PROBABILISTIC STRENGTH ASSESSMENT OF STEEL POLES FOR POWER TRANSMISSION AND DISTRIBUTIONS

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DECLARATION

I declare that this research work entitled '**Probabilistic Strength Assessment of Steel Poles for Power Transmission and Distributions**' is my own research work and has been composed by me. It has not been presented in any previous work for the award of a degree.All sources of information utilized in this dissertation that are not part of my conception are specifically acknowledged by means of references.

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CERTIFICATION

This Dissertation entitled **'Probabilistic Strength Assessment of Steel Poles for Power Transmission and Distributions'** by MUSTAPHA ALIYU, has been read and approved as meeting the partial requirement of the Postgraduate School of Ahmadu Bello University, Zaria, for the award of thedegree of Masters of Science (M.Sc.) in Civil Engineering.

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NOTATIONS

$A - A =$ Line of application of resultant of wind loads on wires and poles

$BM =$ Bending Moment.

$d_c =$Diameter of conductor

$D_1, D_2, D_3 =$ Outside diameter of top, middle and bottom sections of pole

$f_y =$ Design strength of steel

$GL =$ Ground Level

$G_k =$ Dead Load

$h =$ Height of conductors on cross-arm from GL

H = Height of pole above ground level

I = Moment of Inertia

$l_1, l_2, and\ l_3$ = Distance in m from A-A to bottom of each section

$M_r =$ Resistance moment of the pole

$M_a =$ Applied moment of the pole

$n =$ Number of conductors

$P_1 =$ Equivalent wind load on pole

$P_2 =$Equivalent wind load on conductor calculated as acting at A-A

$P =$Total wind load as acting at A-A, $P = P_1 + P_2$

$P_f =$ Failure Probability

$R_{(t)}$ = Resistance at time t

s = Sum of half the spans on each side of pole

$S_{(t)}$ = Load effects at time t

V_{res} = Ultimate shear resistance

V_{app} = Applied shear force

W_k = Wind Load

ρ = Wind pressure on flat surface, N/m^2

δ_{max} = Maximum deflection

δ_{per} = Permissible deflection

φ_v = Shear resistance factor

β = Reliability index

φ = Standardized normal distribution function

ABSTRACT

Electric transmission structures are unique civil structures used to support conductors and shield wires of a transmission line. In this research work, a probabilistic assessment of the strength of steel poles in service was studied. Resistance of the steel pole, ultimate strength of steel, section modulus, cross sectional dimensions of the pole, distance at which the load acts on the pole and the magnitude of the load acting on the pole were treated as random variables, which can be significantly influenced by time and location since the subject of structural reliability offers rational framework to quantify uncertainties mathematically. The safety levels of electric steel poles under uncertain loadings using First Order Reliability Method (FORM) in MATLAB were obtained. The determination of the response of the pole to loads was accomplished by the aid of Finite Element (FE) coded in software, ABAQUS/CAE.The stress distributions were effectively measured in the software. The result in ABAQUS shows that the steel pole did not suffer any reasonable deformation when loaded as stresses obtained are far less than the yield strength 500N/mm^2. However, thereliability analyses in MATLAB gave lower values of reliability index, β, (1.4802E+00) and probability of failure P_f (6.9407E-02) for moment failure mode while higher values of β (2.339E+01 and 5.1245E+01) were obtained respectively for deflection and shear failure, with both negligibleP_f of $0.100E-10$. The effect of variation of parameters like thickness, diameters and length of steel poles were also studied by sensitivity analysis.

CHAPTER ONE

INTRODUCTION

1.1 Preamble

Electric transmission structures are unique civil structures used to support conductors and shield wires of a transmission line. They are either lattice type or pole type structures. Transmission poles can be wood, steel, reinforced concrete and composite material poles.Pole type structures are generally used for voltage of 345-kV or less, while lattice steel structures canbe used for the highest of voltage levels. Tubular steel pole structures are commonly used in the power industries mainly due to itsconstruction efficiency and light weight product (Kaoshan, 2009).

In the event of natural disasters, the continuous supply of electricity is essential for the welfare, economy, and security of our societies. Steel poles are by far the strongest material used in power pole construction but require less maintenance than the other varieties. Consequently, steel poles are of great significance (Kenney, 2017).

Generally, the power system includes three main components: generation, transmission, and distribution. Of these three components, the distribution systems (lines and poles) are the most susceptible to wind damage. This is due to the fact that distribution lines and poles are more exposed to winds than the generation plants and the transmission systems. Furthermore, the distribution poles are often not designed to withstand high wind speeds. However, when it comes to failure due to natural hazards, the distribution system is the most vulnerable (Davidson *et al*., 2003).

In recent years, utility companies have been searching for cost-effective alternatives totimber poles and other pole materials due to environmental concerns, high cost of maintenance, and need forimproved aesthetics (Lacoursiere, 1999). In North America, for example, steel poles usage has more than tripled, as reported among the estimated 185 million electric distribution poles that crisscross the United States and Canada; 600 utilities use steel pole alternative.

Polesmay have one of the three basic configurations: horizontal, vertical, or delta, depending on the arrangement of the phase conductors. In areas with severe climatic loads and/or on higher voltage lines with multiple subconductors per phase, designing wood or concrete structures to meet the large loads can be uneconomical. In such cases, steel poles become the cost-effective option (Stewart and Goodman, 1990).

Steel poles hold up the wires and cables that bring electricity and other modernamenities from the power and cable companies to our homes. These poles help provide for the growing network of telephones, televisions, computers and satellite gadgets. Though wires are frequently covered underground in new grid expansions, there are still roughly millions of utility poles in service in the world(AISI, Environmental Literacy Council, 2005).

The overall sustainable economic growth and the well-being of a nation depend heavily on the functionality, reliability and durability of its constructed facilities;it is for this reason that the strength of these steel poles in service need to be assessed in a stochastic manner, while accounting for all the parameters associated with their performance in service during their design life.

There is considerable investment now in steel poles worldwide, and there is a need to examine the structural reliability and probability-based assessment of these power distribution poles. Given the scale of this infrastructure,it is reasonable to assume that even a small improvement in the design and maintenance strategies would lead to considerable cost savings and improved strength of these poles and safety of lives (Paraic *et al*., 2014).

In a research conducted by Mankowski*et al*. (2002), 261 North American utility companies were surveyed. 116 of the companies reported that they had employed steel poles as substitutes for timber poles within five (5) years before the survey. Other materials such as fiberglass, concrete, and laminated poles were also reported to have been used by some companies. Steel however, was the most commonly used substitute.

Steel poles have several advantages over timber poles, including reduced maintenance cost, predictability of behavior, consistent performance, insusceptible to wood-pecker attacks and rottening, light weight; factory pre-drilling is possible, environmentally friendly, recyclable, no toxic preservatives, no disposal concerns, and superior life-cycle cost (Lacoursiere, 1999).

Collapse of transmission poles is not a well understood phenomenon. These poles are subjected to various loads like wind, snow, icing and earthquake. Comparatively, wind loads are more complex for these poles due to high geometric non-linearity and randomness of wind turbulence（AlokDua*et al*., 2015）.

Another important consideration is the consequence of power pole failures, which are primarily caused by wind loading of deteriorated poles (Winkler *et al*., 2010).

The most appropriate approach for improving failure rates for poles is probabilistic assessment. Probability-based methods provide essential asset management tools in other areas of civil engineering, such as road management, bridge management, etc., due to the ability to incorporate and quantify uncertainty and variability acrossaninfrastructurenetwork(O'Connor and Enevoldsen, 2009). Surprisingly,however,there is littleresearch utilizing probabilisticmethodstoexamine the structuralreliability of steelpowerdistributionpoles. This representsanimportant gapintheexisting literature as probabilisticanalysisisishighlyappropriate to the management of steel powerpolesnetworks, whichexhibit high elemental variation within other types of material poles duetodifferencesin strength characteristics, durability,loadinganddeterioration conditions polesaresubjectedto, since it is recognized that all materials are susceptible to environmental degradation in service. The degradation processes for various pole materials are such that it is very difficult to develop accurate predictive models that can be used to modify design strength factors(Paraic *et al.,* 2014).

Properly estimating the reliability of a transmission pole structure is a complex problem. It requires knowledge of the joint Probability Distribution Function (PDF) of the load-producing events such as ice, wind, temperature, wind direction, and the PDF of the strength properties of the pole, and the evaluation of multiple correlated failure modes including bending, compression, buckling, connections failures, and foundation failures (Dagher, 2001).

In the past, naturally occurring events such asearthquakes have caused massive damages to poles and disruption of power supply, in addition to direct loss to property and lives inflicted by earthquake by damaging the infrastructure; disruption in electricity supply results in huge loss of revenues and interruption of industrial activities. The reliability and safety of electrical transmission and distribution systems after earthquake depend on the seismic response of individual components such as substation equipment, poles, etc. Hence, steel poles located in seismically sensitive regions have to be designed to withstand possible earthquakes and the most commonly used methods for seismic qualification are Finite Element Analysis and the shake table tests.A Finite Element Analysis (FEA) program would be used to validate the strength of the pole. If the pole were too weak, it would not hold up under the stress of the attached wires, if it were over designed, its lifecycle would require more energy(Ramesh andPanneer, 2012).It is for this reason that the probabilistic strength of steel poles for power transmission and distributions is evaluated in this work in order to improve on their use and efficiency in service.

1.2 Problem Statement

According to Ditlevsen and Madsen (2005) engineering judgment is the art of being able to decide whether results obtained from a structural analysis or design model is sufficiently realistic and cost effective that the engineer dare base his or her practical decisions on these results. With the increasing demand for electricity around the world and during peak weather inclemency, such as during heavy storms, rain, strong winds, and sometimes owing to line snaps or as a result of damages due to collision by vehicles, the need to provide a reliable pole material such as steel will sufficiently help in distributing power to meet various needs.

1.3 Justification of the Research

Over the last decade, many electricity companies have started to convert wood and concrete poles to steel. Steel poles are strong, durable and cost effective option for distribution systems. However, when considering the cost of steel poles, it is important not only to consider the initial cost, but more so its life cycle cost due to its maintenance free longevity. Traditionally, wood poles have been used predominantly to carry electric wires and cables from point to point, but the changing market is beginning to openly embrace steel distribution poles as a viable and cost-effective alternative. The cost usually accounts for 30 to 40% of the total cost of a transmission line (Shu-jin*et al.,* 1999).

Steel poles have consistently proved to stand tall in the event of a pole failure. An engineered product, each steel pole is designed to meet specific strength and load requirements with uniform dimensions and strength, without twists, knots, splits, or bows, all of which can lead to failure in severe situations. Steel poles can save time and money; cost savings can be significant when the following factors are considered (Synder, 2005):

(a) Steel poles are at least 30% lighter than wood poles - lighter weight can reduce the cost of transportation, handling, and construction. Hauling the materials to the site must also be considered in evaluating constructability. Transporting concrete structures, which weigh at least five times as much as other types of structures, will be difficult and will increase the construction cost of the line. Heavier equipment, more trips to transport materials, and more matting or temporary roadwork will be required to handle these heavy poles.

(b) Steel poles require little maintenance, greatly reducing the costs associated with up-keep; there is little need for tightening hardware to compensate for pole shrinking.

(c) At the end of its long service life, a steel pole can be sold to scrap dealers and completely recycled, eliminating the costs of pole disposal.

1.4 Aim and Objectives of the Research

1.4.1 Aim

The aim of this research is to evaluate the strength and safety level of steel poles used for power transmissions and distribution.

1.4.2 Objectives of the Research

The objectives of this research are:

(i.) To determine the statistical properties of the basic design variables for the failure modes of electric transmission poles.

(ii.) To determine the stress distributions in the poles using Finite Element Method (FEM) coded in ABAQUS.

(iii.) To develop the limit state function for various modes of failures of electric transmission poles.

(iv.) To perform structural reliability analysis of steel poles using First Order Reliability Method with the aid of FERUM, in order to determine the safety level of electric transmission poles.

1.5 Scope and Limitations

1.5.1 Scope

This research work focuses on executing probabilistic analysis on the strength of steel poles to determine the implied safety levels and checks the requirements for the design and erection of the poles; while using relevant codes and software.

1.5.2 Limitations

This study is limited to finite element analysis and reliability/stochastic method of analysis;also lack of some relevantdata not available for the estimation of some random variables.

CHAPTER TWO

LITERATURE REVIEW

2.1 Concept ofPower System

Electricity plays an important role in the socio-economic and technological development of every nation. A fast developing country like Nigeria needs adequate supply of electricity not only for industrial usage but also for domestic purposes; and with the level of development, the use of electric poles has been on the increase as a result of electrifications everywhere (Oyejide*et al., 2014*).

Research conducted by Adejumobi and Adebisi(2001) reported that the electricity utility is far from within the reach of the majority of the populace with so many challenges facing the country as a result of inadequate and low power supply.

The power system is one of the instruments used for converting and transporting power to meet various demand. The main function of the power system is to convert energy from one form to another and distribute it to the consumers. Traditionally, the electric power systemhas three main components that can be broadly divided into three subsystems, they are: generation, transmission, and distribution (William, 2005).

