NON-LINEAR ANALYSIS OF CHAINETTE TOWER

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CERTIFICATE

This is to certify that the dissertation entitled **"Non-linear Analysis of Chainette Tower"** submitted by **Mr. Mahendrasinh P. Makwana**, towards the partial fulfillment of the requirements for the award of Degree of **Master of Engineering (CIVIL)** in field of **Computer Aided Structural Analysis and Design (CASAD)** of Gujarat University is the record of work carried out by him under my supervision and guidance. The work submitted has in my opinion reached a level required for being accepted for examination. The results embodied in this dissertation, to the best of my knowledge have not been submitted to any other university or institution for award of any degree or diploma.

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ACKNOWLEDGEMENT

I would like to thank my guide Shri Urmil V. Dave for his valuable guidance, his patience and moral support. I would also like to thank Prof. G. N. Patel and Dr. H. V. Trivedi for giving me constant support and suggestions throughout the work.

I m sincerely thankful to Shri S. J. Mohan, scientist at S.E.R.C., Chennai for his guidance and support and sparing his valuable time for me.

I would also like to thank Shri M. C. Mehta, vice president (Design) of Kalpataru Power Transmission Ltd., Shri N. P. Mehta and Shri Nitin Patel, (both) senior design manager of Kalpataru Power Transmission Ltd. Gandhinagar, for giving me valuable information regarding the topic.

I would like to thank my parents and my wife for their constant moral support and help throughout the work. It would have been impossible to reach at this stage without their presence.

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ABSTRACT

With the advancement of technology it has become a necessity to transmit extra high voltage and Ultra high voltage. The self-supporting towers prove to be very heavy and hence the concept of chainette type of guyed tower was introduced. The components and the configuration of the chainette tower are discussed in detail. The experience of using the chainette tower in the James-Bay line, Quebec, Canada [11] has given positive reasons for implementing this concept.

The behaviour of the chainette tower is highly non-linear due to the presence of pretensioned cables. The non-linearity is due to the geometry and material. The large displacements due to the heavy loads are responsible for geometric non-linearity and the stress-strain behaviour of the pretensioned cables are responsible for material non-linearity. In this work only the geometric non-linearity is considered.

ANSYS, a finite element package has been used to do the linear static and geometric non-linear static analysis. The necessity of going for the non-linear analysis has been discussed. A computer program has been developed to perform a non-linear analysis. The results of linear and non-linear static analysis, displacements from ANSYS and computer program, forces in masts and cable guys from ANSYS and computer program are compared.

The design guidelines are given for the design of mast member and finally the concluding remarks are made at the end of the work.

In chapter 1, different types of transmission line towers are introduced along with the objectives of study, action plan and problem formulation.

In chapter 2, the literature review is given.

In chapter 3, the chainette tower is briefly discussed.

In chapter 4, the non-linear analysis is discussed in detail.

In chapter 5, the types of load and load combinations are explained.

In chapter 6, the use of ANSYS software is briefed along with steps involved in non-linear analysis.

In chapter 7, algorithm used for the computer programming is given.

In chapter 8, the results obtained from ANSYS and the computer program are discussed.

In chapter 9, the guidelines for the design of mast members are discussed.

In chapter 10, the concluding remarks and the further scope of work is given.

INDEX

Certificate
Acknowledgement
Abstract
Index
Index of figures
Index of tables
Nomenclature

Chapter 1 Introduction

1.1 General
1.2 Self-supporting towers
1.3 Conventional guyed towers
1.4 Chainette guyed towers
1.5 Objectives of study
1.6 Action plan
1.7 Problem formulation

Chapter 2 Literature review

2.1 General
2.2 Chainette tower
2.3 Non-linear analysis
2.4 Design
2.5 Codal provision for transmission line towers
2.6 Computer program

Chapter 3 About chainette towers

3.1 General
3.2 Why use chainette towers?
3.3 Component details
3.4 External components
3.5 Case-study details (735 kV Hydro-Quebec, Canada)
3.6 Codal provisions

Chapter 4 Non-linear analysis

4.1 General
4.2 Types of non-linearity
4.3 Non-linear analysis
4.4 Types of geometric non-linearities
4.5 Stress stiffening

Chapter 5 Types of loads and load combinations

5.1 General
5.2 Requirements of loads on transmission lines
5.3 Nature of loads
5.4 Loading criteria
5.5 Load computations
5.6 Loads while broken wire condition
5.7 Combinations of loads
5.8 Other types of loads

Chapter 6 Software implementation

6.1 General
6.2 Procedure of non-linear static analysis
6.3 Build the model
6.4 Setting solution controls
6.5 Setting additional solution options
6.6 Applying the loads
6.7 Performing the analysis
6.8 Reviewing the results

Chapter 7 Computer program for analysis

7.1 General features
7.2 Flow chart
7.3 Steps involved in programming
7.4 Variables used in computer program
7.5 Features of the program

Chapter 8 Results and discussions

8.1 General
8.2 Non-linear analysis
8.3 Output for the vertical loads (Ansys)
8.4 Output for the vertical and horizontal loads (Ansys)
8.5 Discussion on results
8.6 Comparison of linear static and non-linear static analysis results
8.7 Comparison of analysis results form computer program and Ansys

Chapter 9 Design guidelines

9.1 Design guidelines
9.2 Selection of material
9.3 Slenderness ratio limitations

9.4 Permissible stresses in tower members
9.5 Design of members
9.6 Connection design
9.7 Foundation

Chapter 10 Conclusion and further scope of work

10.1 General
10.2 Concluding remarks
10.3 Further scope of work

References

Appendix I Computer program for analysis

CHAPTER – 1 INTRODUCTION

1.1 General

For the development and growth of economy of a country, great attention has to be given to the generation, transmission and utilization of the most convenient and cleanest form of energy, the electrical energy. The technical, environmental and economic considerations involved in sighting and development of power generation projects required for meeting the demand for electrical energy are gradually resulting in longer transmission distances and introduction of higher and higher transmission voltage, and use of high voltage direct current transmission systems. In India, 66kV, 132 kV/110 kV, 230 kV/220 kV, and 400 kV a.c. and +/- 500 kV d.c. systems are already in service and 800 kV a.c. systems are introduced nowadays [2].

With the increase in transmission voltage levels, the heights as well as weights of towers have also increased and so has their cost. The transmission line towers constitute about 28 to 42 percent of the cost of transmission line. Therefore optimization of designs of towers can bring about significant economy in the cost of transmission lines [2].

Depending upon the manner in which the towers are supported they fall in the following three categories: -

1) Self-supporting towers.
2) Conventional Guyed towers.
3) Chainette Guyed towers.

1.2 Self-supporting towers

Self-supporting broad or narrow based lattice steel tower are used in India and other countries. It can be used for a.c. and d.c. extra high voltage (EHV) transmission. Self-supporting towers are with square or rectangular base and four separate footing are required. Some of the inherent drawbacks of these towers are higher steel consumption and higher foundation volume. Some of these towers are shown in figure 1.1.

1.3 Conventional Guyed towers

The features of these types of towers are the portal structures fabricated in 'Y' and 'V' shapes and have been used in some of the countries for EHV transmission upto 735 kV. The guys are either internal or external. The guyed towers including guy anchors occupy much larger land as compared to self-supporting towers and as such this type of construction finds application in long unoccupied waste lands. Guyed towers are much lighter with substantial saving in steel and therefore fabrication, assembly and erection are easier, faster less expensive compared to that of self supporting towers. The analysis of these guyed towers is usually more complicated when compared to self-supporting towers, due to the presence of pre-tensioned guys. The guyed towers exhibit both geometric and material non-linearity [16]. Dependence of stress-strain behavior of the guys on the tensile forces in them is a type of material non-linearity and the large deflections and large rotations compels to go for a geometric non-linear analysis. Some of these towers are shown in figure 1.2.

(1) Guyed single poles or masts

Single guyed masts or poles find many applications on modern lines and are often the most efficient means of handling the large horizontal loads of large angle positions or at pints where dead ending is required. The more common use of guyed single poles or masts relies on an effective pinned connection to the footing. Attempts to apply guys to fixed masts or poles will, of course, introduce indeterminacies and predictable performances will depend on the degree and control of pre-tensioning of the guys and close control on any future movement of guy anchors or rotation of the mast foundation.

When the poles or masts are used as ends, the line or wire tensions are usually taken by in-line back guys, while at line angles, the angle pulls are resisted by bisector guys or by pairs of approximately in-line guys.

Galloping can be a problem [16]. Galloping is caused by strong winds acting on the airfoils of ice-coated wires. Single or two loop galloping can produce tension changes of 2:1 in the conductors. At a dead end or 90 degree guyed single mast, the in-line back guy can respond as a spring system to these pulsating tension changes and/or pole, the result can be amplified motions that can lead to the destruction of line elements.

(2) Guyed Rigid Frames

Guyed rigid frames are very popular and are frequently used a voltages from 69 kV to 230kV and infrequently as high as 500 kV. A single mast and a set of guys and anchors (usually four) replaces the lower tower body and foundations while the upper part of the structure remains practically the same as it would for a totally rigid framing. The significant difference between these guyed rigid frames and the guyed single masts mentioned earlier is that each pair of side guys is attached at widely separated points so that torsional stability is inherent, an imperative with at least three phases attached to the structure.

(3) Guyed and hinged (or pinned) masted structures

The guyed portal, guyed V and the cross-rope suspensions are the three most used versions of this family, each of them comprising two tripods of a mast and guys, the apexes of which are held apart by the cross-arm of the portal tower, held together by the tensioned wire rope suspension assembly of the cross-rope suspensions.

1.4 Chainette Guyed towers

Chainette guyed tower also known as cross rope suspension tower, consists of two masts each of which is supported by two guys and a cross rope which is connected to the tops of two masts and supports the insulator strings and conductors bundles in horizontal formation. As one of the objectives of the study is also dealt with these types of structures, more detailed study about the tower is done in subsequent chapters.

1.5 Objectives of Study

The structural response of chainette tower to externally applied loads is always nonlinear because of the large displacements, which occur, substantially changing the geometry of the structure.

Following are the objectives of the study:-

- To develop geometry of the chainette type of tower in a general-purpose finite element program ANSYS version 7.1 and perform a linear static analysis.

- To perform a geometric non-linear static analysis for the same geometry with the same loads and compare the results. For this purpose a 735 kV chainette tower at Hydro-Quebec (Canada) is taken as a case study [11].

- To develop a computer program for analysis of chainette tower considering the geometric non-linearity only.

- To perform analysis of the same problem with the help of the computer program developed and compare the results with the ANSYS results.

- To design the mast members and compare its weight with the weight of a self-supporting structure required for the same design parameters.

1.6 Action Plan

(1) To collect the relevant data to formulate the design problem.

(2) To understand the behavior of the structure against the various loadings.

(3) To study the concept of non-linear problems and the solutions technique to handle the problem.

(4) To interact with the ANSYS_7.1 software and understand the working of the software and how the structure would behave.

(5) To develop a computer program for analysis using Visual C ++.

(6) To develop the geometry of the mast in STAAD-Pro 2003 and design the mast members.

(7) Collect relevant data for the weight comparison of the self-supporting structure required for the same design data.

1.7 Problem Formulation

Data of the problem [11]: -

1. A line of 735 kV.

2. Phase conductors consist of a bundle of four conductors of the type aluminium conductors with steel reinforced (A.C.S.R.), 42/7 Bersimis (690mm^2).

3. Average span = 490mts.

4. Span factor = 0.95

5. Ground Wire diameter = 13 mm.

6. Design wind pressure = 0.8 kPa on cables and 1.8 kPa on masts.

7. Maximum ice thickness = 32mm radially.

8. The height of the tower = 35 mts vertically.

9. The slope of the guys = 5.75:10 in the transverse direction to the line and 5.25:10 in the longitudinal direction.

10. The spacer cable is 13mm in diameter.

11. The diameter of the guy cables and cross-rope assembly is 25.4 mm.

The figure 1.3 indicates the tower geometry.

CHAPTER – 2 LITERATURE SURVEY AND REVIEW

2.1 General

The first stage of getting introduced to the concept of chainette type of transmission line tower was through literature survey. Intensive literature survey was then carried out to get more information on non-linear analysis and detailed literature on chainette tower. The following books, journals, codes and proceedings were reviewed.

2.2 Literatures reviewed for chainette tower configuration

- Ghannoum, E. et al. [11] has discussed his experience of the first chainette transmission line in this proceeding along with the detailed study of the line and the results of the line.

- Peyrot, A.H. et al. [18] has discussed about the concept of cable element to the Cross Rope Suspension Structure (C.R.S.S.) (another name of chainette tower), transmission line. The concept of modeling the cable elements as long curved cable elements has been principally introduced in this reference.

- Tsui, Y.J et al. [20] has explained the advantages of using chainette tower in this proceeding along with detailed results of the study.

- Vinh, T. et al [21] has discussed about the phenomena of Corona loss and audible noise in this reference, for a 9-conductor bundle, which determines the tower structure configuration. Extrapolations of single-phase test results to evaluate the audible noise and corona loss performance of an assumed three-phase line is also explained.

2.3 Literatures reviewed for non-linear analysis

- Ansys incorporated [3] has explained the procedure of non-linear analysis and the steps involved in it. Finite element method has been used for analysis. Types of non-linearity, solutions for the non-linear problems, types of elements and their properties, importance of convergence criteria and other methods of structural analysis has been well explained in the help system of the software.

- Bathe, K.J. et al. [5] has discussed the theory and implementation of the procedures for finite element analysis, both static and dynamics in this reference.

- Cook, R.D. et al. [6] has discussed about the finite element method of analysis used for structures. In this reference, types of non-linearity, methods of solutions for the non-linear problems and the use of various techniques for improving the solution of the non-linear problems.

- Desai, C.S. et al. [7] has discussed the fundamentals of finite element method of analysis in this reference. The chapter on techniques for non-linear analysis has been the major source of getting the know-how of the non-linear problems and the different solution techniques for the same. Detailed literature on both the geometric and material non-linearities has been explained.

- Foster, E.P. et al. [9] A non-linear iterative technique has been discussed to analyze a cable net-panel system in this reference.

- Gere, J. et al. [10] has discussed the study of direct stiffness matrix method used for computer programming has been studied to help with the programming part.

- Mohan, S.J. et al. [15] has discussed the behavior of the chainette tower under different loadings, modeling the problem in ANSYS program and the steps involved in the non-linear analysis of the chainette tower have been well explained in this reference. The methodology of modeling the cable element as a bar element (3D-truss) between the nodes has been explained.

- Mohan, S.J. [16] has discussed the types of conductor loads on transmission line towers, wind loads, stringing loads and the conceptual background of guyed and special type of tower configurations in this reference.

- Peyrot, A.H. et al. [17] has discussed a numerical procedure to analyze complex 3-dimensional assemblies of substructures and cables in this reference.

- Raman, N.V. et al. [19] has discussed a general finite element method using sub structuring technique for large displacement analysis of guyed towers in this reference.

2.4 Literatures reviewed for design

- Ahluwalia, P.M. et al. [2] has discussed all the features regarding self supporting transmission line tower structures in this reference. The types of hardware used in the assembly of the line, the combinations of different types of loads, the various checks and the general outline of the planning, analysis, design, construction and maintenance of self-supporting towers has been well explained in this reference.

- Dulis, J.E. et al. [8] has discussed the procedures for the design of mast and rope along with the construction and maintenance this reference.

2.5 Codal review for transmission line towers

- A.S.C.E. 91 [1] has been specially dealing with the guyed type of transmission line structures. As there is no Indian code dealing in the guyed towers, this code has been referred for the allowable stresses and the design considerations.

- I.E.C. 826 [12] is accepted as international code to refer for the analysis and design of transmission line tower structures. Most of the derivatives of the I.S. 802 are from this code.

- I.S. 5613 [13] gives the procedure of calculation of sag and tension for conductors and ground wires.

- I.S. 802 [14] has been prepared to give uniform practices for design, fabrication, inspection and testing of overhead transmission line towers. This code consists of the materials, loads and permissible stresses used for the tower structure. The wind load calculation on the tower, conductors and the ground wire is done using this code.

2.6 Literatures reviewed for the computer program

- Bates, J. et al. [4] has explained the fundamentals of programming in Visual C++ 6.0 in this reference. The use of single document interface, use of property sheet technique, files input-output technique, dialog box utilities, and the use of Microsoft Foundation Classes (M.F.C.) has been well explained.

CHAPTER – 3 ABOUT CHAINETTE TOWERS

3.1 General

In this chapter, a brief discussion is made about the looks and effects of chainette towers. During the forthcoming sections, the necessity to use chainette towers, component details of the structure and the experience of James-Bay, Hydro-Quebec, Canada line [11] is discussed.

The chainette tower can be thought of as consisting of two tripods between which the three phases are suspended on a slack cable system [18]. Each tripod has one compression member, the mast, and two tension members, the guys. The ground wires are attached to the top of each mast. A spacer cable is installed between the tops of the mast to simplify tower erection. It becomes slack after the weight of the conductors is applied on the structure and has no structural value; however, it facilitates access to the phases during construction and maintenance operations.

3.2 Why use chainette towers?

Transmission of electrical energy over greater distances is inevitable. For the development of a country, the transmission and distribution is a must and hence transmission line system is required. With the need of extra high voltage (EHV) and ultra high voltage (UHV) lines the design of self-supporting towers would be uneconomical, hence the concept of guyed towers was introduced. A lot many configurations of the guyed towers were introduced. But still the cross-arm could not be omitted, whose design was very heavy for supporting the heavy lines transmitting more than 700 kV line.

3.2.1 Comparison of chainette tower and other guyed towers

- A brief review of existing guyed towers revealed that, for extra-high voltages, 50 percent of the weight of steel of the tower is in its cross-arm and that, as voltages increase, much heavier cross-arms have to be installed at much greater heights. For instance, at 239 kV, the cross-arm weighing only one ton has to be installed at 25 meters, while at 315 kV, a two ton cross-arm has to be installed at 35 meters and at 735 kV, a five ton cross-arm has to be installed at 45 meters [20]. For extra-high voltages, this makes the raising of guyed towers very difficult and costly.

- The weight of steel for the cross-arm of a 735 kV is reduced to only 10 percent of the total weight, and the total weight is reduced by about 40 percent compared with the guyed V – tower.

