ANALYSIS AND DESIGN OF TRANSMISSION LINE TOWERS

A DISSERTATION

submitted in partial fulfilment of
the requirements for the award of the degree
of

MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
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CERTIFICATE

Certified that the dissertation entitled "ANALYSIS AND DESIGN OF TRANSMISSION LINE TOWERS" which is being submitted by Er. Niladri Behari Misra in partial fulfilment for the award of the degree of Master of Engineering (Water Resources Development) of University of Roorkee, is a record of student's own work carried out by him under our guidance and supervision. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for a period of six months from 1.10.1978 to 31.3.1979 for preparing this dissertation for Master of Engineering at this University.

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SYNOPSIS

The electrical transmission line serves as a vital link between the generating stations and load centres and also between different generating stations in an inter-connected system. With the rapid growth in generating capacities as well as load demand, large blocks of power are required to be transmitted over long distances, which poses a challenging task for the structural engineer to design relatively large transmission structures to support heavy conductor loads, with a high degree of reliability and economy. This requires more efficient and accurate methods of analysis and design for the towers.

Transmission line towers are highly redundant, statically indeterminate structures and their analysis by classical structural analysis procedures is very tedious and time consuming and indeed, impossible in certain cases. In the conventional methods of analysis, the tower is reduced to several statically determinate planar frames together with the resolution of the loads into the planes in which they act, ignoring entirely or approximating the effect of the redundant systems in the distribution of forces. Thus conservative assumptions are made in the conventional methods resulting in costlier structures.

In this dissertation, the analysis and design of trans- mission line towers has been presented, treating them as pin-jointed space structures, using digital computer. The analysis has been based on the Stiffness Matrix approach. A general digital computer programme has been developed for analysing and designing single circuit as well as double circuit self supported towers. The input to the programme consists of the geometrical configuration of the tower, the conductor and groundwire sizes, the design spans, the climatic conditions, and sizes and properties of available steel sections. The programme first determines the maximum working tensions for the conductor and groundwire by carrying out nécessary sag-tension calculations. All the design loads are then computed and the tower is analysed for different load combinations corresponding to the normal and brokenwire conditions, and the maximum tensile and compressive member forces are determined. The programme then estimates the minimum cross sectional areas for the members and selects appropriate sizes from a list of available steel sections.

Using this programme, a single circuit 132 kV trans- mission line tower has been designed and a comparison is made with the conventional design. It is noted that a saving of 5.8 percent in steel is possible by analysing this tower as a three dimensional space structure . From the results of member forces, it is also seen that the three dimensional analysis through computer gives a more realistic picture of stress distribution. Any design based on this will therefore produce a more economical as well as safer tower in minimum possible time. Further, the programme developed in this work, with the ability of analysing and designing a tower in a single run, entirely within the core memory of the computer is expected to be quite useful.

C O N T E N T S

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CHAPTER-1

INTRODUCTION

1.1. General

The Electric Power System consists of three principal components, that are the generating stations, the transmission lines and the distribution systems. The transmission line, besides serving as the connecting link between the generating stations and the distribution systems, it also serves in transferring bulk power between large power stations and also between large receiving stations in any interconnected grid. The development of large mine-mouth Super Thermal Stations, remote Hydro Electric power plants and the inter-connection of state grids to form regional grids and further interconnection of regional grids have given rise to the need for Extra High Tension transmission lines. The capacity of these E.H.T. lines has also been increasing steadily with the increase in the installed capacity and rapid growth in load demand. It is felt that in order to cater such huge quantity of power to the load centres, the existing transmission lines are to be strengthened and new lines are to be laid. Estimates based on long-term system planning studies indicate that, even with the supplemental H.V.D.C. transmission, over 16,000 km of 400 kV A.C. lines would have to be built in this country during the next 5-6 years[16].

The length of other H.V. A.C. transmission lines will naturally be several times more than this. Thus the investment on the electrical transmission lines goes on multiplying at an accelerated rate. This requires every effort in optimisation of the cost of E.H.T. lines. Besides the necessity of observing economy in construction, it is also required to build larger, stronger and safer towers now, for the electrical transmission lines, as the transmission voltage rises and the conductor size increases for evacuation of large blocks of power. The recent development of self-damping conductors, which renders longer spans feasible and more economical[17], necessitates the design of higher and more heavily loaded towers. Great responsibility therefore rests on the transmission lines engineer, who has to prepare not only economical but also dependable design to meet the increased load requirements of the modern transmission lines. This naturally calls for accurate and efficient methods of design and analysis for the transmission line towers. Since cost of the transmission line towers are about 35-45 percent of that of the total transmission line[5], considerable economy can easily be achieved by suitably designing the towers.

In the design of many civil engineering structures, it is quite often more economical to design oversafe some elements of the structure, rather than spending time in executing a precise analysis, since, a comparatively heavier steel section is probably much less costly than the engineering hours spent to save the material. This however is not the case with the design of transmission line towers. In this case the engineer has, beside ensuring the safe performance of the structure, to keep constantly in mind that a small percentage of excess weight per tower will result in a substantial unnecessary steel tonnage on the line, ultimately affecting its construction cost.

1.2. Present Practice

A review of the present practice of analysis and design of towers, shows that it has not benefitted much from the rapidly expanding knowledge of computer technique. For many decades, these structures have been designed and built on the basis of simplified methods of analysis, such as the familiar graphic analysis method. In these traditional methods, the space structure is reduced into statically determinate plane trusses together with resolution of vertical, transverse and longitudinal loads into the planes in which they act, ignoring totally or approximating the effect of the redundant systems on the distribution of these forces. One of the major concerns resulting from the use of these methods is the distribution of shear arising from torsional moment induced by a broken conductor, ice shedding off one span or pull imposed during stringing or maintenance operations. These methods therefore require either a rigorous approach or a traditional approach with empirical formulae to the distribution of these torsional moments to the faces of the tower by means of one or more horizontal bracing systems. In addition, these methods also require conservative assumptions to be made resulting in use of more material in the structure and thereby increasing both weight and cost. On the other hand an accurate indeterminate stress analysis of the tower including the effect of redundant members and considering the loads at the true points of application, instead of resolving them into different planes, can be done by analysis of the tower as a space truss with the help of digital computer.

