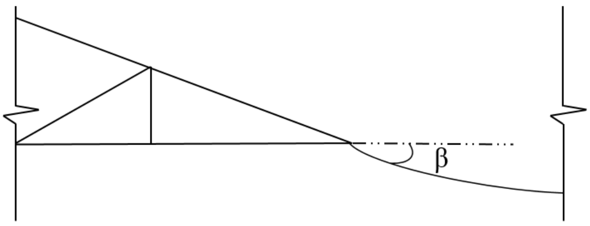


Figure 3-7 Tower cross arm and conductor connection angle

Due to the sag of the conductor being small relative to span, the trigonometric equation can give a good result of the cable tangent angle. This angle is used to dissolve the pretension force of the cable into a horizontal and vertical one.

Table 3-5 Dissolved forces in the tower cross arm due to conductor

Parameter	β (Degree)	Tension (kN)	Fy (kN)	Fz (kN)
300 m Span conductor sag	0.9625	91.8	1.5421	91.787
300 m Span earth wire sag	0.9548	17.62	0.2946	17.6175
250 m Span conductor sag	0.8021	91.8	1.2851	91.791
250 m Span earth wire sag	0.7883	17.62	0.2424	17.6183

When modeling the conductor in Dlubal RFEM5 the sag and pre-tension values calculated above are assumed in the model.

3.3 Verification of Modelling Technique

Verification of a software program is necessary so that we can get a correct answer, that corresponds close to reality. Dlubal RFEM5 software is a program that solely calculates input parameters given. The accuracy of our modeling technique should be evaluated by comparing it with an experiment output. Here below the modal behavior predicting the capacity of the software is evaluated by comparing the natural vibration of a previously done experiment on cable and tower structure.

Many applications of structural dynamics rely on their success upon having a faithfully representative model. Modal analysis is an effective technique of system identification by determining the inherent dynamic characteristics of a system. Itis based upon the philosophy that the vibration response of a linear time-invariant (LTI) system can be expressed as the linear combination of a set of vibration modes. Modal testing is an experimental technique to obtain modal data representative of a physical model in reality. Based on the concept that error-free measurements truly represent a structure, the mathematical model can be verified or updated (He J, 2001).

3.3.1 Tower Verification

To verify the modeling and dynamic analysis capacity and of Dlubal RFEM5 a previously done experiment was replicated. The experiment conducted in Hydro-Québec TransÉnergie (Xiaohong Z, 2009) was on a prototype power transmission tower. A series of characterization tests (static and dynamic) in elastic and post elastic regime was conducted.

The test tower prototype was the upper section (8-panel sections in total) of the standard Hydro-Québec TransÉnergie suspension tower with a narrow base for single-circuit 235 kV lines. The outline drawing of the prototype is shown in Figure 3-8. Several dynamic tests were done within the elastic regime to get the natural frequencies of different vibration modes and the structural damping ratios. Under each loading scenario, torsion, and bending, two free vibration tests were done to measure the natural frequencies and damping ratios (Xiaohong Z, 2009).

The tower was first excited by tensioning a cable attached to the tip of the tower cross arm with a mass of approximately 1000 kg (a concrete block) and suddenly dropping it.

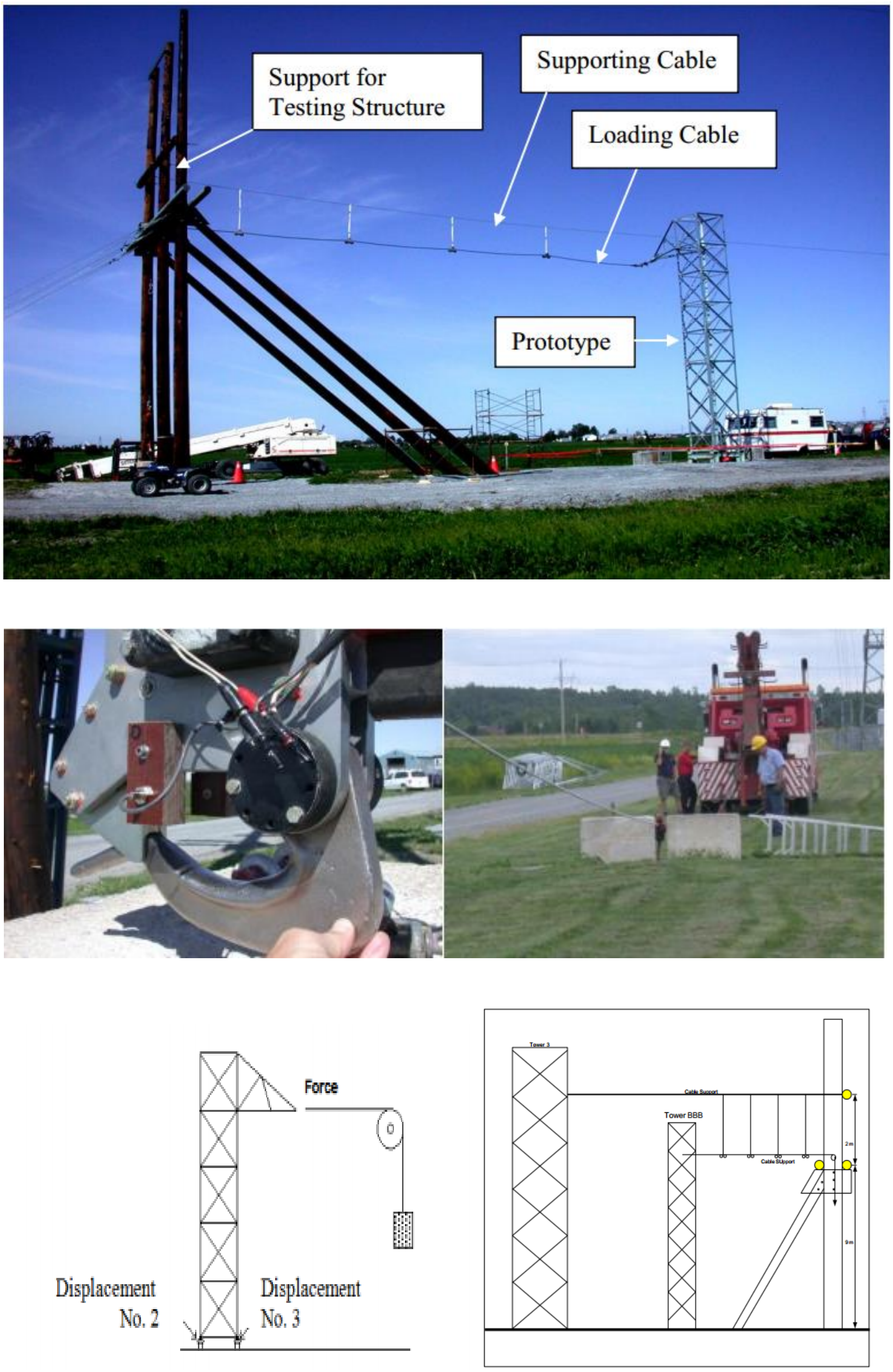


Figure 3-8: Prototype tower configuration (Xiaohong Z, 2009)

The tower was modeled in Dlubal RFEM5. The software uses a finite element method based code. The tower components are meshed based on the optimization of the program offers. Figure 3-8 (left) shows an isometric view of the detailed tower in the program.

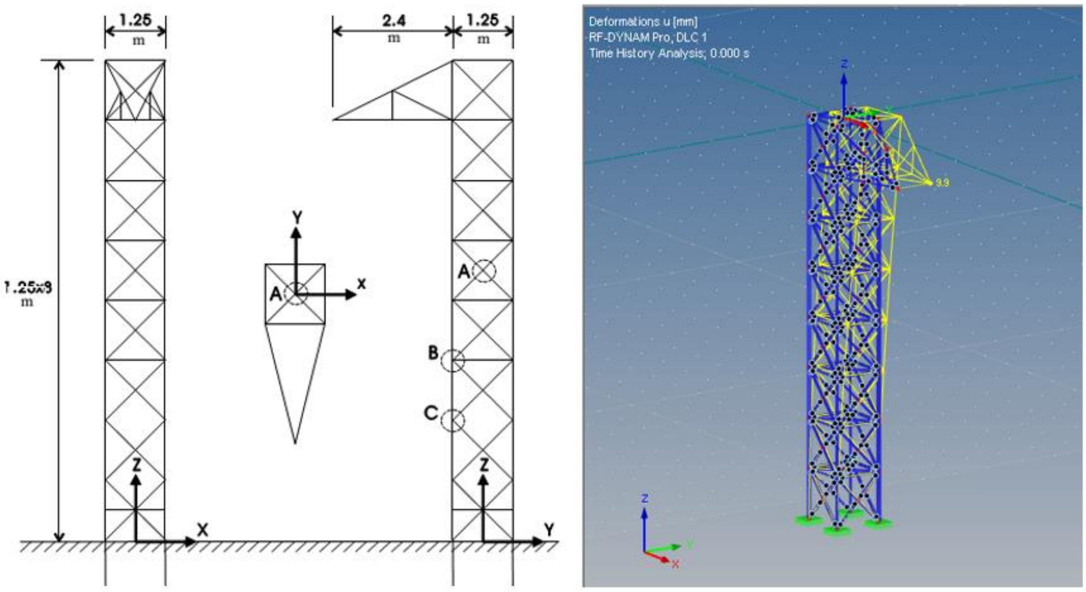


Figure 3-9: Tower BBB geometry (left), Tower model on Dlubal RFEM5 (Right)

Table 3-6: Natural vibration analysis result

Mode No.	Natural Frequency f [Hz] Model	Natural Frequency f [Hz] experiment	Natural period T [s] Model	Natural period T [s] experiment	Mode Shape
1	9.176	7.4 (9.1)	0.109	0.135 (0.110)	Transverse Bending
2	9.231	8 (9.4)	0.108	0.125 (0.106)	Longitudinal Bending
3	19.326	14.8 (18.8)	0.052	0.068 (0.053)	Torsion

In Table 3-6 comparison is made between numerical analysis and experiment. The values in the bracket are from the numerical analysis of the experimental research author. Xiaohong (Xiaohong Z, 2009), explains the difference between the experiment and numerical tests as: “The natural frequencies measured from the pluck tests for all the three lowest-frequency modes were approximately 20% smaller than calculated values, which confirms that the real test structure is more flexible than its numerical model, mainly because foundation flexibility was not modeled”. Thus the difference in the modal behavior of the tower arises from experimenting error.

3.3.2 Cable Verification

Dlubal RFEM5 has a very powerful finite element solution method. It can account for geometric and material nonlinear properties of cables. In this paper, only geometry non-linearity is considered. To verify the capability of the software as well as the geometry and modeling technique of the program, a previously conducted experiment was replicated.

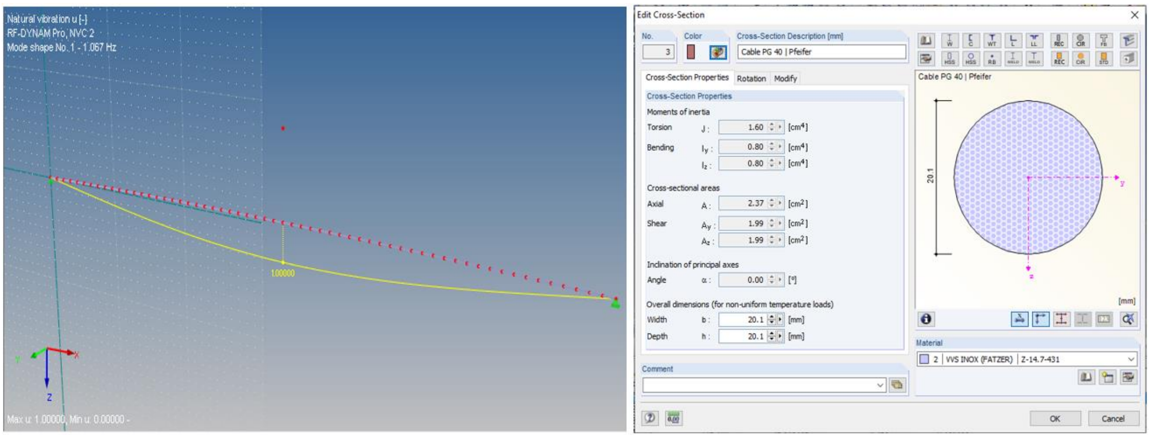


Figure 3-10: Model and cross-section parameter of cable experiment

Barbieri et al (Barbieri N, 2002), experimented on the dynamic behavior of electric cables of a transmission line. Three sample lengths were used: 13, 30, and 65 meters. The force responses were obtained through an impulse excitation. The experimental data were acquired using five accelerometers placed along with the sample. Figure 3-9 shows the schematic and basic components of the overhead line conductor testing system.