- Furthermore, as the weight of each element is relatively low, this type of tower is highly suitable for helicopter erection. The mast can be assembled at a site where the working conditions are favourable and the complete masts can be flown to the tower site. Since there is no cross-arm in the tower, tower erection by conventional methods is simplified.

- Large construction tolerances can be accepted because they have negligible effect on the reactions in the members.

- In the case of broken conductors, the internal damping reduces the dynamic peak load. In addition the static residual load is also reduced because the sag of the chainette cables is added to the length of insulator strings.

- In addition, the construction of the chainette tower is much simpler than other more conventional towers, and live maintenance is found to be very safe.

- The additional arguments that helped the acceptance of this tower were its insensibility to frost heave of foundations and the short repair time of a damaged section.

Table 3.1 Theoretical cost comparison between chainette and guyed V towers

	Guyed V	Chainette	Difference
• Purchase of material	49%	43%	- 6%
• Shipping, storage distribution	10%	8.4%	-1.6%
• Foundation installation	18%	20%	+ 2%
• Tower assembly and erection	23%	15.6%	-7.4%
Total	100%	87%	-13%

3.3 Component Details of chainette tower structure

3.3.1 Guys

Guys resist the inward pull of the masts due to the cross-rope assembly supporting the conductors. It also provides longitudinal strength to the tower. The guys are designed so that axial forces at ultimate load would not exceed 80 percent of rated breaking strength.

3.3.2 Masts

Masts resist the compressive loads. The chainette tower masts are conventional steel latticed masts with a staggered single bracing system. The design of the mast was done using non-linear P-Delta analysis [8]. In addition, it is necessary to include the effect of joint and bolt slippage. Due to these, the maximum forces in the chords increases by 5 to 10% when compared to the linear analysis.

The global buckling of the mast is also to be checked, which is marginally critical for the tallest mast.

3.3.3 Spacer Cable

The spacer cable, also known as maintenance cable or construction cable, stretching between mast peaks, stabilizes the tower during construction. Once the conductors are strung in, this spacer cable becomes slack and does not contribute to the tower's structural stiffness or strength.

In order to prevent the construction cable from falling on the conductors when ruptured by frost heave action, a mechanical fuse [11] (figure 3.1) is added to the end fitting. It consists of a bolt designed to break in shear at 50 to 70% of the tensile strength of the construction cable. A slack jumper will prevent the broken cable from falling on the conductors after the weak bolt is sheared. The substitution of the regular bolt by the weaker bolt is done after conductor stringing, in order not to jeopardize the safety of the tower during construction.

3.3.4 Cross ropes

The cross rope assembly supports the bundled conductors through the insulators. The ropes are of cables of equal properties of that of the guys.

3.3.5 Foundation

The foundation of the masts is subjected to compressive forces and a small shear. Steel grillage foundations are used.

Grouted steel bars are used as guy anchors. A typical grouted anchor is shown in the figure 3.2. The theoretical length of guyed V anchors is 14 % more in the rock and 28 % more in normal soil than the chainette anchors [11].

3.4 External Components

3.4.1 Conductors

Conductors are the main electrical part for conveyance of electrical energy from production point to the receiving station.

Mainly there are four types of conductors [2], which are listed below:

1) Copper conductors
2) Aluminium conductors
3) Steel conductors
4) Aluminium conductors with steel reinforced (A.S.C.R.)

Copper conductors are most common conductors used for transmission because it has higher current density, so for the given current rating lesser cross sectional area is required and hence wind load will be less. Again copper metal is quite homogeneous and it has low specific resistance. But because of the shortage of copper-ores in India, the use of aluminium in transmission and distribution lines has been adopted.

Aluminium conductor is next to the copper as far as its conductivity is concerned. Also it is cheap and light in weight. But due to low tensile strength, diameter provided is more and so wind pressure increases. Also swing of the conductor will be more and hence requires larger width. Melting point of aluminium is also low, so the short circuit may damage it. Steel conductors have got the greatest tensile strength but are least used for transmission of electrical energy as steel has got high resistance. Also in damp atmosphere it rusts.

Aluminium conductor having a central core of galvanized steel is used for high voltage transmission purposes. This is done to increase the tensile strength of aluminium conductors. One or more aluminium conductors cover the galvanized steel core. The steel conductors are galvanized in order to prevent corrosion and rusting. The cross sections of the two metals are in the ratio 1:6, but in case of high strength conductors their ratio is 1:4. Thus the A.C.S.R. has less sag and longer span than the copper conductor line since it has high tensile strength. The various properties of these types of conductors are presented.

Table 3.2 Conductor Details

Sr.No.	Code	Strands (No./mm.)		Ultimate Strength (N.)	Overall Diameter (mm.)	Unit Wt. (N/m.)	Coefficient Of linear Expansion (/ deg C.* 10e-06)	Modulus of Elasticity (N/sq.mm.) * 10e05
		Al	Steel					
1	Dog	6/4.720	7/1.570	33050	14.15	3.94	19.1	0.806
2	Leopard	6/5.283	7/1.753	41400	15.85	4.935	19	0.806
3	Coyote	26/2.540	7/1.904	46550	15.9	5.215	18.99	0.773
4	Tiger	30/2.362	7/2.362	58000	16.5	6.061	17.8	0.816
5	Wolf	30/2.590	7/2.590	68670	18.13	7.261	17.8	0.816
6	Lynx	30/2.794	7/2.794	79650	19.58	8.455	17.8	0.816
7	Lark	30/2.924	7/2.924	90800	20.47	9.231	17.8	0.816
8	Panther	30/3.000	7/3.000	91440	21	9.741	17.8	0.816
9	Bear	30/3.353	7/3.353	113300	23.5	12.195	17.8	0.816
10	Goat	30/3.708	7/3.708	138000	26	14.915	17.8	0.816
11	Sheep	30/3.980	7/3.980	159000	27.93	17.26	17.8	0.815
12	Kundara	42/3.595	7/1.960	90540	26.88	12.81	21.2	0.632
13	Zebra	54/3.160	7/3.180	132890	28.62	16.21	19.3	0.704
14	Deer	30/4.267	7/4.267	182000	29.84	19.8	17.8	0.816
15	Camel	54/3.353	7/3.353	147600	30.2	18.1	19.3	0.704
16	Drake	26/4.442	7/3.454	141770	28.14	16.28	18.99	0.773
17	Moose	54/3.530	7/3.730	164380	31.77	20.04	19.3	0.704
18	Canary	54/3.280	7/3.280	146500	29.51	17.21	19.3	0.704
19	Dove	26/3.720	7/2.890	101800	23.55	11.37	18.99	0.773
20	Redwing	30/3.920	19/2.350	156900	27.46	16.46	17.5	0.738
21	Bersimis	42/4.570	7/2.540	157340	35.1	21.85	21.2	0.632
22	Curles	54/3.510	7/3.510	163500	31.62	19.75	19.3	0.704
23	Duck	54/2.690	7/2.690	102100	24.18	11.58	19.3	0.704
24	Leg Horn	1/22.69	7/2.690	58600	13.46	5	15.3	1.05

3.4.2 Groundwires

Groundwire having sufficient mechanical strength is situated in such a manner so as to shield the line conductors from direct strokes. The groundwires are selected from mechanical strength point of view rather from electrical considerations [2]. Also groundwires must be non-corrosive.

Table 3.3 Groundwire Details

Sr.No.	Strands (No./dia in mm.)	Weight per meter length (N)	Overall Diameter (mm.)	Total c/s Area (sq.mm.)	Ultimate tensile strength (N)		
					700 N/sq.mm.	1100 N/sq.mm.	1570 N/sq.mm.
1	7/3.15	4.28	9.45	54.552	36990	58130	82970
2	7/3.50	5.28	10.5	67.348	45670	71770	102430
3	7/3.66	5.83	10.98	73.646	49940	78480	112010
4	7/4.00	6.9	12	87.965	59650	93740	133790
5	19/3.15	11.63	15.75	148.069	100410	157780	-
6	19/3.50	14.36	17.5	182.801	123960	194790	-
7	19/3.66	15.7	18.3	199.897	135550	-	-
8	19/4.00	18.75	20	238.761	161910	-	-

Strands	Modulus of Elasticity (N/Sq.mm.)	Coefficient of Linear Expansion (/degree C)
7	$1.969*10^6$	$11.50*10^{-6}$
19	$1.898*10^6$	$11.50*10^{-6}$

3.4.3 Insulators

In order to prevent the flow of current to the earth from support, the transmission lines are all secured to the supporting towers or poles with the help of insulators. Thus the insulators play an important part in the successful operation of the lines.

Material

Porcelain is the most common material used for insulators. Mostly three types of materials are used for insulators:

1) Porcelain, 2) Glass, 3) Stealite.

Types

Many types of insulators are available in the market but each have a specific purpose. They are discussed below:

1) Pin type insulator: Used for low voltage lines.

2) Suspension type insulator: Used for high voltage and EHV lines. They can be used in a series and each is designed for 11 kV.

3) Strain insulator: When there is dead end of tower or sharp curve or line crosses river etc., line is to withstand great strain and at that time strain insulators are used.

4) Stay insulator: For low voltage lines, the stays are to be insulated from ground. Insulators used in stay wires are called stay insulators and are so designed that in case any insulator breaks, the guy wire would not fall to the ground.

5) Shackle insulator: Used for low voltage distribution lines. Such insulators can either be used in a horizontal position or in a vertical position.

3.5 Case-study Details (735kV Hydro-Quebec, Canada)

3.5.1 General

This portion deals with the design and construction features and experience that Hydro-Quebec acquired with 1500 km of 735 kV chainette lines already in service. The design features of this type of tower are discussed as well as the parameters of the chainette tower used on James Bay lines. In this portion, the construction of the chainette tower is compared to the guyed V tower and the productivities of constructions crews are discussed. Comparative material and construction costs are also given.

The chainette tower concept was thoroughly validated by Hydro-Quebec in 1976 and an experimental line of nine 735 kV chainette towers was built to test construction methods and perform dynamic and static full-scale tests on the line.

A 1/50 scale model of a chainette line was built according to principles of dynamic similitude. This model was used to reproduce different dynamic and static load conditions and proved to be very reliable when compared with theoretical analysis and full scale testing.

3.5.2 Comparison with the Guyed V towers

Table 3.1 represents the studies carried out during this project [11]. The overall percentage of the profit, reduced to 4 % after the completion of the project.

The analysis of the chainette tower under frost heave conditions indicated that an upward movement of 30 cm of both foundations reduces the capacity of the tower by an amount of 10% to 15% except for the construction cable, which will fail. By comparison the guyed V tower will fail under a frost heave of 15 to 30 cm.

The chainette tower accepts large tolerances without any significant influence on its strength as shown in table 3.4. Compared to a self-supporting tower where foundation tolerance is within a few mm, the chainette tower tolerances are considerably more (about 500 mm).

Hydro-Quebec's construction tolerances were set as the basis of the precision that can be achieved without undue costs. The adopted tolerances are shown in table 3.5.

Table 3.4 Consequences of construction tolerances (35 m mast)

Tolerance	Max loads (kN)		Clearance (m)	
	Mast	Guys	No Wind	Limit wind
• Ideal structure	375	900	5089	1.33
• Foundation +1 m inwards	378	906	5.54	1.05
• Pretension -6 kN	373	897	5.88	1.31
• Foundation +0.75m upwards	376	908	5.76	1.19

Table 3.5 Specified construction tolerances of the chainette tower

Item	Tolerance
Foundation	
- Vertical	25 mm
- Horizontal	50 mm radius
Anchors, horizontal	300 mm radius
Top of masts	300 mm radius
Pretension	± 5 kN

Table 3.6 Man-days per km required for construction

Construction Activity	Self-supporting	Guyed V	Chainette
	Tower types		
• Material distribution	18	12.3	10.5
• Foundations (grillage+piles+concrete)	62	23.9	39.7
• Anchors assembly	-	42.8	42.8
• Tower assembly	42	32.2	16.0
• Tower erection (+ guys + chainette + insulator + inspection)	53	23.1	26.2
• Conductors + g.w. + jumpers + counterpoise	37	36.5	36.5
Total	212	170.8	171.7

As shown in the table 3.6, the construction of the chainette tower and the guyed V are equivalent and 20 % cheaper than the self-supporting tower. A thorough analysis was done in order to explain the difference between this result and the theoretical analysis that indicated a 4.5 % advantage in favour of the chainette tower.

The cost of material was compiled for guyed V and chainette tower lines, in which a reduction of 10% was obtained with the use of the chainette tower. One of the important savings comes from the steel cost, not only from weight reduction, but also from the lower cost per unit weight due to the simplicity of fabrication.

The chainette tower construction proved to be very satisfactory after the completion of more than 1500 km of 735 kV lines.

3.6 Codal Provisions

A.S.C.E – 91 [1], provides with some information on how to model the structure on computer. It is strongly recommended that a geometrically nonlinear computer analysis be used to determine final design forces and capacity in guyed structures.

Guy pretension of about 5 % to 10 % of Rated Breaking Strength (RBS).

Allowable Tensions

Guy stress under service loads such as extreme wind, wind and ice, and extreme ice is usually limited to 65 % of the guy RBS to prevent exceeding the elastic limit of the guy. Some line designers will allow the guy (and fitting) tensions to reach 85% of RBS under longitudinal or failure containment loading that results from a failed tower or broken conductor.

CHAPTER – 4 NON-LINEAR ANALYSIS

4.1 General

The analysis of the guyed towers is usually more complicated when compared to self-supporting towers, due to the presence of pre-tensioned guys. The guyed towers exhibit both geometric and material non-linearity. The structural response of chainette tower to externally applied loads is always nonlinear because of the large displacements, which occur, changing the geometry of the structure substantially [15].

The chainette tower can be analyzed conservatively as a statically determinate truss composed of linearly elastic members. However, very little understanding of its structural behavior would be gained through such a simplified approach. This is due to some of the ropes becoming slack under certain loading conditions. Further changes in the tower geometry introduce large degree of geometric non-linearity. As a result, large displacements occur in the structure, due to large rotations experienced by the elements of the system. At element level, these large deformations also introduce a rigid body rotations and bowing actions. Under these conditions, any further change in loading increases the deflection, necessitating the consideration of the changes in the geometry. Hence a nonlinear iterative solution is necessary for the analysis of chainette tower.

Generally Newton-Raphson method is used in solving non-linear structural problems wherein during each stage of iteration the stiffness matrix is also reworked to account for the nonlinear behavior of the structure. The stiffness matrix thus worked out during each iteration, is expected to reach the exact tangent stiffness of the nonlinear structure after a few cycles of iterations [7]. Further improvement of the solution may be possible by applying the loads in several steps instead of applying in a single step.

4.2 Types of nonlinearity

In structural mechanics, material may yield or creep; local buckling may arise; gaps may open or close. Nonlinear problems pose the difficulty of describing phenomena by realistic mathematical and numerical models and the difficulty of solving nonlinear equations that result. Effort required of the analyst increases substantially when a problem becomes nonlinear.

In structural mechanics, types of nonlinearity [6] include the following:

1) Material Nonlinearity, in which material properties are functions of the state of stress or strain. Examples include nonlinear elasticity, plasticity, and creep.

2) Contact Nonlinearity, in which a gap between adjacent parts may open or close, the contact area between parts changes as the contact force changes, or there is sliding contact with frictional forces.

3) Geometric Nonlinearity, in which deformation is large enough that equilibrium equation, must be written with respect to the deformed structural geometry. Also, loads may change direction as they increase, as and when pressure inflates a membrane.

Problems in these categories are nonlinear because stiffness and perhaps loads as well, become functions of displacement or deformation. Thus, in structural equations [K]{u}={P}, [K] and {P} become functions of {u}(refer nomenclature section for the meanings of the used symbols). We cannot immediately solve for {u} because information needed to construct [K] and {P} is not known in advance. An iterative process is required to obtain {u} and its associated [K] and {P} such that the product [K]{u} is in equilibrium with {P}.

When equations [K]{u}={P} are nonlinear the principle of superposition does not apply. That is, we cannot scale results in proportion to load or superpose results of different load cases. Each different load case requires a separate analysis. Also, for a given set of loads there may be more than one solution for {u}. If a load case consists of (say) two portions that are sequentially applied, reversing the sequence of application may produce different results.

Successful nonlinear finite element analysis requires a grasp of the physical problem, but also more understanding of equation-solving procedures than is required for linear analysis, because a single strategy may not work for all problems. Satisfactory results may appear only after several attempts and a change of strategy.

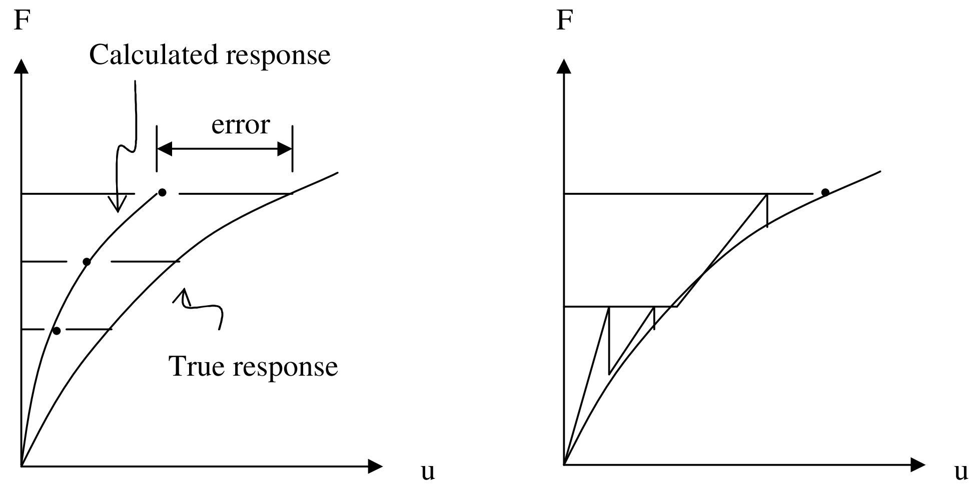
Geometric nonlinearity is ascribed to large-deflection problems in which the deformed configuration must be used to write the equilibrium equations, and to problems related to structural stability. This nonlinearity is introduced into the theory of elasticity through the equilibrium equations and by the inclusion in the strain-displacement relation of higher-order terms [3].

4.3 Nonlinear Analysis

The nonlinear behavior of structures cannot be represented directly with a set of linear simultaneous equations to predict the response of an engineering system. A series of successive linear approximations with corrections are needed to solve nonlinear problems.