A digital computer analysis of a structure invariably considers a much more complicated and complete model than a manual analysis. It also becomes practicable to study several alternatives with different material or configuration to achieve better economy.

It is however seen that most of the available programmes for frame analysis have merely automated the old static methods of analysis[6]. Few of the computer programmes which are available now for three dimensional frame analysis, are general purpose type programmes, which considers six degrees-of-freedom[15]. Since the transmission line towers are most ideal pin-jointed space structures, only three degrees-of- freedom are required. Use of these programmes for tower analysis therefore requires longer execution time on computer than what is actually necessary. Besides, the input data preparation is difficult and interpretation of the output requires greater effort.

It is further felt that computerisation of only the analysis part does not actually help much in saving man hours, unless computation of tower loads and also the design calculations are computerised. Advanced design programmes for design of towers are apparently existing, but these are proprietory programmes for which little details are known, and is only available to their owners.

1.3. Scope of Investigation

A general computer programme has been developed in this work and presented for automated design of self supported transmission line towers. From the considerations of simple input data and efficient computer utilisation, the iterative design procedure, that is, the design by repeated analysis has been adopted in preference to the direct method of Structural Synthesis. Structural optimisation by variation of geometry and/or topology has been excluded from the scope of study for this work. It is assumed that the structure outline and member arrangements are predetermined, and only the member sizes are to be found out.

In this dissertation, the Chapter-II gives a review of the present practices in establishing the tower configuration in respect of the self supported, broad based steel towers for transmission lines. Various electrical and structural factors which are taken into consideration for the purpose, have been quoted.

Chapter-III covers the conventional procedure for calculation of tower design loads. The manner in which calculation of design loads for tower has been incorporated in the automated analysis and design programme, is also discussed in this chapter.

Chapter-IV is devoted to the review of the conventional methods of tower analysis. The limitations or inadequacies of these conventional methods have also been discussed.

Chapter-V explains the three dimensional analysis procedure which has been used for the automated design of the tower anda comparison of the results obtained through conventional and three dimensional analyses has been made.

Chapter-VI deals with the automated design procedure developed in this work for designing of transmission line towers. An existing 132 kV, Single Circuit, 30^{o} angle tower designed by conventional method has been redesigned here, and a saving in steel has been shown.

The last chapter gives the conclusions drawn from the studies made in this work.

CHAPTER-II

ESTABLISHING THE TOWER CONFIGURATION

2.1. Introduction

The design of Extra High Tension electric transmission towers is an interesting and challenging task for the transmission engineer. Ever higher voltages, and the pressure to keep costs down, are incentives to develop transmission structures which are stronger, more reliable and have optimum cost.

In the design of transmission line structures the selection of an optimum outline and system of bracing patterns contributes to a great extent in developing an economical design of a transmission line tower and thereby reducing their overall cost. The selection of an outline is of course, flexible in nature. Various configurations have been tried, from considerations of (i) voltage, (ii) number of circuits, (iii) type of material used for the structure, (iv) transmission efficiency, (v) overall economy and also, to some extent,(vi) aesthetic values. For a particular tower configuration selected, the outline decided shall satisfy both, electrical and structural requirements in consistent with economy.

The tower configurations can be widely different, depending upon the type of material used for the structure. The various kinds of material used today for the transmission line structures for H.V./ E.H.V. lines are wood poles, laminated wood, prestressed concrete, steel and aluminium[6]. The scope of discussion is restricted to steel structures only in this work.

The steel structures used in the H.V./E.H.V. transmission lines can be broadly classified as (1) Guyed structures and (2) Self-supported structures. The guyed towers require less structural steel, but more space and often more construction effort. There are however some erection advantages to the guyed tower, as the steel can often be completely assembled on the ground, after which the tower as a whole can be lifted in place with a crane. The self-supported structure, on the other hand, is heavier but has the advantages of occupying less ground, and often providing greater reliability. In the case of guyed towers, the guy anchors require excellent compaction of the back filling of its footings, where as the self-supported tower can tolerate, to some extent, sub-standard backfilling of its footings. These considerations should not be overlooked in the remote areas, where transmission lines are often constructed and inspection is difficult and costly. The guyed type of structures have been used mostly in U.S.A. and Canada, while self-supported structures have been extensively used all over the world[17]. In India the self-supported structures only have been used, as these are more robust, need lesser right of way, and specially more reliable, compared to the guyed structures. In this work, the analysis and design of self-supported steel transmission line towers have been dealt with.

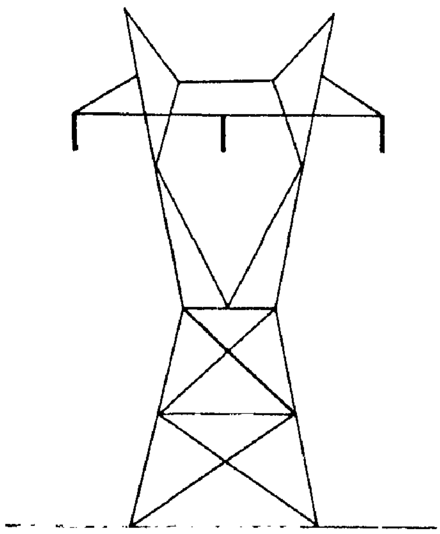
The configuration of a self-supported steel lattice tower is dependant on various electrical and structural requirements such as -

1) Conductor arrangement
2) Minimum ground clearance
3) Length of insulator assembly
4) The air gap clearances
5) Mid span clearances between conductor and ground-wire
6) Nature of shielding provided, that is, the number and height of groundwires
7) The system of bracing patterns

All the above factors are therefore to be considered from transmission efficiency, structural soundness and personnel safety point of view.