The parameters used for the modal analysis are from Barbieri's experiment (Barbieri N, 2002). To allow the comparison of the obtained simulation results to the corresponding experimental results. The conductor has the following characteristics:

- Mass per unit length, m = 0.8127 kg/m.
- Flexural rigidity EI min = 11.07 N.m
- Mechanical load (tension) in the cable, T = 15860 N
- The span length considered, L = 65 m

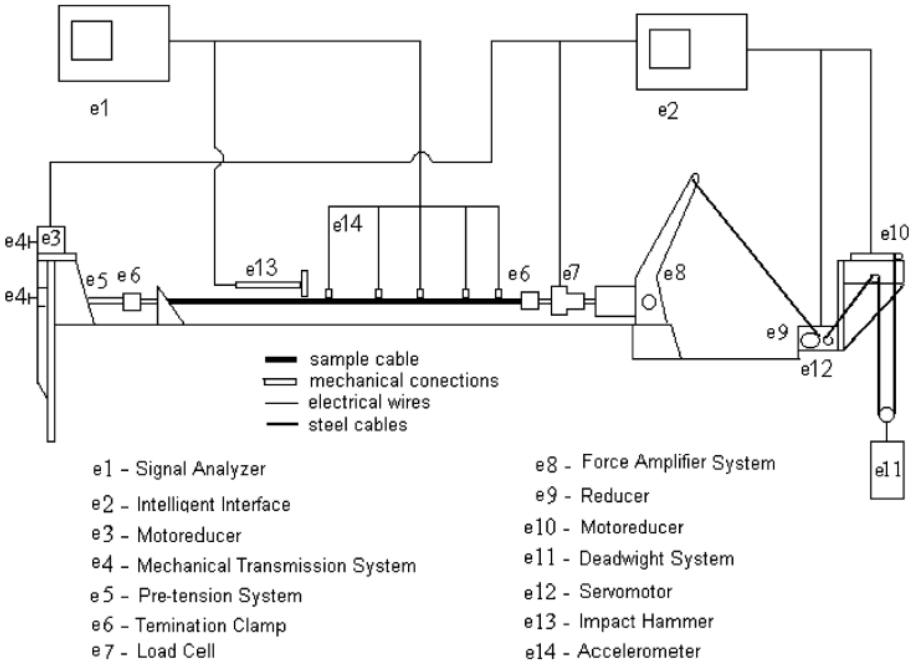


Figure 3-11: Schematic view of the testing system for overhead line cable (Barbieri N, 2002).

The conductor is modeled in Dlubal RFEM5 as a geometrically non-linear cable. The cable was subjected to a pre-stress of 15.86 kN. The program utilized the root of characteristic polynomial to solve the eigenvalue problem. Material and cross-section of the cable were chosen from the vast material library of the program.

Table 3-7: Simulated and experiment comparison of natural frequencies

Mode No.	Natural Frequency f [Hz] Model	Natural Frequency f [Hz] experiment
1	1.068	1.1159
2	2.133	2.1234
3	3.192	3.1829
4	4.242	4.2509
5	5.281	5.3081

The comparison of experimental and finite element results from Dlubal RFEM5 is illustrated in Table 3-7. From this table, it is evident that both results are very close. The percentage of error is less than 5%. The discrepancies in the results are due to some unavailable experiment data. when modeling on the software some conditions were not modeled because of data unavailability. But from the results, we can be sure that the software program is capable of capturing the dynamic response of cable structures adequately.

Barbieri et al (Barbieri N, 2002), concluded from their experimental study about the effects of boundary condition, damping characteristics, and tension load properties of conductor cables. The existence of axial tension in cables will lead to a considerable difference in the conductor's mode shapes, regardless of making a small difference in the values of natural frequencies. The properties of the conductor with different boundary condition has little effect on modal properties of the cable. It will increase its stiffness by a small amount. A damper will have a huge effect on the properties of cables. The author has demonstrated the significance of the difference in natural frequencies when a single damper is there.

3.4 Damping

Damping in steel latticed power transmission tower and conductor arises from different components of the structure. Each component has a different type and magnitude of damping. The steel tower and conductor cables consist of individual steel members with different sizes, bolts, nuts, cables, spacers, insulators, and damping instruments. All of which contribute to the general damping of the system differently. It is very difficult to model a different kind of damping due to incomplete experimental data and software incapability. Even with experiments done it is extremely difficult to determine damping value's coming from the different components of the tower-line system.

Damping should be introduced so that it contributes to filtering out spurious high-frequency components of the finite element model response due to discretization (Syed F, 2018). Meng et al in their research (Meng Z, 2017), when modeling a three-dimensional finite element model of a 500kV high-voltage transmission tower-line coupling system, the damping assumption was made based on Rayleigh orthogonal damping matrix. Which is expressed as:

$$[C] = \alpha[M] + \beta[K]$$

Where α and β are the system's mass and stiffness coefficients respectively. The equations developed by Rayleigh are as follows:

$$\alpha = \frac{2\omega i\omega j(\varepsilon j\omega i - \varepsilon i\omega j)}{\omega i^2 - \omega j^2}$$

$$\beta = \frac{2(\varepsilon i\omega i - \varepsilon j\omega j)}{\omega i^2 - \omega j^2}$$

In the above equations, ωi and ωj are the i-th order and j-th order circular frequencies and £i and £j are the systems i-th order and j-th order damping ratios. As long as any two modes are determined the coefficients α and β can be calculated. The circular frequencies are chosen based on the maximum mass participation factor.

In researches of Meng et al (Meng Z, 2017), they compared damping results obtained from finite element simulation and field experiment measurement. The variation between the two is averagely around 20 percent. The discrepancies in the result arise from factors that are hard to incorporate in simulations. The critical damping ratio of tower and conductor combined varies between 1 to 3%. For this study, it was taken as 2%.

In RF-DYNAM PRO (subset module in the Dlubal-RFEM software program) (Dlubal Software, 2018), structural viscous damping is available. It is defined by using Rayleigh coefficients α and β or the Lehr's damping Di.

In Rayleigh damping, it is normally recommended that the two specific frequencies for determining Rayleigh damping should ensure reasonable damping values in all the modes significantly contributing to the vibrations. At the frequency outside the range of these two frequencies, the damping will dramatically increase and the modal responses at the corresponding frequency range will almost be eliminated. Practically, this can be used to damp out the high and low-frequency vibrations/noises that are outside the frequency range of interests. In many cases, the variation of damping ratio with frequency is not available, and one can then assume the damping at the two specific frequencies to be identical (Jia J, 2014). On this paper viscous damping is considered. The figure below depicts how damping was inputted in the analysis.

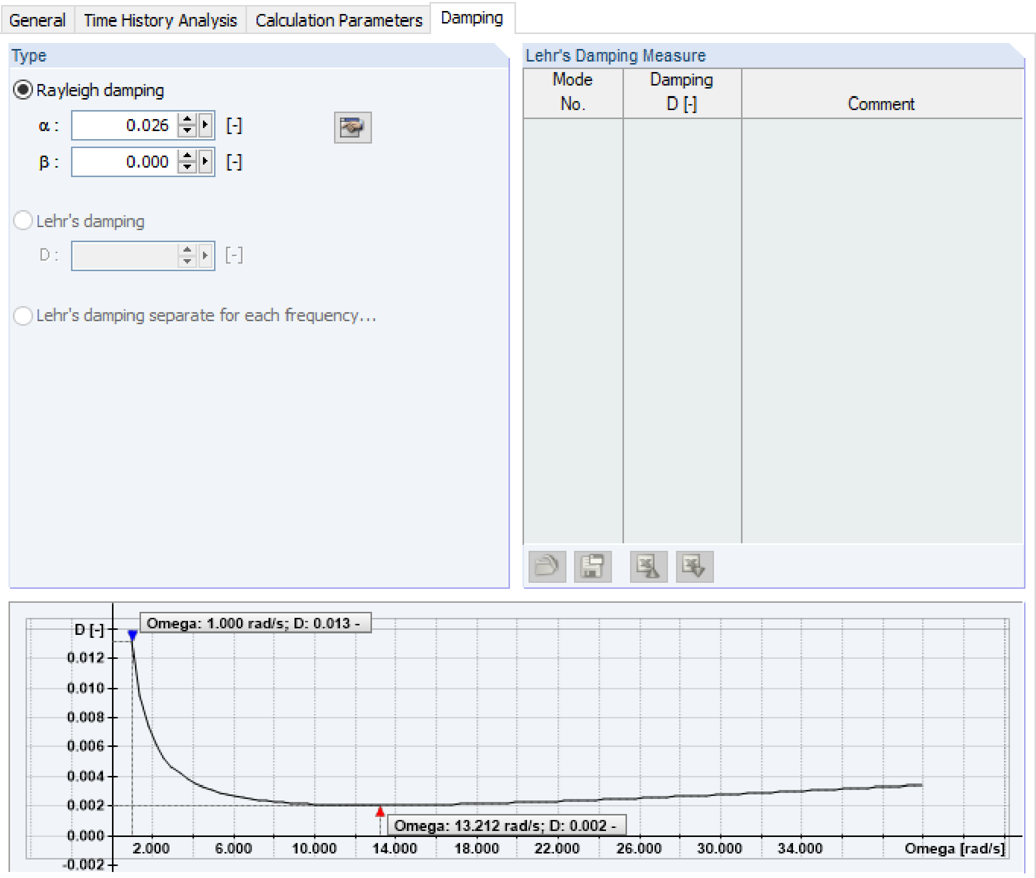


Figure 3-12 Damping parameter input window

When the actual tower-line system is erected additional damping devices will be installed. There is a multitude of damping devices that uses different kinds of mechanism to dissipate vibrational energy. Because it is not the focus of this research the type and method of damping will not be discussed.

3.5 Dynamic Analysis

Very few research papers have addressed the dynamic analysis of blast loaded steel lattice towers coupled with a conductor cable. Because most design codes don't require dynamic analysis. The dynamic work here reviewed was limited to the linear elastic regime due to the complexity of predicting the post elastic behavior of steel angle shapes and the unavailability of experiments to verify numerical calculation. The current approach studies forces in the structure with different capacity explosives and standoff distance.

Table 3-8 The parameters considered in the dynamic analysis

Tower designation	Description	Type
T 1	3 kg 10 m	Single Tower 1
T 2	3 kg 20 m	Single Tower 2
T 3	3 kg 50 m	Single Tower 3
T 4	5 kg 10 m	Single Tower 4
T 5	5 kg 20 m	Single Tower 5
T 6	5 kg 50 m	Single Tower 6
T 7	10 kg 10 m	Single Tower 7
T 8	10 kg 20 m	Single Tower 8
T 9	10 kg 50 m	Single Tower 9
TL 1	3 kg 10 m	Tower-line coupled 1
TL 2	3 kg 20 m	Tower-line coupled 2
TL 3	3 kg 50 m	Tower-line coupled 3
TL 4	5 kg 10 m	Tower-line coupled 4
TL 5	5 kg 20 m	Tower-line coupled 5
TL 6	5 kg 50 m	Tower-line coupled 6
TL 7	10 kg 10 m	Tower-line coupled 7
TL 8	10 kg 20 m	Tower-line coupled 8
TL 9	10 kg 50 m	Tower-line coupled 9

The above parameters are studied with and without considering damping. To see the effect of viscous damping on the structure.