The generation aspect deals mostly with the source of power, ideally with specified voltage and frequency. For example, the voltage levels for transmission system ranges from 34.5 kV to as high as 1100 kV in the United States of America (Brown, 2008). The transmission usually are the links between the generating stations and the distribution system in terms of voltage levels, ittransports the power in bulk using the transmission system that uses wires supported by steel towers that are about 45m high and spaced about 240m. apart (Willis andPhilipson, 2005). In other words, the distribution system is part of the system between transmission and the consumer service point (Pabla, 2008).

The distribution component deals with the distribution of electricity at lower voltages for daily consumption via poles. Poles are of two genres, first the utility poles which are grouped into two kinds; utility transmission and utility distribution and the second genre includes poles for lightening, traffic and even for intelligent traffic structures. Both genres of poles are analyzed and designed by the same structural principles, but they differ in governing codes and industry practice. (Crosby, 2001).

The advancement in science and technology has drastically changed over the decade, and has added a striking contrast to older technology, for instance, Uchida (1991), reported the progress of human civilization which has depended largely on the science and technology of materials for the manufacture of tools and facilities, particularlythe contributions of high-performance electrical steels to power productions.

George and Stetson (1999) identified the importance of the use of steel poles for the construction of electrical power poles in central Nebraska, which can be used for a single 69kV pole line. They further reported that steel poles were initially considered to extend the life of distribution lines and cheaper than timber poles with better bending capacity and weight even 50% less than timber poles. They both concluded that the strength of all poles deteriorates with time, which reduces their reliabilities and makes them more susceptible to damage. For timber poles, the reduction of strength is mainly due to decay caused by fungi at areas where the pole is in contact with the ground. The strength of steel poles, however, deteriorates due to corrosion of the steel at or below the ground level caused by moisture and other chemicals in the soil. Corrosion reduces the strength of the poles by reducing their cross-sectional areas.

Little research has been done to investigate the advantages of using steel poles in place of other poles to minimize the damage to the distribution system, when subjected to winds and naturalhazards especially as the pole ages. As companies and the world increasingly adopt the use ofsteel poles, there is a need to look at their long-term structural behavior, foundation design and strength effectiveness and how it compares to other poles in terms of performance.

Researchershavepublished and reported so manystudiesexamining structural reliabilityandwindvulnerability for timber poles inanAustralian context (Stillmanand Darveniza, 1991; and Stillman *et al*., 1997), withmorerecentstudies being carriedoutintheUnited States of America andEuropean contexts (Gustavsen and Rolfseng., 2004;Bjarnadottir and Stewart., 2014). Although thesestudiesrepresentvaluablecontributions tothe existingsparseliteratureinthisarea,continuedresearchare necessary to help improve power poles in service.A realistic assessment of the structural reliability of a distribution pole must incorporate the effect of inspections and subsequent maintenance replacements.

Gustavsen and Rolfseng(2004),also carried out probabilistic studies on poles considering maintenance, where they examined the effect of no network maintenance and suggested that once a pole deteriorates to 90% of its original capacity,it should be replaced.This 90% capacity replacement criterion seems somewhat stringent when it is considered that the correspondingfigures fortheUnitedStates of AmericaandAustraliaare66%and50%, respectively (NESC, 2002; NS, 2001).

Quadri and Afolayan (2016) conducted a probability-based assessment of power distribution concrete pole in Southwestern Nigeria. They reported that these poles have non uniform height as the reliability increases at any given reinforcement ratio, with the criterion for safety also violated. They suggested that when comparing the levels of reliability associated with the effects of overturning moment and deflection, the most critical failure mode for tapered power distribution concrete poles is deflection and advocated for newly designed facilities.

Phoon(1995)conducted a reliability-based design of foundations for transmission structures and concluded that a target reliability index of 3.2 is the most appropriate for foundation ultimate limit state design, while a lower reliability index of 2.6 was chosen for serviceability limit state. He also concluded that his research provided a promising foundation design, because they can account for the uncertainties in determining soil properties at investigation sites and in translating these properties to each foundation locations.

2.2 Utility Distribution Poles and Structures

Utility distribution poles often support wires and other components for many utilities such as electric power, telecommunications, and cable television. Transmission lines typically carry the electric power from the source to substations where distribution lines branch off to supply the surrounding businesses and homes with power (RUS Bulletin 1724E-20,2009).Other components of the structure include various utility wires spanning from poles to poleand other equipment (Crosby, 2001). Electric wires and cables are located overhead on utility poles as an expensive way to keep them insulated and out of the way of people and vehicles.

2.2.1 Types of utility distribution poles

Generally, utility distribution poles comprise of three common types:Tangent poles, Guyed poles and Self-supporting poles (Theresa, 2017). They are discussed briefly below;

2.2.1.1 Tangent poles

Tangent poles (almost always wood) can be identified as the poles that do not have down guy wires and are in a straight line with other poles. Tangent poles act as simple cantilever beams and/or slender columns. According to Rural Utilities Service construction standards, tangent poles may have a maximum line angle of 5 degrees (RUS Bulletin 1724E-150, 2009). Because tangent poles are not to be located at a sharp angle turn in the line, they typically resist only the forces due to wind, ice, gravity, and the forces from unbalanced tension in the conductors or other utility wires.

2.2.1.2 Guyed poles

The purpose of pole guying is to support a fully loaded design tension of conductors, and help also in supporting applied wind and even ice loads. The strength of the guy-anchor assembly is dependent on; strength of the guy wire de-rated to 90%, strength of the guy attachment including the bolt and washer; Strength of the anchor and rod, holding power of the soil in which the anchor is installed. When a pole is guyed, it is safe to say the guy will satisfy all the transverse or dead-end loads and this will mean the pole acting as strut. Horizontal loads also exist above a guy and moments at the point of guy attachment should be checked. Even if a pole can withstand certain loads,the soil may not generate sufficient resistance to prevent the pole from overturning. In this case, the best approach will be to guy the pole (www.redvector.com).

2.2.1.3 Self-supporting poles

Self-supporting poles are utilized when guy-wires cannot be used to compensate for additional load;they are typically made of steel or concrete (Theresa, 2017). Self-supporting poles are not common on distribution lines, but are required where there is no guying option.Another instance when self-supporting poles are used in lieu of tangent or guyed poles is where the grade of construction increases from the typical grade C to the more stringent grade B, such as crossing of distribution lines. The higher strength is required because of theincrease in grade of construction, which may make a self-supporting pole the most cost effective option (Crosby, 2001).

2.3 Power Distribution Poles

2.3.1 Materials of poles

Poles are supporting structures which carry the overhead conductor. They are of different types; they could be of wood, concrete, steel, or composite material types, which are becoming more prevalent. (Oyejide*et al.,* 2014). This research focuses on steel distribution poles, but the other types of poles will be discussed briefly.

2.3.1.1 Steel distribution poles

In transmission line construction, the use of steel poles has become the product of choice because of its construction efficiency and the use of these poles is just beginning to make in-roads in distribution lines. The steel distribution pole is an engineered product, each designed to meet specific strength and load requirements. The strength of these steel poles is based on the minimum values of ultimate tensile strength of steel. They are tested and fabricated using several code specifications such as the American Society of Civil Engineers(ASCE) tolerance, National Electricity Safety Code (NESC), Indian Standard (I.S) specification for tubular steel poles for overhead power lines load requirements.

Steel as a construction method requires little maintenance, and this durability is further strengthened when the distribution pole is hot-dip galvanized. Not only is there little maintenance, but it will remain durable in all different atmospheric conditions. Galvanizing is used in substations, transmission structures, as well as transmission and distribution poles to provide maintenance-free functionality without interruption for decades. Inspections for damage caused by rot, insects, or woodpeckers are eliminated with galvanized steel poles, because they are impermeable to these forces of nature. Not only is there a high capacity and availability for steel, but with the combination of steel and hot-dip galvanizing there is a heightened level of durability, sustainability, and technical feasibility as well as improved life-cycle cost. The demand and production of steel around the world is at an increasing rate, and as steel pole manufacturers are located throughout the world, the positive economic impact has become more widespread around the world(SMDI, 2011).

The type of metal used for most steel poles is hot-dipped galvanized steel (Zamanzadeh*et al*., 2006). It is that type of steel that has been coated with zinc to reduce corrosion. Zinc has the ability of protecting steel from corrosion in moderately corrosive soils by acting as a galvanic (sacrificial) metal. The base metal used for steel poles is usually 11-gauge sheet steel (Bolin and Smith, 2011). The steel material conforms to the American Society for Testing and Materials (ASTM) specifications A572-04 (2004). The minimum average coating thickness of the poles is governed by ASTM A123 (2013) and to mitigate potential brittle fracture, use of steel with good impact toughness in the longitudinal direction of the pole is necessary. Since the majority of pole structures are manufactured from steels of yield strength of 344.75-489.18N/mm^2, it is advantageous to specify a minimum Charpy-V-notch impact energy for plate andhigh strength anchor bolts made of ASTM A615-87 (1987) grade 75 steel.There are many applications for which alternate pole materials, typically engineered products such as steel, which represent an optimum solution can be incorporated into distribution systems,for example, areas where access of heavy machines is difficult, however, it is important to maintain the structural reliability of the alternative steel poles relative to the existing other types of poles. The NESC (2002) has served to provide basic safety rules to ensure such reliability, the designer would then supply the pole manufacturer the forces, moments, and design criteria for the manufacturer to design the pole itself.The NESC (2002) strength and loading rules are specified as a function of “grade of construction” (that is, reliability level), which will determine the appropriate size (strength) of pole to withstand wind and ice storms, to meet the basic safety requirements. Three grades of construction are defined relevant to pole lines (Outside Plant Consulting Service, OPCS, 2002):

(i.) Grade B: the highest grade; typically corresponds to crossings (highway, railroad, pole lines carrying varying power supply voltage levels).

(ii.) Grade C: lower grade of construction than Grade B; typical power or joint use (telecommunications and power) distribution pole applications.

(iii.) Grade N: lowest grade of construction; typical sole use in telephony applications.

Studies show that wood poles can be classified as Class 1, Class to Class 10that defines the size and strength of the pole. A class 1 pole is of larger diameter and stronger than a higher class pole number. The different grades of construction will require a different capacity or strength, and therefore a different class pole. Utilities and company’s responsible for pole installations are familiar with the wood pole classification system; it is therefore, convenient for steel poles to be classified similarly, to facilitate their selection and use.However, due to the inherently different characteristics (statistical distribution) of naturally grown wood and steel pole structures, the equivalency of a given non-wood pole size/strength (pole class number) for a

Grade B construction application will not be the same as for a Grade C construction application(Rollins, 2001).The grade of construction is usually a key parameter used to describe the degree of importance of the utility line, and the desired reliability level, for grade Bthe reliability is higher than that of grade Cwhich in turn is of higher reliability than grade N. The Grade B is often the objective for transmission line structures.

(i.) Condition of Manufacturing Steel poles

Pole manufacturers are available around the world, and the method of manufacturing and production is almost similar everywhere.Steel poles are fabricated from uniformly tapered hollow steel sections and for structural efficiency, the structures taper over their height to a smaller tip diameter at the top. The poles are cold formed by standard methods such as bending, stretch, roll forming, etc; from plates of thickness of 0.48 mm until 12.7 mm (because the maximum plate thickness that a hydraulic press is able to bend is around 12.7 mm). For these structures the industry practice is that the analysis, design and structure detail are usually performed by a steel pole supplier in order to develop the cheapest structures (lightest weight) and more compatibles with fabrication practice and available equipment. Because the selling price of the poles is directly proportional to the weight, companies spend time and money in the engineering to greatly simplify the manufacturing process with the minimal waste. For this reason, the structural design of the poles is highly dependent on the cuts in the plates as can be seen in the Plate I.

Plate I: Cutting of plate for poles**(**Edgar, 2016)

The tapered steel pole is formed from a trapezoid-shaped plate and depending on the dimensions; the pole might be formed with a single longitudinal weld seam. This is a common practice to reduce material waste and, therefore, reduce the cost of the poles. Once cutting has been performed, the rough cross-section is formed by presses (Plate I), and then the cross-section is detailed through small presses (Plate II). Finally, the shell is long-seamed with vertical weld along pole axis, through submerged arc welding techniques or electric resistance welding methods (Plate III) **(**Edger, 2016).

Plate II: Bending of the plate with hydraulic press

Plate III: Detail of the cross-section through smaller presses

Plate IV: Longitudinal weld seam**(**Edgar, 2016)

(ii.) Deterioration of Steel Poles

Steel pole industries and manufacturers are becoming aware of the potential problems of these poles and solution associated with underground corrosive soils. (Zamanzadeh*et al*., 2006).

Robinson(2005) reported that the primary main cause of deterioration of steel poles is corrosion. The corrosion rate of galvanized steel is usually higher at or near the soil surface because of the availability of oxygen. These layers result in a much slower rate of corrosion in galvanized steel than other zinc-coated steels. Corrosion reduces the strength of steel poles by reducing the cross-sectional area and bearing capacity at the location of corrosion. He therefore, reported that there are two main issues that determine the service life of steel poles. The first is the corrosion rate of the galvanized zinc coating and the second is the corrosion rate of the underlying steel.