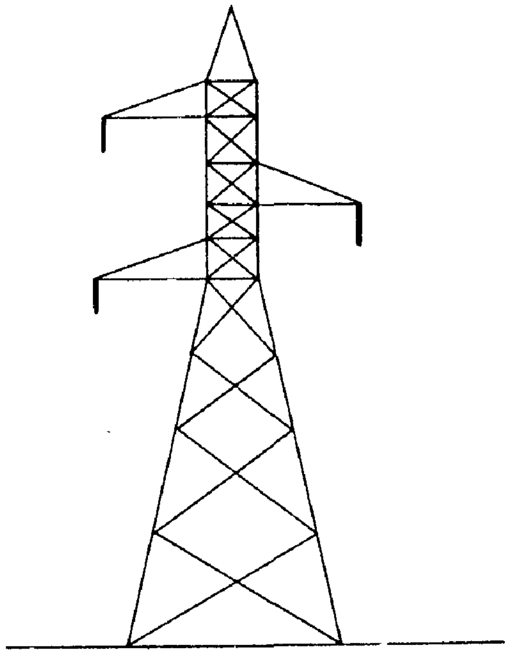
2.2. Conductor Arrangement

In a single circuit transmission line tower the conductor arrangement can be either horizontal or triangular as shown in Fig.2.1a and 2.1b. Both have some advantages and disadvantages. The height of towers with triangular



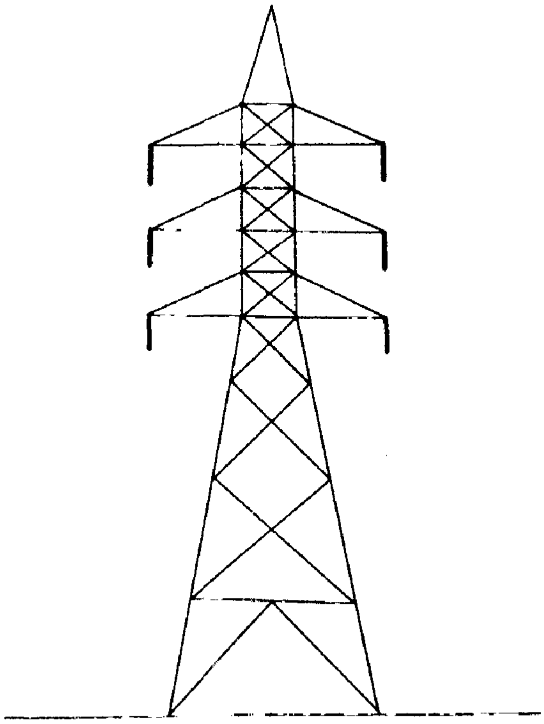
SINGLE CIRCUIT, HORIZONTAL

2·1 a



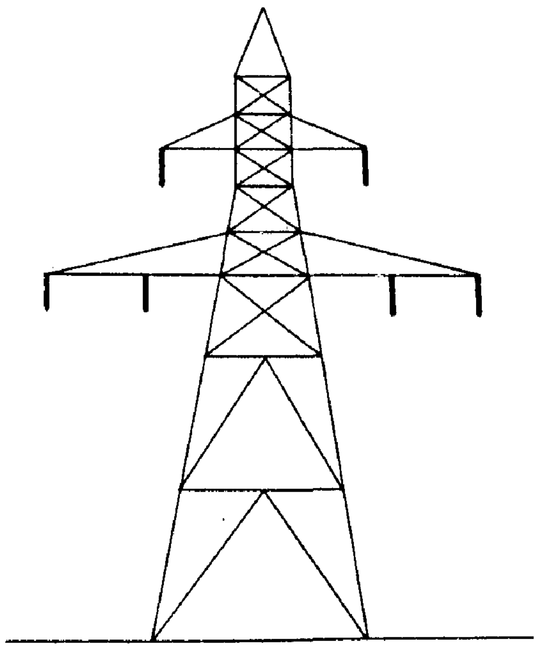
SINGLE CIRCUIT, TRIANGULAR

2·1 b



DOUBLE CIRCUIT, VERTICAL

2·1 c



DOUBLE CIRCUIT, TRIANGULAR

2·1 d

GENERAL ARRANGEMENT OF CONDUCTORS ON SINGLE AND DOUBLE CIRCUIT TOWERS

FIG. 2·1

conductor arrangement, is more than that of the towers with horizontal arrangement, resulting in more weight. The effect of wind on triangular tower will also be more, resulting in further increase in the tower weight. The probability of lightning flashover is also increased with higher structures. The triangular tower however, provides balanced reactances, and transmission capability is improved with the reduction of the geometric mean distance between the conductors. There is distinct advantage in the triangular formation of the conductor when the right-of-way requirements are considered. The usual practice is to use the triangular configuration for the single circuit towers upto and including the transmission voltage of 220 kV, and use the horizontal configuration for transmission voltages for 220 kV and upwards. For the 220 kV single circuit towers both the types of conductor arrangements have been successfully adopted.

On double circuit towers, the conductors are hung one above another in a more or less vertical plane from three horizontal cross arms, as shown in the Fig.2.1c. Another arrangement shown in Fig.2.1d. uses two cross-arms, the upper of which support two conductors, while the lower support four conductors[6]. In this arrangement each of the two circuits is placed in a triangular configuration.
In either case, it is important to maintain sufficient vertical and horizontal separation between conductors to prevent mid span interference under dynamic conditions.

These conditions may be brought about by nonuniform ice dropping or aerofoil behaviour of the conductor under certain icing and wind conditions. In this country the vertical arrangement of the conductors has been adopted for all double circuit lines.

2.3. Minimum Ground Clearance

Power conductors, along the entire route of the transmission lines, should maintain requisite clearance to ground over open country, national highways, important roads, electrified and unelectrified railway tracks, navigable and non-navigable rivers, telecommunication and power lines etc., as laid down in the various National Standards issued by the respective Authorities.