3.5.1 Blast loading

Blast load imparts energy to the structure, causing it to behave dynamically. How the structure (or structural element) responds to blast loading is linked closely to the ratio between its natural period and the duration of the blast. Three response regimes are defined as impulsive, quasi-static, and dynamic (Feng F, 2015). Based on the above three responses, SCI Publication 244 (Yandzio E, 1999) further defines it based on the natural period of the structure as follows:

Impulsive $td/T<0.4$

Dynamic $0.4<td/T<2$

Quasi-static $td/T>2$

Where: td is the duration of the blast load and T is the natural period of vibration of the structural element

To define blast loads, the web-based Kingery-Bulmash (Kingery C, 2016) blast parameter calculator was used for determining the loads on the structure. This program calculates reflected (Pr), time duration (td), and other parameters as well. Before obtaining peak pressure and time duration; the type of blast, the direction of the target, and standoff distance were selected as given conditions. The input procedures and blast impact force-time diagram are shown in Figure 2-2.

The type of explosion is a free air blast that will apply pressure loads on the steel tower and conductor. The blast will be considered as a hemispherical surface burst. The weight and standoff distance will vary to study their effect.

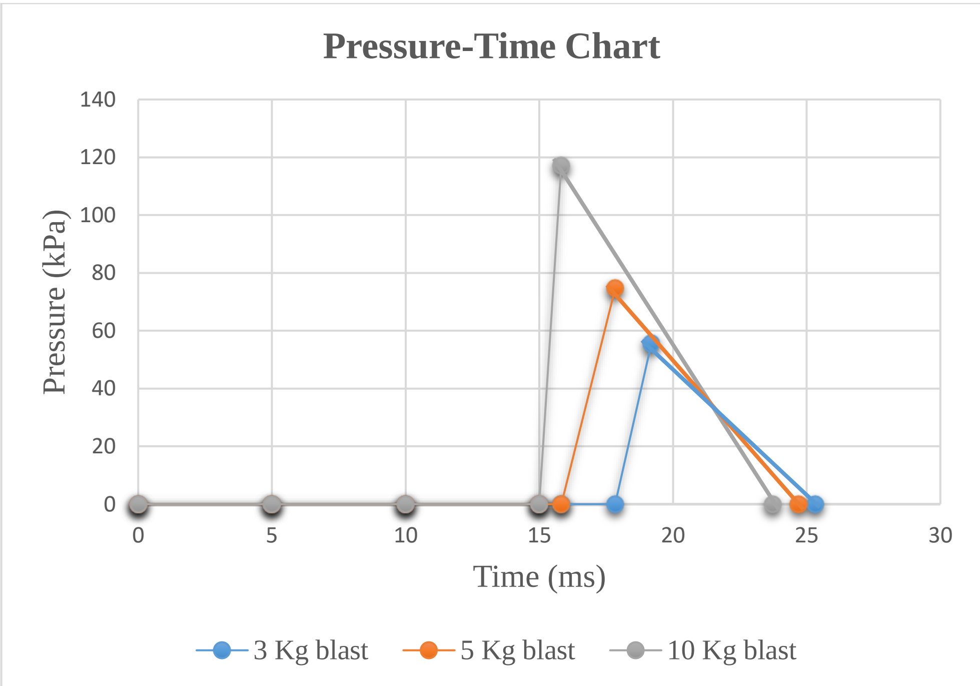


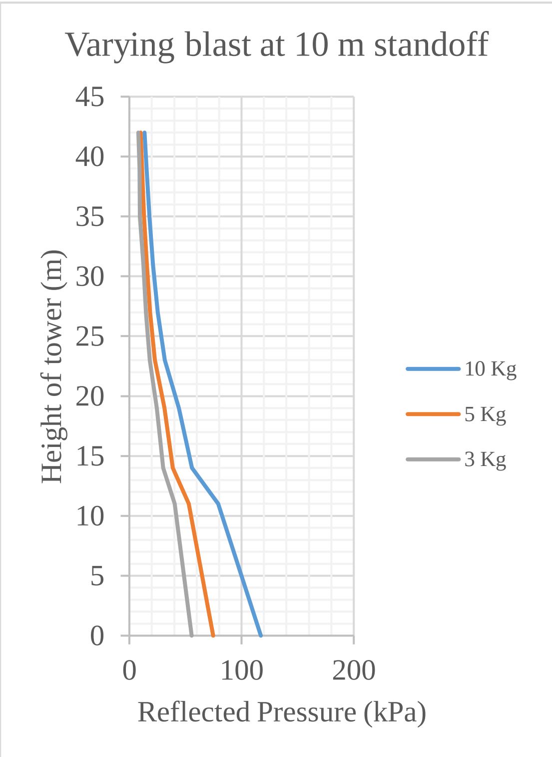
Figure 3-13: Blast loading triangular pressure-time diagram

The tower will be loaded with a lightweight TNT explosion. This is done so that we can study the vibration response of the tower. Material nonlinearity is not considered for both tower and conductor. But the conductor is modeled as a geometrically non-linear member. This is because cables behave in a non-linear manner. The blast loads are quantified such that they don't cause member failure. Failure theory and mechanism has a different concept for calculation. This is not covered in the paper.

Table 3-9: Blast parameters varying with distance

	5	Kg TNT	20	m Standoff
Range (m)	Reflected Pressure (kPa):	Time of Arrival (ms):	Positive Phase Duration (ms)	Incident Impulse (kPa-ms)
20	25.25	45	8.61	45.58
21.18962	23.3	48.34	8.77	43.08
22.82542	21.1	52.81	8.98	40.15
24.41311	19.26	57.3	9.17	37.6
27.58623	16.37	66.36	9.53	33.38
30.4795	14.39	74.55	9.82	30.34
33.6006	12.69	83.56	10.1	27.6
36.89173	11.29	92.81	10.36	25.26
40.31129	10.04	102.97	10.62	23.11
43.82921	8.99	113.15	10.85	21.29

Table 3-7 shows the variation of the arrival time of loads and pressure quantities as we move along the tower. We can also see the variance of the arrival time of blast load with changing height. The difference in pressure arrival time is in the microsecond, the structure is loaded in the same instant.



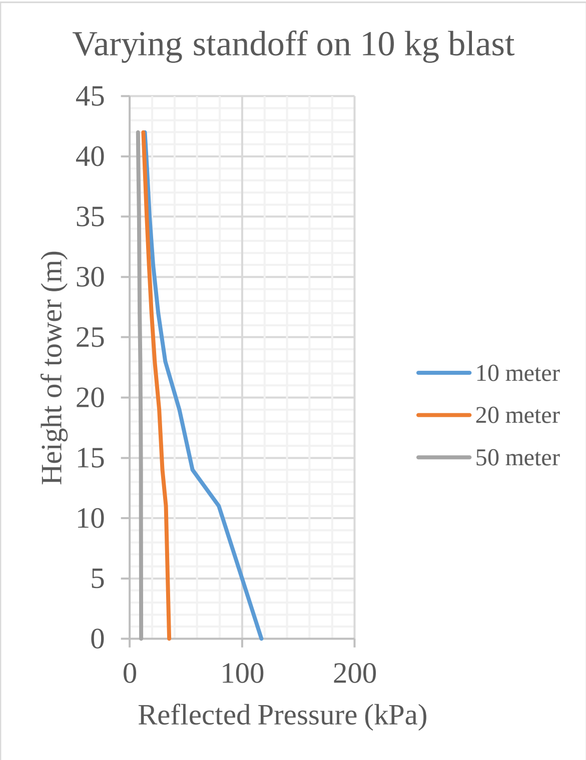


Figure 3-14: Properties of varying blast and stand-off properties along the height of the tower

3.5.2 Free Vibration Analysis

Modeling and calculating the vibration frequency of towers is relatively simple. When coupling it with conductors its analysis becomes complicated. Measurements from a full-scale transmission tower have indicated that conductors affect the vibration of the towers (Momomura Y, 1997) and (Meng Z, 2017). The transmission line structure system is an extensive and large scale that physical experiments are difficult. Due to cost and safety reasons associated with large scale testing, numerical analysis is widely used by researchers to study the dynamic characteristics of transmission structures. Detailed numerical modeling is also inefficient and requires a lot of computing power. Most researchers simplify the tower and conductor into simpler lines and truss elements to ease upon computation requirements. Dlubal-RFEM gives a fair result while modeling the members (tower angles and cable) as they are.

Dynamic loads transmitted to tower structures may be damaging when the effects are amplified by the properties of the members. Studying the natural frequencies of towers and their components is very useful in understanding the dynamic properties of the structure. Towers have many different ways of vibration. These movements of a tower under dynamic load can be discretized into the contribution of several natural modes of vibration.

Table 3-8 lists frequency for the first mode of vibration of different types of power transmission structures. Including the towers studies in this thesis.

Table 3-10 Summary of natural vibration frequencies of transmission towers in previously conducted researches

Study	Tower Type	Height (m)	Mass (kg)	Natural Vibrational Frequency
(Long L, 1974)	Self-Supporting Tower	43	-	5.0 Hz (1st structure mode of vibration)
(El-Attar M, 1997)	Self-Supporting Tower 1	41.6	11,100	1.8 Hz (1st Structure mode of vibration)
	Self-Supporting Tower 2	29.6	7,000	3.5 Hz (1st structure mode of vibration)
(Ghobarah A, 1995)	Self-Supporting Tower	41.6	-	1.8 Hz (1st Structure mode of vibration)
(Yasui H, 1999)	Self-Supporting Tower	71	-	1.3 Hz (1st flexural mode - transversal)
				1.3 Hz (1st flexural mode - longitudinal)
(Mara T, 2013)	Self-Supporting Tower	46.9	-	1.1 Hz (1st flexural mode - transversal)
				1.2 Hz (1st flexural mode - longitudinal)
(Rodrigo F, 2016)	Self-Supporting Tower 1	36.6	23,680	3.9 Hz (1st flexural mode - transversal)
				4.9 Hz (1st flexural mode - longitudinal)
	Self-Supporting Tower 2	57.1	48,277	4.1 Hz (1st flexural mode - transversal)
				4.1 Hz (1st flexural mode - longitudinal)
(Khedr M, 1998)	Self-Supporting Tower 1	48.2	13,500	3.4 Hz (1st flexural mode - transversal)
				3.5 Hz (1st flexural mode - longitudinal)
	Self-Supporting Tower 2	41.6	6,500	3.3 Hz (1st flexural mode - transversal)
				3.3 Hz (1st flexural mode - longitudinal)
	Self-Supporting Tower 3	48.5	5,000	2.4 Hz (1st flexural mode - transversal)
				2.4 Hz (1st flexural mode - longitudinal)
(Meng Z, 2017)	Self-supporting tower coupled with conductors	47.1	-	0.38 Hz Bending in the longitudinal direction
	Self-Supporting single Tower	47.1	-	1.75 Hz Bending vibration in the z-direction

The present study (2020)	Self-Supporting single Tower	42	17,712	2.07 Hz Bending vibration in the x-direction
	Self-supporting tower coupled with conductors	42	60,246	0.37 Hz Bending in the longitudinal direction

According to the information given on the above table, self-supporting power transmission towers have a first mode vibration that is relatively in the same range. When a transmission tower is coupled with conductor cable its frequency decreases significantly. This shows cable lines influence the response of transmission tower in a great deal.

3.5.3 Time History Analysis

To study transmission structure response under blast loading dynamic analysis is performed using the finite element method. Spectrum and time history analysis are two typical approaches used to obtain structural behaviors under dynamic loads. Spectrum analysis is used for earthquake loads. Time history analysis or transient dynamic analysis is a technique used to obtain time-varying displacement, stresses, force, etc. of a structure excited by time-dependent loads.

The linear implicit analysis was performed numerically using Dual-RFEM. The transient dynamic equilibrium equation of the pole was directly solved by Newmark time integration method.

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \{F\}$$

Where [M] is mass matrix; [C] is damping matrix; [K] is structural stiffness matrix; $\{\ddot{x}\}$, $\{\dot{x}\}$ and $\{x\}$ represent the acceleration, velocity, and displacement vector. {F} is an excitation function. Damping [C] is Rayleigh damping discussed in the previous subtopic.