The rate of corrosion of steel poles depends on several factors such as quality of initial corrosion prevention measures, soil type, mechanical damage, atmospheric chemical attack, fatigue, height of water table, metallurgical structure of galvanized layer, protective painting, duration of storage, and the presence of bacteria in soil. These parameters cannot be described with adequate accuracy and consequently, any corrosion rate model can only be a rough estimate. The most extensive and comprehensive research on underground corrosion of plain and galvanized steel was conducted by the National Bureau of Standards between 1910 and 1955 and the results collated and published by Romanoff (1957). More than 36,500 specimens representing 333 varieties of materials were buried in 128 locations throughout the United States America. The burial depths range from 0.45m to 1.8m. Out of all the specimens, 14,260 were ferrous materials consisting of wrought materials (steels and plain irons) and cast irons which were used to study corrosion in the primary metals. To study the corrosion of protective metallic coatings, 1,639 steel specimens coated using the hot-dipped galvanization method were buried in different types of soils and the rate of corrosion studied. Five different base metals were used that include Bessemer steel, wrought iron, plain and copper bearing steel, and open-hearth steel.The results from the plain steel and iron specimens showed that the properties of the soils control the rate of corrosion as specimens supplied by different manufacturers showed similar corrosion pattern. The loss of thickness with time was found to conform to theequation 2.1 (Robinson, 2005).

$$P = KT^n \qquad (2.1)$$

Where:P = thickness loss (mils),T = time in years, K and n are constants

Romanoff (1957) and his associates reported and discovered that the performance of steel poles deteriorated with increased temperature,sulphate and chloride concentrations, while they had little influence on galvanized steel. In support of Romanoff's (1957)work, Darbin (1988) noted that plain carbon steel in aggressive soil produced an n value of 0.6 to 1, while galvanizedsteel gave a significantly lower n value of 0.33. The authors found that the minimum percentage error in k was obtained if it is calculated for an average exposure time of 5.3 years.

Corrosion protection must be considered for steel poles. Selection of a specific coating or use of weathering steel depends on weather exposure, past experience, appearance, and economics. Weathering steel is best suited for environments involving proper wetting and drying cycles. Surfaces that are wet for prolonged periods will corrode at a rapid rate. A protective coating is required when such conditions exist. When weathering steel is used, poles should also be detailed to provide good drainage and avoid water retention. Also, poles should either be sealed or well ventilated to assure the proper protection of the interior surface of the pole. Hot-dip galvanizing is an excellent alternate means for corrosion protection of steel poles above ground.

Visual inspection remains the most reliable method for assessing corrosion in steel poles.This includes digging below ground to inspect underground corrosion of the steel. More advanced non-destructive methods are also available such as electrochemical field potential monitoring and the ground line corrosion meter (Ostendorp, 2003; Zamanzadeh*et al.,* 2006).

2.3.1.2 Wood poles

Woods are composed of naturally grown, biological material, which exhibits inconsistent material properties. These inconsistencies can have direct impact on the strength of pole. There are different kinds of wood available in Nigeria, irrespective of their location and usage (Ohagwu and Ugwuishiwu, 2011). The most common type of woods are Apa, BlackAfara, White Afara, Iroko, Abura,etc; and these wood can be used for power pole construction and lines. Traditionally, timbers have been used to a significant extent in construction, over 80% of timber products in Nigeria are utilized for different purposes.

Wood poles are the most common because they are generally the most cost effective and functional material. When codes, cosmetics, or conditions are an influence factor, the engineer may use a material other than wood. The criticism of wooden poles is mostly environmental stemming from logging required to harvest the material and the energy required to manufacture and install them (Kenny, 2017). This has led to the use of alternative materials such as steel.

2.3.1.3Concrete poles

Generally, concrete poles are the most frequently used poles, and are sometimes guyed, concrete poles are fabricated from engineered materials, and these poles have consistent material properties throughout their length. There is very little maintenance for concrete poles, concrete is poor in tension, the tensile strength only accounted for about 10% of the compressive strength. Due to this, nearly all reinforced concretes are designed on the assumption that concrete does not resist any tensile force. Concrete poles are usually used in areas that are swampy and persistently wet; they are used in areas where the soils around the areas shorten the life expectancy of other pole materials to be used. This uncertainty makes alternate pole material choice unwise (Oritola, 2006). Standard concrete poles are limited by their ground line moment capacity.Concrete may be very costly to transport depending on the distance from the concrete plant to the installation site. Concrete poles have different classification than wood poles.For instance, a Class 2 wood pole, with a 1678.29kg tip load may be considered equivalent to a Class H concrete pole. It is also possible to use a ratio of the material strength reduction factors to get a Class G concrete pole; for instance, the concrete and steel products have approximately the same dimensions and generally are used interchangeably with wooden poles.While wood poles use a numbering system, concretepoles use the alphabet that is Class G, Class H. Also concrete and steel products have approximately the same dimensions and are generally used interchangeably with wood poles.

2.3.1.4Composite Poles

Composite power poles are made from a combination of materials ranging from reclaimed wood or medium-density fibre to plastic and even fiberglass. Fibre-glass poles are very light and thus can be carried by hand to remote locations where a vehicle cannot go such as the side of a mountain. Fibre-glass poles have few maintenance issues, but because fiberglass is not a commonly used material, it requires installers who have experience with fibre-glass. Conversely,when manufacturing the composite material they require more energy than milling a wooden pole according to the Environmental Literacy Council, (2005). Other types of composite poles are laminated wood poles, and typically not used around the world, as with self-supporting steel poles, the utility company designer would give the design criteria to the laminated wood pole manufacture to design the pole. These poles are typically set deeper than ordinary wood poles, and backfilled with gravel (RUS, 2008).

2.4 Pole Foundations

Foundations are provided to transfer load from the structure to the ground. According ASCE 48-11(2011), this implies that three methods are normally used to place a steel transmission pole in the ground; they are, drilled shaft, direct embedded foundation and casing foundation. Usually before deciding which foundation method to use, certain considerations must be addressed initially in the design as well as restrictions to pay attention to,such as type of the structure to be built, importance of the structure, soil properties, foundation loads, design limitations, environmental restrictions, availability of materials, allowable foundation movement or rotation, geological conditions and of course, cost (Barone, 2014).

Direct embedment of poles tends to be more economical over the other types, because it essentially just requires digging a hole, dropping the pole into the ground. This method is also the oldest form of foundation as it has been used on steel pole transmission lines since early times; with directly embedded poles the setting depth is usually 10 percent of the total pole length plus 0.6m. This is the same 'rule of thumb' used for tangent or guyed distribution poles (Crosby, 2011). Thus, the structure acts as its own foundation transferring loads to the in-situsoil through the backfill. The backfill can be a stone mix, stone-cement mix, excavated material, polyurethane foam, or concrete. One can directly embed steel poles the same way as one can with wood poles. These methods of pole foundations have become popular because of their relatively low installation cost. In general, non-woodpole suppliers also attempt to test and characterize the strength of their poles in a manner consistent with that of Figure 2.1.,

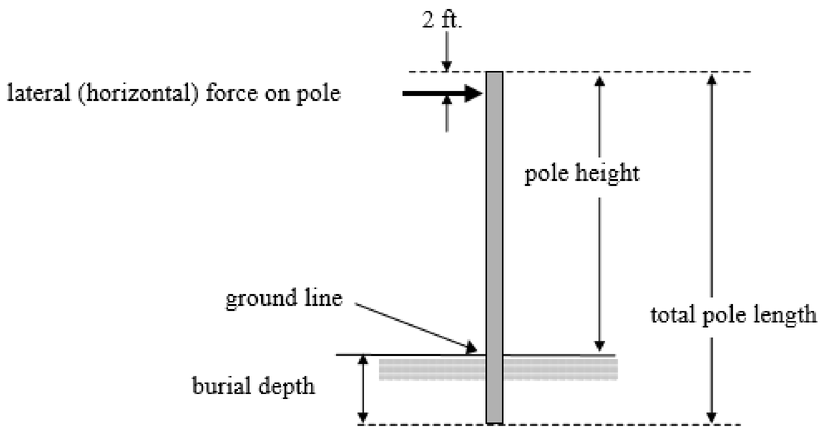

Figure2.1: Lateral Force Applied to Installed Pole (Outside Plant Consulting Service,2002).

Irrespective of the foundation type to be used, the main advantage that steel poles provide is their dimensional consistency. There is another option for setting steel poles as well. Where direct embedment is not practical, base plates can be welded to the bottom of steel poles so that they can be mounted on anchor bolts that are either set in concrete foundations or grouted directly into rock.The disadvantage of direct embedment is the dependency on the quality of backfill material. To accurately get deflection and rotation of direct embedded structures, the stiffness of the embedment must be considered. The performance criteria for deflection should be for the combined pole and foundation. Instability of the augured hole and the presence of water may require a liner or double liners.

2.5 Structural Loadings on Poles

Pole design is usually dependent on the power and voltage that is to be transmitted; the design is based on the principle of conductor loading. Steel poles are sometimes specified in situations where poles of high strength are required, stress imposed on poles are calculated as if they were cantilever beam fixed at one end, also ground line moments as well as cross-arms if attached must be designed to withstand the load of conductor and equipment, Most of the forces on a pole are from the vertical loading comprising of dead weight of conductors, cross-arms, insulators and associated hardware, the horizontal loading due to wind pressure on conductors and pole (Oritola, 2006).But, using engineered poles may be a good option for a project, if the owner and/or line designer is confident with the approximate ground line moment calculation on simple line designs or ifthe line designer is plugging in the structural properties of a pre-engineered steel pole from a catalog into line design software.

According to IS: 2713(1980) Indian Standard "Specification for Tubular Steel Poles for Overhead Power Lines", Poles are to be designed generally for the following critical loading conditions

(a) Bending due to wind load on the pole and cables on the exposed transverse face.

(b) Combined bending and torsion due to eccentric snapping of wires on either side of the pole.

(c) Maximum torsion due to skew snapping of wires on either side of the pole.

(d) Production, handling and erection stresses.

The IS: 2713 (I-III): 1980; 2008classified all steel poles into several lengths with various cross sectional properties and carrying load capacity for each section. The strength of the poles have been based on the minimum values of ultimate tensile strength of steel and designed such that when it is vertical, its strength in the transverse direction shall be sufficient to make an ultimate load equal to the horizontal wind load on wires and pole multiplied by the load factors.Figure 2.2 is a schematic representation of a standard steel distribution pole with point A-A the point of application of the load, 0.3m from the tip of the pole.

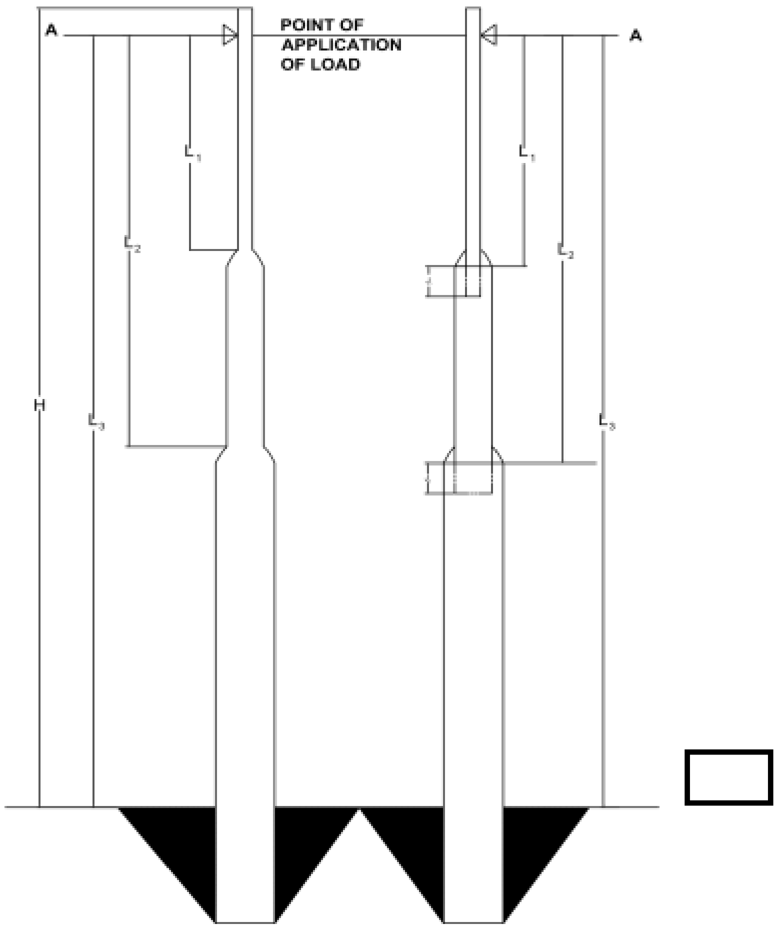

Figure 2.2 Steel Distribution Pole Layout (I.S 2713)

Where;$A - A =$ line ofapplication ofresultant of wind loads on wires and pole,$H =$ overall height above ground in m,$l_1, l_2, and\ l_3$ = Distance in m from A-A to bottom ofeach section (in case of bottom section, it is up to GL only), $D_1 D_2 D_3$ =Outside diameter of top, middle and bottom sections of pole, p = Wind pressure on flat surface N/m^2, $h =$ height of conductors on cross-arm from GL, $n =$ Number ofconductors,$d =$ diameter of conductors, $s =$ Sum of half the spans on each side of pole, $P_1 =$ equivalent wind load on pole, calculated as acting at A-A,P_2 =equivalent wind load on conductor calculated as acting at A-A, P =total wind load as acting at A-A, $P = P_1 + P_2$ in N,$GL =$ ground level,$BM =$ bending moment.