Clause No.77 of the Indian Electricity Rules -1956 stipulates the following clearances above ground for the lowest point of the conductor. For extra high voltage lines, this clause stipulates that the clearance above ground shall not be less than 5.1 metre plus 0.3 metre (17 ft plus 1 ft) for every 33,000 volts or part thereof by which the voltage of the line exceeds 33,000 volts. The permissibles minimum ground clearances for different voltages therefore work out as follows -

66 kV	-	5,490 mm
132 kV	-	6,100 mm
220 kV	-	7,015 mm
400 kV	-	8,840 mm

The above minimum ground clearances are applicable for transmission lines running in the open country. Extra allowance over this may be made to provide for creep.

The minimum clerance of conductors over rivers is specified as 3,050 mm over maximum flood level for rivers which are not navigable[5]. For navigable rivers, clearances are fixed in relation to the tallest mast in consultation with the concerned navigating authorities. In case the power line crosses over a telephone line, the minimum clearance between the conductors of the power line and telecommunication wires are specified as follows[5] -

66 kV	-	2,440 mm
132 kV	-	2,745 mm
220 kV	-	3,050 mm
400 kV	-	4,880 mm

Between power lines upto 220 kV, crossing over another power line of any other voltage upto 220 kV, the clearance shall not be less than 4,550 mm, between 132 kV the clearance is 2,750 mm and for 66 kV line the figure is 2,440 mm. For 400 kV, the clearance may be assumed as 6,000 mm tentatively[5].

The minimum height above rail level of the lowest portion of any conductor under condition of maximum sag are as follows[5], in accordance with the regulations for Electrical Crossings of Railway Tracks, 1963.

Table 2.1

For Unelectrified Tracks or Tracks Electrified on 1500 volts D.C.

Voltage	Broad Gauge		Metre and Narrow Gauge	
	Inside station limits (mm)	Outside station limits (mm)	Inside station limits (mm)	Outside station limits (mm)
66 kV	10,300	7,900	9,100	6,700
132 kV	10,900	8,500	9,800	7,300
220 kV	11,200	8,800	10,000	7,600
400 kV	13,600	11,200	12,400	10,000

Table 2.2

For Electrified Tracks on 25 kV A.C.

Voltage	Broad, Metre and Narrow Gauge	
	Inside station limits (mm)	Outside station limits (mm)
66 kV	13,000	11,000
132 kV	14,000	12,000
220 kV	15,300	13,300
400 kV	16,300	14,300

No conductor of an extra high voltage overhead line crossing a tramway or trolley bus using trolley wires should have a clearance less than 3,050 mm above the trolley line.

2.4. Length of Insulator Assembly

For selecting the number of insulator discs in one insulator string, the under-noted factors have been considered for voltages upto 220 kV.

The insulator string selected should have adequate impulse strength so as to prevent trip out due to back flashover during lightning strokes. This consideration has usually given adequate insulation strength as well as adequate air gap for power frequency over voltages. One or two discs have been usually allowed extra so that in case of the failure of a disc during service, the insulator string maintains the designed impulse strength.

The tower inductance is usually neglected in computing back flashover voltage for voltages upto 220 kV[17].

At 400 kV the switching surges become more critical. It has been found desirable to reduce the per unit switching surge for 400 kV system to a maximum level of 2.5 p.u., and design the insulation of transmission line for this value. This insulation is sually found adequate for temporary over voltages at power frequency and also atmospheric over-voltages.

In designing the external insulation for a transmission tower, advantage is taken of the possibility that the maximum over voltage occurs rarely, and the lower possibility of its coinciding with the assumed withstand and unfavourable weather conditions. Thus the design is based on a given risk of flashover which can be calculated by combining the flashover voltage distribution function of an insulation structure (in this case the insulator string) with the over voltage probability density function. The latter gives the probability $P_o(U)dU$ of a surge of value between U and $U + dU$ occurring. Multiplication of the probability $Pd(U)$ of flashover at this voltage gives the possibility of both events occurring simultaneously. The integral over the whole voltage range is the risk of failure represented by the hatched area in Fig.2.2. If a number of strings are subjected to the same switching surge, the distribution $Pd(U)$ has to be adjusted according to the equation

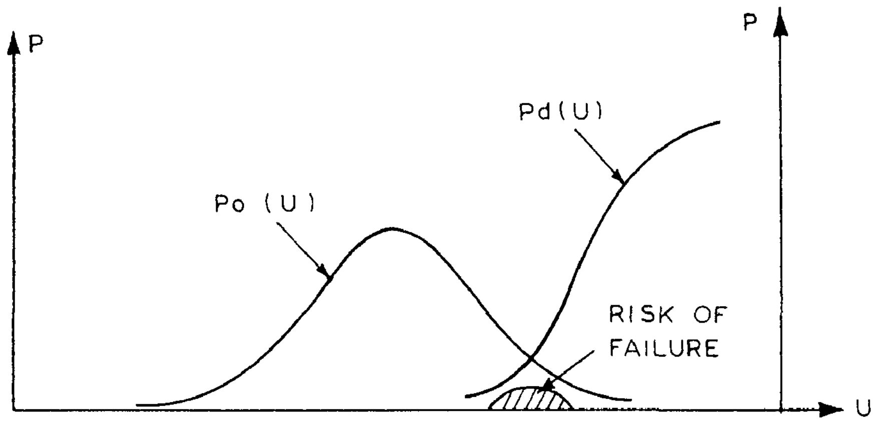
$$q_n = q_1^n \tag{2.1}$$

where

q_1 = Probability of flashover of one string

q_n = Probability of flashover of n strings in parallel

According to the above statistical methods, the insulation is selected in such a way as to obtain a calculated probability of failure lower than, or equal to, a pre-established value that characterizes the required safety level. The usual acceptable risk of switching surge flashover index, i.e. flashover per number of circuit breaker operations per circuit, is between 50-200. The choice of exact number will depend on the importance of a line in a system[17].