The tower and conductor structure was modeled with and without damping. The damping ratio was assumed to be 2%. This value was taken based on previous works of literature recommendation. The method of damping consideration is clearly stated in the damping subtopic.

Input excitation (blast) was modeled as a triangular impulse loading with a very short amount of time. In microseconds. The duration of the load is equal to the amount of positive phase duration of the blast load.

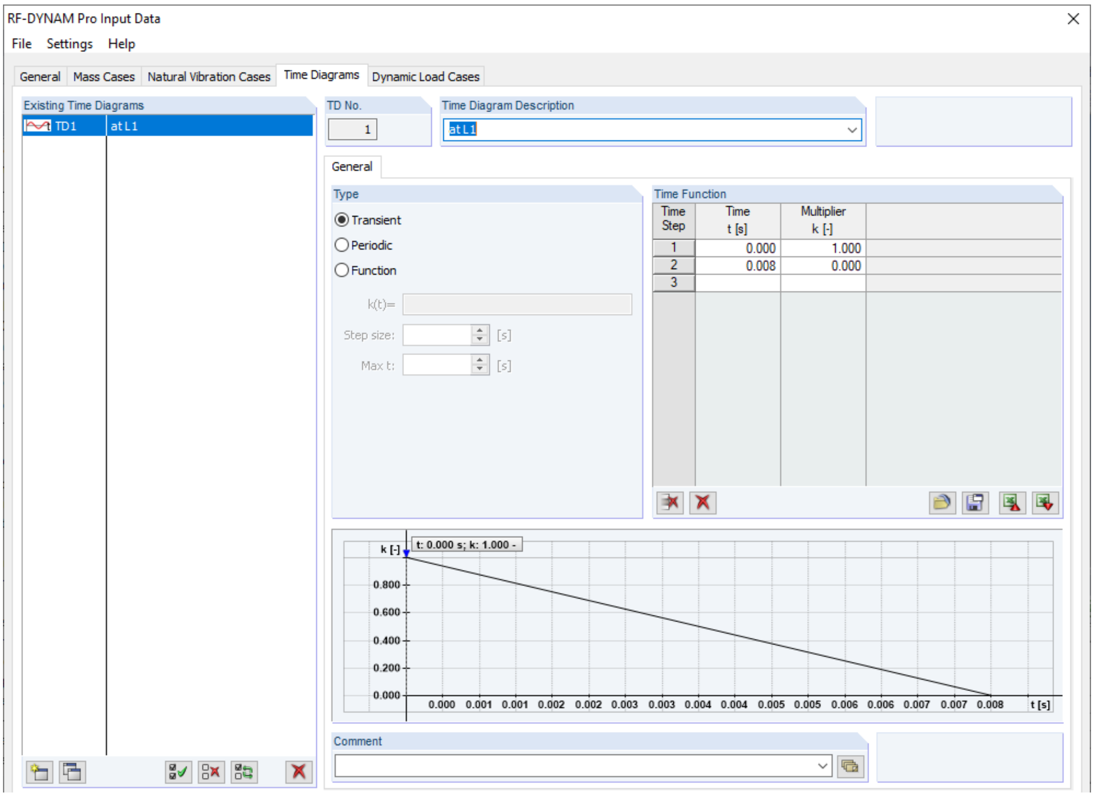


Figure 3-15 Time diagram input of blast load for 3 Kg TNT at 10 m Standoff

Pre-stresses in conductors due to sag and tension are considered within the model.

Chapter 4 Analysis of Result and Discussion

4.1 Modal Behaviors

When numerically modeling and analyzing the structure (both tower-line coupled and uncoupled), three-dimensional natural vibration analysis was performed. The masses act on all three (x, y, and z) directions. A consistent matrix was used in the calculation so that the masses would be distributed following finite element shape functions. In the software program, the Lanczos solver is used for analysis. This solver is recommended for most large structures. The mode shapes are scaled with respect to unity. Global stiffness modification was considered.

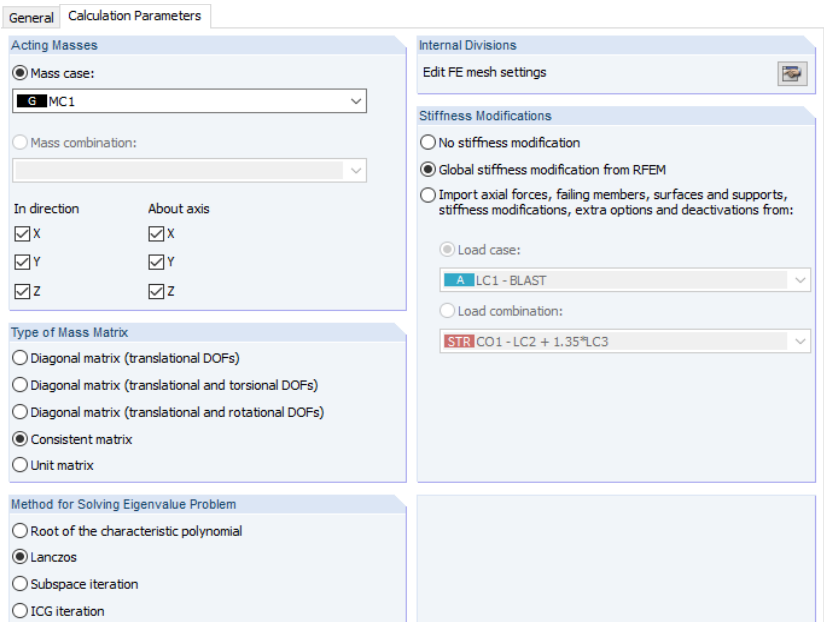


Figure 4-1 Natural vibration case settings input window

The free-standing uncoupled tower has a first mode of vibration frequency 0.648 Hz which is in the transverse direction. In the longitudinal direction, it is 0.675 Hz. The third mode in the torsional direction has a frequency of 1.232 Hz. A slight difference due to the asymmetry of the tower. The first 25 modes of vibration of the uncoupled tower along with effective modal mass are listed in Table 4-1.

Free vibration of the transmission tower-line system was analyzed using the finite element method, as done for the uncoupled tower. The initial 300 orders of the natural frequency of the tower-line coupled system and corresponding vibration characteristics were simulated. The first modes of vibration conform to the conductors behaving in an uncoupled manner. Their natural frequency varies between 0.019 Hz to 0.218. These modes happen in both the short and long span of the structure system. Natural vibration mode number 239th up to 300th orders mostly represents the overall vibration model of the tower-line coupling system. Their frequency ranges between 1.281 Hz to 9.61 Hz.

Table 4-1 Natural vibration modes of the single tower

Mode	**Eigenvalue**	**Natural frequency**	**Natural period**	**Effective modal mass factor**		
No.	**l [1/s^2]**	**f [Hz]**	**T [s]**	**fmex**	**fmey**	**fmez**
1	16.584	0.648	1.543	0.681	0	0
2	18.005	0.675	1.481	0	0.759	0
3	59.906	1.232	0.812	0	0	0
4	166.716	2.055	0.487	0.181	0	0
5	289.063	2.706	0.37	0	0.109	0
6	458.025	3.406	0.294	0	0	0
7	599.848	3.898	0.257	0	0	0.155
8	609.442	3.929	0.255	0	0.001	0.57
9	690.193	4.181	0.239	0	0.007	0.056
10	719.894	4.27	0.234	0	0	0
11	766.533	4.406	0.227	0.002	0	0
12	819.838	4.557	0.219	0	0.007	0
13	837.222	4.605	0.217	0.01	0	0
14	924.879	4.84	0.207	0.025	0	0
15	926.601	4.845	0.206	0	0.001	0
16	1173.966	5.453	0.183	0	0	0
17	1376.952	5.906	0.169	0	0.001	0.01
18	1393.543	5.941	0.168	0	0.001	0.01
19	1539.328	6.244	0.16	0	0	0.066
20	1942.613	7.015	0.143	0	0.008	0
21	2097.021	7.288	0.137	0	0.04	0
22	2414.512	7.821	0.128	0	0	0.006
23	2417.672	7.826	0.128	0	0	0
24	2608.009	8.128	0.123	0	0	0.002
25	2705.888	8.279	0.121	0.054	0	0

NOTE: The effective modal mass participation factor you get for each mode and each direction is a reflection of how the center of gravity of the structure gets excited by that mode. Mode 3, 6, 10, 16 and 23 are symmetric modes. For a symmetric mode, the center of gravity experiences no movement in the direction of symmetry. So as a percentage of effective mass participating in the mode the solution will show zero (Alon F, 2019).

Among the vibration modes, 239th mode represents the tower-line coupled model vibration in a longitudinal direction. Modes from number 240 to 242 represent tower and conductor vibration in transverse direction individually. Whereas 257th mode up to 259th corresponds to the torsional vibration of the tower-line system.

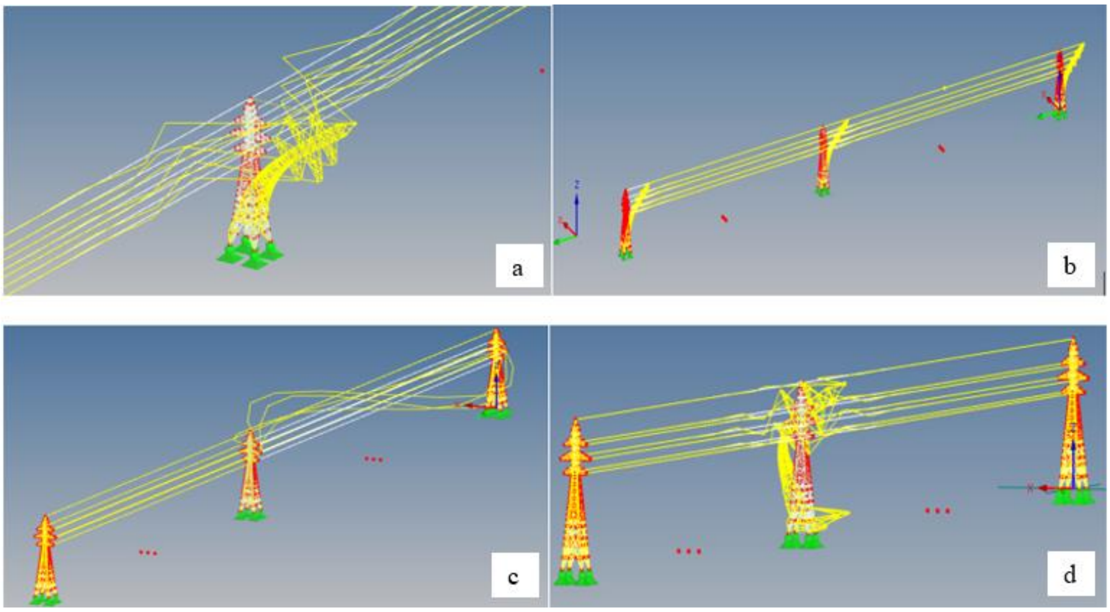


Figure 4-2 Few of the vibration modes of tower-line coupled system a) Mode 240th transverse bending of tower b) Mode 239th longitudinal translation of conductor and bending of towers c) Mode 31th transverse conductor vibration d) Mode 257th torsion of the central tower

Some of the natural frequencies and characteristics of the vibration modes of the tower-line system are depicted in Figure 4-2. When the tower-line coupling system is vibrating as a whole, the dynamic characteristics of the transmission tower are pronouncedly different from that of an individual single transmission tower. The natural frequency of the transmission tower in the tower-line coupling system is much lower than that of the individual single tower in the corresponding vibration direction. In the tower-line system, the vibration of the conductors is more prone to occur at low frequencies, whereas the vibration in the transmission tower occurs at high frequencies. In the tower-line system, the overall vibration of the tower and the line is characterized by low-frequency, dense-mode vibration, which is different from the vibration characteristics of a single tower, demonstrating that, in the design of transmission tower-line structures, the tower-line coupling effect on the dynamic characteristics of transmission tower should be considered.