Incremental loading and equilibrium iterations:

One approach to nonlinear solutions is to break the load into a series of load increments. The load increments can be applied either over several load steps or over several substeps within a load step. At the completion of each incremental solution, the stiffness matrix is to be adjusted to reflect the nonlinear changes in structural stiffness before proceeding to the next load increment. Unfortunately, a pure incremental approach inevitably accumulates error with each load increment, causing the final results to be out of equilibrium, as shown in figure 4.1.



(a) Pure Incremental Solution

(b) Full Newton-Raphson Iterative Solution (2 load increments)

Figure 4.1 Pure Incremental Approach V/S Newton-Raphson Approach.

The Newton-Raphson equilibrium iterations overcome this difficulty, which drives the solution to equilibrium convergence (within some tolerance limit) at the end of each load increment. The figure 4.1 gives an illustration of a single DOF nonlinear analysis.

Before each solution, the Newton-Raphson method evaluates the out-of-balance load vector, which is the difference between the restoring forces (the loads corresponding to the element stresses) and the applied loads. Then a linear solution is performed, using the out-of-balance loads, and checks for convergence. If convergence criteria are not satisfied, the out-of-balance load vector is re-evaluated, the stiffness matrix is updated, and a new solution is obtained. This iterative procedure continues until the problem converges.

A number of convergence-enhancement and recovery features, such as adaptive descent, line search, automatic load stepping, and bisection, can be activated to help the problem to converge.

The arc-length method causes the Newton-Raphson equilibrium iterations to converge along an arc, thereby often preventing divergence, even when the slope of the tangent stiffness matrix becomes zero or negative. This iteration method is represented schematically in the figure 4.2.

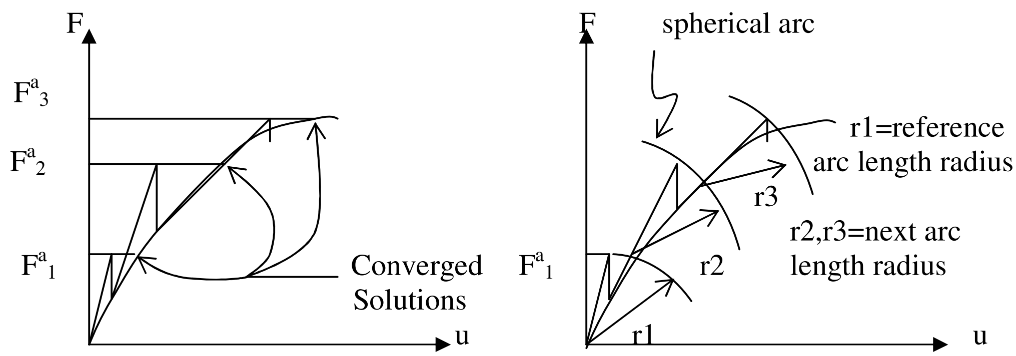


Figure 4.2 Traditional Newton-Raphson Method v/s Arc-length Method.

4.4 Types of Geometric Nonlinearities

Geometric nonlinearities refer to the nonlinearities in the structure or component due to the changing geometry as it deflects. That is, the stiffness [K] is a function of the displacements {u}. The stiffness changes because the shape changes and /or the material rotates. There are four types of geometric nonlinearities:

(1) Large Strain: - In this, the strains are no longer infinitesimal. Shape changes (e.g. area, thickness, etc.) are also accounted for. Rotations may also be large.

(2) Large Deflection: - In this, the rotations are large but the mechanical strains (those that cause stresses) are small. The structure is assumed not to change shape except for rigid body motions.

(3) Stress Stiffening: - In this, both the strains and rotations are small. A 1st order approximation to the rotations is used to capture some nonlinear rotation effects.

(4) Spin Softening: - In this, both the strains and rotations are small. This option accounts for the radial motion of a body's structural mass as it is subjected to an angular velocity. Hence it is a type of large deflection but small rotation approximation.

4.5 Stress stiffening

Stress stiffening (also called geometric stiffening, incremental stiffening, initial stress stiffening, or differential stiffening by different authors) is the stiffening (or weakening) of a structure due to its stress state. This stiffening effect normally needs to be considered for thin structures with bending stiffness very small compared to axial stiffness, such as cables, thin beams, and shells and couples the in-plane and transverse displacements. This effect also augments the regular nonlinear stiffness matrix produced by large strain or large deflection effects. The effect of stress stiffening is accounted for by generating and then using an additional stiffness matrix, hereinafter called the "stress stiffness matrix". The stress stiffness matrix is added to the regular stiffness matrix in order to give the total stiffness. Stress stiffening may be used for static or transient analysis.

The stress stiffness matrix is computed based on the stress state of the previous equilibrium iteration. Thus, to generate a valid stress-stiffened problem, at least two iterations are normally required, with the first iteration being used to determine the stress state that will be used to generate the stress stiffness matrix of the second iteration. If this additional stiffness affects the stresses, more iterations are needed to be done to obtain a converged solution.

Theory

The strain-displacement equations for the general motion of a differential length fiber are derived below. Two different results have been obtained and both of these are discussed here. Consider the motion of a differential fiber, originally at dS, and then at ds after deformation.

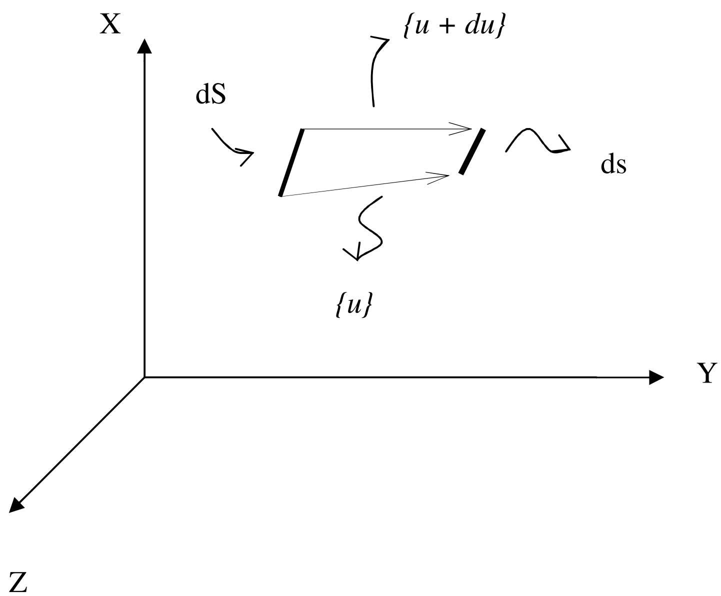


Figure 4.3 General Motion of a Fiber

One end moves *{u}*, and the other end moves *{u + du}*, as shown in figure 4.3. The motion of one end with the rigid body translation removed is *{u + du} - {u} = {du}*.

$$\{du\} \begin{bmatrix} du \\ dv \\ dw \end{bmatrix} = \qquad \text{--------------------------(1)}$$

where u is the displacement parallel to the original orientation of the fiber. This is shown in the figure 4.4. Note that X, Y, and Z represent global Cartesian axes, and x, y, and z represent axes based on the original orientation of the fiber.

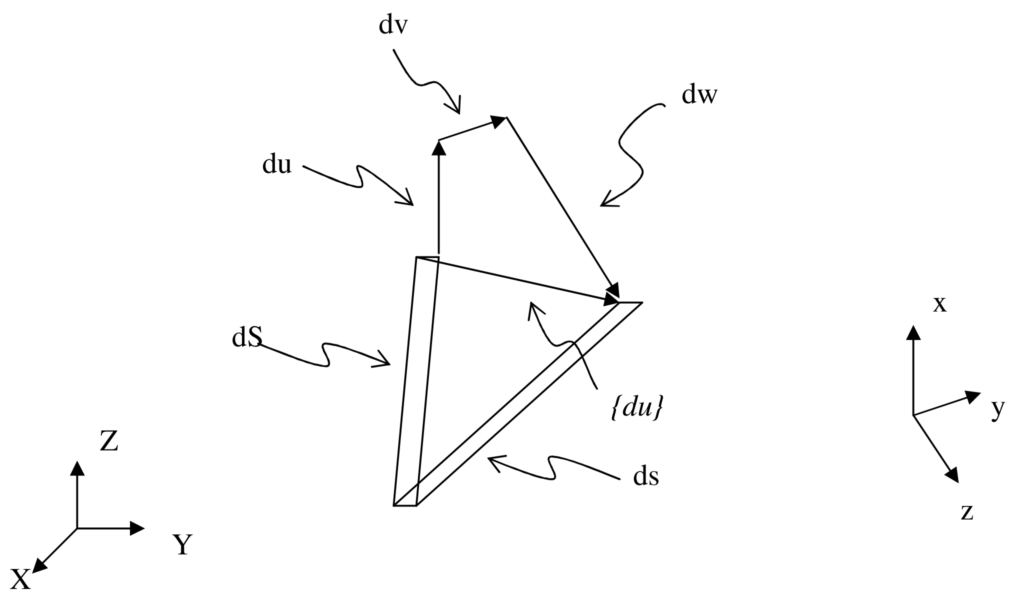


Figure 4.4 Motion of a fiber with rigid body motion removed.

By the Pythagorean theorem,

$$ds = \sqrt{(dS + du)^2 + (dv)^2 + (dw)^2} \quad \text{-------------------}(2)$$

The stretch, Δ, is given by dividing ds by the original length dS:

$$\Delta = ds/dS = \sqrt{(1+du/dS)^2 + (dv/dS)^2 + (dw/dS)^2} \quad \text{----------}(3)$$

As dS is along the local x axis,

$$\Delta = \sqrt{(1+du/dx)^2 + (dv/dx)^2 + (dw/dx)^2} \quad \text{-------------------}(4)$$

Next, Δ is expanded and converted to partial notation:

$$\Delta = \sqrt{1+2*\partial u/\partial x + (\partial u/\partial x)^2 + (\partial v/\partial x)^2 + (\partial w/\partial x)^2} \text{ ---------(5)}$$

The binominal theorem states that:

$$\sqrt{1+A} = 1 + A/2 - A^2/8 + A^3/16 \ldots \text{ ------------------------------(6)}$$

when A < 1. One should be aware that using a limited number of terms of this series may restrict its applicability to small rotations and small strains. If the first two terms of the series in (6) are used to expand the equation (5),

$$\Delta = 1+\partial u/\partial x + {}^1/_2*\left((\partial u/\partial x)^2 + (\partial v/\partial x)^2 + (\partial w/\partial x)^2 \right) \text{ ----------(7)}$$

The resultant strain (same as extension since strains are assumed to be small) is then

$$\varepsilon_x = \Delta - 1 = \partial u/\partial x + {}^1/_2*\left((\partial u/\partial x)^2 + (\partial v/\partial x)^2 + (\partial w/\partial x)^2 \right) \text{ -------(8)}$$

For better accuracy, the first three terms of equation (6) are used and displacement derivatives of the third order and above are dropped, equation (6) reduces to:

$$\Delta = 1+\partial u/\partial x + {}^1/_2*\left((\partial v/\partial x)^2 + (\partial w/\partial x)^2 \right) \text{ ----------(9)}$$

The resultant strain is:

$$\varepsilon_x = \Delta - 1 = \partial u/\partial x + {}^1/_2*\left((\partial v/\partial x)^2 + (\partial w/\partial x)^2 \right) \text{ ---------------------(10)}$$

For most 2-D and 3-D elements, equation (8) is more convenient to use as no account of the loaded direction has to be considered. The error associated with this is small as the strains were assumed to be small. For 1-D structures, and some 2-D elements, equation (10) is used for its greater accuracy and causes no difficulty in its implementation.

Implementation

Using the nonlinear strain-displacement relationships given in equation (8) or equation (10) and on doing some rigorous mathematical evaluations, the solution obtained for a spar such as LINK 8 (ANSYS) the stress-stiffness matrix is given as:

$$[Kg] = F / L \left[\begin{array}{ccc|c} (1-l^2) & -lm & -ln & \\ -lm & (1-m^2) & -mn & (-) \\ -ln & -mn & (1-n^2) & \\ \hline & & & \\ & & & \end{array}\right]$$

where: [Kg] = stress stiffness matrix.

F = force in member.

L = length in member.

Applicable Input

In a nonlinear analysis, the stress stiffness contribution is activated and added to the stiffness matrix. When not using large deformations, the rotations are presumed to be small and the additional stiffness induced by the stress state is included. When using large deformations, the stress stiffness augments the tangent matrix, affecting the rate of convergence but not the final converged solution.

Applicable Output

In a small deflection/small strain analysis, for 2-D and 3-D elements strains are computed using equation (8). The strains (output) therefore include the higher-order terms in the strain computation. Also, nodal and reaction loads (output) will reflect the stress stiffness contribution, so that moment and force equilibrium include the higher order (small rotation) effects.

CHAPTER – 5 TYPES OF LOADS AND LOAD COMBINATIONS

5.1 General

Tower loading is most important part of tower design. Any error in load assessment will make the tower design erroneous and, because, the percentage of the tower cost is high of the whole line cost, it makes more important for a designer to calculate the loads on the tower so that the design becomes economical and safe against the loads. In the load calculations the wind plays a vital role. The correct assessment of wind will lead to proper load assessment and reliable design of tower structure.

5.2 Requirements of loads on transmission lines

Overhead transmission lines are subjected to various loads during their life span, which are classified into three distinct categories [2]: -

(a) Climatic loads related to reliability requirements.
(b) Failure containment loads related to security requirements.
(c) Construction and maintenance loads related to safety requirements.

5.2.1 Reliability requirements – Climatic loads under Normal Condition

- Wind Loads (Non-Snowy Regions).
- Wind Loads with Ice (Snowy Regions).
- Wind Loads without Ice (Snowy Regions).

5.2.2 Security requirements – Failure Containment loads under Broken Wire Condition

- Unbalanced Longitudinal Loads and Torsional Loads due to broken wires.
- Anti-Cascading Loads due to failure of items like insulators, hardware joints etc. as well as failure of major components such as towers, foundations and conductors may result in cascading conditions. In order to prevent the cascading failures angle towers shall be checked for anti-cascading loads for all conductors and earthwires broken in the same span.

5.2.3 Safety requirements – Loads during construction and maintenance

- As an important and essential requirement, construction and maintenance practices should be regulated to eliminate unnecessary and temporary loads that would otherwise demand expensive permanent strengthening of towers.

5.3 Nature of loads

5.3.1 Transverse Loads (T)

- Wind load on tower structure, conductor, ground-wire and insulator strings.
- Component of mechanical tension of conductor and ground-wire.

5.3.2 Vertical Loads (V)

- Loads due to weight of each conductor, ground-wire based on appropriate weight span, weight of insulator strings and fittings.
- Self-weight of structure.
- Loads during construction and maintenance.

5.3.3 Longitudinal Loads (L)

- Unbalanced horizontal loads in longitudinal direction due to mechanical tension of conductor and / or ground-wire during broken wire condition.

5.4 Loading Criteria

Loads imposed on tower due to action of wind are calculated under the following climatic criteria:

- Everyday temperature and design wind pressure.
- Minimum temperature with 36 % of design wind pressure.

5.5 Load computations

All the three loads namely longitudinal, transverse and vertical loads are further subdivided into three categories:

(1) Random loads
(2) Permanent loads
(3) Maintenance loads

5.5.1 Transverse loads

[I] Random transverse loads

These loads are imposed on tower due to action of wind on transmission line and do not act continuously. The random transverse load shall be the sum of the horizontal loads due to:

(1) Wind action on conductor, groundwire, insulator string and tower structure computed as per procedure detailed below.
(2) Mechanical tension of conductors and groundwires.

Calculations of various random transverse loads are discussed below in detail:

(1) Wind load on conductor or groundwire: The load due to wind on each conductor and groundwire applied at supporting point normal to the line shall be determined by the following expression;

$F_{wc} = P_d * L * d * G_c * C_{dc}$

Where,

F_{wc} = Wind load at conductor or groundwire point in N,

P_d = Maximum wind pressure in N/Sq.m.

d = Diameter of conductor or groundwire in meters.

C_{dc} = Drag Coefficient which is 1.0 for conductor and 1.2 for groundwire.

G_c = Gust response factor which takes into account the turbulance of the wind and the dynamic response of the conductor.

(2) Wind load on insulator string: Wind load on insulator string shall be determined from the attachment point to the center line of the conductor in case of suspension tower and up to the end of clamp in case of tension tower in the direction of wind. It can be determined by,

$F_{wi} = P_d * A_i * G_i * C_{di}$

Where,

F_{wi} = Wind load on insulator string in N,

P_d = Maximum wind pressure on insulator string in N/Sq.m

A_i = Area of insulator string projected horizontally on the vertical plane. In absence of actual value it may be taken as half of the gross projected area of the insulator string.

G_i = Gust response factor depending upon terrain category and height of insulator attachment above ground.

C_{di} = Drag coefficient of insulator is taken as 1.2

(3) Wind load on tower: Wind load on tower for each member can be calculated by formula given below:

$F_{wt} = P_d * A_e * C_{dt} * G_T$

Where,

F_{wt} = Wind load on particular member in N,

P_d = Wind pressure on particular member in N/Sq.m.,

A_e = Net effective area of the member in Meters. It is 1.5 times area of the member projected on vertical plane, which is perpendicular to the wind direction.

C_{dt} = Drag coefficient pertaining to wind blowing against any face of the tower. It also depends upon the solidity ratio.

G_T = Gust response factor depending upon terrain category and height of C.G. panel above ground level.

After calculating the wind load on each member, it can be further distributed equally at both the appropriate nodes in horizontal direction.

(4) Mechanical tension of conductor and groundwire: This load is called deviation load, which act, on the tower as component of tension of conductor/groundwire. It can calculated by,

$F_{wd} = 2 * T * \sin (L/2)$

Where,

F_{wd} = Load due to deviation in N,

T = Maximum tension of conductor or groundwire in N,

L = Angle of line deviation

[II] Permanent transverse load

These load are horizontal loads due to mechanical tension of conductors/groundwires at everyday temperature and no wind. These loads act on the tower due to deviation of the line and can be calculated by,

$T_p = 2 * T_1 * \sin (b/2)$

Where,

T_p = Permanent transverse load in N.,

T_1 = Tension in conductor/groundwire in N calculated at everyday temperature and no wind condition,

b = Angle of line deviation.