(i.) Wind Load

The principal wind loadings on poles can be grouped into two categories; wind loading concentrated along the height of the pole and wind loading concentrated at the level of the cables usually 0.3m from the tip of the pole.The load on the wires is calculated by multiplying the wind pressure by the diameter of each wire, by the length of the span and the result by 2/3 to allow for circular section. Then the equivalent load acting at A-A can be calculated using equation (2.2) to (2.7) respectively.

Wind load on conductors $= \frac{Pnsd\,h}{100} \times \frac{2}{3}\ N$, acting at h meters from GL(2.2)

BM due to wind load at GL $= \frac{Pnsd\,h}{100} \times \frac{2}{3}\ N.m.(2.3)$

Equivalent load acting at A-A $= \frac{Pns\,dh}{100\,l_3} \times \frac{2}{3}\ N(2.4)$

Therefore,

$$P_2 = \frac{Pnsdh}{150 l_3}\ N \tag{2.5}$$

The wind load on pole is next calculated and expressed as the equivalent load acting at the same point as the load imposed by wires.

Wind load on pole $= \frac{2PD_1}{300}[H - (l_3 - l_1)]$ (2.6)

Acting at a distance of,

$$H - \frac{H - (l_3 - l_1)}{2} + \frac{2PD_2(l_3 - l_1)}{300} + \frac{2PD_3(l_3 - l_2)}{300} \tag{2.7}$$

Where: $H - \frac{H-(l_3-l_1)}{2}$ is from G.L, $\frac{2PD_2(l_3-l_1)}{300}$ acting at a distance $\left[(l_3 - l_2) + \frac{l_{2-}l_1}{2}\right]$ from G.L and $\frac{2PD_3(l_3-l_2)}{300}$ is acting at a distance $\frac{l_3-l_2}{2}$ from GL.

Therefore, Bending Moment at ground line is given as

$$BM = \frac{2\rho}{300}\left[D_1\{H-(l_3-l_1)\}\left\{H - \frac{H-(l_3-l_1)}{2}\right\} + D_2(l_2-l_1)\left(l_3 - \frac{l_1}{2} - \frac{l_2}{2}\right) + D_3(L_3 - L_2)\frac{l_3-l_2}{2}\right] \quad (2.8)$$

Say $BM = WM,$

Equivalent load acting at A-A

$$= P_1 = \frac{WM}{L_3} \quad (2.9)$$

So that total load

$$P = P_1 + P_2 \quad (2.10)$$

(ii.) **Pole deflection**

Deflections of poles are due to the lateral loads from the span wires, bracket arms, street lights, other attachments and wind loading. Pole deflection can vary during the service life of the pole, also wind loading, span wire loading, and bracket arm loading can vary due to seasonal and temperature changes. The pole should be designed so that maximum deflections will not be exceeded**.** Expressions for the deflection of poles are given in Equation(2.11):For a single section pole, the deflection is expressed as:

$$D = \frac{PL^3}{3EI} \quad (2.11)$$

Where: D = deflection, L = length of pole at applied load, P = load applied,E = modulus of elasticity, E = 2.1 x 10^5N/$mm^2$for Steel, I = moment of inertia.

The methodology applied for the evaluation of the probabilistic strength of steel poles for power transmission and distribution is given in chapter three.

(iii.) **Steel Poles Parameters and Statistics**

Many uncertain parameters exist in a structure including geometric dimensions, material properties, load and initial conditions and boundary conditions (Xing and Hong-Nan, 2018). To calculate the reliability of a structural element or system, statistical information for all the random variables related to capacity and load are required. The variables related to capacity depends on the material used and can be estimated from test data. Variables related to load can be obtained from historical data such as maximum wind speeds, earthquake intensities, and so on. Data may not be available for the estimation of some variables, in this case subjective estimates can be made and the effect of such assumptions on the reliability can be studied using sensitivity analysis (Salman, 2014). The geometrical properties, loads (dead and wind loads) acting on a pole varies as most of the variables are uncertain in there manifestations. To properly capture the variation of the force, the distributions and coefficient of variations (C.O.V) of the random variables are considered in this research and are summarized in Table 2.1. The relationship between the mean, standard deviation and coefficient of variations are related by equation (2.12).

/N	Design Variables	unit	Mean E(x)	Coefficient of variation (C.O.V)	Distribution Model	Source

	Steel Strength (f_y)	N/mm^2	500	0.10	Normal	Salman, 2014
	Pole Thickness (t)	mm	3.65	0.04	Normal	Shehu,1997
	Diameter of Pole (d)	mm	114.3	0.04	Normal	Shehu,1997
	Dead load (G_k)	kN/m^2	79.515	0.10	log normal	Afolayan, 2005
	Height above ground (h)	mm	7500	0.03	Normal	Salman, 2014/Paraic *et al.*, 2014
	Wind load (W_k)	kN/m^2	5.51×10^{-3}	0.03	Log-normal	Salman, 2014

$$COV = \frac{\sigma}{\mu} = \frac{standard\ deviation}{mean} \quad (2.12)$$

Table 2.1 Stochastic Model Parameters and Their Statistical Values for Steel Pole.

2.6 Finite Element Analysis Applied to Poles

The use of Finite Element Method (FEM) has gained abundant popularity in recent years due to advances in high-speed computers. Finite Element Analysis (FEA) is a numerical method for solving problems of engineering and mathematical physics; these methods originated from the need for solving complex structural analysis problems in civil and aeronautical engineering. The Finite Element Method (FEM),its practical application often known as Finite Element Analysis (FEA) is a numerical technique for finding approximate solutions of Partial Differential Equations (PDE) as well as integral equations. In a structural simulation, FEM helps tremendously in producing stiffness and strength visualizations and also in minimizing weight, materials, and costs.FEM allows detailed visualization of where structures bend or twist, and indicates the distribution of stresses and displacements. FEM software provides a wide range of simulation options for controlling the complexity of both modeling and analysis of a system. Similarly, the desired level of accuracy required and associated computational time requirements can be managed simultaneously to address most engineering applications. The introduction of FEM has substantially decreased the time to take products from concept to the production line. It is primarily through improved initial prototype designs using FEM that testing and development have been accelerated.

2.7 Finite ElementModelling of Steel Poles

The objective of analysis using ABAQUS finite elements programing is to examine the formulated behavior of a steel pole due to its applied loads. Consequently, the stresses and deformations of the pole under various load effects will be determined. The modeling for a function of an existing structure, assumed to be close to the real responses as in the available structural situation as possible as it can be done. In addition, it is an important procedure to figure material properties as they appear in the target structure. As a result, the cross section modeling will be highly influenced by material modeling.The design of the model should reflect closely the pole structure behavior and approaching exactly in addressing a problem. The specifications of a model have been represented by many attributes such as the targeted structure type, the form of loads and the resistance under investigation, the analysis pattern (design or an existence structure), and the expecting results that will be determined from modeling, which refers to the accuracy needed from this task and a user friendly model.The extent of failure, stresses and displacements will be effectively measured with the aid of Finite Element (FE) which is simulated in ABAQUS software.The calculations will then be determined using the results obtained from ABAQUS finite element programming.A Finite Element Analysis (FEA) program can be used to validate the strength of the pole, however, the safety of the steel pole maychange during the lifetime as degradation and corrosion of the pole might occur.

(a) Mathematical Formulations of Finite Element (F.E)

Finite Element method is a special form of the well-known Galerkin and Rayleigh-Ritz methods of finding approximate solution of differential equations. In both methods the governing differential equation first is converted into an equivalent integral form. The Rayleigh-Ritz method employs calculus of variations to define an equivalent variation while the Galerkin method uses a more direct approach. An approximate solution, with one or more unknown parameters, is chosen, but in general, this assumed solution will not satisfy the differential equation. The integral form represents the residual obtained by integrating the error over the solution domain. Employing a criterion adopted to minimize the residual gives equations for finding the unknown parameters. For most practical problems, solutions of differential equations are required to satisfy not only the differential equation but also the specified boundary conditions at one or more points along the boundary of the solution domain. In both methods some of the boundary conditions must be satisfied explicitly by the assumed solutions, while others are satisfied implicitly through the minimization process. The boundary conditions are thus divided into two categories; essential and natural. The essential boundary conditions are those that must explicitly be satisfied, while the natural boundary conditions are incorporated into the integral formulation. In general, therefore, the approximate solution will not satisfy the natural boundary conditions exactly.

In a method known as the least–squared weighted residual method the error term is squared to define error as shown in equation (2.13).

$$e_T = \int_{x0}^{x1} e^2\,dx \quad (2.13)$$

The necessary conditions for the minimum of the total squared error give *n* equations that can be solved for unknown parameters and expressed by equation (2.14).

$$\frac{\partial e_T}{\partial a_i} = 2\int_{x0}^{x1} e\,\frac{\partial_e}{\partial a_i}\,dx = 0; \qquad i = 0,1,.... \quad (2.14)$$

Thus in the least-squares method the weighting functions are the partial derivatives of the error term *e(x)*

Least-squares weighting functions: shown by equation (2.15).

$$(X) = \frac{\partial_e}{\partial a_i}; \qquad i = 0,1,\ldots.n \qquad (2.15)$$

A more popular method in the finite element applications is the Galerkin method. In this method, instead of taking partial derivatives of the error function, the weighting functions are defined as the partial derivatives of the assumed solution.

Galerkin weighting functions are expressed by equation (2.16).

$$W_i\,(X) = \frac{\partial_u}{\partial a_i}; \qquad i = 0,1,\ldots.n \quad (2.16)$$

Thus the Galerkin weighted residual method defines the following *n* equations to solve for the unknown parameters (equation 2.17).

$$\int_{x0}^{x1} e\,\frac{\partial u}{\partial a_i}\,dx = 0; \qquad i = 0,1,... \quad (2.17)$$

It turns out that for a large number of engineering applications the Galerkin method gives the same solution as another popular method, the Rayleigh-Ritz method, presented in a later section. Furthermore, since the least –squares method has no particular advantages over the

Galerkin method for the kinds of problems we encounter; only the Galekin method is presented in detail. So far in developing the residual we have considered error in satisfying the differential equation alone. A solution must also satisfy boundary conditions. In order to be able to introduce the boundary conditions into the weighted residual, we use mathematical manipulations involving integrations by parts.

The integration-by-parts formula is used to rewrite an integral of a product of a derivative of a function, say f(x), and another function, say g(x), as shown by equation (2.18).

$$\int_{x0}^{x1} [\frac{d}{dx}(f(x))]\, g(x)dx = f(X_1)g(X_1) - f(X_0)g(X_0) - \int_{x0}^{x1} [\frac{d}{dx}(g(x))] f(x)dx \qquad (2.18)$$

2.8 Structural Reliability Analysis Applied to Poles

Reliability simply refers to some probabilistic measure of satisfactory (or safe) performance, and as such, may be viewed as a complementary function of the probability of failure. The reliability is the compliment of the probability of failure (Shu-jin*et al*., 1999) that is expressed by equation (2.19).

Reliability$=$

$$1 - P_f \qquad (2.19)$$

Afolayan (2005), observed that natural phenomena show that loadings and other parameters with which we are concerned in structural design vary in value such as the strength of any given material and the sizes of the identical units. It is therefore necessary to systematically quantify uncertainties and apply them in the design process.

2.8.1 Methods of structural reliability analysis

Madsen *et al*.(1986) identified methods to measure the reliability of a structure into four groups as follows:

(i.) *Level I methods*: The uncertain parameters are modeled by one characteristic value, as for example in codes based on the partial safety factor concept.

(ii.) *Level II methods*: The uncertain parameters are modeled by the mean values and the standard deviations, and by the correlation coefficients between the stochastic variables. The stochastic variables are implicitly assumed to be normally distributed. The reliability index method is an example of a level II method.

(iii.) *Level III methods*: The uncertain quantities are modeled by their joint distribution functions. The probability of failure is estimated as a measure of the reliability.

(iv.) *Level IV methods*: In these methods the consequences (cost) of failure are also taken into account and the risk (consequence multiplied by the probability of failure) is used as a measure of the reliability. In this way different designs can be compared on an economic basis taking into account uncertainty, costs and benefits.

For a structural component for which the uncertain resistance, R, may be modeled by a random variable with probability density function $F_R(r)$ subjected to the load, S, the probability of failure may be determined byequation (2.20) (Gollwitzer, *et al*; 1988; Afolayan, 2005).

$$P_f = P(R \leq S) = F(S) = P(R/S \leq 0) \qquad (2.20)$$

In case also the load is uncertain and modeled by the random variable S with probability density function $F_S(s)$ the probability of failure is expressed by equation (2.21).

$$P_f = P(R \leq S) = P(R - S \leq 0) = \int_{-\infty}^{\infty} F(x) d_x \qquad (2.21)$$

In a structural reliability evaluation, each possible failure mode can be expressed in the form of a failure function. The failure function is an explicit or implicitfunction of many random variables such as wind, ice, wind direction, temperature,mechanical properties and pole dimension. In the simplest case, failure is a function of tworandom variables, the strength R and the load S; thus, (Dagher, 2001) noted in equation (2.22) that:

$$G = R - S \tag{2.22}$$

The objective of reliability analysis is to evaluate the probability of occurrence of a specified scenario (Neves*et al*., 2006), In all special cases, where the failure surface is linear and all basic variables are normally distributed, it is easy to show that there is direct relationship between the failure probability, (P_f), and the reliability index, (β) That is as shown by equation (2.23) and (2.24).

$$P_f = \varphi(-\beta) \tag{2.23}$$

Thus,

$$\beta = -\varphi^{-1}(P_f) \tag{2.24}$$

Where φ is the standardized normal distribution function.