Due to the complexity of the structure and coupling effect, the tower-line system has a large number of vibration modes. The conductors constitute more than half of the natural vibration frequency mode. When proceeding in time history analysis, damping has a use in filtering out redundant high-frequency modes of vibration.

4.2 Blast load time history analysis

The blast load time history analysis was performed for two seconds. This is because blast loads occur in a very small amount of time (microseconds). The tower was divided along with its height and varying blast load was applied. The explosion was kept minimum because the scope of paper was limited to linear response of the structure. The varying load was applied on the transmission line as well until blast pressure becomes insignificant. Some components of the tower such as insulators and damping devices were not considered in the model.

When modeling blast load for tower and tower-line coupled system there is an obvious difference. Blast load applied on conductors and its effect on a tower is not modeled. The horizontal load effect of the blast will not be accounted for in single tower modeling.

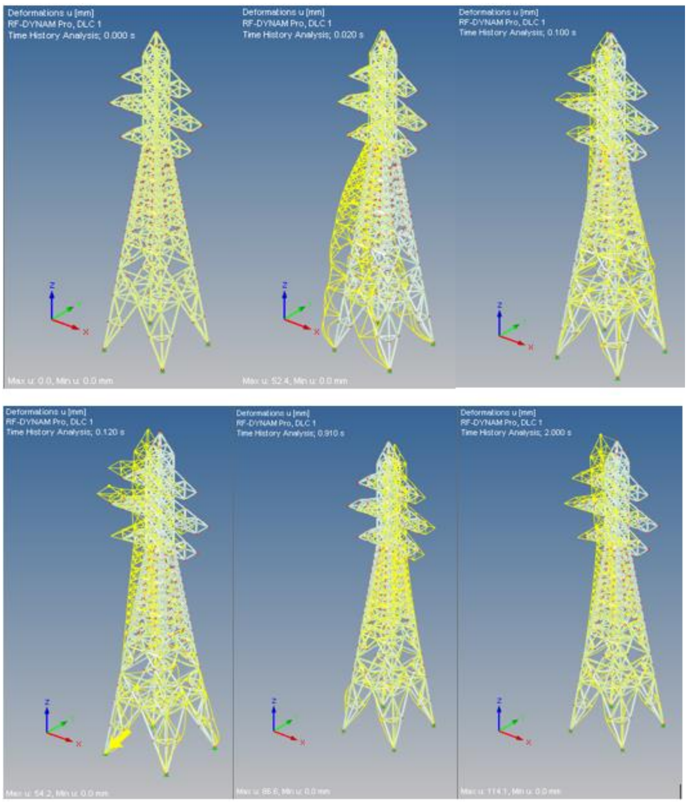


Figure 4-3 Blast loaded uncoupled transmission tower displacement at different time step

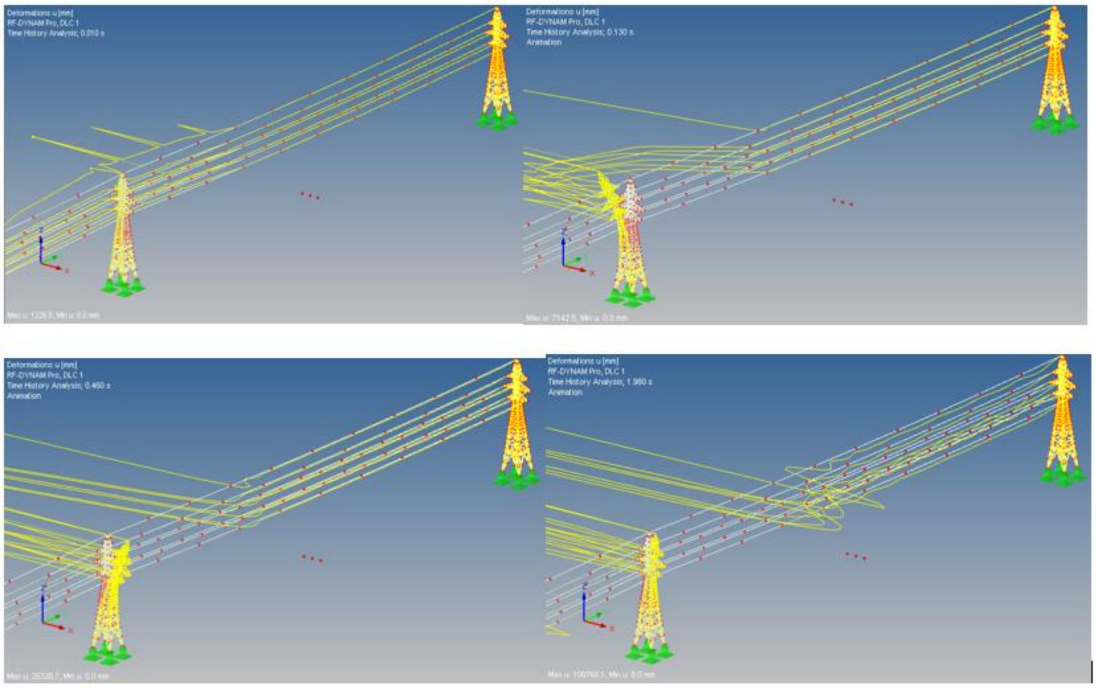


Figure 4-4 Blast loaded coupled transmission tower-line displacement at different time step

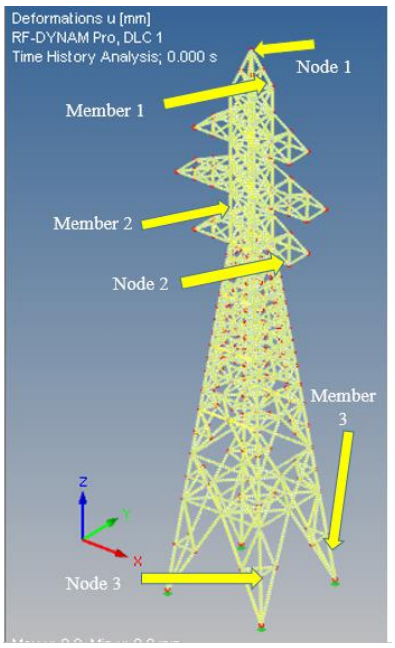


Figure 4-5 Members and nodes studied for different responses

4.3 Effect of different standoff distance with the same charge mass

4.3.1 For single tower

Among the analysis of transmission towers done (36 in total), results of structures with the same blast weight but different standoff distance was studied. As figure 4-6 and 4-7 implies, when standoff distance decreases, that is if blast origin closes in on tower the displacement, torsional rotation and normal force throughout the structure increases significantly. One thing observed from the graphs is that peak response occurs at the same instance for the close-range blast. As explosive loads become further away, the curve flattens out. This is due to the loading of the structure. As blast origin furthers away from the tower, the variance of loading along the height of the tower becomes small and it will be loaded uniformly. The arrival of the last load will also be identical throughout the entire structure.

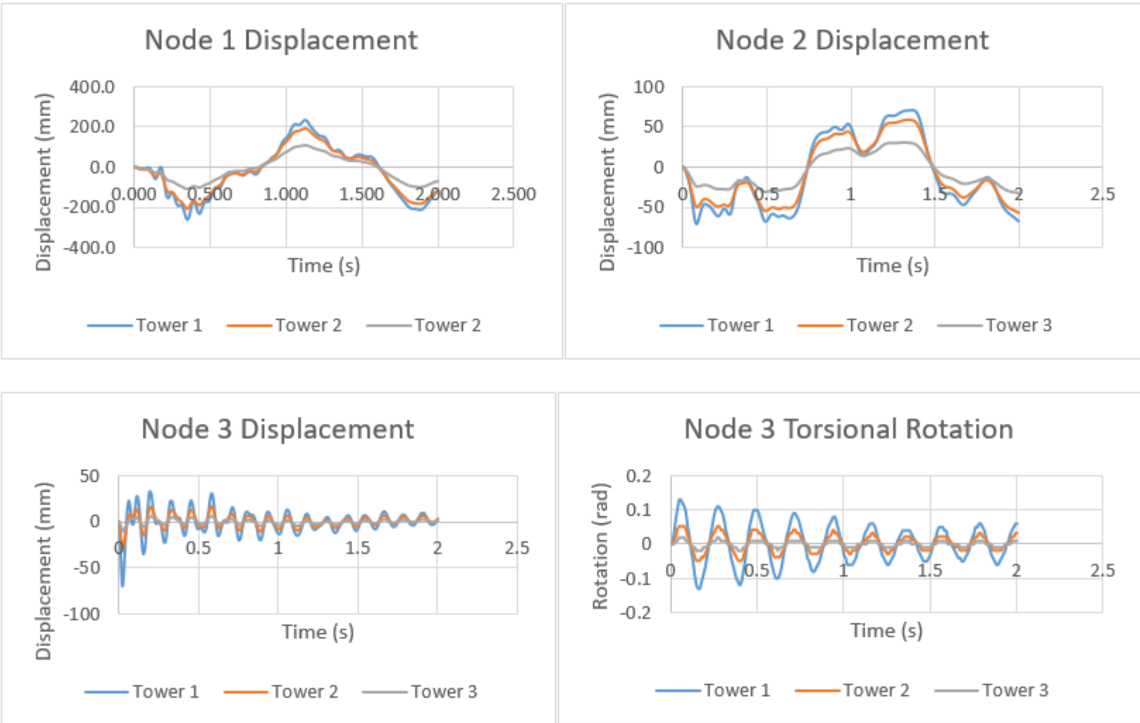


Figure 4-6 Time history response of towers with identical blast weight and varying standoff distance. Tower 1) 3kg and 10m, Tower 2) 3kg and 20m, Tower 3) 3kg and 50m.

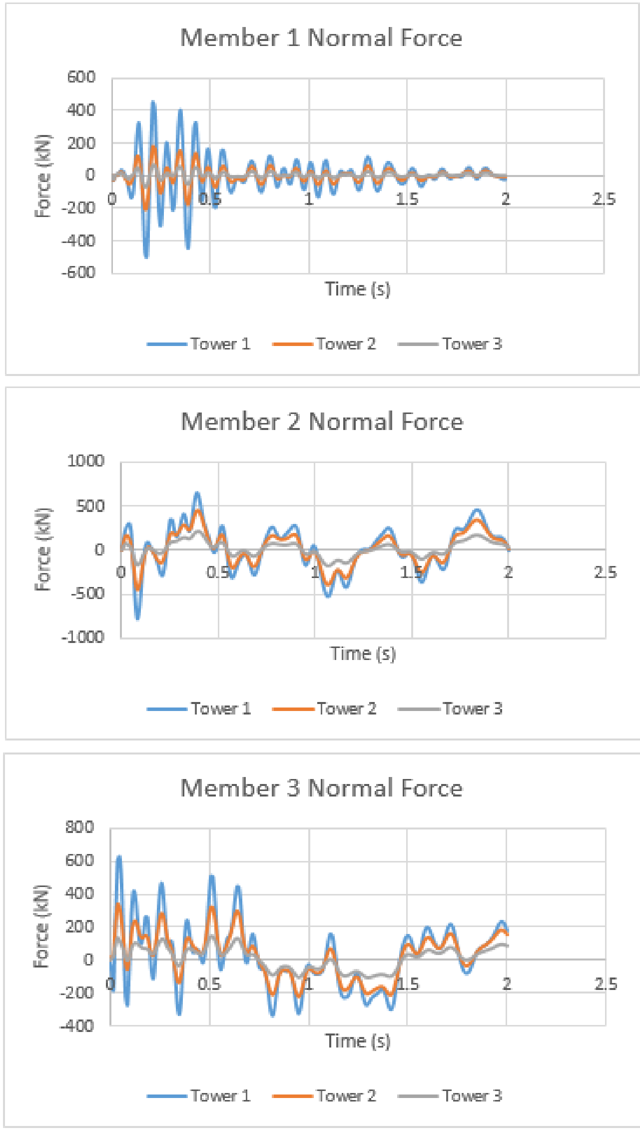


Figure 4-7 Member force response of towers with identical blast weight and varying standoff distance. Tower 1) 3kg and 10m, Tower 2) 3kg and 20m, Tower 3) 3kg and 50m.