[III] Transverse loads during maintenance

No extra transverse loads will be assumed to be acting on tower during maintenance of line and these may be taken as nil.

5.5.2 Vertical loads

[I] Random vertical loads

No element of vertical loads is considered random in nature and they may be taken as nil.

[II] Permanent vertical loads

These loads comprises of:

(1) Loads due to weight of each conductor or groundwire based on appropriate weight span.

(2) Loads due to weight of insulator strings and fitting can be assumed as 600 N.

(3) Loads due to weight of structure.

[III] Vertical loads during maintenance

Load of 1500 N shall be considered acting anywhere on the tower crossarm point as a provision of weight of lineman with tools.

5.5.3 Longitudinal loads

[I] Random longitudinal loads

Horizontal loads in longitudinal direction are due to mechanical tension of conductor/groundwires. In normal condition, and if the ruling spans are same on either sides, the tension due to the conductor/groundwire is balanced coming from either sides of the tower. Differential tension occurs due to different ruling span on either side of the tower. In broken wire condition, tension will act on one side of tower.

[II] Permanent longitudinal load

Horizontal load due to mechanical tension of conductor or groundwire at everyday temperature and no wind condition.

[III] Longitudinal load during maintenance

No additional load is imposed on tower in this condition and may be taken as nil.

5.6 Loads while broken wire condition

Loads in broken wire condition are very different compared to normal condition. Discussion about the changes is indicated below for various loads:

(1) Transverse loads: The transverse loads at broken wire condition to be calculated in similar way as explained above for normal condition except the wind on conductor/groundwire would be 60% of normal condition.

(2) Vertical loads: The vertical loads at broken wire condition to be calculated in similar way as explained above for normal condition except the weight of conductor/groundwire would be 60% of the normal condition.

(3) Longitudinal loads: Unbalanced longitudinal pull due to broken wire conditions shall be considered as follows;

(i) Suspension towers

a. 50 % of phase tension

b. 100% of groundwire tension

(ii) Tension towers

a. 100% of phase tension

b. 100% of groundwire tension

5.7 Combination of loads

The safest tower would be the one which is designed to withstand the worst loading condition. But, for economy combined with reliability, we must consider the probable combinations of loads that are likely to occur. This will depend upon the importance of line, type of tower etc.

IS: 802 [14] lays down the broken wire conditions for which various types of towers are to be designed. The broken wire conditions are to be assumed in the design.

The following table 5.1 speaks about it.

Table 5.1 Combination of loads

[I] Single circuit towers	Any one-power conductor or one groundwire broken whichever is more stringent for a particular member.
[II] Double circuit towers (1) Tangent tower with suspension strings (0° to 2° deviation)	Any power conductor broken or one groundwire broken whichever is more stringent for a particular member.
(2) Small angle tower with tension strings. (2° to 15° deviation) (3) Medium angle tower with tension strings. (15° to 30° deviation)	Any two of the power conductors are broken on the same side and on the same span or any one of the power conductors and any one groundwire broken on the same span whichever combination is more stringent for a particular member.
(4) Large angle and dead end tower with tension strings. (30° to 60° deviation)	Three power conductors are broken on the same side and on the same span or any two of the power conductors and any one groundwire broken on the same span, whichever combination substitutes the most stringent condition for a member.

5.8 Other types of loads

Apart from wind loads and self weight of the tower all other loads are one way or other related to the presence of conductor / earth wire only. Based on the operating voltage of the line and average and peak loads to be transmitted, the size, material and the type of the conductors are selected. The tower is exposed to stringing loads and construction / maintenance loads while the conductors are being installed. After the stringing operations the other components of loads coming on the tower are the gravity load in vertical direction due to self weight of the conductor and transverse / longitudinal horizontal loads form conductor tension and line deviations if any. Unbalanced longitudinal loads may occur in case of any mechanical rupture in tower accessories or insulator. Further, the towers are also subjected to other types of loads like galloping loads, ice loads and short circuit loads. All these types of loads are briefly explained in this section [16].

5.8.1 Sag and Tension calculation for conductor / earth wire

If a uniform, perfectly flexible and inelastic length of material, such as a chain or cable, hangs in still air between two fixed supports, it will take the form of a catenary. For the catenary, the mass of the conductor is assumed to be uniformly distributed along the arc of the conductor. The minimum tension in the cable will be at the lowest point of the arc, and the maximum tension will be at the points of support. The tension at any point in the cable will consist of two components: 1) a horizontal component, which is uniform throughout the length of the cable, and 2) a vertical component, which varies along the curve. Hence the total tension in the cable will also vary along its length. The vertical component of the tension at the lower most point of the cable is zero. If it is assumed that the mass of the cable is uniformly distributed along a horizontal line between the points of support, instead of along the cable itself, the resultant mathematical equation for the curve of the cable is that of a parabola. The results of the two methods of calculation (catenary and parabola) are almost identical when the sag is small; however, the difference in results becomes increasingly greater as the sag increases. Since longer spans have larger sags, the difference increases as the span length increases. (Figure-5.1 and Figure-5.2)

Within a limited range of values of the ratio of sag to span, either the catenary or the parabolic method may be used for calculations. Generally, the use of the parabolic method should be limited to spans where the value of this ratio is less than 0.05. The catenary method can also be used for ratios less than 0.05, and should be the method used for ratios between 0.05 and 0.20. For ratios greater than 0.20, the catenary may present difficulties. Fortunately, most transmission lines encountered in practice will involve a sag – to – span ratio of less than 0.20. The error inherent or introduced in sag and tension computations should not be greater than the tolerance allowed in stringing the conductor. In general, the error allowed in stringing is 10 to 15 mm per 30 mts of span length for spans up to and including 350 mts, and 150 mm maximum error for spans greater than 350 mts.

Calculations of sag and tension as given in Appendix A of IS: 5613 [13] may be followed to obtain these values to a reasonable accuracies in the event of rigorous analysis are not wanted.

5.8.2 Broken Conductor Loads

In the broken conductor problem the initial condition, that is before the conductor breaks, is one of static equilibrium with all known quantities such as sags, tensions, and span lengths. The final condition is likewise one of static equilibrium; however, in this case all such quantities are unknown. The diagrams shown in figures 5.3 and figures 5.4 have been given to illustrate in general these initial and final conditions. Referring to figure 5.4, the horizontal force designated P is the force, which retards or damps the effects of the broken conductor and may be considered as the equal and opposite of the force required to deflect an insulator string by any angle θ while a vertical load is acting. The relation between the force, P, and the vertical load can be determined from figure 5.5. Again, assuming that the insulator string acts as a rigid body and also that its gravity axis is midway between the conductor and the attachment hinge, the following relations can be written: for equilibrium at any angle θ;

$Wd = P'v$ or

$P' = bWd/v$.

Also, $\cos\theta = v/I$ and $\tan\theta = d/v = P/W$ from which $P = W\,d\,/\,v$;

therefore, $P = P'$.

Also, $P = Wd\,/I\cos\theta$, which is a form more convenient for calculations.

As far as the broken conductor problem is concerned, this relation should be interpreted as; P is the horizontal force, which resists the movement of an insulator string of length (I) from the vertical to any angle θ while a vertical load W is acting.

The horizontal force designated H in figure 5.4 is the horizontal component of tension acting in the conductor or cable. As far as the broken conductor problem is concerned, the relation between this force and a change in span length, which might be caused by the deflection of an insulator string, is of primary importance and can be developed from figure 5.5 as follows:

The length of conductor in the initial span is:

$L_0 = (2H_0/w)^* \sinh (wL_0/2H_0)$

The length of conductor in the span after a change of Ø is:

$L_1 = (2H_1/w)^* \sinh \{w (L_0 - Ø)/2H_1\}$

The change in conductor length due to the elastic properties of the conductor in changing the tension from H_0 to H_1 is:

$(H_0 – H_1) (L_0) / AE$

Then, barring temperature and / or loading changes, the final conductor length must equal the initial conductor length plus or minus the elastic change in the conductor depending on whether the span length is increased or decreased, i.e.

$L_1 = L_0 \pm (H_0 – H_1) (L_0) / AE$

By substituting the values found above for L_1 and L_0,

$\text{Sinh} \{w (L_0 - Ø)/2H_1\} = \{[H_0 \sinh (wL_0/2H_0)] [1 \pm (H_0 - H_1)/AE]\} / H_1$

And solving for Ø:

$Ø = L_0 – 2H_1 / w * \sinh^{-1} \{[H_0 \sinh (wL_0/2H_0)] [1 \pm (H_0 - H_1)/AE]\} / H_1$

By way of interpretation, this means that when a span of length L_0 is changed in length by an amount Ø, the horizontal tension in the cable changes from H_0 to H_1, which gives the final broken conductor load after a conductor breakage for that particular span.

5.8.3 Icing Load

An icing situation may be defined as the situation where the humidity of air is 100 % and the temperature is between 0 ° and –10 ° centigrade. Severe icing problem takes place when the air contains small droplets of super cooled water. When this droplet hits an obstacle then they immediately freeze into ice and gradually cover the obstacle. The particle collection theory, which seems to be developed on a sound physics basis, originates from Albrecht. According to this theory, the ice deposit on circular cylinder is a function of certain parameters like duration of icing, water content in the air, wind velocity, size of water droplets and diameter of the cylinder. The amount of ice due to a given situation is calculated considering the water content (based on temperature and wind direction) and the wind velocity. The theoretical amount of ice thus calculated, can be used directly to establish the desired distribution of yearly maximum ice load. However, the line route examination will still be a necessary part of the procedure in estimating ice loads. Using various other methods based on the data of yearly maximum mass of ice at ice measuring load stands and using statistical means like distribution function for duration, wind velocity water content and probability for a specified value not to be exceeded etc. the ice loads may be estimated rationally.

The variation of load due to the ice on conductor may be from 12 N/m to as high as 250 N/m depending on the parameters like duration of icing, wind velocity, diameter of wire etc. Cases have been reported on the failures of electrical transmission lines caused by the formation of large glaze-type (ice) deposits on conductors / towers and the simultaneous or subsequent action of wind pressure. Hence when icing increases with hazardous wind velocities then to protect the line from failure, removal of the deposited ice is done by heating the conductors. A specific note of IS: 802 [14] indicates that the ice loading have to be handled as follows: " Ice loading on towers and conductors/ground wires for lines located in the mountainous regions of the country subjected to snow fall, may be taken into account on the basis of available meteorological data both for ice with wind and without wind."

5.8.4 Conductor Galloping Loads

A galloping conductor is a phenomenon usually caused by a relatively light wind of about 48 to 56 km/hr blowing on an iced conductor. The ice may be in the form of either rime or glaze. Galloping was the result of a glaze formation which, with the conductor, has an airfoil cross section capable of causing a certain type of aerodynamic instability. An airfoil cross section may easily be formed at a freezing-thawing temperature of about 0 ° C (32 ° F) when the sun is shining and a light wind is blowing. The sun will melt a portion of the radial ice, the water will run down and around the conductor, and the water will be blown back by the light wind into the shade at the bottom of the conductor where it will again freeze. As this process continues, the airfoil takes shape and the possibility of galloping conductors becomes very real. The path of conductor oscillation approximates a loop of elliptical shape. The major axis of the ellipse is slightly larger than the resultant conductor sag under the loading condition described above, and is inclined from the vertical in a direction opposite the direction of the conductor side swing by an angle equal to the angle of side swing. In level spans, the highest point of the full ellipse is only a small distance above the normal level of the points of attachment of the conductor to the insulator string. As long as the ellipses for all conductors and overhead ground wires of a transmission line do not overlap, galloping will not cause the conductors to come in contact with each other or with the overhead ground wires. Contact should result in outages and possible damage to the conductors. Observations of lines with long spans and heavy conductors indicate that the conductors may dance in two loops in which the magnitude of oscillations, or size of ellipse, is approximately half size; that is, the major axis of the ellipse is approximately one-half of the total conductor sag in the span. Application of this method to several existing lines, in regions subject to sleet conditions, has shown that those lines with sufficient spacing between conductors and between conductors an overhead ground wires to prevent overlap of the ellipses, had no outages under the sleet conditions. The lines that showed an overlap of the ellipses had a record of many outages under sleet conditions. There have also been observations where iced conductors have galloped in a manner similar to a skip rope being turned, with the midpoint rising as far above the points of support as it hangs below those points when at rest. This sort of galloping is rare, fortunately, because clearance limits are indicated which a designer cannot economically be expected to meet.

There in no definite length of span where galloping will change from full-sag ellipses to half-sag ellipses. However, experiences indicate that for line locations and conditions, we should use full-sag ellipses in spans up to 180 m in length. In longer spans, the conductors are likely to gallop in two or more loops so one-half size ellipses, with the major axis equal to 53 % of the sag, shall be used. If these ellipses do not overlap, the probability of contact between conductors and overhead ground wires, as a result of galloping, is greatly reduced.

There are three types of effects due to galloping on transmission lines:

(1) Reduction of the electrical safety due to reduced conductor separations causing flashovers.

(2) Reduction of the mechanical safety due to thermal damage and mechanical wear

(3) Additional dynamic loads on structural members of the transmission line.

The dynamic processes of conductor galloping induce, in addition to the design loads, dynamic stresses on the components of the transmission line. Suspension strings have been observed to be shaken back and forth most violently by galloping conductors, considerably increasing the stresses on conductors, clamps, insulators and towers. So far this has led to total failure in only a few cases. It does, however, mean that additional loads are acting continuously on these components, which may lead to fatigue failures of conductor strands at the suspension clamps. At these points the conductors are already severely stressed by Aeolian vibrations, tension, bending and compression. It is known that galloping of sub-conductors can produce signs of wear and tear on conductors and damage to spacers. The effects of conductor galloping accumulate during the life span of a component, causing loss of its mechanical properties.

5.8.5 Short Circuit Forces

The phase conductors are exposed to electromagnetic forces, which cause them to repel one another resulting in conductor movement and tensile forces in the conductor. The resulting tensile forces in the conductor lead to change in stresses in insulators and tower members. Electrical testing on certain line elements like 16mm diameter pin when exposed to current test of 90 A/sq. mm., for 1 second, the pin fractures for 3 kN force whereas its expected minimum load carrying capacity is 4.5 kN in tension. Tensile tests on certain elements after current flow with 100 A/sq.mm. , for 1 second showed a permanent reduction in breaking load from about 450 to 500 N/sq.mm, probably due to brittleness caused by zinc molecules moving in to the steel. Further the zinc layers of hot-dip galvanized steel wires can be damaged at a temperature of approximately 400 ° C (due to electrical short circuit) resulting in a risk of corrosion damage. (The melting point of zinc is 419 ° C) Mechanical short circuit test on conductor spans indicated that the highest change in stress was obtained in the cross-arm, 170 N/sq mm and conductor movement resulted in phase clashing for 40 kA, for 0.1sec. The additional factor for consideration is heating of phase conductor and earth wire, which generally results in increasing sag due to increasing temperature. For example, for a 330 m normal span of 593 / 68 sq.mm. ACSR conductor, the sag increases from 10.70m to 15.00m when the conductor temperature rises from +50 ° C to 200 ° C. The necessary short-circuit current is approximately 50kA for 1 second.

For the horizontal conductor configuration, conductor movement is directed outwards, involving a risk of flashover to earth, another line or another phase when the conductors swing back again. Even personnel risk can rise during live-line working. For bundled conductors, the fault current can give rise to attraction and possibly to clashing which can result in short lived tensile forces in the conductors.

5.8.6 Stringing Forces on Tower

Stringing load on towers is considerable in case of large differences in height between positions of towers or where there are large variations in span length. The phenomena shown in the following sketches can occur when stringing the conductors over rollers.

In the design situation of figure.5.7, the suspension chains are vertical, i.e. in all spans of the section the same horizontal stress exists in the conductor. During line stringing when the conductors are still on rollers figure.5.8, the suspension chains are not vertical, but are inclined towards the mountain. In the lower lying spans the sag is greater, a conductor stress lower than designed; in the higher spans on the other hand, the sag is smaller and the stress is larger than designed. If the sag is adjusted in one of the lower lying spans by pulling the conductor until the design sag is reached, the total conductor length over the section will be too short. When finally by trial and error the suspension chains are made to assume vertical position, the same horizontal stress exists in all spans built this stress is too large and there is a danger of vibrations. If, on the contrary, the sag in one of the higher laying spans is adjusted to the design value, then the total conductor length is too long, the stress too small, the final sag too large and the planned height above ground is no longer obtained.

The suspension insulator strings should be positioned vertically by repeated repositioning of the clamps. If this, however, is not done with sufficient patience and care, one chain or another will still not hang verticality. Therefore, at least part of the above-described danger of vibration for the higher laying spans will still remain. Apart from vibrations, the conductor stress caused by non-vertical chains in the higher laying spans can be so high that wind and ice conditions which are taken into account the design, let alone a natural catastrophe, can cause the permissible conductor stresses to be far exceeded. A conductor break or tower collapse could then occur.

For stringing the conductors over extremely large spans, special capstan winches with a high tractive power and pulling wires of high mechanical strength have to be employed.

Figure 5.9 shows the point of application of loads for self-supporting structures with a single circuit and double circuit whereas figures 5.10 show for chainette towers.

6.1 General

Today, many software are programmed to include the finite element method. The use of ANSYS_5.4 and ANSYS_7.1 is done for the design problem chosen here. ANSYS can analyze problems of many fields such as structures, thermal analysis, electromagnetic field analysis, coupled-field analysis, etc. The software includes the h-method as well as p-method. The problems can be analyzed for many conditions like static, transient, modal analysis, spectrum analysis, buckling analysis, harmonic response, contact, nonlinear etc [3].

6.2 Procedure of Nonlinear Static Analysis

The procedure for doing a nonlinear static analysis consists of following tasks:

- Building the Model
- Setting Solution Controls
- Setting Additional Solution Options
- Applying the Loads
- Solving the Analysis
- Reviewing the Results

6.3 Building the Model

This step is essentially for both linear and nonlinear analysis, although a nonlinear analysis might include special elements or nonlinear material properties. If the analysis includes large-strain effects, the stress-strain data must be expressed in terms of true stress and true (or logarithmic) strain. After creating a model in ANSYS, set solution controls (analysis type, analysis options, load step options, and so on), apply loads, and solve. A nonlinear solution will differ from a linear solution in that it often requires multiple load increments, and always requires equilibrium iterations. The general procedure for performing these tasks is as follows.