RISK OF FAILURE EVALUATION

FIG. 2·2

Application of the rigorous statistical method to insulation design as explained above is however often laborious without use of digital computer. A simplified statistical method is therefore used where the risk of failure is **correlated** with the margin between the **Statistical** Over-voltages (U_o) and the Statistical Withstand Voltage(U_w). The voltages U_o and U_w correspond to given reference probailities on the two probability functions, that are the probability of occurrence of the overvoltages and the probability of flashover respectively. The ratio between the voltages U_w and U_o is defined as the Statistical Factor of Safety (Y). The given reference probabilities for the two voltages as per the I.E.C. standard are, U_o having only a 2 percent probability of being exceeded and U_w having a 10 percent probability of failure. Once the correlation between the statistical safety factor and risk of failure is established for these given probabilities, the Statistical Withstand Voltage (U_w) can be determined by the equation -

$$U_w = U_o \times \text{ Statistical Factor of Safety} \qquad (2.2)$$

The 50 percent Critical Flash Over value can then be obtained [12] from the relationship -

$$\text{50 percent C.F.O. voltage} = \frac{U_w}{(1-1.281\,\sigma)} \qquad (2.3)$$

Where, σ is the standard deviation of the reclosing overvoltages, which follows a Gaussian distribution, and has a value of .06 for switching surge.

The number of insulator discs and the length of insulator string are then finalised depending upon this 50 percent C.F.O. value and the size and rating of each disc.

The present practice in this country is however to provide a standard length of insulator strings with disc size of 255 mm x 145 mm for single and double suspension strings depending upon different transmission voltages, irrespective of the magnitude of switching surge anticipated[5].

The following table gives the length of insulator strings and number of discs normally used at present for various voltages[5].

Table 2.3

Voltage	Suspension String				Tension string		
		No. of discs	Length mm	Hanger mm		No.of discs	Length mm
66 kV	S/S	5	965	75	S/T	6	1,070
	D/S	2x 5	1,255	-	D/T	2x6	1,575
132 kV	S/S	9	1,630	445	S/T	10	1,820
	D/S	2 x9	1,915	450	D/T	2x10	2,175
220 kV	S/S	14	2,340	350	S/T	15	2,915
	D/S	2x14	2,640	-	D/T	2x15	3,345
440 kV	S/S	21	3,740	350	D/T	2x22	4,700

S/S = Single Suspension D/S = Double Suspension

2.5. The Air Gap Clearances

The air gap clearance refers to the minimum distance which must be maintained between the live conductor and the earthed metal parts of the tower to avoid flashover between them. The minimum air gap clerances under different conditions of overvoltages can be computed. As stated under the procedure for determining the length of insulator assembly, although the atmospheric overvoltages are the governing criteria for transmission lines upto 220 kV, overvoltages due to the switching surges become critical at and above 400 kV. The air gap clearance has therefore to be designed to withstand switching surges. The probabilistic approach explained earlier can also be applied for this purpose. In order to determine the air gap clerance the Statistical Withstand Voltage (U_w) for the specific surge, either lightning or switching has to be determined first as in the previous case. The air gap clearance can then directly be obtained[12] from the equation -

$$d^n = \frac{U_w}{500\, K\, (\, 1-1.3\, \sigma\,)} \qquad (2.4)$$

Where,

d	=	clearance in metres
n	=	1 for lighting and 0.6 for switching
U_w	=	switching/lightning impulse withstand voltage
K	=	gap factor depending upon the electrode configuration
σ	=	standard deviation (.06 for switching and .03 for lightning).

The above minimum air clearance has to be maintained under the conditions of system over voltages with the insulator strings in the deflected position due to the action of wind pressure. Assumptions regarding swings of insulator strings made in the design of transmission line towers vitally affects the tower weights and, it is therefore necessary to make realistic assumptions in this regard consistent with acceptable norms of line performance.

No uniform practice has been followed in this country. For instance, the insulator swings, for reckoning clearance to tower steel work are specified without reference to the wind pressure regions in which the line is to be erected, and this has resulted in the adoption of the same swings for all lines of the same voltage class, using the same size of conductor in widely varying wind pressure regions.

It has been estimated that the tower weights can be reduced by about 5 percent for 220 kV single circuit tower[4] if the angle of insulator swing is reduced from 45° to 35°. While lower air clearances will adversely affect the full utilisation of the line insulation provided, unnecessary large air-clearances will mean longer cross-arms and some what heavier towers. A proper co-ordination between the air clearances and insulator swings is therefore essential.

The problem of fixing up a specific angle of suspension insulator swing for which a suspension tower should be designed, is fairly complex as the computations involve a number of variables. The angle of swing depends upon the following factors -

i) Transverse wind velocity

ii) Diametre of conductor

iii) The weight of conductor

iv) The angle of line deviation and conductor tension corresponding to the wind pressure on the conductor.

The swing angle can be expressed[4] by the following relationship -

$$\text{Tan } \theta = \frac{2\ T \sin \frac{\phi}{2} + HW_h + \frac{1}{2} \text{(wind load on insulator)}}{V.W_v + \frac{1}{2}\ W_i} \tag{2.5}$$

where,

θ	=	Angle of insulator swing from vertical
T	=	Maximum working tension of the conductor, corresponding to the wind pressure and temperature conditions
ϕ	=	Angle of line deivation
W_h	=	Wind load per unit length of conductor
H	=	Wind span
V	=	Weight span
W_v	=	Weight per unit length of conductor
W_i	=	Weight of insulator string

The air clearances between the conductor and the earthed steel work corresponding to the three overvoltages have to be co-ordinated with the corresponding deflected position of the insulator string. The problem therefore lies in foreseeing, as far as possible, realistic atmospheric conditions and the simultaneous occurrence of system overvoltages.