The response of the tower along its height is depicted in figure 4-8. Nodes at three different locations respond in an out of phase manner. This behavior arises from a change in stiffness and loading along with the height of the tower. From the graphs, it can be noted that higher modes of vibration are in utilization. Displacement and rotation around the bottom of the tower decrease due to fixity to ground. At the same time, the internal load increases significantly. From the x-axis displacement depicted in Figure 4-8, we can see vibrational difference along with the height of the tower. As node 1 (top of the structure) undergoes two cycles of motion, bottom of the tower has gone more than ten oscillations. Loads sustained at cross arms exceeds from any other location on the structure. This is caused due to additional loads assumed in place of conductors. Due to the symmetrical placement of blast load, no torsional rotation was read around the top of the tower.

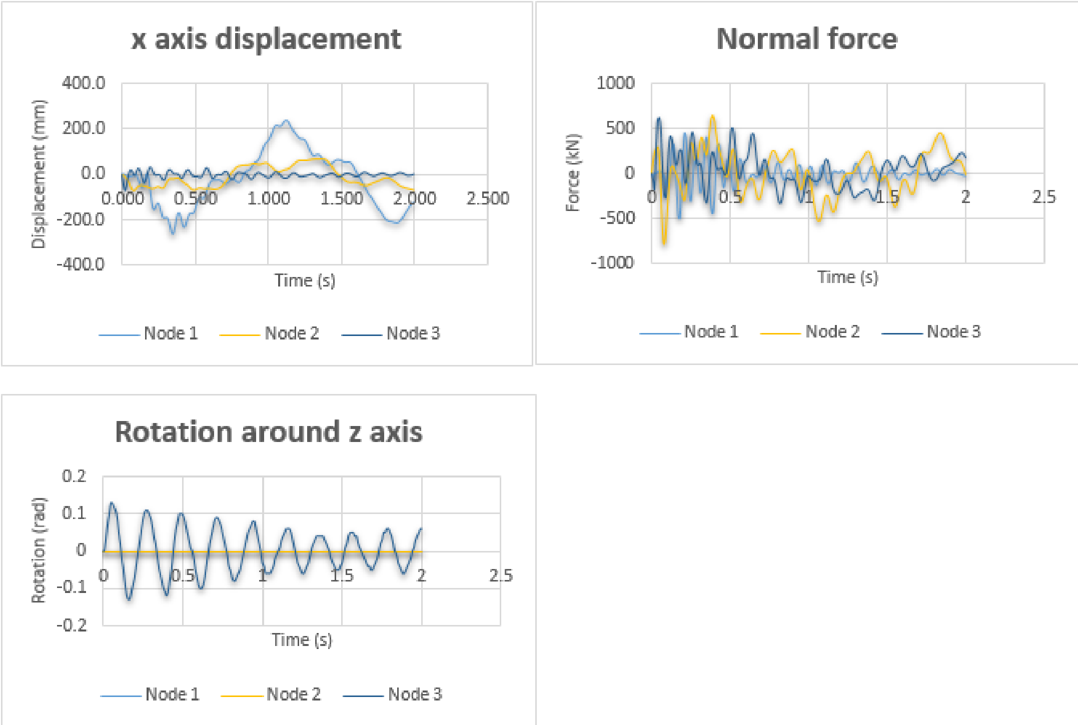


Figure 4-8 Time history response of tower 1 (3kg at 10m) at a different location along with the structure

4.3.2 For tower-line coupled system

The time history response of a tower-line coupled system is depicted in Figure 4-9 up to 4-10. As standoff distance increases response result curve flattens out. The oscillation cycle decreases. Displacement at top of the tower is shown to be greater than other locations on the tower. The displacement time history between 10m and 20m standoff distance is identical in terms of vector and frequency. But they differ in peak magnitude values. As it is for a single tower, when blast origin furthers away variance of loading along the height of the tower and length of the conductor becomes uniform.

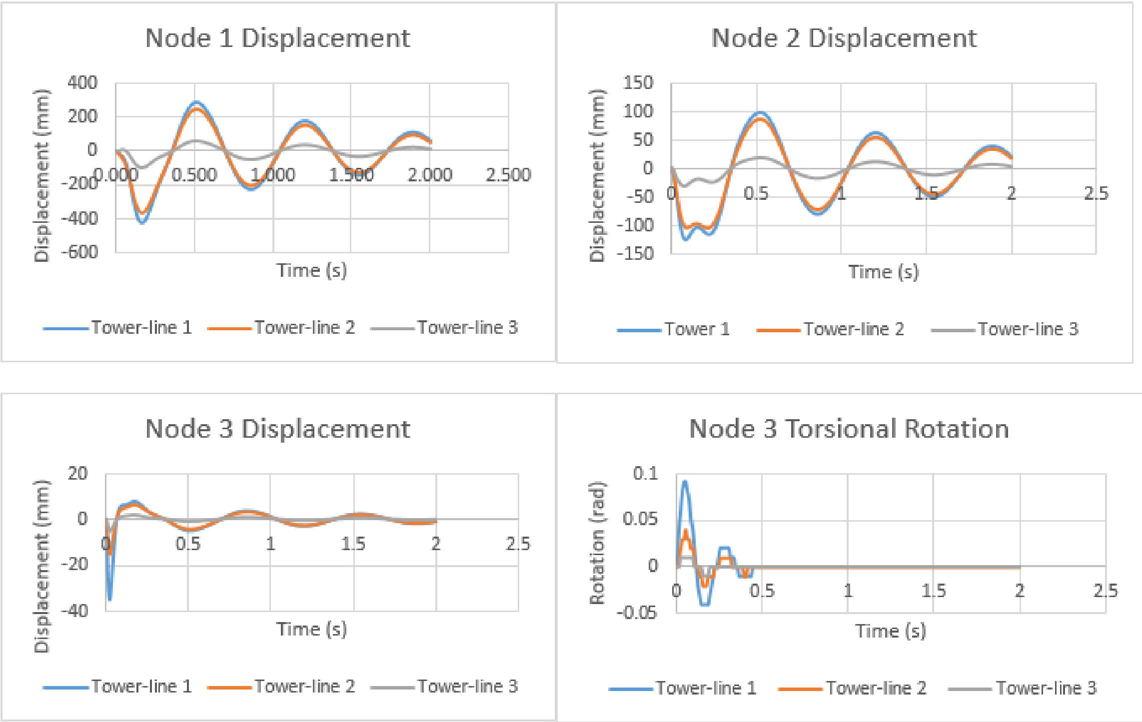


Figure 4-9 Response of tower-line system with identical blast weight and varying standoff distance. Tower-line 1) 3kg and 10m, Tower-line 2) 3kg and 20m, Tower-line 3) 3kg and 50m

Torsional response in the tower-line coupled system is limited to the base of the tower. This might be due to the symmetry of blast loading. As standoff distance increases rotation around the z-axis of the tower seems to diminish. The response ends before reaching a 0.5-second mark.

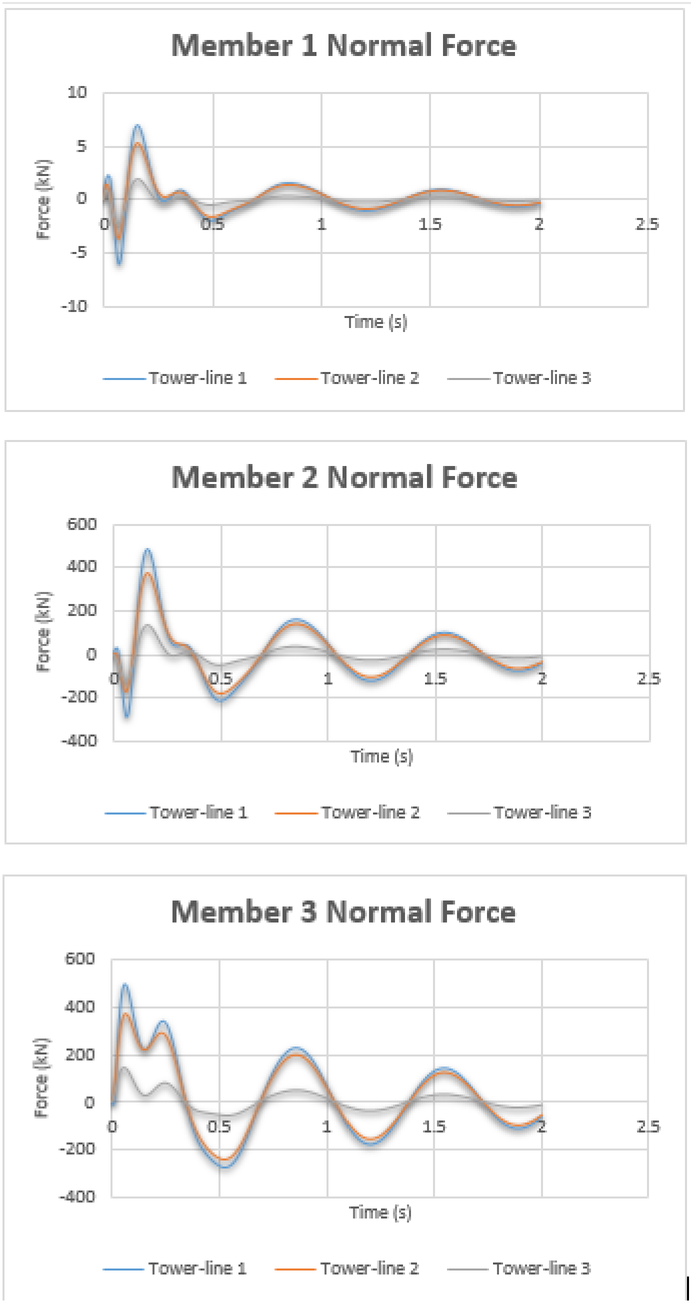


Figure 4-10 Member force response of a tower-line system with identical blast weight and varying standoff distance. Tower-line 1) 3kg and 10m, Tower-line 2) 3kg and 20m, Tower-line 3) 3kg and 50m

4.4 Effects of different charge mass at the same standoff distance

4.4.1 For single tower

To study the effect of charge mass concerning standoff distance tower (1,4,7), (2,5,8) and (3,6,9) result was compared. As the weight of explosives (severity) increases, the response of the tower becomes more violent. This is observed at node 1 (tip of the tower). This behavior arises due to a lack of fixity at the top of the tower. From Figures 4-11 and 4-12 we can observe that mode of displacement the tower undergoes does not change but magnitude increases significantly.