6.4 Setting Solution Controls

Setting solution controls for a nonlinear analysis involves the same options and method of access (the Solution Controls dialog box) as those used for a linear structural static analysis.

- Set Solution Controls
- Access the Solution Controls Dialog Box
- Using the Basic Tab
- The Transient Tab
- Using the Solution Options Tab
- Using the Nonlinear Tab
- Using the Advanced NL Tab

6.4.1 Using the Basic Tab: Special Considerations

Special considerations for setting these options in a nonlinear structural static analysis include the following:

- When setting analysis type and geometric non-linearlity, choose Large Displacement Static, on performing a new analysis.
- When working with time settings, one should remember that these options can be changed at any load step.

A nonlinear analysis requires multiple sub steps (or time steps; the two terms are equivalent) within each load step so that ANSYS can apply the specified loads gradually and obtain an accurate solution. The NSUBST and DELTIM commands both achieve the same effect (establishing a load step's starting, minimum, and maximum step size). NSUBST defines the number of substeps to be taken within a load step, whereas DELTIM defines the time step size explicitly. If automatic time stepping is off [AUTOTS], the starting substep size is used throughout the load step.

6.4.2 Equation Solver

ANSYS' automatic solution control activates the sparse direct solver (EQSLV, SPARSE) for most cases. The sparse direct solver is the default; except for the generation pass of a substructure analysis (which uses the frontal direct solver). Other choices include the frontal direct and PCG solvers.

The sparse direct solver, in sharp contrast to the iterative solvers included in ANSYS, is a strong solver. Although the PCG solver can solve indefinite matrix equations, when the PCG solver encounters an ill-conditioned matrix, the solver will iterate to the specified number of iterations and stop if it fails to converge. When this happens, it triggers bisection. After completing the bisection, the solver continues the solution if the resulting matrix is well conditioned. Eventually, the entire nonlinear load step can be solved.

Use the following guidelines for selecting either the sparse or the PCG solver for nonlinear structural analysis:

- If it is a beam/shell or beam/shell and solid structure, choose the sparse direct solver.
- If it is a 3-D solid structure and the number of DOF is relatively large (that is, 200,000 or more DOF), choose the PCG solver.
- If the problem is ill conditioned (triggered by poor element shapes), or has a big difference in material properties in different regions of the model, or has insufficient displacement boundary constraints, choose the sparse direct solver.

6.4.3 Automatic Time Stepping

ANSYS' automatic solution control turns automatic time stepping on [AUTOTS, ON]. An internal auto-time step scheme ensures that the time step variation is neither too aggressive (resulting in many bisection/cutbacks) nor too conservative (time step size is too small). At the end of a time step, the size of the next time step is predicted based on four factors:

- Number of equilibrium iterations used in the last time step (more iterations cause the time step size to be reduced)
- Predictions for nonlinear element status change (time step sizes are decreased when a status change is imminent)
- Size of the plastic strain increment
- Size of the creep strain increment

6.4.4 Convergence Criteria

The program will continue to do equilibrium iterations until the convergence criteria are satisfied (or until the maximum number of equilibrium equations is reached). One can define his own criteria if the default settings are not suitable.

ANSYS' automatic solution control uses L2-norm of force (and moment) tolerance equal to 0.5%, a setting that is appropriate for most cases. In most cases, an L2-norm check on displacement with tolerance equal to 5% is also used in addition to the force norm check. The check that the displacements are loosely set serves as a double-check on convergence.

By default, the program will check for force (and, when rotational degrees of freedom are active, moment) convergence by comparing the square root sum of the squares (SRSS) of the force imbalances against the product of VALUE x tolerance. The default value of VALUE is the SRSS of the applied loads (or, for applied displacements, of the Newton-Raphson restoring forces), or MINREF (which defaults to 0.001), whichever is greater. The default value of tolerance is 0.005. If, SOLCONTROL is off then the value of tolerance is default value, which is 0.001 and MINREF defaults to 1.0 for force convergence.

One should almost always use force convergence checking. One can also add displacement (and, when applicable, rotation) convergence checking. For displacements, the program bases convergence checking on the change in deflections (Δu) between the current (i) and the previous (i-1) iterations: $\Delta u = u_i - u_{i-1}$.

6.4.5 Maximum Number of Equilibrium Iterations

ANSYS' automatic solution control sets the value of NEQIT to between 15 and 26 iterations, depending upon the physics of the problem. The idea is to employ a small time step with less quadratically converging iteration.

This option limits the maximum number of equilibrium iterations to be performed at each sub step (default = 25 if solution control is off). If the convergence criteria have not been satisfied within this number of equilibrium iterations, and if auto time stepping is on [AUTOTS], the analysis will attempt to bisect. If bisection is not possible, the analysis will either terminate or move on to the next load step, according to the instructions you issue in the NCNV command.

6.5 Setting Additional Solution Options

This section discusses additional options that one can set for the solution. These options do not appear on the Solution Controls dialog box because they are used infrequently, and their default settings rarely need to be changed. ANSYS menu paths are provided in this section to help one access these options for those cases in which one chooses to override the ANSYS-assigned defaults.

6.5.1 Stress Stiffening

To account for buckling, bifurcation behavior, ANSYS includes stress stiffness in all geometrically nonlinear analyses. If you are confident of ignoring such effects, you can turn stress stiffening off (SSTIF, OFF). This command has no effect when used with several ANSYS elements.

6.5.2 Newton-Raphson Option

ANSYS' automatic solution control will use the FULL Newton-Raphson option with adaptive descent off if there is a non-linearity present. However, when node-to-node, node-to-surface contact elements are used for contact analysis with friction, and then adaptive descent is automatically turned on.

This option is only to be used in a nonlinear analysis. This option specifies how often the tangent matrix is updated during solution. If you choose to override the default, you can specify one of these values:

- Program-chosen (NROPT, AUTO): The program chooses which of the options to use, based on the kinds of nonlinearities present in your model. Adaptive descent will be automatically activated, when appropriate.

- Full (NROPT, FULL): The program uses the full Newton-Raphson procedure, in which the stiffness matrix is updated at each equilibrium iteration. If adaptive descent is on (optional), the program will use the tangent stiffness matrix only as long as the iterations remain stable (that is, as long as the residual decreases, and no negative main diagonal pivot occurs). If divergent trends are detected in iteration, the program discards the divergent iteration and restarts the solution, using a weighted combination of the secant and tangent stiffness matrices. When the iterations return to a convergent pattern, the program will resume using the tangent stiffness matrix. Activating adaptive descent will usually enhance the program's ability to obtain converged solutions for complicated nonlinear problems.

- Modified (NROPT, MODI): The program uses the modified Newton-Raphson technique, in which the tangent stiffness matrix is updated at each sub step. The matrix is not changed during equilibrium iterations at a sub step. This option is not applicable to large deformation analyses. Adaptive descent is not available.

- Initial Stiffness (NROPT, INIT): The program uses the initial stiffness matrix in every equilibrium iteration. This option can be less likely to diverge than the full option, but it often requires more iterations to achieve convergence. It is not applicable to large deformation analyses. Adaptive descent is not available.

6.6 Applying the Loads

Apply loads on the model. One should remember that inertia and point loads will maintain constant direction, but surface loads will "follow" the structure in a large-deformation analysis.

6.7 Solving the Analysis

You solve a nonlinear analysis using the same commands and procedure as you do in solving a linear static analysis. If you need to define multiple load steps, you must respecify time settings, load step options, and so on, and then save and solve for each of the additional load steps.

6.8 Reviewing the Results

Results from a nonlinear static analysis consist mainly of displacements, stresses, strains, and reaction forces. You can review these results in POST1, the general postprocessor, or in POST26, the time-history postprocessor.

Remember that in POST1, only one sub step can be read in at a time, and that the results from that sub step should have been written to Jobname.RST.

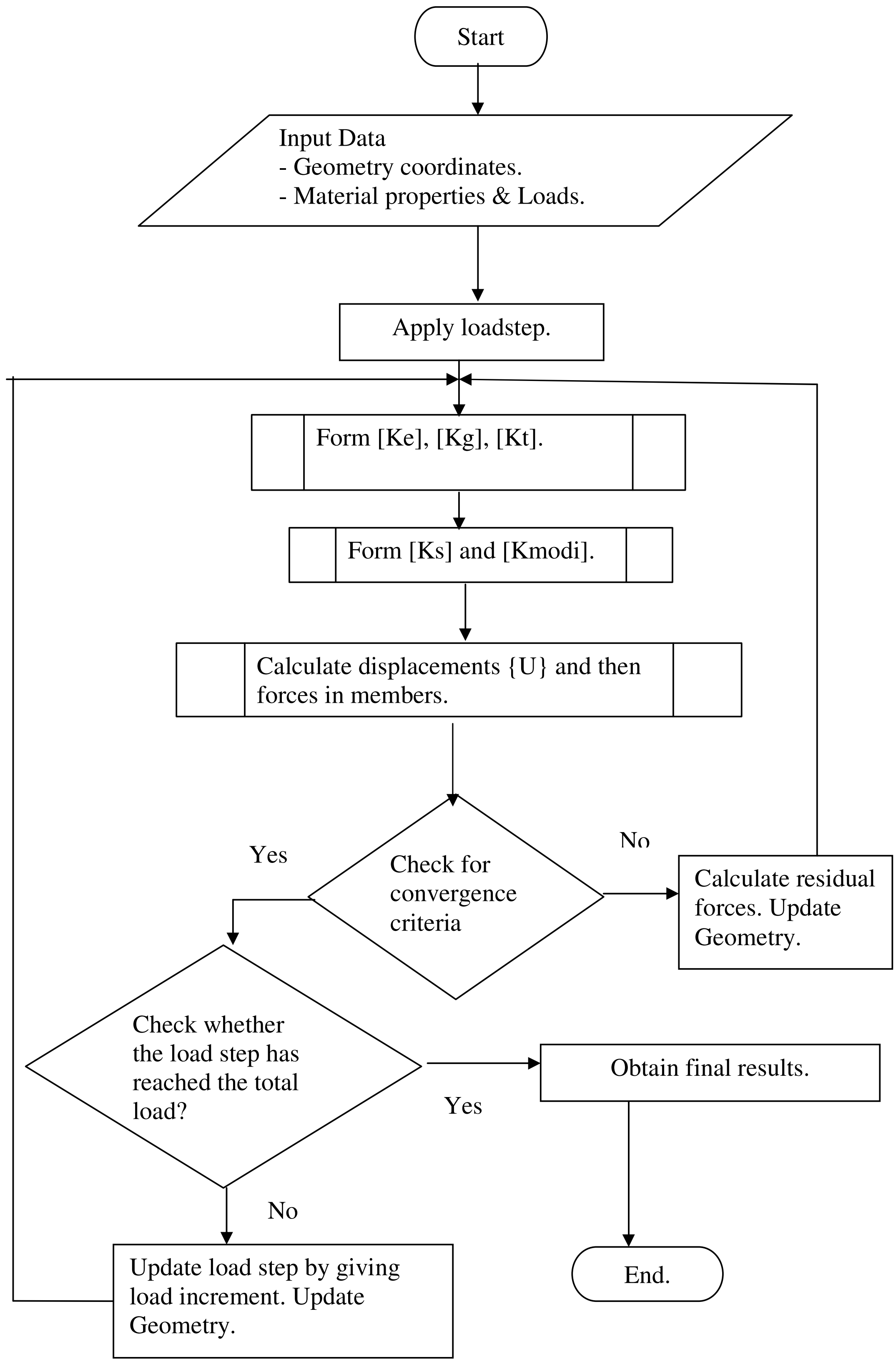
CHAPTER – 7 COMPUTER PROGRAM FOR ANALYSIS

7.1 General

A computer program is developed using Visual C++ for the approximate analysis of the chainette type of transmission line tower. The program is meant to do static analysis only. The program is not finite element based. The results have been checked by doing manual calculation using Microsoft Excel spreadsheets. The program is a single document interface (SDI) program [16]. Microsoft Foundation Classes (MFC) are used to enhance the facilities provided by the program. The use of property sheet and dialog boxes are used to feed the data required for the analysis.

7.2 Flow chart

Figure 7.1 Flow chart diagram explaining the program algorithm.



7.3 Steps involved in programming

- The data of the problem is inputted through three sheets, which are,
  - –Geometry i.e. Coordinates of the nodes.
  - –Loads.
  - –Material properties.

- With the use of the initial geometry the elastic stiffness matrices of each member are calculated.

$$[Ke] = AE/L \left[\begin{array}{ccc|c} l^2 & lm & ln & \\ lm & m^2 & mn & (-) \\ ln & mn & m^2 & \\ \hline & & & \\ & & & \end{array}\right]$$

A = C/S area of members.

E = modulus of elasticity.

L = length of member.

l,m, n = direction cosines in the global X, Y and Z directions.

- For the first iteration the geometric stiffness matrices of each member is formed with the help of initial length and the prestressing force in the cables.

$$[Kg] = F / L \left[\begin{array}{ccc|c} (1-l^2) & -lm & -ln & \\ -lm & (1-m^2) & -mn & (-) \\ -ln & -mn & (1-n^2) & \\ \hline & & & \\ & & & \end{array}\right]$$

where F = Force in member. (For first iteration force is zero for mast, hence this matrix is not formed for the mast members for the first iteration and for the cable elements it is the prestressing force.)

- The initial tangent stiffness matrices of all members are formed by adding the above two matrices.

$[Kt] = [Ke] + [Kg]$ where $[Kt]$ = tangent stiffness matrix.

- The structural stiffness matrix is formed by assembling the member tangent stiffness matrices.

$[Ks] = \sum [Kt]$ where $[Ks]$ = assembled structure stiffness matrix.

- The boundary conditions are applied and the modified structure stiffness matrix, [Kmodi] is formed.

- The resulting matrix is inverted and multiplied by the load vector to get the first set of displacements.

$\{U\} = [Kmodi]^{-1} \{P\}$ where $\{U\}$ = displacement vector
$\{P\}$ = load vector

- The member forces are found corresponding to these displacements.

$F = AE\{[DL - PL]/OL + e\}$

Where DL = Displaced length of the member after the particular iteration.
PL = Previous length of the member before the particular iteration.
OL = Original length of the member.
e = Initial strain if any.(For the cable this strain is due to the pretension)

- These member forces are used to calculate the geometric stiffness matrices of the members for the next iteration. The lengths and direction cosines used in calculation of these matrices is with respect to the deformed geometry.

- Now the next solution is obtained for the next load-step.

- Care has to be taken to remove the element stiffness matrix of the spacer cable element, which goes slack when the elastic strains become compressive.

- The complete solution is obtained when all the load-steps are done.

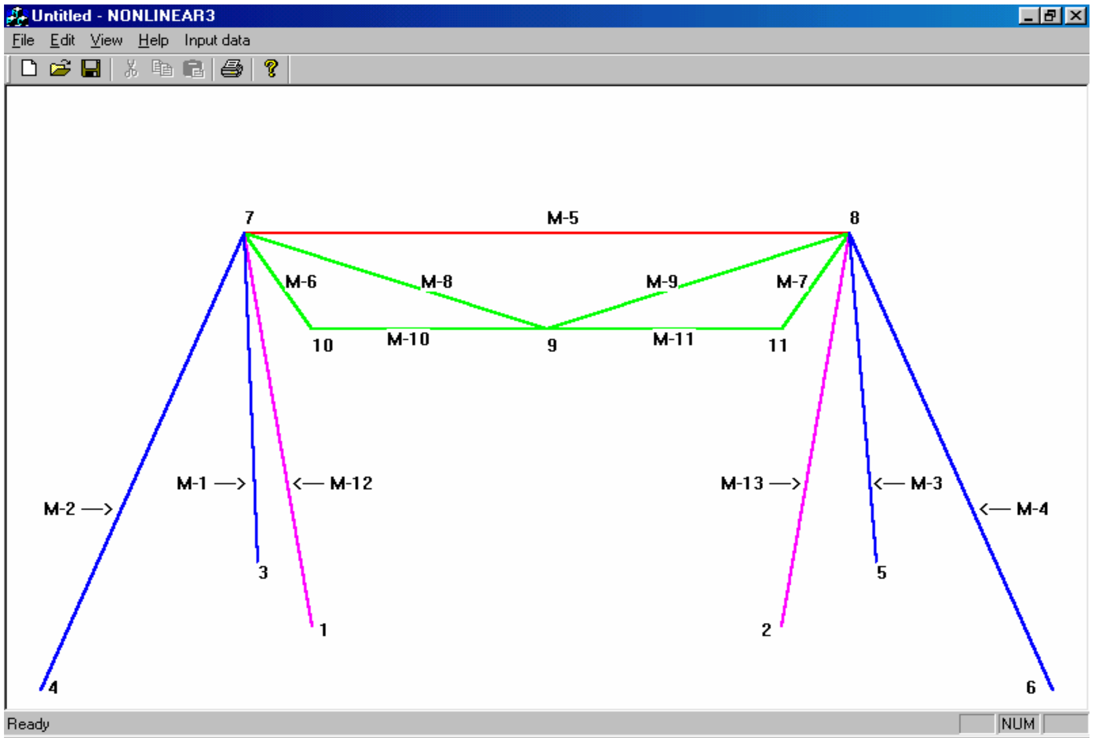
7.4 Variables used in computer program

The variables used in programming are listed in the appendix I.