In general, the failure surface is non-linear and the basic variable non-normal, the generalized reliability index is expressed by equation (2.25).

$$\beta_g = -\varphi^{-1}(P_f) \tag{2.25}$$

2.9 First Order Reliability Method (FORM)

This sectionpresents an introduction to the FORM concept; the reliability analysis procedure will be conducted here to evaluate the strength of steel pole according to the function performance level represented as reliability indices of steel poles.Recent researches in the area of structural reliability and probabilistic analyses have centered on the development of probability-based design procedures**.**These include loading modeling, ultimate and service load performance, and evaluation of current level of safety/reliability in design (Afolayan, 2014).

Generally, in a reliability-based approach, uncertainty associated with material properties, loads, environmental conditions, models etc., are taken into account by treating the parameters as random variables. The condition of a structure is assessed by probability of failure, P_f, or related to the reliability index, β,The First Order Reliability Method (FORM) is a convenient tool to assess the reliability of structural elements. It also provides means for calculating the partial factor of safety.

FORM has been designed for the approximate computation of general integrals over given domains with locally smooth boundaries, but especially for probability integrals occurring in structural reliability. For FORM, it is required that F(x)is at least locally continuously differentiable, that is, the probability densities exist. The random variables $X = (X_1 \ldots X_n)$are called basic variables. The locally sufficiently smooth (at least once differentiable) state function is denoted by $G(x)$, and is defined, such that; (Gollwitzer*et al.*, 1988. Ditlevson and Madsen, 2005):

$\mathrm{G}(x) > 0$Corresponds to safe sate.

$\mathrm{G}(x) = 0$Corresponds to limiting state (failure boundary).

$\mathrm{G}(x) < 0$Corresponds to the failure domain.

In the context of FORM, it is convenient, but necessary that only locally can $G(x)$ be a monotonic function in each component X.

In the context of examination of structural systems such as power poles, bridge networks, road networks, etc., network maintenance must also be considered. This network maintenance plays an important role in reducing the effect of deterioration on the probability of failure of elements in the network at a given time, t. Consequently, in order to realistically represent the performance of an infrastructure over time, a probabilistic model must incorporate (Paraic *et al.*, 2014); Resistance, Applied load, Deterioration of infrastructure elements over time and effects of network maintenance.

As pointed out by Barone and Frangopol (2014), however, resistance and indeed load, are often time-dependent variables with resistance generally deteriorating over time for engineering systems. Thus, Equation (2.22) becomes equation (2.26)

$$G_{(t)} = R_{(t)} - S_{(t)} \qquad (2.26)$$

Where$R_{(t)}$ and $S_{(t)}$are instantaneous resistance and load effects at time t, respectively. Figure 2.3shows a general case of distributions for load carrying capacity and load effect (Bergstrom, 2006).

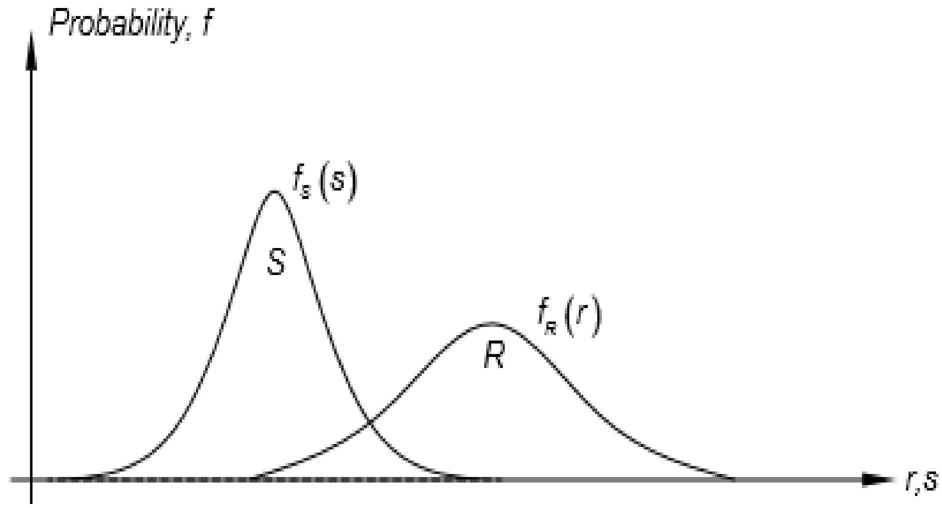

Figure 2.3: Distributions for R and S (Bergstrom, 2006).

Considering Figure 2.3, the probability that a load effect, S, falls into an infinitesimal interval d at s is given by equation (2.27).

$$f_s(s).\, d_s \qquad (2.27)$$

The probability that R falls in or under this interval is given by equation (2.28):

$$\int_{-\infty}^{s} f_r(r) d_r \qquad (2.28)$$

The probability that S falls in this interval ds when R ≤ S; combining Equation (2.27) and (2.28), results to equation (2.29).

$$F_s(s) d_s \int_{-\infty}^{s} f_{R(r)}\, d_r \qquad (2.29)$$

The equation, finally, giving the probability thatR ≤ S is giving in equation (2.30).

$$P_f = P(R - S) \leq 0 = \int_{-\infty}^{\infty} f_s(s) \left[\int_{-\infty}^{s} f_R(r)\, d_r \right] d_s = \int_{-\infty}^{\infty} f_s(s) f_R(r)\, d_s \qquad (2.30)$$

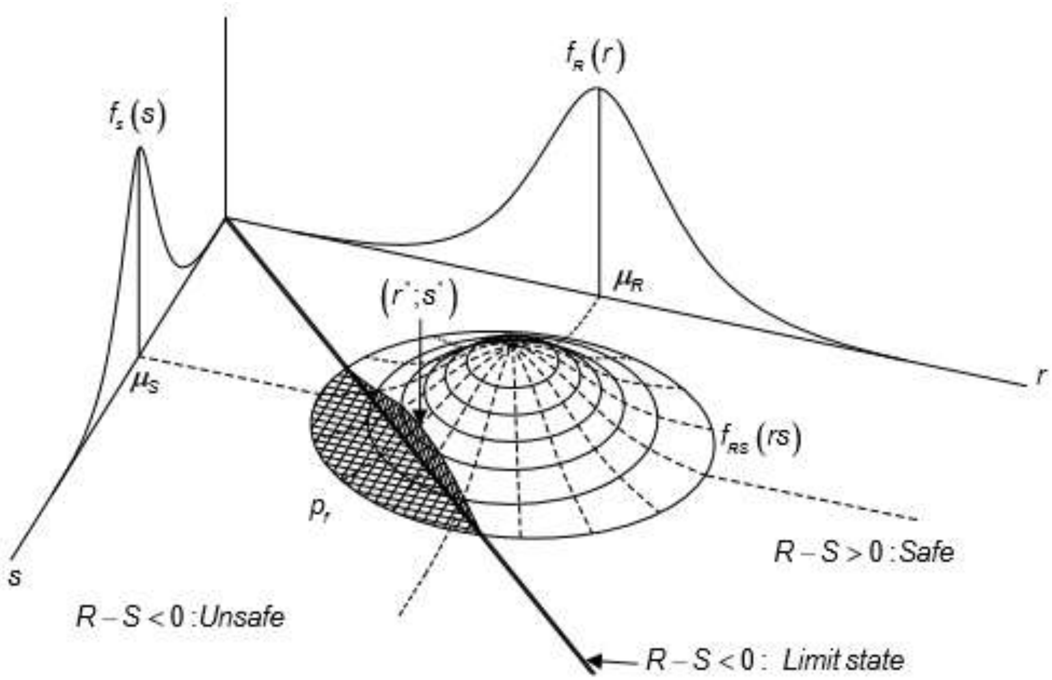

Figure 2.4:The 3-D view of two random joint density function $f_{RS}(rs)$ (Bergstrom, 2006).

This could be viewed upon as the volume of the two dimensional joint density functions from the failure surface, and as can be seen in Figure 2.4 above, R and S are plotted as probability functions on the r and s axes. The limit state equation, $G = R = S = 0$, separates the safe from the unsafe region, dividing the volume into two parts. The volume of the part cut away and defined by s > r corresponds to the probability of failure. The design point (r*; s*) is located on this straight line where the joint probability density is greatest. If failure is to occur, it is likely to be there. The desired safe state is above the failure surface, defined as $R - S = 0$, and undesired failure state is below.

If R and S are normally distributed and statistically uncorrelated, then, equations (2.31 to 2.32) are validated.

$$R \epsilon N(M_R, \sigma_R) (2.31)$$

$$S \epsilon N(M_S, \sigma_S) \tag{2.32}$$

The safety margin, M is defined as:

$$M = R - S \tag{2.33}$$

Then it is also valid that:

$$M \epsilon N(M_M, \sigma_M) \tag{2.34}$$

Where; $M_M = M_R - M_S \; and \; \sigma_M = \sqrt{\sigma_R^2 + \sigma_S^2}$

The distribution of M is schematically given in Figure 2.5

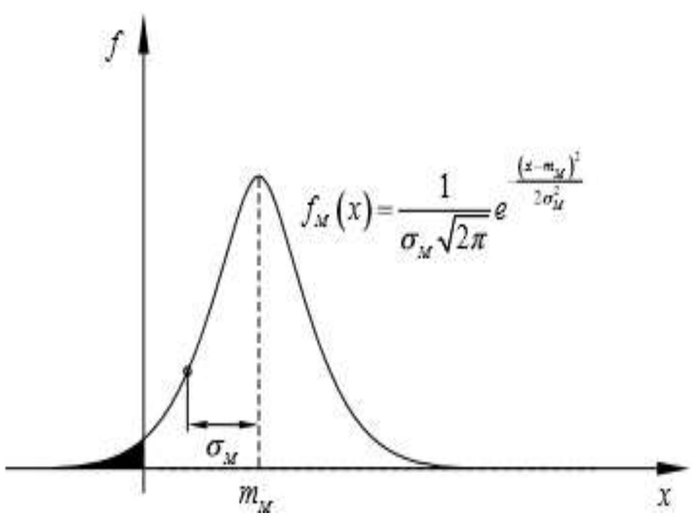

Figure 2.5:Distribution for the variable M. Failure occurs when M< 0 (Bergstrom, 2006)

Thus, the probability of failure can be expressed by equation (2.35) and further expressed into equation (2.36) and (2.37) respectively.

$$P_f = P(M < 0) = \int_{-\infty}^{0} f_M(x) d_x = \frac{1}{\sigma_m \sqrt{2\pi}} \int_{-\infty}^{0} e^{\frac{x - mM}{2\sigma^2 M}} d_x \tag{2.35}$$

Now let,

$$\frac{x - mM}{\sigma_m} = y, \rightarrow \frac{1}{\sigma_M} d_x = d_y \tag{2.36}$$

$$P_f = P(M < 0) = \frac{1}{\sqrt{2\pi}} \int e^{\frac{y^2}{2}} d_y = \varphi\left(\frac{-mM}{\sigma_M}\right) = \varphi(-\beta) \tag{2.37}$$

$\varphi(-)$is the standardized normal distribution function, this function given in figure 2.5 has mean value 0 and standard deviation 1. If R and S are normally distributed, the safety index,β, is calculated with equation (2.38):

$$\beta = \frac{m_M}{\sigma_M} = \frac{mR - mS}{\sqrt{\sigma_{R^2} + \sigma_{S^2}}} \tag{2.38}$$

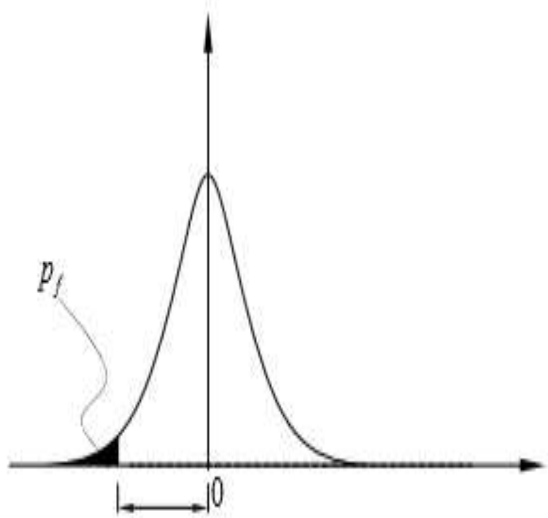

Figure 2.6:The standardized normal distribution function (Bergstrom, 2006).

Safety index, E, is connected to a certain probability of failure$P_{F\beta}$.