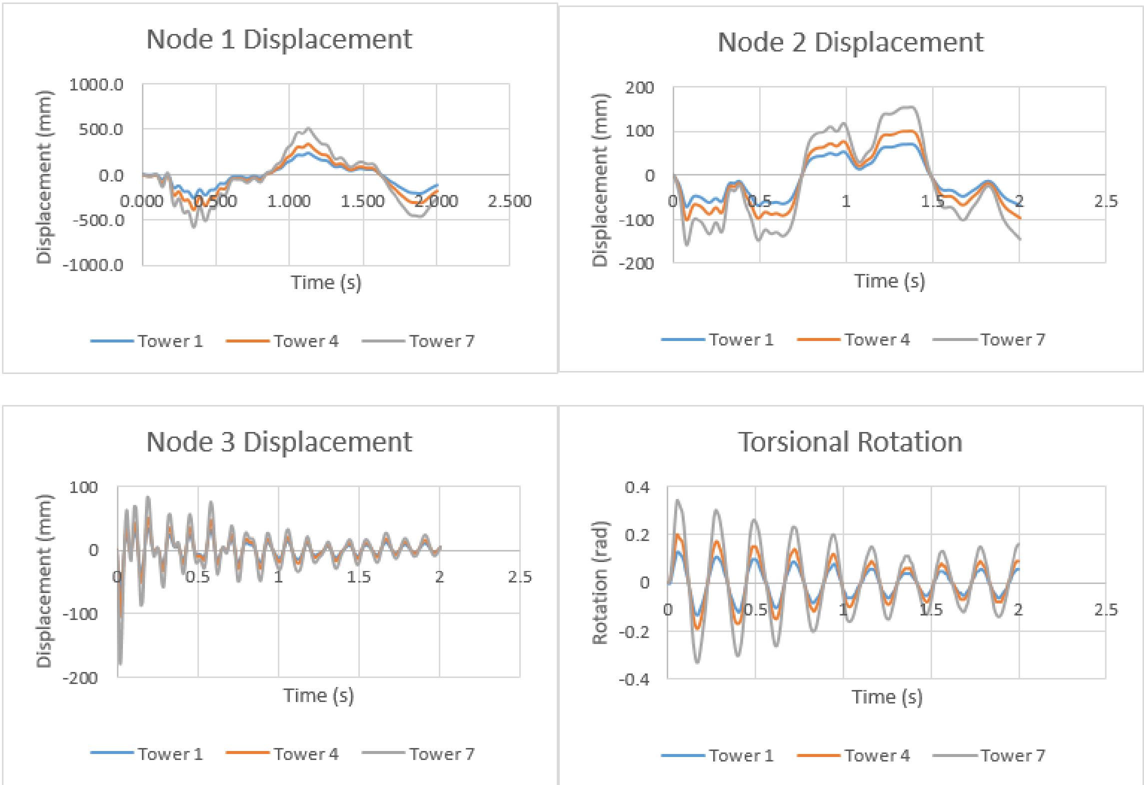


Figure 4-11 Response of tower with identical standoff distance and varying blast weight. Tower 1) 3kg and 10m, Tower 4) 5kg and 10m, Tower 7) 10kg and 10m

Figure 4-12 Member force response of the tower with identical standoff distance and varying charge weight. Tower 1) 3kg and 10m, Tower 4) 5kg and 10m, Tower 7) 10kg and 10m

4.4.2 For tower-line coupled system

The effect of charge weight when the tower is coupled with conductors remains the same as a single tower in terms of response mode. But torsional response diminishes early. From figure 4-13 it can be seen that node 3 displacement makes one significant oscillation then dies down immediately.

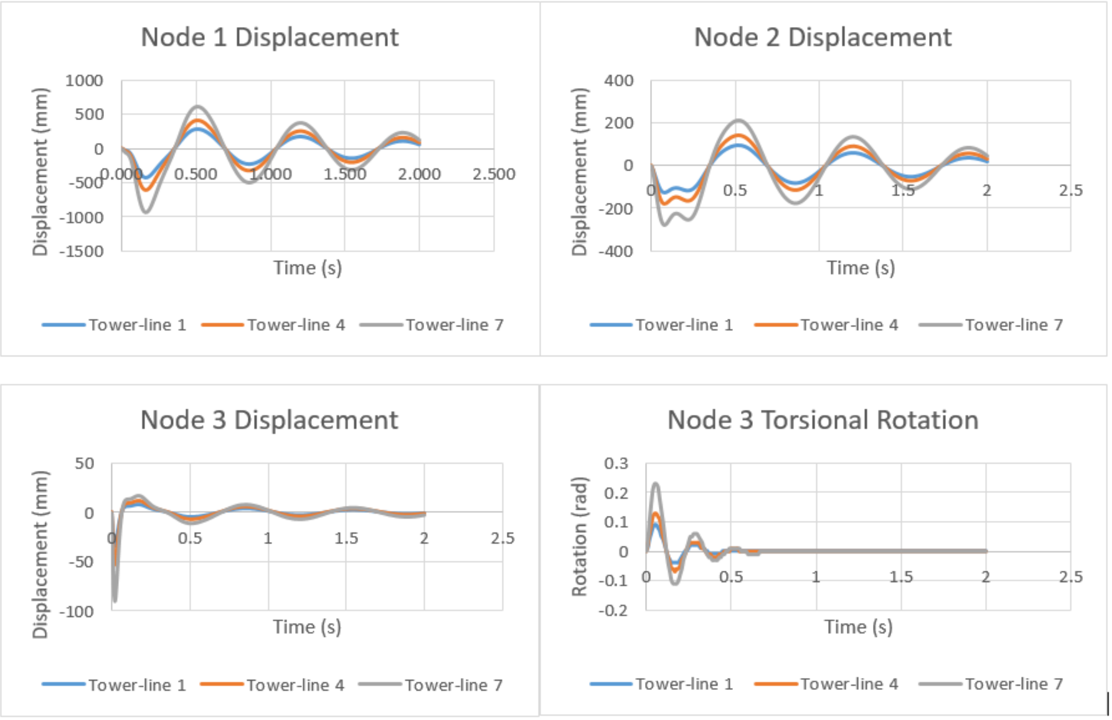


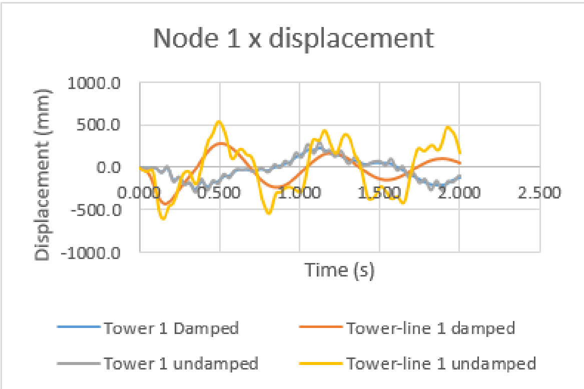
Figure 4-13 Response of tower-line system with identical standoff distance and varying blast weight. Tower-line 1) 3kg and 10m, Tower-line 4) 5kg and 10m, Tower-line 7) 10kg and 10m

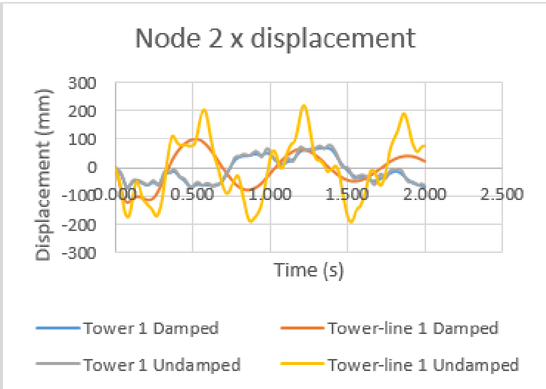
Note: Up on hand calculation for tower 7 it is found that the member has failed. Which goes into failure theory. The member is considered as a marginal element.

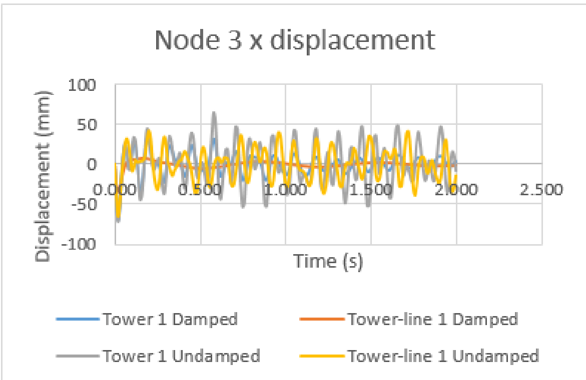
Figure 4-14 Member force response of a tower-line system with identical standoff distance and varying charge weight. Tower-line 1) 3kg and 10m, Tower-line 4) 5kg and 10m, Tower-line 7) 10kg and 10m

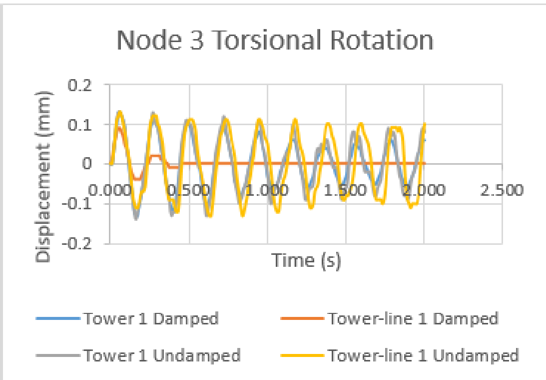
4.5 Effect of modeling the structure as a single tower versus tower-line coupled system

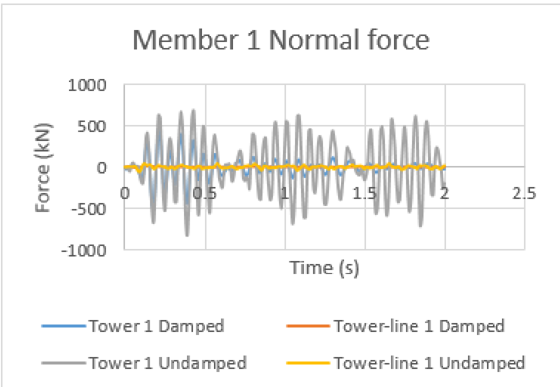
Transmission tower-line systems are complex coupling systems. The coupling effect between the towers and the lines under dynamic blast load has a great influence on the force applied to the transmission towers. However, most existing design codes for overhead transmission lines separate the design of the transmission towers and lines. Blast load sustained by the transmission tower and line was applied as a triangular time history blast load. The linear design theory was implemented when proportioning the tower. The linear design theory is easily implemented, but it underestimates the negative effect of the tower-line coupling vibration on the transmission tower. If a design is done based on transmission tower we cannot get a true response, especially when considering extreme actions. These might lead to an unsafe design. To quantitatively analyze the effect of the tower-line coupled vibration on the transmission tower, this section further calculates the dynamic response of the transmission tower in the transmission tower-line system under different blast load conditions. The results are compared for different parameters of blast between single tower and tower-line coupled.











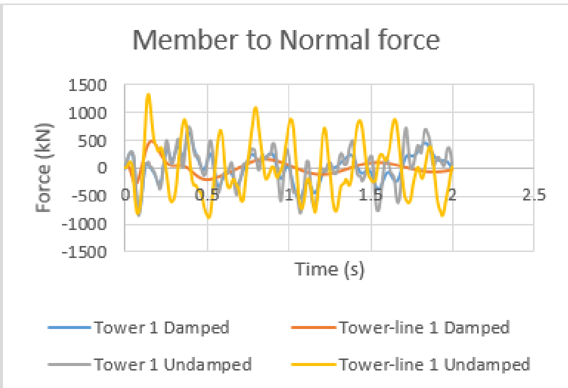


Figure 4-15 Transmission tower displacement and normal force response for coupled and uncoupled tower 1 (3kg and 10m).

We can generally understand from Figure 4-15 that displacement is amplified when tower-line coupling exists. This is due to the non-linear behavior of cable structures. The damping also plays a general role in the response of the tower. Both tower and tower-line system behaved more smoothly when 2% viscous damping was considered. And there is also a decrement in displacement magnitude values. At some points, there was a magnitude change of up to 300%. This shows the vitality of the damping properties of these structures when subjected to impact load.

When it comes to internal forces sustained, the difference between results is amplified even more. A comparison between damped and un-damped structures clearly shows an increment in response values. Figure 4-15 depicts a blast scenario corresponding to 3 kg blast weight to 10 meters' standoff distance. As blast charge weight increases the response becomes more sporadic. Along with the height of the structures as well, the pulse of the tower changes significantly.