Statistics have revealed that when the insulator string is in its maximum deflected position (which theoretically occurs at maximum wind velocity) there is more possibility of occurrence of Power Frequency overvoltage[4]. The switching over-voltage usually occurs under reduced wind velocity conditions, that is 50 percent to 70 percent of the maximum wind velocity. The occurrence of lightning impulse flash over is generally accompanied by wind velocity much less than that occurring under steady conditions. It would be reasonable therefore to provide for air gap clearance under maximum insulator swing conditions corresponding to P.F. overvoltage. Further, statistics also show[4] that switching overvoltages usually occur in the system simultaneously under wind pressure conditions, which are less stringent than maximum wind conditions. The reduced wind velocity may correspond to about 50 percent to 70 percent of the maximum wind velocity. The clearance for switching overvoltage is therefore to be provided with respect to the angle of swing under corresponding reduced condition. Experience has also shown that occurrence of lightning impulse flash over is generally accompanied by wind velocity much less than that occurring under steady wind conditions.

Keeping the above in view, the authors of the article in ' Electrical India '[4] have tried to calculate the insulator swing on a more realistic basis and coordinate the same for the three overvoltage conditions. The swings for different wind pressure regions have been worked out for various conductors usually adopted for 66 kV, 132 kV, 220 kV, and 400 kV lines. A slightly conservative assumption has been made in coordinating the wind velocities with the conditions of overvoltages for the initial stages. A figure of 70 percent of the maximum wind velocity for working out insulator swings for reckoning clearances from switching surge considerations and 50 percent of maximum wind velocity for working out the swings under atmospheric overvoltage condition is chosen. The swings under the maximum wind velocity on the conductor is coordinated with the clearances required under power frequency overvoltages, particularly for 400 kV.

Presently the practice is to specify two swing angles and the corresponding electrical clearances under these deflected position of the strings. The swing angle is assumed to be same, irrespective of the wind pressure zone or conductor size.

The swings and clearances normally adopted at present for 66 kV to 400 kV transmission lines are given in the following tables[5].

Table 2.4

Suspension Insulator Strings

Sl.No.	Line voltage kV	Assumed value of swing of suspension string from vertical (degrees)	Min.clearance specified (mm)
i.	66	30	760
		45	610
ii.	132	30	1,525
		60	1,070
iii.	220	20	1,980
		35	1,400
iv.	400	22	3,000
		45	1,860

Table 2.5

Jumper on Tension tower

Sl.No.	Line voltage kV	Assumed value of swings of jumper from vertical (degrees)	Max.clearances specified (mm)
i.	66	10	760
ii.	132	10	1,525
		30	1,070
iii.	220	15	1,980
		30	1,400
iv.	400	18	3,000
		36	1,860

2.6. Mid Span Clearance between Conductor and Groundwire

In case of direct lightning stroke on the mid-span of overhead earthwires, the potential of the mid-span is built up during the duration of propagation of surge current, and the mid-span flashover may occur from groundwire to conductor before the current is discharged through the tower. The mid-span clearance between the earthwires and conductors is, therefore, kept more than the clearance at the tower. The usual practice in this regard is to maintain the sag of ground wire at least 10 percent less than that of the power conductor under all temperature conditions in still wind at the normal spans, so as to give a mid-span separation greater than at the supports[5]. It is, however, ensured that under minimum temperature and maximum wind conditions, the sag of the ground wire does not exceed the sag of power conductor. It is under the minimum temperature condition that the problem of coordination of conductor and groundwire sags generally arise, whereas sufficient sag differential is available at maximum or other temperature conditions. It is the practice therefore to assume the same temperature and wind pressure conditions for working out the sags for the groundwire as those adopted for power conductors, although the maximum temperature of the groundwire attainable is less. The table 2.6 for example gives the coordinated sags of power conductors and groundwire[5]. The power conductor is ' Panther' 30/3 mm Al + 7/3 mm st. (IS : 398) and the groundwire is 7/3.15 mm, 110 kg/mm^2 quality(IS: 2141).

Table 2.6

Coordination of Power Conductor and Groundwire Sags

Condition	Conductor		Earthwire		Differential Sag
	Tension (kg)	Sag (mm)	Tension (kg)	Sag (mm)	$=(\frac{3-5}{5})$x100
1	2	3	4	5	6
Min.temp.+ Max.wind	3,882	6,690+ 3,530++	1,870	6,220+ 3,200++	7.03
Min.temp.+ Still wind	3,090	4,830	1,380	4,340	10.0
Everyday temp. + still wind	2,282	6,000	1,175	5,100	15.0
Max. temperature + still wind	1,938	7,060	1,030	5,730	18.8

\+ Loaded incline sag

++ Vertical sag

In the case of stroke to mid-span on one of the earthwires when two earthwires are used, it is preferable, if the stricken earthwire flashes over to the second groundwire instead of to the conductor. For this, it is necessary that the spacing between the two earthwires is less than the mid-span clearances between earthwires and conductors. Mid-span clearances vary with the span lengths. Increased spans increase the mid-span clearances.

The design spans normally adopted are 250 m for 66 kV, 305 to 335 m for 132 kV, 350 m for 220 kV and 350-400 m for 400 kV lines[5]. The following are the vertical clearances generally permitted at the middle of the span between the earthwire and the conductors.

Table 2.7

Span (m)	Vertical clearance permissible at the middle of the span (mm)
200	4,000
300	5,500
400	7,000
500	8,500

These figures are based on the rate of rise of 50 kA/micro-second of lightning current with a probability of flashover of 3 percent, surge impedance of the groundwires as 400 ohms and coupling coefficient as 0.25. Further, it is the practice to keep the angle of shield lower at the mid-span than that provided at the tower[5].

2.7. Nature of Shielding

Earthwire provides protection against direct stroke of lightning. It intercepts the direct lighting strokes and conducts the charge to the nearest ground connections. The location of the groundwire/groundwires determines the height of groundwire peak portion or the earth cone of the tower. The height and location of overhead groundwires shall be such that the line joining the groundwire to the outer most conductor shall make angles of approximately 20 to 30 degrees with the vertical. This angle is called the shield angle. The smaller the angle, the better the shielding provided. The practice is to specify 30 degrees for 66 kV and 132 kV lines and 25 or even 30 degrees for 220 kV lines. A lower angle of 20 degrees is suggested for 400 kV lines. The protective value against direct strokes to the phase conductors approaches 100 percent if the protective angle is less than 20 degrees, but it may not be advisable to keep smaller angles from economic considerations. The shield angle is to be taken from outer conductor of the bundle in the case of 400 kV lines. The position of the conductors with reference to which, groundwire position is determined is fixed depending upon the length of the insulator strings to which the conductors are attached, their swings and the corresponding electrical clearances etc., as already discussed.