In FORM it is also assumed that R and S are independent and both normally distributed (or have been transformed to normally distributed variables). The FORM will be used to calculate the reliability index from which the probability of failure of the steel pole will be obtained.In First Order Reliability Methods, the limit state function (failure function) is linearized and the reliability is estimated. Generally, the main steps in a reliability analysis are:

(i.) Select a target reliability level.

(ii.) Identify the significant failure modes of the structure.

(iii.) Decompose the failure modes in series systems or parallel systems of single components (only needed if the failure modes consist of more than one component).

(iv.) Formulate failure functions (limit state functions) corresponding to each component in the failure modes.

(v.) Identify the stochastic variables and the deterministic parameters in the failure functions. Further specify the distribution types and statistical parameters for the stochastic variables and the dependencies between them.

(vi.) Estimate the reliability of each failure mode.

(vii.) In a design process, change the design if the reliabilities do not meet the target reliabilities. In a reliability analysis, the reliability is compared with the target reliability.

(viii.) Evaluate the reliability result by performing sensitivity analyses.

2.10 Finite Element Reliability Using MATLAB (FERUM)

FERUM is a general purpose structural reliability method,whose first developments started in 1999 at the University of California at Berkeley, finite element Reliability will be used to carry out analysis Using MATLAB.FERUM 4.x now offers Reliability-Based Design Optimization (RBDO) capabilities. It offers new features such as simulation-based techniques (Directional Simulation, Subset Simulation), Global Sensitivity Analysis, Reliability-Based Design Optimization, Subset Simulation and global Sensitivity Analysis can be carried out either using the original physical model or a Support Vector Machine surrogate. If the physical model is really computationally demanding, then distributed computing is now available, either virtually through vectorized calculations in MATLAB or for real with multi-processor computers, provided that a suitable interface is developed (Bourinet*et. al*, 2009).

2.11 Limit State

The purpose of design is to achieve acceptable probabilities that a structure will not become unfit for its intended use, that is, it will not reach a limit state. Thus any way in which a structure may cease to be fit for use will constitute a limit state and the design aim is to avoid any of such conditions being reached during the expected life of the structure, limit states can be divided into two categories; Ultimate limit states and Serviceability limit state (Mosley *et al.*, 1999). Ultimate limit state requires that the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed. The possibility of bending is taken into account while Serviceability Limit States (SLSs) are related to gradual degradation and user’s comfort. These limit states are usually not associated with an immediate structural collapse (Mosley *et al.*, 1999). . The limit state function can be written as expressed by equation (3.1).

$$g(x) = g(x_1, x_2, x_3, \dots, x_n) = R - Q \qquad (3.39)$$

Where; R represents the resistance (carrying capacity), Q represents the load effect and x_n represents the random variables of load and resistance such as dead load, wind load, length, depth, thickness, etc.

CHAPTER THREE

METHODOLOGY

3.1 Finite Element Modelling Using ABAQUS/CAE 6.13.1

ABAQUS/CAE 6.13.1is a general-purpose FEM program. The aim is to analyse how reliable is or otherwise the failure regions in the pole structure. The ABAQUS adopts techniques for evaluating failure regions and stresses.

Below is a summary of the steps involved in running the ABAQUS software for the steel pole;

(a) *Part module*: this module provides the tools for creating, editing and managing the elements of the steel pole. Its features capture the design intent and geometry information of all of the elements listed above.

(b) *Property module*: this module specifies the different characters and material properties of the elements created in the part module. Table 3.1 gives the summary of parameters used in the part module session.

(c) *Assembly module*: the element parts of the modeled are assembled in this module.

(d) *Step module*: this module is used to create the analysis type and sequence. Specifying the sequence provides a convenient way to capture changes in the loading and boundary conditions of the model.

(e) *Load module*: this module is also step dependent. The boundary conditions and load types are specified. For this study, the steel pole is fixed at the bottom and free at the top where the load is acting.

(f) *Job module*: this module helps to do the job and submits it for analysis while you monitor its progress.

(g) *Visualization module*: this module provides the graphical display of structural mechanics and the finite element results of the model. It obtains its result from the output data.

Table 3.1 indicates input parameters for Finite Element Analyses of the steel poles, the typical pole was subjected to load of 20kNfor 7, 8, 9, 10 and 12m height of poles; the boundary conditions for the steel poles are fixed at one end and analyzed just as a cantilever beam with the load acting at its free end.

Table 3.1 Input Parameters for Finite Element Analysis (FEA)

Material	Yield Strength (N/mm^2)	Young Modulus (N/mm^2)	Density (Kg/m^3)	Poissons ratio	Thermal Coefficient
Steel	500	209E+03	7850	0.3	12×10^{-6}per ^{o}C

3.2 MATLAB based FERUM program

The MATLAB-based FERUM program was used in this research; it reads other parameters which are also supplied by the user. FORM is subsequently called to calculate the reliability index for the data provided. The program flow chart is as follows;

(i.) Main Directory: The main directory contains the main program. This program will be developed to perform the reliability analysis in this study. While running this program, the user will be required to select the failures mode in question with its input and the capacity of execution of specific function.

(ii.) FORM directory: The form directory contains subroutines that calculate the safety index for each of the failure mode considered.

(iii.) Distribution Model Setup Directory: The distribution model setup directory contains the MATLAB function files that assign appropriate distribution model to each random variable. The distribution model includes normal, lognormal, gumbel, weibull distribution, etc.

(iv.) Coefficient of variation directory: The coefficient of variation directory containing the function file that store the values of the coefficient of variation of each random variable is based on the test result and data for the steel pole.

(v.) Probability of failure and safety index directory: Probability of failure directory contains the MATLAB function that computes probability of failure with safety index as an output.

3.3 Collection of Data for Steel Poles Modelling

The design data of a typical steel pole of 9m high will beused;the steel pole carries electric cables made of aluminum spanning 65m. The loads on the steel pole include: the cross-arms, insulators, street lamp, self-weight of the steel and the effect of wind load. Only un-guyed poles are considered in this research as they are the majority of the poles. Guyed poles tend to have lower probability of failure due to the extra support provided by the guy wires. The geometrical properties of the steel pole are also shown below in Table 3.2.

Table 3.2 Geometric properties of steel poles for FEM analysis

Pole length	9.0m
Step length	$4.8 \times 2.1 \times 2.1$
Outside diameter of steps	$114.3 \times 88.9 \times 76.1$mm
Pole plate thickness	3.65mm
Planting depth	1.5m
Spacing between poles	65m
Diameter of Conductor	13.7mm
$l_1, l_2, and\ l_3$ = Distance in m from A-A to bottom of each section of the length.	1.4m, 3.1m, 5.45m
Point of Application of load from the top	0.3m

3.4 Limit State Equations

The following suitable equations were obtained and will serve as the limit state function in the reliability analyses

Moment: The difference between the resistance of the steel poles and the applied load effect is the limit state function G(x), given by equation (3.1).

$$G(x) = [M_{per} - M_a] \quad (3.1)$$

The moment capacity for a steel pole is expressed by equation (3.2).

$$M_r = 0.66 f_y Z \quad (3.2)$$

While applied moment due to load effect while designing the pole as a cantilever beam is expressed by (3.3).

$$M_a = \frac{-WL^2}{2} \quad (3.3)$$

So that; equation (3.2) and (3.3) becomes equation (3.4)

$$G(x) = 0.66 f_y Z - \frac{(1.35G_k + 1.5W_k)l^2}{2} \quad (3.4)$$

f_y = Steel strength, Z = section modulus given by $Z = \frac{\pi(D_o^4 - D_i^4)}{32D_o}$ and$D_i = D_o - 2t$,

$D_O = outer\ diameter\ of\ the\ pole,\ D_i = inner\ diamter\ t = thickness\ of\ pole$

Thus, the basic variables for the design are $[f_y,\ d, t, G_k, W_k,\ l\,] = [x_1, x_2,\ x_3, x_4,\ x_5,\ x_6\,]$

Deflection:equation (3.5) shows the equation of deflection.

$$G(x) = [\delta_{per} - \delta_{max}] \quad (3.5)$$

The maximum deflection of the steel pole designed as cantilever is given by equation (3.6).

$$\delta_{max} = \frac{wl^3}{3EI} \tag{3.6}$$

While permissible deflection $\boldsymbol{\delta_{per}}$ is given by equation (3.7).

$$\delta_{per} = \frac{span}{250} \tag{3.7}$$

Therefore; the limit state equation for deflection is obtained by substituting equation (3.6) and (3.7) is given by equation (3.8).

$$G(x) = \frac{span}{250} - \frac{wl^3}{3EI} \tag{3.8}$$

Where: δ_{max} = maximum deflection; δ_{per} = permissible deflection; w = design load; l = pole length, E= modulus of elasticity and I= moment of inertia.

Thus, the basic variables for the design are $[l,\ G_k, W_k,\ I,\ d,\ ] = [x_1, x_2,\ x_3, x_4,\ x_5]$

Shear: The limit state equation for the pole to be structurally safe against shear failure due to yielding is expressed by equation (3.9) and (3.10) respectively.

$$G(x) = V_{res} - V_{app} \tag{3.9}$$

$$G(x) = \varphi_v 0.66 f_y A_v C_v - (1.35G_k + 1.5W_k) \tag{3.10}$$

Where: V_{res} = ultimate shear resistance, V_{app} = is the applied shear force due to applied load

φ_v =Shear resistance factor; A_v = shear area; C_v = shear coefficient.

Thus, the basic variables for the design are $\left[\varphi_v,\ f_y,\ d,\ t, G_k, W_k,\ \right] = \left[x_1, x_2,\ x_3, x_4,\ x_5,\ x_6\ \right]$

3.5 Load Data Analysis (Dead and wind load)

The load combination for the ultimate limit state at adverse level from EN 3 (2004) is given by equation (3.11).

$$W = 1.35G_k + 1.5W_k \tag{3.11}$$

(i.) Dead Load

Weight of steel $= 78.5kN/m^2$

Bracket arms and Insulator $= 0.15kN/m^2$

Street Lamp $= 0.005kN/m^2$

The self-weight of aluminum$= 0.86kN/m^2$

Total dead load $= 79.515kN/m^2$

(ii.) Wind Load

BS 6399-2 (1997) Code of Practice for wind loading was used for the analysis of wind load; the dynamic pressure is given by equation (3.12) and (3.13).

$$q_e = kV_e^2 \tag{3.12}$$

$$V_e = V_s K_1 K_2 \tag{3.13}$$

Where; k=0.613, $k_1$is the risk coefficient taken as 1.0, $k_2$is the terrain factor taken as 1.0 for flat terrain, the wind speed will be taken as 3m/s.

According to Adaramola and Oyewola, (2011) the average wind speed across Nigeria is about 3.0m/s with the northern part of the country having higher wind speed value than the southern part; therefore, the average wind speed of 3.0m/s is adopted in this research.

Therefore,

$q_s = 0.613 \times 3.0^2 = 5.512 N/m^2$

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 General

The steel pole was analyzed using the input parameters of Table 3.1into Finite Element Analyses design software,ABAQUS CAE 6.13.1 (2018). The steel poleresult output are generated based on the design input from the ABAQUS design environment, the model of the steel poles is given as shown in Plate V-VI. The analysis of the pole following a non-linear stress pattern and deformations are discussed further. The loads acting on pole comprises of wind loads concentrated along the height of the pole, wind loads concentrated at the level of the cables usually 0.3m from the tip of the pole and dead loads which comprises of self-weight ofaluminium, street lampon the pole, cross arms and pin connection attached and of course self-weight of steel, For the purpose of this study, the typical pole was subjected to load of 20kNfor

7, 8, 9, 10 and 12m height of poles; the boundary conditions for the steel poles are fixed at one end and analyzed just as a cantilever beam with the load acting at its free end. Probabilistic analysis was also conducted to quantify the probability of failure of the steel poles. The reliability levels were calculated using the deterministic and statistical parameters to thedeveloped computer program in MATLAB in order to compute the safety indices based on the limit states equations.

4.2 Results

4.2.1 Finite element results ofSimulia ABAQUS CAE 6.13.1

The results obtained for the steel pole FE analysis coded in ABAQUS are shown in the Plates below, the results presented here are, the maximum principal stresses and von mises stresses.