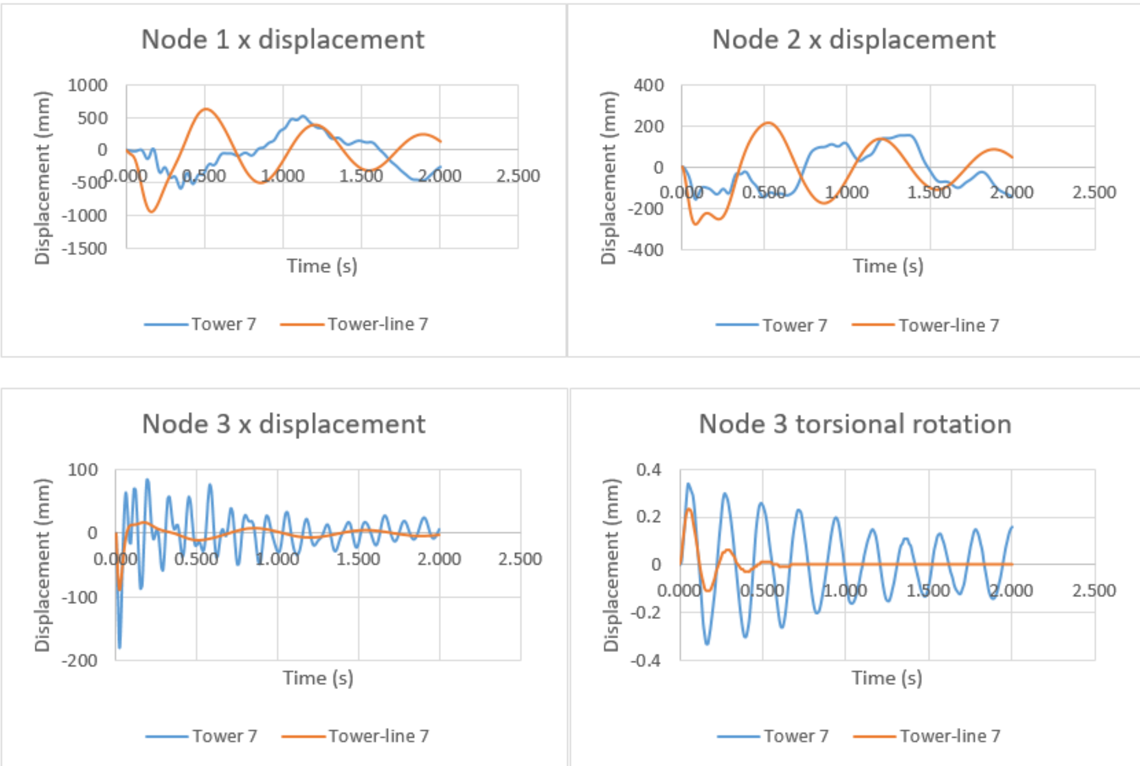


Figure 4-16 Tower and Tower-line 7 damped displacement and internal force response along with the height of the structure

Note: Up on hand calculation for tower 7 it is found that the member has failed. Which goes into failure theory. The member is considered as a marginal element.

For comparison between single and coupled tower-line system, time history response is shown in Figure 4-16. At the top of the tower, displacement is greater in the tower-line system. Around the bottom of the tower, the single tower displacement magnitude becomes larger. Also, the frequency response is higher for a single tower system. When there is a presence of conductor cable, tower response becomes smoother.

When considering normal force created in angle members of the tower, high fatigue creating internal loads occurs in the single tower. Thus, the tower-line system has a relatively smoother force-time history response. The coupling effect has changed the behavior of tower significantly.

Table 4-2 Maximum force comparison between single tower and tower-line coupled system

	Member 1 Normal force (kN)	Member 2 Normal force (kN)	Member 3 Normal force (kN)	Max support force in (x) axis (kN)	Max support force in (y) axis (kN)	Max support force in (z) axis (kN)
TD 1	496.02	791	625.75	139.37	58.8	370.7
TLD 1	6.54	482.31	483.71	438.7	252.43	1758.51
[%] Difference	**98.68%**	**39.03%**	**22.70%**	**-214.77%**	**-329.30%**	**-374.38%**
TD 2	209.43	445	345.5	29.6	54.9	230.6
TLD 2	4.92	373.95	365.76	45.42	171.59	973.27
[%] Difference	**97.65%**	**15.97%**	**-5.86%**	**-53.45%**	**-212.55%**	**-322.06%**
TD 3	72.92	204.68	135.08	90.5	24.2	172.4
TLD 3	2.21	131.22	141.05	172.63	64.93	392.78
[%] Difference	**96.97%**	**35.89%**	**-4.42%**	**-90.75%**	**-168.31%**	**-127.83%**
TD 4	752.35	1153.37	1032.83	903.57	124.61	1331.31
TLD 4	9.39	693.7	694.57	1442.3	362.7	2590.5
[%] Difference	**98.75%**	**39.85%**	**32.75%**	**-59.62%**	**-191.07%**	**-94.58%**
TD 5	261.46	620.33	486.17	364.42	79.88	639.1
TLD 5	6.96	528.7	516.8	668.4	221.1	1407.2
[%] Difference	**97.34%**	**14.77%**	**-6.30%**	**-83.41%**	**-176.79%**	**-120.18%**
TD 6	96.57	311.72	207.36	126.65	34.14	240
TLD 6	3.4	265.25	255.55	247.84	92.9	561.43
[%] Difference	**96.48%**	**14.91%**	**-23.24%**	**-95.69%**	**-172.11%**	**-133.93%**
TD 7	1207.04	1873.56	1719.84	1471.38	198.38	2138.47
TLD 7	15.58	1069.78	1072.2	2457.96	575.3	4259.4
[%] Difference	**98.71%**	**42.90%**	**37.66%**	**-67.05%**	**-190.00%**	**-99.18%**
TD 8	438.58	1004.48	801.1	540.33	124.21	978.27
TLD 8	10.74	813.86	796.6	1056.6	338.3	2181.4
[%] Difference	**97.55%**	**18.98%**	**0.56%**	**-95.55%**	**-172.36%**	**-122.99%**
TD 9	153.33	498.67	330.43	179.87	48.64	339.51
TLD 9	5.62	432.99	416.17	325.76	169.72	914.04
[%] Difference	**96.33%**	**13.17%**	**-25.95%**	**-81.11%**	**-248.93%**	**-169.22%**

Maximum response values are compared for a tower and tower-line system in the above table. An additional table is provided in Appendix A. Percentage difference for both displacement and internal force changes without an obvious observable pattern. For displacement comparison, the percentage of data might be misleading because of the difference in millimeters. Both coupled and uncoupled towers have a relatively similar magnitude of displacement when it comes to internal forces. Contrary to what was expected displacement of tower-line system is greater than single tower in almost all cases studied. When it comes to internal forces in lattice members, again single tower exhibits higher amounts of loads. The big difference occurs at support loads. Difference of up to 374% was recorded when support load single and coupled tower system was compared.

Chapter 5 Conclusions and Recommendations

5.1 Conclusion

Current design codes and guidelines don't have methods or recommendations on blast loaded power transmission towers. Even if there is a recommendation it does not comprehend tower-line coupled effects. Analyzing and designing of transmission towers should include the effects of conductor coupling. This thesis tried to show the effect of tower-line coupling on the response of towers.

The main conclusions that can be drawn from the analytical investigation conducted in the current research project are presented below.

- The dynamic properties of the tower and tower-line system is different. The coupling greatly influences their response to abnormal force. This is observed by their difference in natural vibration mode. For single tower calculated natural frequency ranges from 0.645Hz to 8.279Hz. The tower-line coupled system is in the 0.019 to 9.61 Hz range. The presence of conductors shifts the vibration characteristics of the tower.
- As blast origin furthers away from the tower, the variance of loading along the height of the tower becomes small and it will be loaded uniformly. This has the same effects as wind loading for a very short period. At closer standoff both loading and response become sporadic. For a low level of loading internal forces are halved by just moving blast origin 10 meters further. But this behavior is observed only for the uncoupled tower. For the coupled tower, the response magnitude difference becomes small.
- Charge weight has the same effect as standoff distance. Increasing blast magnitude will make displacement and internal forces greater. Time history response stays the same among the calculated blast load. The difference is observed in their magnitude.
- Damping was not the focus of this thesis. But the towers were calculated with and without damping. Assuming just 2% of viscous Rayleigh damping changed the sporadic response behavior of the tower. The time history responses were smoother. This is observed in Figure 4-15.

- Time history responses for tower and tower-line coupled systems are relatively the same for displacement. It is in the order of hundreds of millimeters. However, their pulse is different. The uncouple tower pulsates more. When charge weight is increased, pulse and displacement magnitude between coupled and uncoupled towers increase greatly.
- Tower 7 member 2 (located at the cross arm) developed a large amount of internal force. Upon checking the member by hand calculation, the member has failed. Which it leads to failure theory (not covered in this thesis paper). It is a marginal member.
- Internal forces in members of the uncoupled tower are generally greater than the coupled system. The main difference arises in the support reaction. There is up to 374% increase in support force. Thus calculating the support force by considering a single uncoupled tower will lead to underestimating critical loads. When designing such structures, coupling effects should be incorporated in the analysis process. This result closely correlates with a research conducted by (Meng Z, 2017). Meng et al has also reported a threefold increase in tower line coupling when subjected to wind loading. This verifies the huge response difference between coupled and uncoupled towers.

5.2 Recommendations

This research provides a significant advancement in characterizing the effect of blast loading in the transmission tower-line system. It also provides a basis for blast load design code development for such structures. Further detailed research is needed to get a comprehensive knowledge in the area. Here are recommendations for future works:

- Studying the effect of blast on foundations of the structure as a huge difference is observed between single and tower-line coupled structure support reactions.
- To include the nonlinear response of transmission tower and conducting failure analysis.
- Study the effect of tower height and type on the response of steel lattice transmission tower. Furthering the study to guyed and tubular towers as well.
- Performing computational fluid dynamic analysis to the model effect of blast load in angle members.

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APPENDIX A

Figure A-1 Maximum displacement comparison between single tower and tower-line coupled system

	Node 1 (x) Displacement (mm)	Node 2 (x) Displacement (mm)	Node 3 (x) Displacement (mm)	Node 1 (y) Displacement (mm)	Node 2 (y) Displacement (mm)	Node 3 (y) Displacement (mm)
TD 1	263.4	70.7	70.5	14.3	3.9	12.1
TLD 1	427.7	125.6	35.2	0.9	0.3	7.1
[%] Difference	**-62.38%**	**-77.65%**	**50.07%**	**93.71%**	**92.31%**	**41.32%**
TD 2	203.6	58.1	29.3	8.6	2.4	5.3
TLD 2	361.1	102.6	15.1	0.4	0.1	5.1
[%] Difference	**-77.36%**	**-76.59%**	**48.46%**	**95.35%**	**95.83%**	**3.77%**
TD 3	108.8	31.9	9.8	3.5	1	2.1
TLD 3	97.7	31.8	5.3	0.1	0	1.9
[%] Difference	**10.20%**	**0.31%**	**45.92%**	**97.14%**	**100.00%**	**9.52%**
TD 4	382.5	102.2	105.4	20.8	5.6	18
TLD 4	615.2	180.9	52.7	1.4	0.5	10.2
[%] Difference	**-60.84%**	**-77.01%**	**50.00%**	**93.27%**	**91.07%**	**43.33%**
TD 5	285.8	81.3	40.9	12.1	3.4	7.5
TLD 5	512	145.7	21.4	0.6	0.1	7.2
[%] Difference	**-79.15%**	**-79.21%**	**47.68%**	**95.04%**	**97.06%**	**4.00%**
TD 6	161.6	47.1	14.8	5.5	1.5	3.1
TLD 6	277.1	82.2	7.2	0.2	0.1	3.5
[%] Difference	**-71.47%**	**-74.52%**	**51.35%**	**96.36%**	**93.33%**	**-12.90%**
TD 7	590.9	160	179.1	33.5	9	30.4
TLD 7	935.1	276.9	89.6	2.3	0.7	16
[%] Difference	**-58.25%**	**-73.06%**	**49.97%**	**93.13%**	**92.22%**	**47.37%**
TD 8	461.8	131	66.6	19.8	5.5	12.4
TLD 8	784.9	223	33.5	0.9	0.2	11.1
[%] Difference	**-69.97%**	**-70.23%**	**49.70%**	**95.45%**	**96.36%**	**10.48%**
TD 9	259.6	75.8	22.7	8.7	2.4	4.8
TLD 9	454	134.9	11.5	0.4	0.1	5.7
[%] Difference	**-74.88%**	**-77.97%**	**49.34%**	**95.40%**	**95.83%**	**-18.75%**