On extra high voltage lines having wide conductor spacings, the use of two earthwires may provide more adequate protection. Two earthwires are also superior in that they have a lower surge impedance and the coupling effect with the conductors is increased. In case of two earthwires, the protective zone between two earthwires forms a semicircle with the line connecting the two earthwires forming the base diameter. Therefore the height of the middle conductor below the earthwire plane should be more than half the spacing between the two earthwires. Having fixed the angle of shield, the location of the groundwire is fixed and height of groundwire peak portion is determined.

Figure 2.3. gives the relationship of the probability of shield failures and the shielding angle between groundwire and top phase conductor for tower of different heights[17].

2.8. The System of Bracing Patterns

Various types of bracing patterns are adopted for transmission line towers. A few of these shown in Fig.2.4. are taken from the ' Manual on Transmission Line Tower' [5] for discussion.

Single Web System

It comprises either diagonals and struts or all diagonals, as shown in Fig.2.4(a) and Fig.2.4(b) respectively. In diagonal and strut system, struts are designed in compression, while diagonals in tension, whereas in a system with all diagonals the members are designed both for tensile and compressive load to permit reversal of the applied external shear. This system is particularly used for narrow base

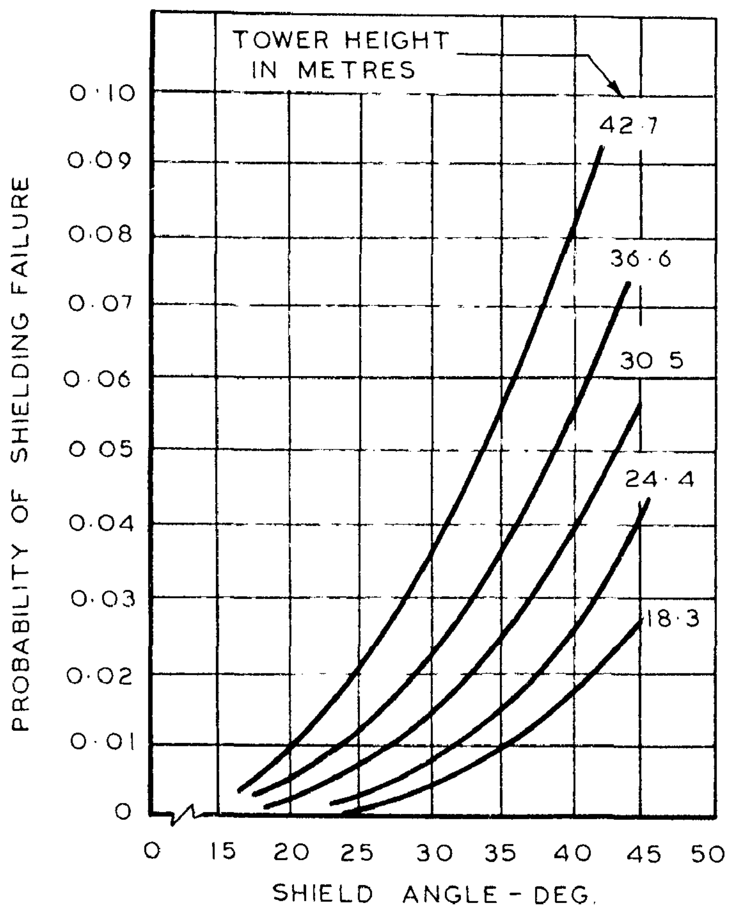

TOWER HEIGHT
IN METRES
42·7
36·6
30 5
24·4
18·3
0·10
0·09
0·08
0·07
0·06
0·05
0·04
0·03
0·02
0·01
0
PROBABILITY OF SHIELDING FAILURE
0
15
20
25
30
35
40
45
50
SHIELD ANGLE - DEG.

FIG. 2·3

towers, in cross arm girders, and for portal type towers. Except for 66 kV single circuit line towers, single web system has little application for wide base H.V. and E.H.V. towers. It is preferable to keep the four faces identical in case of 66 kV single circuit tower using single web system as it results in lighter leg member sizes.