(i.) Maximum principal stress distributionof the poles

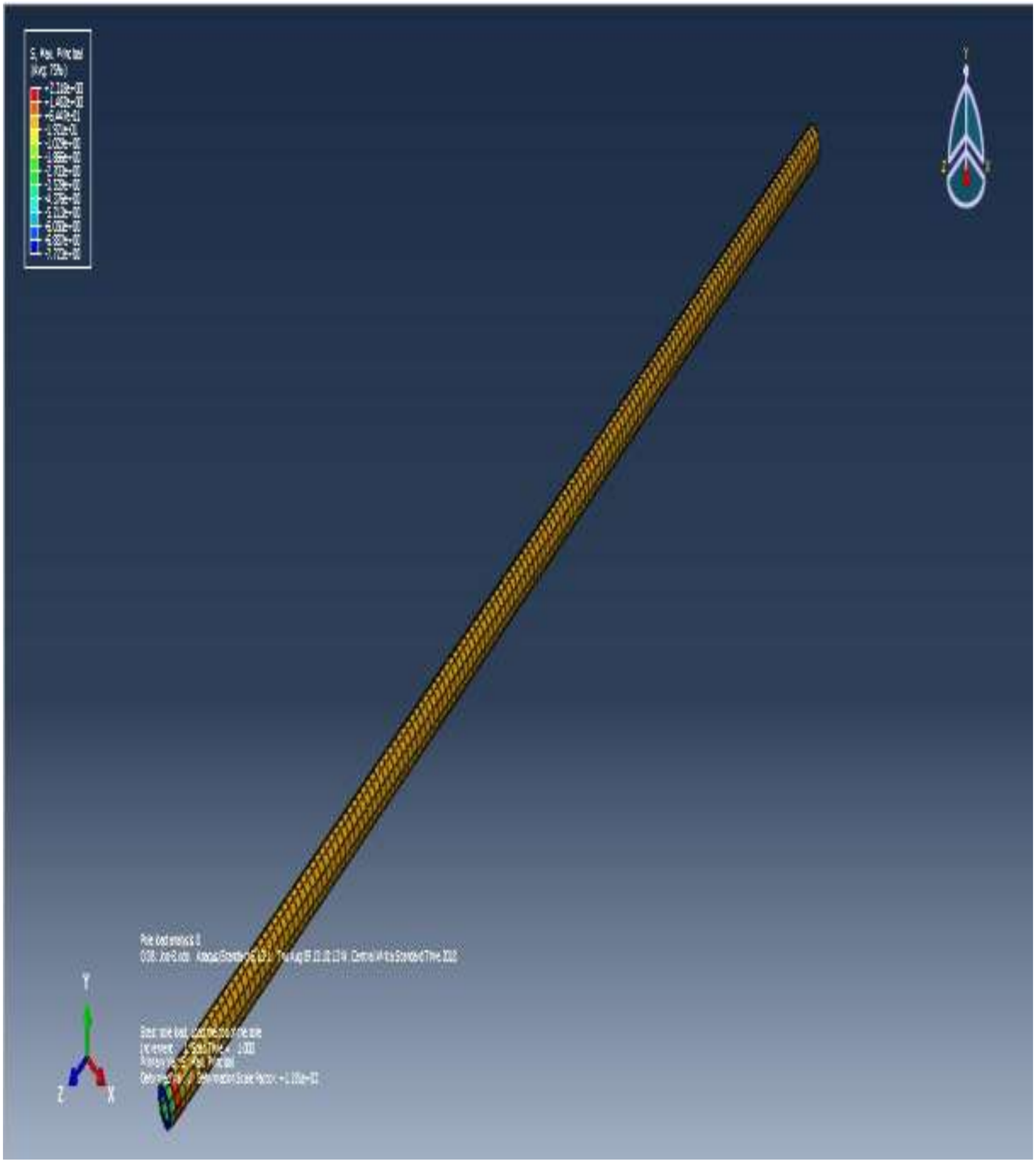

Plate V: Maximum principal stress for the pole (9m)

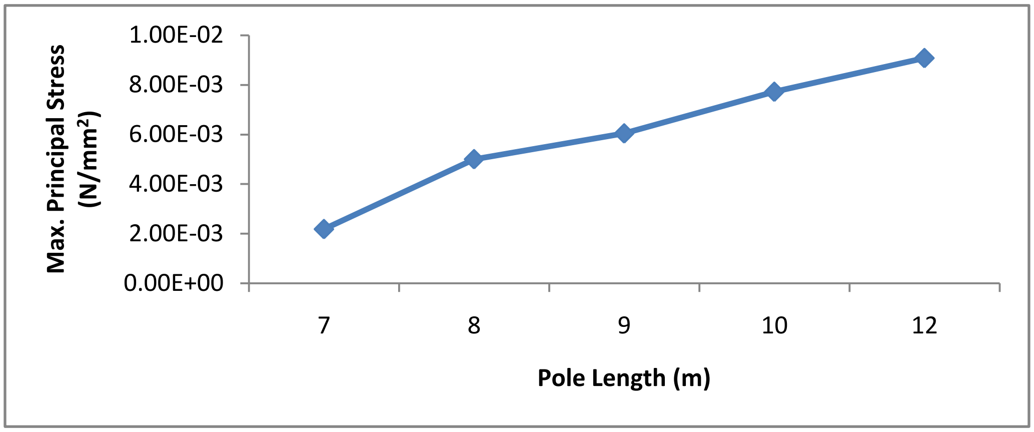

Figure 4.1: Relationship of maximum values of Principal stress (tension) to different pole lengths

Table 4.1 shows the summary results of stresses obtained for maximum principal stress and Von Mises stress distribution.

Pole Length (m)	Maximum Principal stress N/mm^2.	Von Mises StressN/mm^2.
7	2.180E-03	2.180E-03
8	5.001E-03	2.630E-03
9	6.040E-03	3.210E-03
10	7.720E-03	6.040E-03
12	9.08E-03	8.340E-03

(iii.) Von Mises maximum stress distribution for the pole

The Plate II shows the stress distribution for the 9m height of poles.

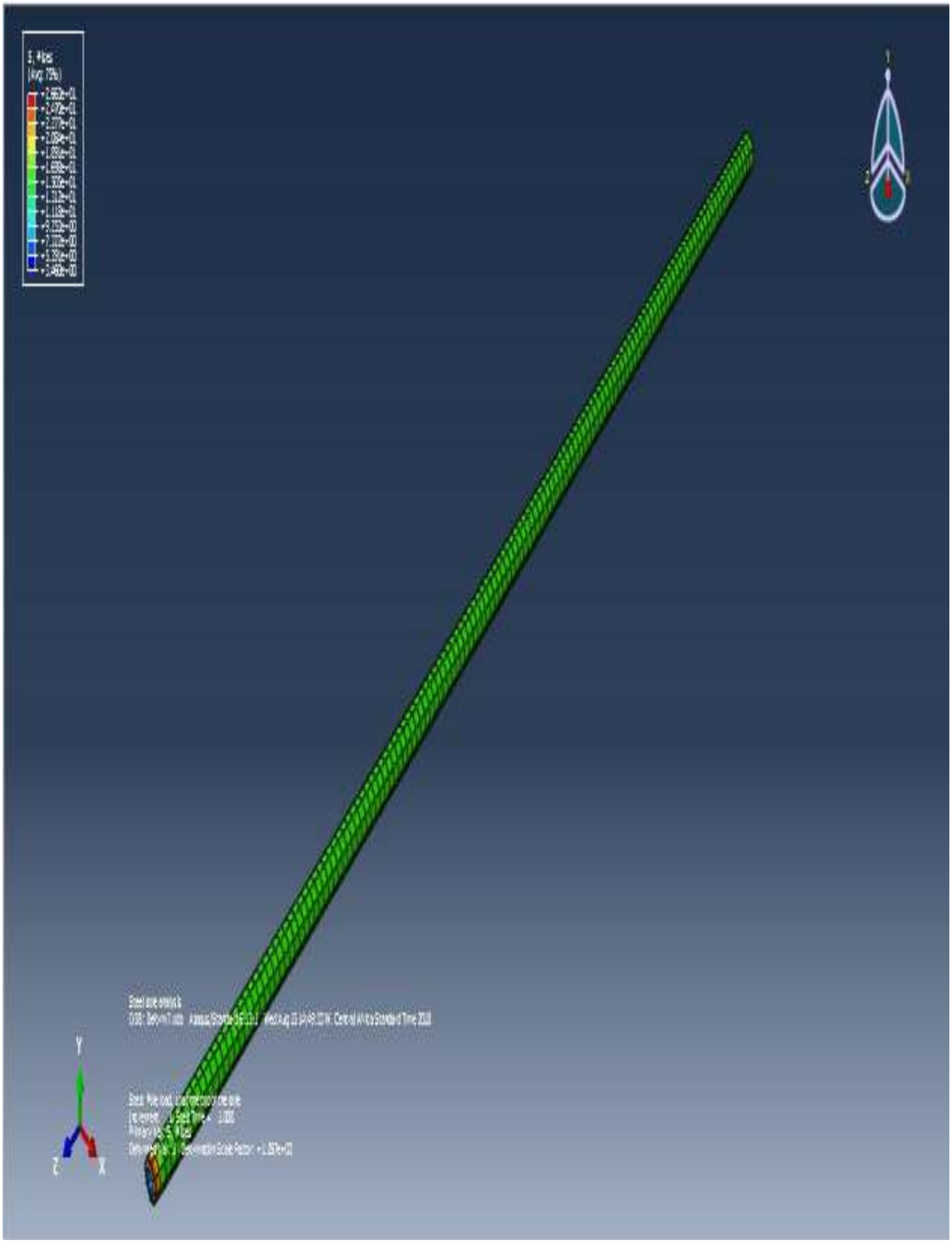

Plate VI: Von Mises stress for the steel pole (9m)

Also, Figure 4.2 indicates the relationship of maximum values of von mises stress to different pole length.

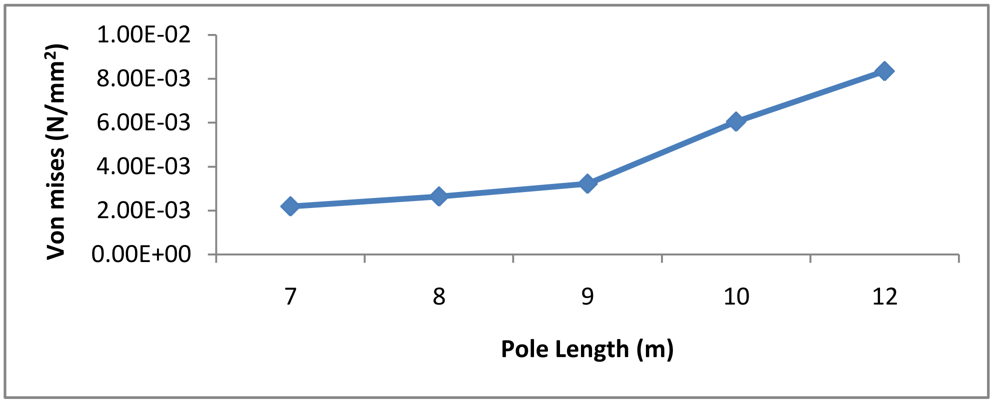

Figure 4.2: Relationship of maximum values of Von-Mises stress to different pole lengths

4.2.1.1 Discussion on Finite Element results using ABAQUS

The ABAQUS displays the solutions in visual expression and with the indication of the numerical values and critical points, these visual expressions are in colours such as green, blue, yellow, red and shades there of which indicates the sensity of the stress in the affected areas, For example red indicates Total failure of the region while blue indicates efficiency in design.The software yielded results indicating that the material properties of the steel poles are sufficient to provide adequate and sufficient resistance against deformation. The steel poles in service did not suffer any stress deformation failure due to the stiffness of the material property; the non-linear finite element stress analysis considering a linear perturbation step perpendicular load yielded a deformation scale factor of 1.00E0 resulting into a minimum principal stress value of 2.322E-03$N/mm^2$and maximum stress value of 9.077E-03N/mm^2.

The von Mises stress pattern obtained after the analyses of the typical pole can also be seen in Figure 4.2 above. The Finite element stress analysis considering a linear perturbation step perpendicular load yielded a deformation scale factor of 1.00E0, resulting to a von Mises minimum stress values of 2.183E-03 N/mm^2 and maximum stress value of 8.34E-03N/mm^2.The reddish yellow colors observed at the bottom of the steel poles indicates where the maximum values of von mises stresses are, however, for all the lengths of the poles considered, the poles did not show any noticeable deformations after the analyses in the software and, hence, it is an indication that the pole will be structurally stable even after deformation caused by the applied loads. The maximum values of the results obtained are usually computational tools which aid designers in showing how the structure will behave when loaded in order to mitigate against actions such as severe winds, deformations and total collapse.The stresses obtained are far less than the yield strength of the steel pole which is $500N/mm^2$.

4.2.2 Results of Reliability Analysis (FERUM IN MATLAB)

This section discusses the findings of the reliability analyses conducted on the steel poles in MATLAB to examine and simulate the behavior of the steel poles at distances above the ground level, the length, diameter, and thickness of the poles were all varied simultaneously, For each of the length considered, the thickness was varied in order to check the effect of the steel pole in service while resisting the effect of moment, deflection and shear failures since the failures of any structure can be measured in terms of the probability of failure; on the other hand, the reliability of a structure is measured by the safety index. The analysis was conducted for the steel poles based on the derived limit states equations. Table 4.2 gives the summary of the result obtained in the program for moment, shear and deflection failures of the poles.

Table 4.2: Results obtained by FERUM in MATLAB (R2015b) for 9m poles

Failure mode	Reliability Index (β)	Probability of Failure (P_f)
Moment	1.4802E+00	6.9407E-02
Deflection	2.3390E+01	0.100E-10
Shear	5.1245E+01	0.100E-10

Based on the result output generated in MATLAB for the steel poles, it was observed from Table 4.2, for moment failure mode the reliability index value was very low (β =1.4802 and P_f =6.9407E-02) as compared to that for deflection and shear failure modes which had higher values (2.339E+01and 5.1245E+01) respectively with both negligible P_f values. It might be an indication that the pole is safe against these failure modes. It might also be due to the fact that deflection and shear failures of steel poles are a rare event; they require the combination of a number of unusual events to occur such as corrosion at or near the ground surface. The consequence of having higher values of reliability index is a conservative design with a very high cost, that is, (uneconomical). Normally, a lower value of the reliability index, β, implies unsafe structure or set of variables as witnessed for moment failure mode while higher values implies safe structure as witnessed for deflection and share failures.Even if the cost of steel poles is high, it is economical in the long run as seen by the higher values of reliability index.