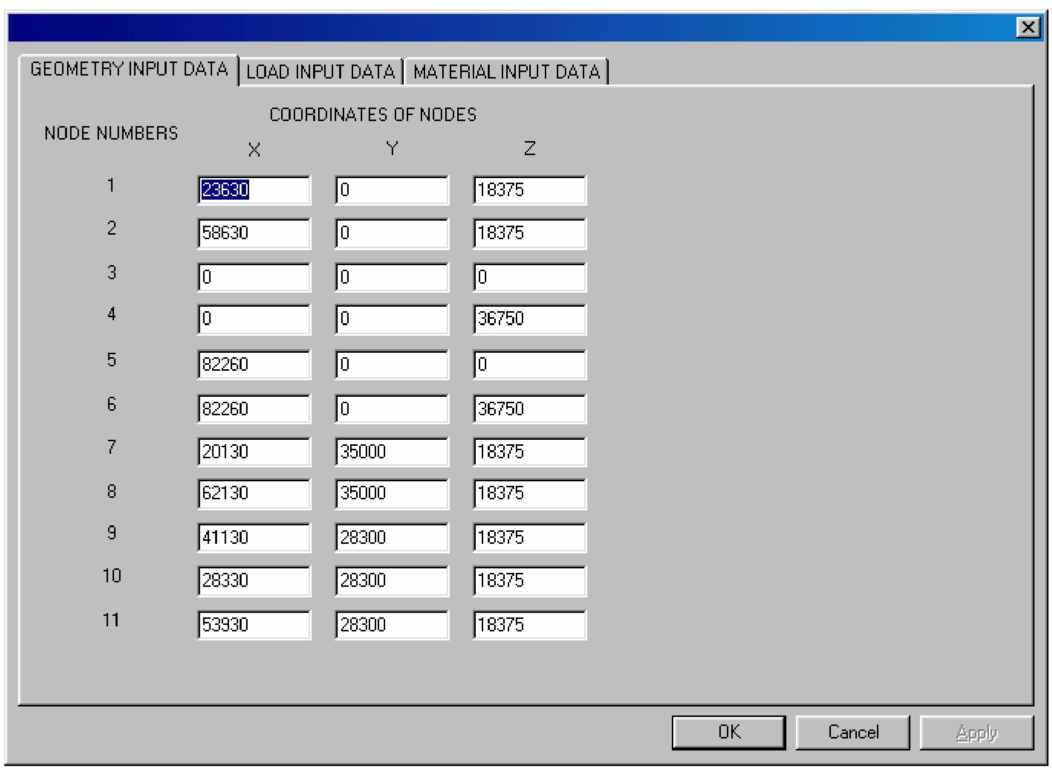
7.5 Features of the program

In the figure 7.2 (a), the initial geometry of the structure is shown. The node numbers are to be given as per the figure. The origin is to be considered at the node 3. This is done so that, all the coordinates entered are positive values. As per the figure 7.2 (b), the input data sheet appears on clicking on the input data menu in the mainframe. This sheet consists of three sub-sheets, which are geometry input data, load input data and the material property input data. On clicking the button 'OK' of this sheet the program is completely executed and the results are written to the output file. The output gives the values of displacements and forces in members for each loadstep. The program has some limitation in itself. The program uses ten loadsteps to do the solution. At the end of each loadstep, it assumes that the force equilibrium is obtained and does not do the intra loadstep iteration to satisfy the convergence criteria, which is done in the general finite element packages.

Figure 7.2 Working Environment of the program



(a) Mainframe of the program.



NODE NUMBERS	COORDINATES OF NODES X	Y	Z
1	23630	0	18375
2	58630	0	18375
3	0	0	0
4	0	0	36750
5	82260	0	0
6	82260	0	36750
7	20130	35000	18375
8	62130	35000	18375
9	41130	28300	18375
10	28330	28300	18375
11	53930	28300	18375

(b) Input data sheet of the program.

CHAPTER – 8 RESULTS AND DISCUSSIONS

8.1 General

The general purpose finite element program ANSYS_7.1 version is used to analyze the chainette tower problem. The structure is loaded with the vertical load of ice and with horizontal load of wind. The vertical loads at the top of the mast are 10000 N and at the insulator junction is 54000 N. In addition to this vertical loads the horizontal wind loads acting at the top of the mast is 11000 N and at the insulator junction is 20000 N [11]. The geometric nonlinear analysis is done for the above two load cases. The following output results are obtained and compared with the published results [11].

As a nonlinear structural analysis proceeds, ANSYS computes convergence norms with corresponding convergence criteria for each equilibrium iteration. The Graphical Solution Tracking (GST) feature displays the computed convergence norms and criteria while the solution is in process.

8.2 Non-linear analysis

As discussed in the chapter 6, the pre-processing involves:

- The formation of the geometry of the problem by giving the coordinates of the nodes. The elements are then connected to proper nodes. The node numbers and the elements are given as per figure 8.1.
- The members of the structure are modeled with specific type of element from the element library of ANSYS. The mast member is modeled using LINK8 type of element. The cable members are modeled as LINK10 – TENSION ONLY. The

properties of the members (elements) like modulus of elasticity, cross-sectional area and initial strain, if applicable, are given.

- The loads are defined along with the end conditions of each element.

Under the solution menu the following options are used:

- The type of analysis used is STATIC.
- The analysis options used are stress stiffness, large deformations and Newton-Raphson
- The loads are applied on the structure. The number of load steps is given as 10.
- In the non-linear solution option the convergence criteria are kept as default and the number of equilibrium iterations given is 25.
- The solution is done for the current loads.

The output of the analysis is discussed in the next section.

8.3 Output for the vertical load (ANSYS)

The solution summary is displayed under the menu of post-processor (POST 1) when an analysis is done. This summary sheet is shown below.

8.3.1 SOLUTION OPTIONS

PROBLEM DIMENISONALITY ....3 – D

DEGREES OF FREEDOM..UX UY UZ

ANALYSIS TYPE ....STATIC (STEADY-STATE)

NONLINEAR GEOMETRIC EFFECTS..........................ON

PRESTRESS EFFECTS CALCULATED........................YES

NEWTON-RAPHSON OPTION.................................PROGRAM CHOSEN

LOAD STEP OPTIONS

LOAD STEP NUMBER………………………………….....…1

TIME AT END OF THE LOAD STEP…………………………1.0000

NUMBER OF SUBSTEPS……………………………………....10

MAXIMUM NUMBER OF EQUILIBRIUM ITERATIONS………25

STEP CHANGE BOUNDARY CONDITIONS…………………NO

TERMINATE ANALYSIS IF NOT CONVERGED……………..YES (EXIT)

CONVERGENCE CONTROLS…………………………...…USE DEFAULTS

PRINT OUTPUT CONTROLS………………………………NO PRINTOUT

DATABASE OUTPUT CONTROLS………………………..ALL DATA WRITTEN FOR THE LAST SUBSTEP

The solution options summary gives the problem dimensionality (whether it is 2D or 3D), degrees of freedom used in the problem (whether it is translational or rotational degrees of freedom in the global x, y and z directions), analysis type (whether it is static or dynamic), non-linear geometric effects (whether the geometric non-linearity is included or not), prestressing effects (whether the initial strain due to the initial prestress is included or not) and the solution technique (Newton Raphson or modified Newton Raphson or other technique is used).

The load step option gives the idea of the number of load steps given, time at the end of each load step, number of substeps in a load step, maximum number of iterations within a substep for the solution to converge, whether step change boundary conditions are applied or not, convergence criteria defined, printing facilities of the results. The results of each converged solution for each load step can be reviewed by using the POST 1 command for the general postprocessor.

Figure 8.2 Graphical Solution Tracking displaying the convergence

Figure 8.2 shows the graphical solution tracking (GST) displaying the convergence of the problem. Absolute convergence norms are scaled on the Y-axis and the cumulative iteration numbers are scaled on the X-axis. The effect of non-converged solution is reflected in the GST when a problem fails to converge. Hence it becomes helpful to decide the convergence norms or criteria to obtain a solution.

Figure 8.2 Deformed and undeformed shape of the tower for vertical loads.

8.3.2 Nodal displacements

The displacements at the end of each load step can be viewed within the POST 1 , general postprocessor. The displacement results of each node of the last load step or substep is presented here. The unit of the displacement is in the terms of millimeter. UX, UY, UZ are the displacements with respect to the global coordinate axis.

PRINT DOF NODAL SOLUTION PER NODE

***** POST1 NODAL DEGREE OF FREEDOM LISTING *****

LOAD STEP= 1 SUBSTEP= 10
TIME= .10000E-01 LOAD CASE= 0

THE FOLLOWING DEGREE OF FREEDOM RESULTS ARE IN GLOBAL COORDINATES

NODE	UX	UY	UZ
1	.00000	.00000	.00000
2	.00000	.00000	.00000
3	.00000	.00000	.00000
4	.00000	.00000	.00000
5	.00000	.00000	.00000
6	.00000	.00000	.00000
7	485.33	42.872	.00000
8	-485.33	42.872	.00000
9	.50156E-11	-1413.3	.00000
10	9.4311	-558.70	.00000
11	-9.4311	-558.70	.00000

MAXIMUM ABSOLUTE VALUES

NODE	8	9	0
VALUE	-485.33	-1413.3	.00000

8.3.3 Nodal forces of each element

The forces at each node in the global coordinates, of each element, can be reviewed. FX, FY, FZ are the forces with respect to the global coordinate system. The element number and the nodes between which the element is connected are also displayed.

```
 PRINT FORC ELEMENT SOLUTION PER ELEMENT

 ***** POST1 ELEMENT NODE TOTAL FORCE LISTING *****

  LOAD STEP=     1  SUBSTEP=    10
   TIME=    .10000E-01  LOAD CASE=   0

  THE FOLLOWING X,Y,Z FORCES ARE IN GLOBAL COORDINATES

  ELEMENT=     1      FX         FY         FZ
               3    48044.      81534.      42777.
               7   -48044.     -81534.     -42777.

  ELEM=     2  FX         FY         FZ
             4   48044.      81534.     -42777.
             7  -48044.     -81534.      42777.

  ELEM=     3  FX         FY         FZ
              5  -48044.      81534.      42777.
              8   48044.     -81534.     -42777.

  ELEM=     4  FX         FY         FZ
               6  -48044.      81534.     -42777.
               8   48044.     -81534.      42777.
```

```
ELEM=    5  FX        FY        FZ
         7   .00000    .00000    .00000
         8   .00000    .00000    .00000

ELEM=    6  FX        FY        FZ
         7   61293.   -58261.    .00000
        10  -61293.    58261.    .00000

ELEM=    7  FX        FY        FZ
         8  -61293.   -58261.    .00000
        11   61293.    58261.    .00000

ELEM=    8  FX        FY        FZ
         7   56547.   -22644.    .00000
         9  -56547.    22644.    .00000

ELEM=    9  FX        FY        FZ
         8  -56547.   -22644.    .00000
         9   56547.    22644.    .00000

ELEM=   10  FX        FY        FZ
        10   61262.   -4231.0    .00000
         9  -61262.    4231.0    .00000

ELEM=   11  FX        FY        FZ
        11  -61262.   -4231.0    .00000
         9   61262.    4231.0    .00000
ELEM=   12  FX        FY        FZ
         1   21705.   -.25405E+06  .00000
         7  -21705.    .25405E+06  .00000
```

ELEM= 13 FX FY FZ
2 -21705. -.25405E+06 .00000
8 21705. .25405E+06 .00000

8.3.4 Element forces

The forces in individual members can be reviewed in the POST 1, post processor. These forces help in designing the members later on. The MFORX indicates the value of the force. The unit here, in this case, is Newtons. Care should be taken on defining the units during the preprocessor. The positive forces indicate that the forces are tensile forces and the negative forces indicate that they are compressive forces.

PRINT ELEM ELEMENT SOLUTION PER ELEMENT

***** POST1 ELEMENT SOLUTION LISTING *****

LOAD STEP 1 SUBSTEP= 10
TIME= .10000E-01 LOAD CASE= 0

EL= 1 NODES= 3 7 MAT= 2 STAT=1 CABLE LINK10
MFORX= .10385E+06

EL= 2 NODES= 4 7 MAT= 2 STAT=1 CABLE LINK10
MFORX= .10385E+06

EL= 3 NODES= 5 8 MAT= 2 STAT=1 CABLE LINK10

MFORX= .10385E+06

EL= 4 NODES= 6 8 MAT= 2 STAT=1 CABLE
LINK10
MFORX= .10385E+06

EL= 5 NODES= 7 8 MAT= 3 STAT=2 CABLE
LINK10
MFORX= .00000

EL= 6 NODES= 7 10 MAT= 2 STAT=1 CABLE
LINK10
MFORX= 84565.

EL= 7 NODES= 8 11 MAT= 2 STAT=1 CABLE
LINK10
MFORX= 84565.

EL= 8 NODES= 7 9 MAT= 2 STAT=1 CABLE
LINK10
MFORX= 60913.

EL= 9 NODES= 8 9 MAT= 2 STAT=1 CABLE
LINK10
MFORX= 60913.

EL= 10 NODES= 10 9 MAT= 2 STAT=1 CABLE
LINK10
MFORX= 61408.

EL= 11 NODES= 11 9 MAT= 2 STAT=1 CABLE
LINK10
MFORX= 61408.

EL= 12 NODES= 1 7MAT= 1
LINK8
MFORX= -.25497E+06

EL= 13 NODES= 2 8 MAT= 1
LINK8
MFORX= -.25497E+06

8.4 Output for the vertical and horizontal loads (ANSYS)

The horizontal loads along with vertical loads are applied and the same procedure for analysis is done. The same options discussed in the previous section are used for this analysis also. The results are discussed below.

8.4.2 Nodal displacements

PRINT DOF NODAL SOLUTION PER NODE

***** POST1 NODAL DEGREE OF FREEDOM LISTING *****

LOAD STEP= 1 SUBSTEP= 10
TIME= .10000E-01 LOAD CASE= 0

THE FOLLOWING DEGREE OF FREEDOM RESULTS ARE IN GLOBAL COORDINATES

NODE	UX	UY	UZ
1	.00000	.00000	.00000

2	.00000	.00000	.00000
3	.00000	.00000	.00000
4	.00000	.00000	.00000
5	.00000	.00000	.00000
6	.00000	.00000	.00000
7	813.10	68.890	.00000
8	-166.18	14.581	.00000
9	385.97	-1427.7	.00000
10	429.94	-433.25	.00000
11	407.65	-688.55	.00000

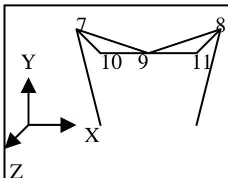
MAXIMUM ABSOLUTE VALUES

NODE	7	9	0
VALUE	813.10	-1427.7	.00000

8.5 Discussion on results obtained after comparison

The results obtained from the above sections are compared with the results obtained from Hydro-Quebec [11]. The displacements are in the units of ‘millimeter’

Table 8.1 Comparison of displacements (mm)

	Displacement in (mm)			
	PUBLISHED RESULTS		ANSYS RESULTS	
	Vertical	Vert + Hori	Vertical	Vert + Hori
Direction, Node				
Transverse, X 7	510	760	485.33	813.10
X 8	-510	-170	-485.33	-166.18
Vertical, Y 9	-1580	-1510	-1413.3	-1427.7
Y 10	-660	-540	-558.70	-433.25
Y 11	-660	-790	-558.70	-688.55

- From the above comparison it is clear that the results obtained from ANSYS are close to the actual values

- As per A.S.C.E. 91 [1], the Rated Breaking Strength of 25.4 mm diameter cable is 464.8 kN. The maximum forces allowed is about 65% of R.B.S. which turns out to be 302.12kN. According to the results obtained the maximum force is 103.85 kN in the cables, which is under the limiting value.

- As a cross check, the value of the axial force in the spacer cable (element EL = 5) is found to be 0, which according to the theory is approved, because the spacer cable becomes slack after the loads are acting and doesnot contribute to the stiffness of the structure.

8.6 Comparison of linear static and non-linear static analysis

It is said that a linear analysis should always preceed before a non-linear analysis. On performing the linear and non-linear static analysis of the problem using ANSYS, the results obtained are tabulated as follows.

Table 8.2 Comparison of deflections (mm.) of linear and non-linear static analysis

NODES	Ux (mm)			Uy (mm)		
	Linear	Non-linear	% age difference	Linear	Non-linear	% age difference
7	750	485.33	35.28	48.28	42.87	11.20
8	-750	-485.33	35.28	48.28	42.87	11.20
9	0	0	0	-2605.5	-1413	45.75
10	24.77	9.43	61.93	-964.13	-558.7	42.05
11	-24.77	-9.43	61.93	-964.13	-558.7	42.05

In the above table Ux and Uy are the deflections (mm.) in the global x and y direction respectively.

Table 8.3 Comparison of forces (kN) of linear and non-linear static analysis

MEMBERS	MAXIMUM FORCE (kN)		
	Linear	Non-Linear	% age difference
1	132.93	104.96	21.04
2	132.93	104.96	21.04
3	132.93	104.96	21.04
4	132.93	104.96	21.04
5	0	0	0
6	85.35	84.57	0.904
7	85.35	84.57	0.904
8	88.83	61.94	30.27
9	88.83	61.94	30.27
10	66.09	61.58	6.81
11	66.09	61.58	6.81
12	-302.26	-256.82	15.03
13	-302.26	-256.82	15.03

From the above results it is clear that when displacements are high, then one should go for a non-linear analysis. The results above clearly indicate that the forces estimated from linear static analysis are high, which actually are overestimated and hence when the members are designed for these forces they prove to be uneconomical.

8.7 Comparison of analysis results from computer program and Ansys

Table 8.4 Comparison of deflection (mm) results of computer program and Ansys

LOAD STEP	DEFLECTION AT THE CRITICAL NODE (9)		
	Ansys Results	Program Results	% difference
1	-467.8	-375.58	18.9
2	-234.81	-277.28	-18.09
3	-75.23	-86.20	-14.57
4	168.77	133.14	21.1
5	397.69	351.73	11.55
6	652.18	712.48	-9.24
7	884.62	930.83	-5.22
8	1098.4	1065.41	3.00
9	1296.90	1173.13	9.55
10	1413.3	1256.25	11.09

Table 8.5 Comparison of forces, in cable guy (N), obtained from computer program and Ansys

LOAD STEP	FORCES IN CABLE GUYS (N)		
	Ansys Results	Program Results	% difference
1	37454	39326.90	-5.00
2	43438	42504.35	2.14
3	49149	47693.85	2.96
4	54447	53375.86	1.96
5	61944	59132.18	4.53
6	71143	71716.20	-0.80
7	79882	80811.42	-1.16
8	88209	88548.79	-0.38
9	96184	95034.12	1.19
10	104001	100386.04	3.33

Table 8.6 Comparison of forces, in Mast (N), obtained from computer program and Ansys

LOAD STEP	FORCES IN MAST (N)		
	Ansys Results	Program Results	% difference
1	-68586	-71155.38	-3.75
2	-87174	-85168.34	2.30
3	-1.05E+05	-102568.94	2.63
4	-1.23E+05	-120157.01	2.16
5	-1.44E+05	-137013.26	4.68
6	-1.67E+05	-159941.03	4.41
7	-1.90E+05	-176184.47	7.34
8	-2.12E+05	-190476.11	10.28
9	-2.34E+05	-202367.11	13.47
10	-2.55E+05	-219300.01	14.00

The above tables indicate that the assumption made of attaining the equilibrium condition after each loadstep creates a reasonable amount of difference with the actual analysis.