4.2.3 Sensitivity study

Sensitivity analyses refer to the evaluation of the response when a design parameter is modified or changed. Thus, to identify the strength behavior of steel poles, design parameters such as the length, diameter and thickness were varied simultaneously on the variability of the reliability indices; the study was carried out to assess the relative impact of the variability and uncertainty of the parameters on the overall output. This was achieved by varying the most important parameters that affect the strength of steel poles, since in steel power poles design; the strength is derived by varying the length, thickness and diameters. The sensitivity study results for each of the parameters are presented in Table 4.3 below.

Table 4.3: Reliability Values for different poles length, thicknesses and diameters for Moment failure mode

Pole Length

Length (m)	Reliability Index (β)	Probability of Failure (P_f)
7.0	1.6943	4.0085E-02
8.0	1.5935	4.5106E-02
8.5	1.5541	5.5524E-02
9.0	1.4802	6.9407E-02
10.0	1.304	9.6114E-02

Thickness of the pole

Thickness (mm)	Reliability Index (β)	Probability of Failure (P_f)
3.6	1.2140	4.086E-03
4.5	1.3802	3.301E-03
4.8	1.4211	3.301E-03
5.9	1.4930	3.301E-03
5.9	1.6222	3.431E-03

Diameters of the pole

Pole Diameters (mm)	Reliability Index (β)	Probability of Failure (P_f)
(7m)114.3 $\times$ 88.9 $\times$ 76.1	2.7152	3.3097E-03
$(8m)139.7 \times 114.3 \times 88.9$	2.3713	3.3100E-03
(9m)165.1 $\times$ 139.7 $\times$ 114.3	2.0867	3.3104E-03
(10m)193.7 $\times$ 165.1 $\times$ 139.7	2.3713	3.3107E-03
$(12m)219.1 \times 193.7 \times 165.1$	1.7153	3.3115E-03

Figures 4.3, 4.4 and 4.5 show the relationship between reliability indices to different pole lengths, thicknesses and diameters. The discussions arising from on these findings are also given below.

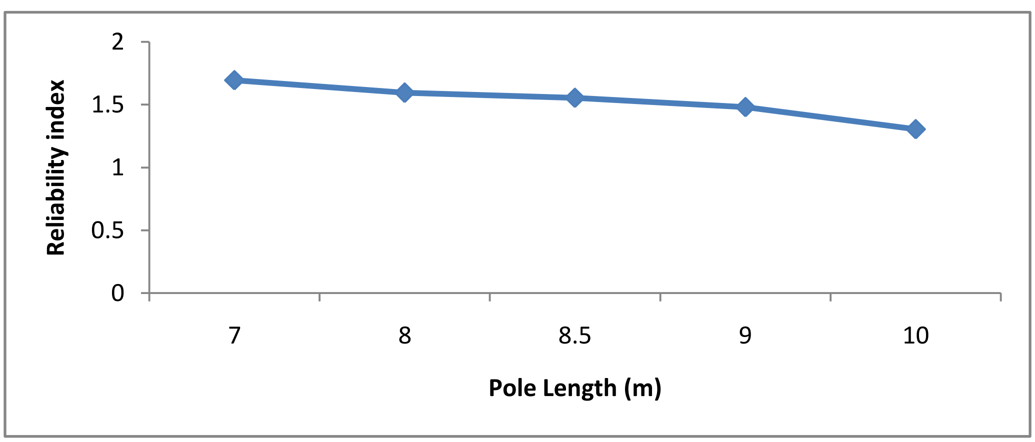

Figure 4.3: Relationship between reliability index and pole length.

As can be seen from Figure 4.3, the reliability index is sensitive to changes in height, where in for an increase in pole length, the β decreases, it can be best explained that even excessive deflections occur at the apex and becomes more rigid down the base of the pole, i.e. the higher the length the more the pole been subjected to wind.

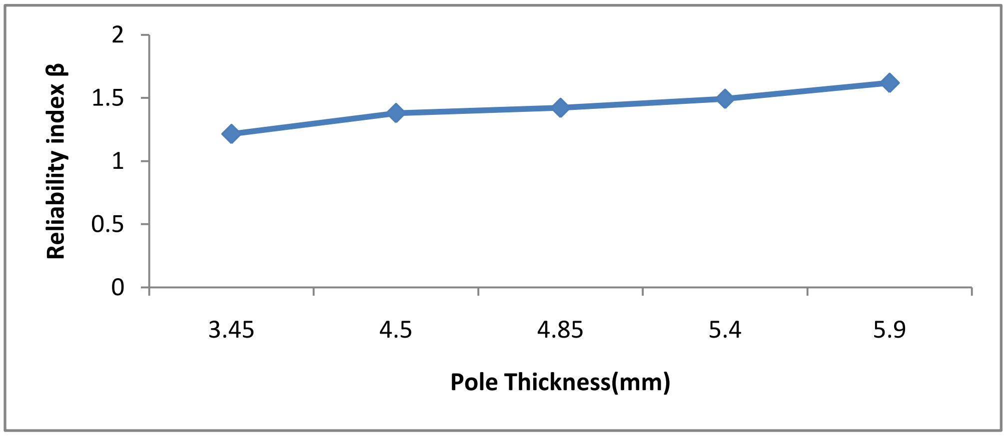

Figure 4.4: Relationship between reliability index and pole Thickness.

As can be inferred from Figure 4.4, the reliability index is highly sensitive to variability in the pole thickness which shows an increasing trend which is logical here that the thicker the pole, the stronger and reliable it will be.

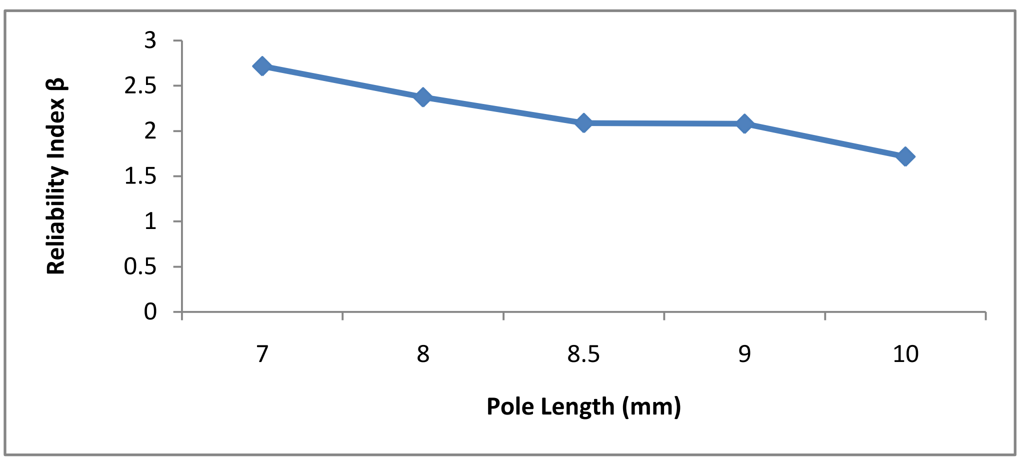

Figure 4.5: Relationship between reliability index anddiameters for different pole length.

Figure 4.5 shows the relationship between the reliability index pole diameters for different pole length, as can be seen from the graph; it is obvious that the structural dimensions plays a significant role in the performance of steel transmission poles as the reliability index decreases drastically.

Based on the above findings,it can be reported that the reliability indices are sensitive to variability in structural dimensions since limit state equations are functions of the geometric variables of the pole. It should also be noted that for deflection and shear failures, higher values of reliability indices, β, were also obtained for the various parameters with also negligible probabilities of failures. Usually, the variability of the statistical parameters significantly affects the magnitude of failures associated with the poles.

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CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATION

5.1 Conclusions

The conclusions of this research are outlined as follows:

(a) The stress distributions in the poles are far less than the yield stress of steel making the poles structurally safe and efficient for use in power transmission and distribution.

(b) The result of reliability analysis of the steel poles using generated limit state functions and First Order Reliability Method with the aid of FERUM in MATLAB have also been presented, the reliability indices for moment had a value of 1.4802E+00, 5.1245E+01 for shear and 2.339E+01 for deflection failure of the pole.

(c) The shear and deflection of the poles had the highest reliability index and lower probability of failure0.100E-10 indicating that these failure modes have little effect on the strength of steel poles.

(d) Based on the findings of these work, the length, thicknesses and diameters are the most sensitive parameters that affect the strength of steel poles.

5.2 Recommendations

The following recommendations should be consideredon steel poles for transmission and distribution;

(a) Circular steel poles of yield stress 500N/mm^2 and of height between 7m and 8m with diameters of step $114.3 \times 88.9 \times 76.1mm$ and thickness of 5.9mm are structurally safe and efficient for use in power transmission and distribution purposes.

(b) The choice of material to be used for power distributions and transmissions should be based on several factors such as available resources and functional requirements, but steel poles provide better performance than other types of poles and have the potential to replace other types of poles.

(c) Based on the above points, pole maintenance should be incorporated into a structural reliability analysis before construction is carried out.

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Appendix I

Properties of Conductors

Typical Conductor	Number of Strands	Diameter of conductors(mm)	Overall diameter of conductors(mm)	Strength(N)	Weight (kg/m)
Copper	3	2.64	5.59	6900	0.149
	3	2.74	9.91	18,900	0.445
	7	3.86	11.7	32,20	0.794

				0	
	7	4.57	13.7	44,100	1.04
	7	5.18	15.5	55,650	1.34
	19	3.66	18.3	77,300	1.79
Steel -Cored Aluminium	7	2.39	7.11	9400	0.104
	7	4.09	12.2	25300	0.312
	6A	5.28A	15.7	40,550	0.491
	7S	1.75S	19.6	77,200	0.833
	37	2.79	22.4	98,300	1.09
	37	3.18	25.5	130,200	1.46
	3	3.35	7.34	4350	0.073
	7	3.78	11.4	1290	0.223

				0	
Alu minium	7	4.00	14.7	20,30 0	0.357
	19	3.53	17.8	29,50 0	0.506
	19	3.49	20.1	3700	0.655

Appendix II

Section properties for various steel poles

L ength m	Thickn ess (mm)	Diameter (mm)	P lanting Depth	Section modulus Z_t, Z_m, Z_b	Moment of inertia I_t, I_m, I_b
7. 0	3.65	114.3 × 88.9 × 76.1	1.25	19.8, 27.9, 48	73.34, 123.9, 274.5
8. 0	4.5	139.7 × 114.3 × 88.9	1.5	27.9, 48, 73.7	123.9, 274.5, 514.5
9. 0	4.85	165.1 × 139.7 × 114.3	1.5	27.9, 48, 73.7	123.9, 274.5, 514.5
1	5.9	193.7 × 165.1 × 139.7	1.8	79.6,11	555.8, 953.7,

0				3.3,159	1536
1 2	5.9	$219.1 \times 193.7 \times 165.1$	2.0	113.3,1 59, 205	935.7, 1536, 2247

Computation of loads

From the schematic diagram of the pole; the following computation of loads are applied,

Pole length 9.00m

$l_1 = 2.1 - 0.3 = 1.8m$

$l_2 = 2.1 + 2.1 - 0.3 = 3.9m$

$l_3 = 9.0 - 1.5 - 0.3 = 7.2m$

$H = 9.5 - 1.5 = 7.5m$

Wind load on conductors P_2

$$P_2 = \frac{Pnsdh}{150 l_3} N$$

$$P_2 = \frac{1000 \times 3 \times 65 \times 0.0137 \times 7}{150 \times 7.2} = 17.32KN$$

Loads due to wind on pole after transferring it to act on A-A

$$P_1 = \frac{WM}{L_3}$$

Therefore, Bending Moment due to wind on pole at ground line,

$$WM = \frac{2\rho}{300}\left[D_1\{H-(l_3-l_1)\}\left\{H-\frac{H-(l_3-l_1)}{2}\right\}+D_2(l_2-l_1)\left(l_3-\frac{l_1}{2}-\frac{l_2}{2}\right)\right.$$
$$\left.+D_3(L_3-L_2)\frac{l_3-l_2}{2}\right]$$

$$WM = \frac{2\times 1000}{300}\left[8.89\{7.5-(7.2-1.8)\}\left\{7.5-\frac{7.5-(7.2-1.8)}{2}\right\}\right.$$
$$\left.+11.43(3.9-1.8)\left(7.2-\frac{1.8}{2}-\frac{3.9}{2}\right)+13.97(7.2-3.9)\frac{7.2-3.9}{2}\right]$$

$$WM = \frac{1000}{150}[120.42+104.4+76.1] = 2007.1364N\_M$$

Say $BM = WM$,

Equivalent load acting at A-A

$$P_1 = \frac{WM}{L_3}$$

$$P_1 = \frac{2007.1364}{7.2} = 278.76N = 0.278KN$$

So that total load $P = P_1 + P_2$

$$P = 17.32 + 0.278 = 17.6KN\ Say\ 20KN$$