CHAPTER – 9 DESIGN GUIDELINES

9.1 Design guidelines

Before actually going for the design of the individual mast members, it is necessary to find out the axial forces in these members. For this, the mast has to be modeled with some specific configuration. The chainette tower masts are conventional masts with a staggered single bracing system. The design of the mast has to be done using non-linear P-Delta analysis [8].

9.2 Selection of material

9.2.1 Use of hot-rolled angle steel sections:

The practice is of using the hot rolled angle steel sections in the design of towers.

9.2.2 Minimum flange width:

Minimum flange widths for bolts of different diameters are given below:

BOLT DIAMETER (mm)	FLANGE WIDTH (mm)
12	40
16	45
20	50
24	60

9.2.3 Minimum thickness of members:

As per I.S. 802 [14] the following minimum thickness for members are specified:

a) Leg members	: 5mm
b) Ground wire peak and external members of Horn Peak	: 5
c) Lower members of cross-arm	: 5
d) Upper members of cross-arm	: 4mm
e) Bracings and inner members of horn peak	: 4
f) Other members	: 4

9.2.4 Grades of steel: -

Generally two grades of steel i.e., mild steel (MS) and high tensile (HT) steel are used in the manufacturing of transmission line towers.

9.3 Slenderness ratio limitations (l/r): -

As per I.S. 802 [14], the following limits of l/r ratio are prescribed: -

-	Leg members, G.W. peak.	= 120
-	Bracings	= 200
-	Redundants/Nominal stress carrying members	= 250
-	Tension members	= 400

9.4 Permissible stresses in tower members: -

Various strut formulae for working out the permissible compressive stresses are as per I.S. 802 [14]. This code suggests to 6 different curves for calculation of the permissible compressive stresses in different tower members.

Table 9.1 Utility of curves for calculation of permissible compressive stresses

CURVE NO.	WHEN TO USE
1	is used for leg-members and double-angle sections, connected back-to-back, having concentric loads at both ends and Kl/r upto 120.
2	is used for cross-arm lower members, having concentric loads at one end, eccentric load at the other ends and Kl/r upto 120.
3	is used for bracings with single angle sections having eccentricity at both ends and Kl/r upto 120.
4	is used for bracings with single-bolt connections at both the ends, thus being unrestrained against rotation at both the ends and having Kl/r from 120 to 200.
5	is used for bracings with single-bolt connections at one end and 2-bolt connections at the other end thus being partially restrained against rotation at one end only and having Kl/r from 120 to 225.
6	is used for bracings with 2-bolt connections at both the ends, thus being partially restrained against rotation at both the ends and having Kl/r from 120 to 250.

Suitable reduction in permissible stresses has to be made for limits in b/t ratio.

9.5 Design of members: -

9.5.1 Design of members in compression: -

This design should follow stipulations of curve-1 to curve-6, described above.

9.5.2 Design of members in tension: -

The estimated ultimate tensile stress in a member, should, not exceed 2550 kg/sq.cm. The slenderness ratio of member carrying axial tension should not exceed 400. The net effective areas of angle sections in tension to workout the permissible tensile load in a member shall be determined as under: -

(1) Single angle in tension connected on one flange only.

A+BK, where
A = Net sectional area of the connected flange.
B = Area of the outstanding flange.
= (L-t) t, where L = flange width and t = Thickness of the member.
K = 1/ (1+0.35*B/A)

(2) Pair of angle s back to back: connected on one flange of each angle to the same side of gusset.

A+BK
Where K = 1/ (1+0.2 *B/A)

The back to back angles are to be connected or stitched together throughout their length.

9.5.3 Redundant members: -

Redundant members carry nominal stress. They are used to restrict the slenderness ratio l/ r of the main members. Slenderness ratio of redundant member is restricted to 250. They are also required to carry 2.5 % of the stress in the main members, which are supported by these redundant members. These members, if placed at an angle less than 15 degree are required to be checked to withstand bending also, due to a mid-point concentrated load of 150 kg independent of other loads.

9.6 Connection design: -

Tower structures are usually bolted type. The ultimate stresses in bolts shall not exceed the following values shown in table 9.2

Table 9.2 Ultimate stresses in bolts

No.	Nature of Stress	Ultimate stress Kg./Sq.cm. (N/Sq.mm.) Class 4.6	Class 5.6	Remarks
1.	Shear stress on gross area of bolt	2,220 (218)	3,161 (310)	Gross area of the bolt shall be taken as the nominal area of the bolt.
2.	Bearing stress on gross diameter of bolt	4,440 (436)	6,322 (620)	Bolt area shall be taken as d*t where, d = diameter of bolt and t = thickness of the thinner member.

3.	Bearing on member			
	M.S.	4,440 (436)	4,440 (436)	
	H.T.	4,440 (436)	6,322 (620)	
4.	Tension	1980 (194)	2590 (254)	

The bolt sizes used are 12, 16, 20 and 24 mm diameter. Preferably not more than two sizes of bolts should be used in one tower. Connections are to be designed for the relevant shear and bearing stresses and the class of bolts used. There is no restriction on the number of bolts.

9.7 Foundation: -

Due to the flexibility of the cross-rope assembly, the structure is practically insensitive to foundation movements. For example, a 15 cm upward movement of one foundation will cause only a 3 percent increase in stress levels in the guys and a 5 percent increase in the masts. The tower is also insensitive to movements of foundations in transversal and longitudinal directions.

A study of the geometrical changes in the structure shows that the electrical clearances are not greatly affected by foundation movements. In the case of a simultaneous 15 cm settlement under the two foundations, the clearance of the external phase to the mast is reduced by only 3.5 percent while the insulator string is vertical and by 5 percent at an extreme swing position of 19 degrees. The clearance to ground of the central phase is reduced by only 70 cm.

Similar behavior is produced if creep in guys or slipping of anchors is experienced. Therefore, the electrical clearances and stress levels remain acceptable. It will not be necessary then to adjust the guy tensions as on most guyed towers, particularly during the first years, to remedy the effects of creep in guys or foundation movements.

Insensitivity of the tower to settlement or uplift makes it possible to use surface foundations installed at a depth not exceeding one meter, even in regions where frost reaches 3 meters. Excavation is then reduced to a minimum. In the case of overburden, the foundation consists of a steel grillage with ‘I’ beams which sits on a 25 cm pad of compacted granular material. In the case of surface rock, the foundation consists of a grouted deformed bar topped with a steel plate. For both these types of foundation, a spherical plate is added to insure a perfect hinge of the lower end of the mast [20].

CHAPTER – 10 CONCLUSION AND FURTHER SCOPE OF WORK

10.1 Conclusion

Ansys, a general finite element package has been used to solve a non-linear static analysis problem for the chainette type of transmission line tower. A computer program has been developed to do an approximate non-linear static analysis. The program has limitations of doing fixed ten numbers of loadsteps. The program assumes that the equilibrium condition has been obtained after solving for each loadstep.

10.2 Further scope of work

Static analysis has been done in this work, hence dynamic analysis can be a further scope of work. The condition of broken guy or condition of broken cross rope cable can be taken. The assumption of equilibrium condition obtained after each loadstep causes a fair percentage of error. Hence the program can be modified in this respect.

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APPENDIX I COMPUTER PROGRAM FOR ANALYSIS

- List of variables used in the program are as follows:

  - n [ ][ ][ ] = coordinates of node stored in three dimensional array.
  - cx [ ][ ][ ], cy [ ][ ][ ] and cy [ ][ ][ ] = direction cosines in global x, y and z directions respectively.
  - length[ ] = length of members.
  - previouslength[ ] = length of member before the current iteration.
  - deformedlength[ ] = length of member after the current iteration.
  - tension [ ] = tension in the cables due to pretension.
  - uts[ ] = ultimate tensile strength of the cable.
  - inistrain [ ] = initial strain in the cables due to pretension.
  - initialloadvector [ ] = load vector.
  - kg [ ][ ][ ] = member geometric stiffness matrix.
  - ke [ ][ ][ ] = member elastic stiffness matrix.
  - K [ ][ ][ ] = assembled structure stiffness matrix.
  - Kmodi [ ][ ][ ] = modified structure stiffness matrix.
  - defl [ ] = deflection vector.
  - forceinmem [ ] = force in member.

```
void CNONLINEAR3Doc::OnInputdata()
{

Cgeometryinputdlg f1;
Cloadinputdatadlg f2;
```

```
Cmaterialinputdatadlg f3;
CPropertySheet f4;
f4.AddPage(&f1);
f4.AddPage(&f2);
f4.AddPage(&f3);

int i = f4.DoModal();

if (i==IDOK)
{

        double cx[15],cy[15],cz[15],x1,x2,x3,sum,inistrain[15];
        double forceinmem[15],forceinmember[15][5];
        double s[15][15];
        double  previouslength[15],deformedlength[15],lengthdefiter[15][15];
        int  x,y,z,i,j,u,xy;



// READING DATA FROM THE DIALOG CLASSES AND STORING THEM-----------

        n[0][0][0]=f1.m_en3x;
        n[0][0][1]=f1.m_en3y;
        n[0][0][2]=f1.m_en3z;

        n[0][1][0]=f1.m_en7x;
        n[0][1][1]=f1.m_en7y;
        n[0][1][2]=f1.m_en7z;

        n[1][0][0]=f1.m_en4x;
        n[1][0][1]=f1.m_en4y;
        n[1][0][2]=f1.m_en4z;
```

```
n[1][1][0]=f1.m_en7x;
n[1][1][1]=f1.m_en7y;
n[1][1][2]=f1.m_en7z;

n[2][0][0]=f1.m_en5x;
n[2][0][1]=f1.m_en5y;
n[2][0][2]=f1.m_en5z;

n[2][1][0]=f1.m_en8x;
n[2][1][1]=f1.m_en8y;
n[2][1][2]=f1.m_en8z;

n[3][0][0]=f1.m_en6x;
n[3][0][1]=f1.m_en6y;
n[3][0][2]=f1.m_en6z;

n[3][1][0]=f1.m_en8x;
n[3][1][1]=f1.m_en8y;
n[3][1][2]=f1.m_en8z;

n[4][0][0]=f1.m_en7x;
n[4][0][1]=f1.m_en7y;
n[4][0][2]=f1.m_en7z;

n[4][1][0]=f1.m_en8x;
n[4][1][1]=f1.m_en8y;
n[4][1][2]=f1.m_en8z;

n[5][0][0]=f1.m_en7x;
n[5][0][1]=f1.m_en7y;
```

```
n[5][0][2]=f1.m_en7z;

n[5][1][0]=f1.m_en10x;
n[5][1][1]=f1.m_en10y;
n[5][1][2]=f1.m_en10z;

n[6][0][0]=f1.m_en8x;
n[6][0][1]=f1.m_en8y;
n[6][0][2]=f1.m_en8z;

n[6][1][0]=f1.m_en11x;
n[6][1][1]=f1.m_en11y;
n[6][1][2]=f1.m_en11z;

n[7][0][0]=f1.m_en7x;
n[7][0][1]=f1.m_en7y;
n[7][0][2]=f1.m_en7z;

n[7][1][0]=f1.m_en9x;
n[7][1][1]=f1.m_en9y;
n[7][1][2]=f1.m_en9z;

n[8][0][0]=f1.m_en8x;
n[8][0][1]=f1.m_en8y;
n[8][0][2]=f1.m_en8z;

n[8][1][0]=f1.m_en9x;
n[8][1][1]=f1.m_en9y;
n[8][1][2]=f1.m_en9z;

n[9][0][0]=f1.m_en10x;
```

```
n[9][0][1]=f1.m_en10y;
n[9][0][2]=f1.m_en10z;

n[9][1][0]=f1.m_en9x;
n[9][1][1]=f1.m_en9y;
n[9][1][2]=f1.m_en9z;

n[10][0][0]=f1.m_en11x;
n[10][0][1]=f1.m_en11y;
n[10][0][2]=f1.m_en11z;

n[10][1][0]=f1.m_en9x;
n[10][1][1]=f1.m_en9y;
n[10][1][2]=f1.m_en9z;

n[11][0][0]=f1.m_en1x;
n[11][0][1]=f1.m_en1y;
n[11][0][2]=f1.m_en1z;

n[11][1][0]=f1.m_en7x;
n[11][1][1]=f1.m_en7y;
n[11][1][2]=f1.m_en7z;

n[12][0][0]=f1.m_en2x;
n[12][0][1]=f1.m_en2y;
n[12][0][2]=f1.m_en2z;

n[12][1][0]=f1.m_en8x;
n[12][1][1]=f1.m_en8y;
n[12][1][2]=f1.m_en8z;
```

```
areaguy=f3.m_eguyoverallarea;
areacr =f3.m_ecrcableoverallarea;
areamast=f3.m_emastoverallarea;
areaspace=f3.m_espaceoverallarea;

moduguy=f3.m_eguymodulus;
moducr=f3.m_ecrcablemodulus;
modumast=f3.m_emastmodulus;
moduspace=f3.m_espacemodulus;

pretension[0]=f3.m_epreguy;
pretension[1]=f3.m_epreguy;
pretension[2]=f3.m_epreguy;
pretension[3]=f3.m_epreguy;
pretension[4]=f3.m_epresp;
pretension[5]=f3.m_eprecr;
pretension[6]=f3.m_eprecr;
pretension[7]=f3.m_eprecr;
pretension[8]=f3.m_eprecr;
pretension[9]=f3.m_eprecr;
pretension[10]=f3.m_eprecr;

uts[0]=f3.m_eutsguy;
uts[1]=f3.m_eutsguy;
uts[2]=f3.m_eutsguy;
uts[3]=f3.m_eutsguy;
uts[4]=f3.m_eutssp;
uts[5]=f3.m_eutscr;
uts[6]=f3.m_eutscr;
uts[7]=f3.m_eutscr;
uts[8]=f3.m_eutscr;
```

```
uts[9]=f3.m_eutscr;
uts[10]=f3.m_eutscr;

tension[0]=pretension[0]*uts[0]/100;
tension[1]=pretension[1]*uts[1]/100;
tension[2]=pretension[2]*uts[2]/100;
tension[3]=pretension[3]*uts[3]/100;
tension[4]=pretension[4]*uts[4]/100;
tension[5]=pretension[5]*uts[5]/100;
tension[6]=pretension[6]*uts[6]/100;
tension[7]=pretension[7]*uts[7]/100;
tension[8]=pretension[8]*uts[8]/100;
tension[9]=pretension[9]*uts[9]/100;
tension[10]=pretension[10]*uts[10]/100;
tension[11]=0.00;
tension[12]=0.00;

inistrain[0]=tension[0]/(areaguy*moduguy);
inistrain[1]=tension[1]/(areaguy*moduguy);
inistrain[2]=tension[2]/(areaguy*moduguy);
inistrain[3]=tension[3]/(areaguy*moduguy);
inistrain[4]=tension[4]/(areaspace*moduspace);
inistrain[5]=tension[5]/(areacr*moducr);
inistrain[6]=tension[6]/(areacr*moducr);
inistrain[7]=tension[7]/(areacr*moducr);
inistrain[8]=tension[8]/(areacr*moducr);
inistrain[9]=tension[9]/(areacr*moducr);
inistrain[10]=tension[10]/(areacr*moducr);
inistrain[11]=tension[11]/(areamast*modumast);
inistrain[12]=tension[12]/(areamast*modumast);
```

```
        fout=fopen("matrixoutput.txt","w");

// calculation of length of members at the first step----------////////

        for(x=0;x<13;x++)
        {
                x1 = (n[x][1][0]-n[x][0][0])*(n[x][1][0]-n[x][0][0]);
                x2 = (n[x][1][1]-n[x][0][1])*(n[x][1][1]-n[x][0][1]);
                x3 = (n[x][1][2]-n[x][0][2])*(n[x][1][2]-n[x][0][2]);
                sum = x1+x2+x3;
                length[x] = sqrt(sum);
        }

// calculation of the direction cosines ----/////////////////////

        for(xy=0;xy<13;xy++)
        {
                cx[xy]=(n[xy][1][0]-n[xy][0][0])/length[xy];
                cy[xy]=(n[xy][1][1]-n[xy][0][1])/length[xy];
                cz[xy]=(n[xy][1][2]-n[xy][0][2])/length[xy];


        }
double initialloadvector[15];


                initialloadvector[0]=(-tension[0]*cx[0])+(-
tension[1]*cx[1])+(tension[4]*cx[4])+(tension[5]*cx[5])+(tension[7]*cx[7]);
                initialloadvector[1]=(-tension[0]*cy[0])+(-
tension[1]*cy[1])+(tension[4]*cy[4])+(tension[5]*cy[5])+(tension[7]*cy[7]);
                initialloadvector[2]=(-tension[0]*cz[0])+(-
tension[1]*cz[1])+(tension[4]*cz[4])+(tension[5]*cz[5])+(tension[7]*cz[7]);
```

```
		initialloadvector[3]=(-tension[2]*cx[2])+(-tension[3]*cx[3])+(-
tension[4]*cx[4])+(tension[6]*cx[6])+(tension[8]*cx[8]);
		initialloadvector[4]=(-tension[2]*cy[2])+(-tension[3]*cy[3])+(-
tension[4]*cy[4])+(tension[6]*cy[6])+(tension[8]*cy[8]);
		initialloadvector[5]=(-tension[2]*cz[2])+(-tension[3]*cz[3])+(-
tension[4]*cz[4])+(tension[6]*cz[6])+(tension[8]*cz[8]);
		initialloadvector[6]=(-tension[5]*cx[5])+(tension[9]*cx[9]);
		initialloadvector[7]=(-tension[5]*cy[5])+(tension[9]*cy[9]);
		initialloadvector[8]=(-tension[6]*cx[6])+(tension[10]*cx[10]);
		initialloadvector[9]=(-tension[6]*cy[6])+(tension[10]*cy[10]);
		initialloadvector[10]=(-tension[7]*cx[7])+(-tension[8]*cx[8])+(-
tension[9]*cx[9])+(-tension[10]*cx[10]);
		initialloadvector[11]=(-tension[7]*cy[7])+(-tension[8]*cy[8])+(-
tension[9]*cy[9])+(-tension[10]*cy[10]);


for(iter=1;iter<=10;iter++)
{
		loadvector[0]=initialloadvector[0];
		loadvector[1]=initialloadvector[1]-1000*iter;
		loadvector[2]=initialloadvector[2];
		loadvector[3]=initialloadvector[3];
		loadvector[4]=initialloadvector[4]-1000*iter;
		loadvector[5]=initialloadvector[5];
		loadvector[6]=initialloadvector[6];
		loadvector[7]=initialloadvector[7]-5400*iter;
		loadvector[8]=initialloadvector[8];
		loadvector[9]=initialloadvector[9]-5400*iter;
		loadvector[10]=initialloadvector[10];
		loadvector[11]=initialloadvector[11]-5400*iter;
```

```
for(xy=0;xy<13;xy++)
{
	if(iter==1)
	{
		forceinmem[xy]=tension[xy];
		lengthdef[xy]=length[xy];
	}
	else
	{
	}

	kg[xy][0][0]=(forceinmem[xy]/lengthdef[xy])*(1-cx[xy]*cx[xy]);
	kg[xy][0][1]=(forceinmem[xy]/lengthdef[xy])*(-cx[xy]*cy[xy]);
	kg[xy][0][2]=(forceinmem[xy]/lengthdef[xy])*(-cx[xy]*cz[xy]);

	kg[xy][1][0]=kg[xy][0][1];
	kg[xy][1][1]=(forceinmem[xy]/lengthdef[xy])*(1-cy[xy]*cy[xy]);
	kg[xy][1][2]=(forceinmem[xy]/lengthdef[xy])*(-cy[xy]*cz[xy]);

	kg[xy][2][0]=kg[xy][0][2];
	kg[xy][2][1]=kg[xy][1][2];
	kg[xy][2][2]=(forceinmem[xy]/lengthdef[xy])*(1-cz[xy]*cz[xy]);

	kg[xy][0][3]=-kg[xy][0][0];
	kg[xy][0][4]=-kg[xy][0][1];
	kg[xy][0][5]=-kg[xy][0][2];

	kg[xy][1][3]=-kg[xy][1][0];
	kg[xy][1][4]=-kg[xy][1][1];
```

```
kg[xy][1][5]=-kg[xy][1][2];

kg[xy][2][3]=-kg[xy][2][0];
kg[xy][2][4]=-kg[xy][2][1];
kg[xy][2][5]=-kg[xy][2][2];

kg[xy][3][0]=kg[xy][0][3];
kg[xy][3][1]=kg[xy][0][4];
kg[xy][3][2]=kg[xy][0][5];
kg[xy][3][3]=kg[xy][0][0];
kg[xy][3][4]=kg[xy][0][1];
kg[xy][3][5]=kg[xy][0][2];

kg[xy][4][0]=kg[xy][1][3];
kg[xy][4][1]=kg[xy][1][4];
kg[xy][4][2]=kg[xy][1][5];
kg[xy][4][3]=kg[xy][1][0];
kg[xy][4][4]=kg[xy][1][1];
kg[xy][4][5]=kg[xy][1][2];

kg[xy][5][0]=kg[xy][2][3];
kg[xy][5][1]=kg[xy][2][4];
kg[xy][5][2]=kg[xy][2][5];
kg[xy][5][3]=kg[xy][2][0];
kg[xy][5][4]=kg[xy][2][1];
kg[xy][5][5]=kg[xy][2][2];

}

// calculation of the member stiffness matricess--------/////////////
```

```
for(xy=0;xy<13;xy++)
{

	if((xy==11)||(xy==12))
	{
		area=areamast;
		modulus=modumast;
	}
	else
	{
		if(xy==4)
		{
			area=areaspace;
			modulus=moduspace;
		}
		else
		{
			if((xy==0)||(xy==1)||(xy==2)||(xy==3))
			{
				area=areaguy;
				modulus=moduguy;
			}
			else
			{
				area=areacr;
				modulus=moducr;
			}
		}
	}
ke[xy][0][0]=((area*modulus)/(length[xy]))*(cx[xy]*cx[xy]);
```

```
ke[xy][0][1]=((area*modulus)/(length[xy]))*(cx[xy]*cy[xy]);
ke[xy][0][2]=((area*modulus)/(length[xy]))*(cx[xy]*cz[xy]);
ke[xy][1][0]=ke[xy][0][1];
ke[xy][1][1]=((area*modulus)/(length[xy]))*(cy[xy]*cy[xy]);

ke[xy][1][2]=((area*modulus)/(length[xy]))*(cy[xy]*cz[xy]);
ke[xy][2][0]=ke[xy][0][2];
ke[xy][2][1]=ke[xy][1][2];
ke[xy][2][2]=((area*modulus)/(length[xy]))*(cz[xy]*cz[xy]);
ke[xy][0][3]=-ke[xy][0][0];
ke[xy][0][4]=-ke[xy][0][1];
ke[xy][0][5]=-ke[xy][0][2];
ke[xy][1][3]=-ke[xy][1][0];
ke[xy][1][4]=-ke[xy][1][1];
ke[xy][1][5]=-ke[xy][1][2];
ke[xy][2][3]=-ke[xy][2][0];
ke[xy][2][4]=-ke[xy][2][1];
ke[xy][2][5]=-ke[xy][2][2];
ke[xy][3][0]=ke[xy][0][3];
ke[xy][3][1]=ke[xy][0][4];
ke[xy][3][2]=ke[xy][0][5];
ke[xy][3][3]=ke[xy][0][0];
ke[xy][3][4]=ke[xy][0][1];
ke[xy][3][5]=ke[xy][0][2];
ke[xy][4][0]=ke[xy][1][3];
ke[xy][4][1]=ke[xy][1][4];
ke[xy][4][2]=ke[xy][1][5];
ke[xy][4][3]=ke[xy][1][0];
ke[xy][4][4]=ke[xy][1][1];
ke[xy][4][5]=ke[xy][1][2];
ke[xy][5][0]=ke[xy][2][3];
```

```
ke[xy][5][1]=ke[xy][2][4];
ke[xy][5][2]=ke[xy][2][5];
ke[xy][5][3]=ke[xy][2][0];
ke[xy][5][4]=ke[xy][2][1];
ke[xy][5][5]=ke[xy][2][2];

}
for(x=0;x<13;x++)
{
	for(y=0;y<=5;y++)
	{
		for(z=0;z<=5;z++)
		{
			k[x][y][z]=ke[x][y][z]+kg[x][y][z];
		}
	}
}

if(iter>=5)
{
	for(y=0;y<=5;y++)
	{
		for(z=0;z<=5;z++)
		{
			k[4][y][z]=0.00;
		}
	}
}
else
{
}
```

```
// structure stiffness matrix ------------------//////////////////////////

for(x=0;x<=2;x++)
{
        K[x][x]=k[0][x+3][x+3]+k[1][x+3][x+3]+k[4][x][x]+k[5][x][x]+k[7][x][x]+k[11]
[x+3][x+3];
}
for(x=3;x<=5;x++)
{
        K[x][x]=k[2][x][x]+k[3][x][x]+k[4][x][x]+k[6][x-3][x-3]+k[8][x-3][x-
3]+k[12][x][x];
}
for(x=6;x<=8;x++)
{
        K[x][x]=k[5][x-3][x-3]+k[9][x-6][x-6];
}
for(x=9;x<=11;x++)
{
        K[x][x]=k[6][x-6][x-6]+k[10][x-9][x-9];
}
for(x=12;x<=14;x++)
{
        K[x][x]=k[7][x-9][x-9]+k[8][x-9][x-9]+k[9][x-9][x-9]+k[10][x-9][x-9];
}
for(xy=1;xy<=2;xy++)
{
        K[0][xy]=k[0][3][xy+3]+k[1][3][xy+3]+k[4][0][xy]+k[5][0][xy]+k[7][0][xy]+k[1
1][3][xy+3];
}
for(x=0;x<=2;x++)
```

```
{
	for(xy=0;xy<=2;xy++)
	{
	K[x][xy]=k[0][x+3][xy+3]+k[1][x+3][xy+3]+k[4][x][xy]+k[5][x][xy]+k[7][x][xy
]+k[11][x+3][xy+3];
	}
	for(xy=3;xy<=5;xy++)
	{
		K[x][xy]=k[4][x][xy];
	}
}
	for(x=0;x<=2;x++)
	{
		for(xy=6;xy<=8;xy++)
		{
			K[x][xy]=k[5][x][xy-3];
		}
	}

	for(x=0;x<=2;x++)
	{
		for(xy=9;xy<=11;xy++)
		{
			K[x][xy]=0.00;
		}
	}

	for(x=0;x<=2;x++)
	{
		for(xy=12;xy<=14;xy++)
		{
```

```
                K[x][xy]=k[7][x][xy-9];
            }
    }

    for(x=3;x<=5;x++)
    {
            for(xy=12;xy<=14;xy++)
            {
                K[x][xy]=k[8][x][xy-12];
            }
            for(xy=9;xy<=11;xy++)
            {
                K[x][xy]=k[6][x-3][xy-6];
            }
            for(xy=6;xy<=8;xy++)
            {
                K[x][xy]=0.00;
            }
            for(xy=3;xy<=5;xy++)
            {
                K[x][xy]=k[2][x][xy]+k[3][x][xy]+k[4][x][xy]+k[6][x-3][xy-
3]+k[8][x-3][xy-3]+k[12][x][xy];
            }
    }

    for(x=6;x<=8;x++)
    {
            for(xy=12;xy<=14;xy++)
            {
                K[x][xy]=k[9][x-6][xy-9];
            }
```

```
            for(xy=9;xy<=11;xy++)
            {
                    K[x][xy]=0.00;
            }
            for(xy=6;xy<=8;xy++)
            {
                    K[x][xy]=k[5][x-3][xy-3]+k[9][x-6][xy-6];
            }
    }

    for(x=9;x<=11;x++)
    {
            for(xy=12;xy<=14;xy++)
            {
                    K[x][xy]=k[10][x-9][xy-9];
            }
            for(xy=9;xy<=11;xy++)
            {
                    K[x][xy]=k[6][x-6][xy-6]+k[10][x-9][xy-9];
            }
    }

    for(x=12;x<=14;x++)
    {
            for(xy=12;xy<=14;xy++)
            {
                    K[x][xy]=k[7][x-9][xy-9]+k[8][x-9][xy-9]+k[9][x-9][xy-
9]+k[10][x-9][xy-9];
            }
    }
```

```
        for(x=0;x<=14;x++)
        {
                for(xy=0;xy<=14;xy++)
                {
                        K[xy][x]=K[x][xy];
                }
        }

// boundary conditions to be applied and the resulting structure stiffness matrix formation------

// for vertical loads only------//////////



// modified structure stiffness matrix\\///////////////////////

                for(i=0;i<=7;i++)
                {
                        for(j=0;j<=7;j++)
                                Kmodi[i][j]=K[i][j];
                        for(j=8;j<=9;j++)
                                Kmodi[i][j]=K[i][j+1];
                        for(j=10;j<=11;j++)
                                Kmodi[i][j]=K[i][j+2];
                }

                for(i=8;i<=9;i++)
                {
                        for(j=0;j<=7;j++)
                                Kmodi[i][j]=K[i+1][j];
                        for(j=8;j<=9;j++)
```

```
				Kmodi[i][j]=K[i+1][j+1];
			for(j=10;j<=11;j++)
				Kmodi[i][j]=K[i+1][j+2];
		}



		for(i=10;i<=11;i++)
		{
			for(j=0;j<=7;j++)
				Kmodi[i][j]=K[i+2][j];
			for(j=8;j<=9;j++)
				Kmodi[i][j]=K[i+2][j+1];
			for(j=10;j<=11;j++)
				Kmodi[i][j]=K[i+2][j+2];
		}
//---------------------

// Inversing the matrix////////////////////////////////////////////////////
		int i,j,k;
		double z;
		for(i=0;i<=11;i++)
		{
			z=Kmodi[i][i];
			Kmodi[i][i]=1.0;
			for(j=0;j<=11;j++)
				Kmodi[i][j]=Kmodi[i][j]/z;
			for(k=0;k<=11;k++)
			{
				if((k-i)!=0)
				{
					z=Kmodi[k][i];
```

```
                        Kmodi[k][i]=0.0;
                        for(j=0;j<=11;j++)
                            Kmodi[k][j]=Kmodi[k][j]-z*Kmodi[i][j];
                    }
                }
            }
//-------------------------------------------------------




// Multiplication of matrices//////////////////////////////
        for(i=0;i<=11;i++)
        {
            {
                double sum=0.00;
                for(k=0;k<=11;k++)
                    sum=sum+Kmodi[i][k]*loadvector[k];
                defl[i]=sum;
            }
        }

//-------------------------------------------------------

// Revision of the deformed geometry////////////////////////////////////


                n[0][1][0]=n[0][1][0]+defl[0];
                n[0][1][1]=n[0][1][1]+defl[1];
                n[0][1][2]=n[0][1][2]+defl[2];

                n[1][1][0]=n[1][1][0]+defl[0];
```

```
n[1][1][1]=n[1][1][1]+defl[1];
n[1][1][2]=n[1][1][2]+defl[2];

n[4][0][0]=n[4][0][0]+defl[0];
n[4][0][1]=n[4][0][1]+defl[1];
n[4][0][2]=n[4][0][2]+defl[2];

n[5][0][0]=n[5][0][0]+defl[0];
n[5][0][1]=n[5][0][1]+defl[1];
n[5][0][2]=n[5][0][2]+defl[2];

n[7][0][0]=n[7][0][0]+defl[0];
n[7][0][1]=n[7][0][1]+defl[1];
n[7][0][2]=n[7][0][2]+defl[2];

n[11][1][0]=n[11][1][0]+defl[0];
n[11][1][1]=n[11][1][1]+defl[1];
n[11][1][2]=n[11][1][2]+defl[2];
//----------------------------------------------------------
n[2][1][0]=n[2][1][0]+defl[3];
n[2][1][1]=n[2][1][1]+defl[4];
n[2][1][2]=n[2][1][2]+defl[5];

n[3][1][0]=n[3][1][0]+defl[3];
n[3][1][1]=n[3][1][1]+defl[4];
n[3][1][2]=n[3][1][2]+defl[5];

n[4][1][0]=n[4][1][0]+defl[3];
n[4][1][1]=n[4][1][1]+defl[4];
n[4][1][2]=n[4][1][2]+defl[5];
```

```
n[6][0][0]=n[6][0][0]+defl[3];
n[6][0][1]=n[6][0][1]+defl[4];
n[6][0][2]=n[6][0][2]+defl[5];

n[8][0][0]=n[8][0][0]+defl[3];
n[8][0][1]=n[8][0][1]+defl[4];
n[8][0][2]=n[8][0][2]+defl[5];

n[12][1][0]=n[12][1][0]+defl[3];
n[12][1][1]=n[12][1][1]+defl[4];
n[12][1][2]=n[12][1][2]+defl[5];

//------------------------------------------------------------
n[7][1][0]=n[7][1][0]+defl[10];
n[7][1][1]=n[7][1][1]+defl[11];

n[8][1][0]=n[8][1][0]+defl[10];
n[8][1][1]=n[8][1][1]+defl[11];

n[9][1][0]=n[9][1][0]+defl[10];
n[9][1][1]=n[9][1][1]+defl[11];

n[10][1][0]=n[10][1][0]+defl[10];
n[10][1][1]=n[10][1][1]+defl[11];
//-------------------------------------------------------------
n[5][1][0]=n[5][1][0]+defl[6];
n[5][1][1]=n[5][1][1]+defl[7];

n[9][0][0]=n[9][0][0]+defl[6];
n[9][0][1]=n[9][0][1]+defl[7];
```

```
//--------------------------------------------------------------

                    n[6][1][0]=n[6][1][0]+defl[8];
                    n[6][1][1]=n[6][1][1]+defl[9];

                    n[10][0][0]=n[10][0][0]+defl[8];
                    n[10][0][1]=n[10][0][1]+defl[9];

//---------------------------------------------------------------
// Calculation of the length of the members after deformation////////////////////////

            for(int x=0;x<13;x++)
            {
                    x1 = (n[x][1][0]-n[x][0][0])*(n[x][1][0]-n[x][0][0]);
                    x2 = (n[x][1][1]-n[x][0][1])*(n[x][1][1]-n[x][0][1]);
                    x3 = (n[x][1][2]-n[x][0][2])*(n[x][1][2]-n[x][0][2]);
                    sum = x1+x2+x3;
                    lengthdef[x] = sqrt(sum);

                    lengthdefiter[iter][x]=lengthdef[x];


            }

            for(xy=0;xy<13;xy++)
            {
                    cx[xy]=(n[xy][1][0]-n[xy][0][0])/lengthdef[xy];
                    cy[xy]=(n[xy][1][1]-n[xy][0][1])/lengthdef[xy];
                    cz[xy]=(n[xy][1][2]-n[xy][0][2])/lengthdef[xy];


            }
```

```
//--------------------------------------------------

// coding for the calculations of member forces using elastic strains..////////

                    for(x=0;x<=12;x++)
                          deformedlength[x]=lengthdefiter[iter][x];
                    double elstrain[13];
                    for(x=0;x<=12;x++)
                    {
                          if(iter==1)
                                previouslength[x]=length[x];
                          else
                                previouslength[x]=lengthdefiter[iter-1][x];

                          elstrain[x]=deformedlength[x]/length[x]-
previouslength[x]/length[x]+inistrain[x];
                    }

                    for(x=0;x<=12;x++)
                    {
                          if(x<=3)
                                forceinmem[x]=elstrain[x]*areaguy*moduguy;
                          else
                          {
                                if(x==4)
                                {

      forceinmem[x]=elstrain[x]*areaspace*moduspace;
                                }
                                else
                                {
```

```
                                                if(x<=10)

        forceinmem[x]=elstrain[x]*areacr*moducr;
                                                else

        forceinmem[x]=elstrain[x]*areamast*modumast;
                                        }
                                }
                        }

                fprintf(fout,"\n Deflection Matrix : \n");
                fprintf(fout," ----------------------- \n");
                for(i=0;i<=11;i++)
                        fprintf(fout,"%10.4f\n",defl[i]);
                fprintf(fout,"\n");

                        fprintf(fout,"\n Force in Member Matrix : \n");
                        fprintf(fout," ------------------------- \n");
                        for(i=0;i<=12;i++)
                                fprintf(fout,"%10.4f\n",forceinmem[i]);
                        fprintf(fout,"\n");

        }

        set = true;
}
else
{
        set = false;
}
```

```
UpdateAllViews(NULL);
}
```