Double Web or Warren System

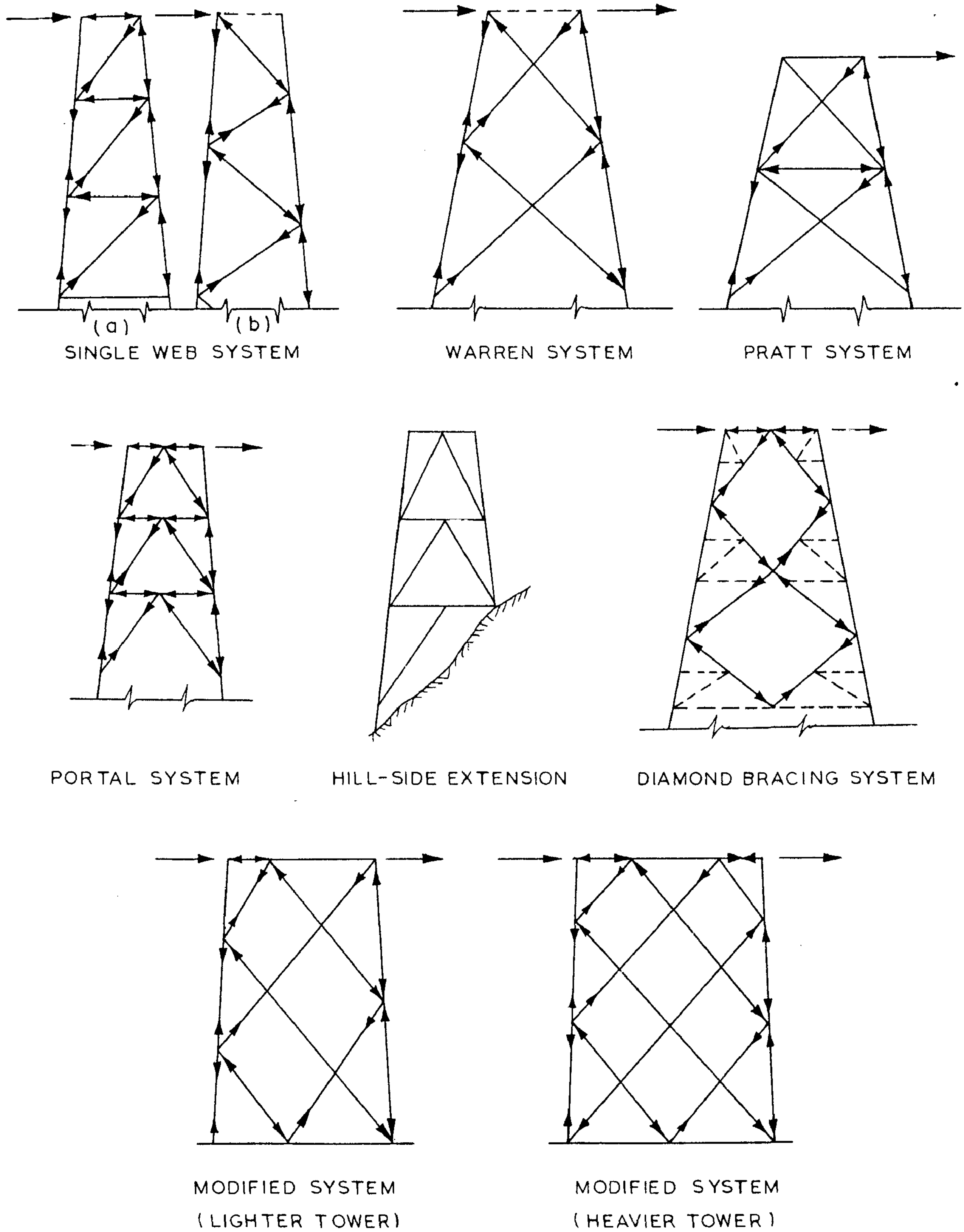
This system is made up with diagonal cross-bracings. Shear is equally distributed between the two diagonals, one in compression and the other in tension. Both diagonals are designed for tensile and compressive loads in order to permit reversal of externally applied shears. The diagonal braces are connected at their cross points. The tension diagonal gives an effective support to the compression diagonal at the point of their connections, and reduces the unsupported length of bracings which results in lighter size of bracing members. This tension-compression system works well where the lateral dimensions of the tower are not too great with respect to the tower loads. This system is used for both large and small towers and can be economically adopted through out the shaft and pedestal of suspension and small angle towers and also in wide base large towers except in lower one or two panels, where diamond or portal system of bracing is more suitable These bracings result in better distribution of loads to the tower footings, and reduces stress in the tower legs, especially that brought on by torsional loads.

Pratt System

This is a tension system in which all the diagonal members have a slenderness ratio high enough to act only in tension. Horizontal members at the junction of each tension, X, are designed for compression. Advantage of this system is that the sizes of diagonal members would be small, because these are designed for high slenderness ratio. This type of bracings result in larger deflection of tower under heavy loadings, because the tension members are smaller in cross-section than compression members would be for similar loading. If such a tower is over loaded, the inactive diagonal will fail in compression due to large deflection in the panel although the active tension member can very well take the tension loads. This system of bracing imparts torsional stresses in leg members of the square based tower and also result in unequal shears at the top of four stubs for the design of foundation.

Portal System or K- Bracings

In this system of bracings the diagonals are designed for both tension and compression. It is stiffer than Pratt system, and has the advantage that the horizontal struts are supported at mid-length by the diagonals and the same are correspondingly smaller than that in Pratt system.



VARIOUS TYPES OF BRACING SYSTEMS
FIG. 2.4

Used for approximately same size of panels as Pratt bracing, portals are used in conjunction with Warren system of bracings. Used when it is desirable to provide clearance between the bottom legs of a tower. It has been found advantageous to use the portal system for bottom panels, extensions, and heavy river crossing towers when rigidity is a prime consideration . If hill-side or corner extensions are anticipated, the portal panel is particularly attractive due to its versatility of application.

Diamond Bracing System

Somewhat similar to the Warren system, this bracing arrangement can also be derived from the portal system by inverting every second panel. As for each of these systems, all diagonals are designed for tension and compression and share the web shear. Applicable to panels of approximately same size as the Pratt and Portal systems, this arrangement has the advantage that the struts carry no primary loads and are designed as redundant supports.

Modified System of Bracings

In H.V./E.H.V. towers where the torsional loads are of high magnitude, the top hamper width is kept large to resist the torsional loads. Standard Warren system, if used there, gives longer unsupported lengths of legs and bracings which increases the weight of tower disproportionately.

For such towers modified system of bracings are used as shown in Fig.2.4. The advantage of the system is that the unsupported lengths of the leg members are reduced substantially thereby increasing their strength and reducing the member sizes. Although there is an increase in the number of bolts, fabrication and erection cost, yet the above system gives overall reduction in weight and cost of steel.

The bracings on the transverse and longitudinal faces may be staggered as slight reduction in tower weight is achieved by this. The system is preferable only for suspension and medium angle towers. In heavy angles and dead end towers, bracings on transverse and longitudinal faces may not, however, be staggered in order to have more rigidity.

2.9. Constituents of an Outline Diagram

Deciding the tower configuration means determining the various dimensions of the tower which define its outline. These dimensions or the constituents of an outline diagram in respect of a transmission line tower are as follows -

a) Tower height considered from ground level
b) Length of the cross-arms, and phase, spacings
c) Tower widths at (i) base and (ii) top hamper of the pedestal.
d) Bracing pattern.

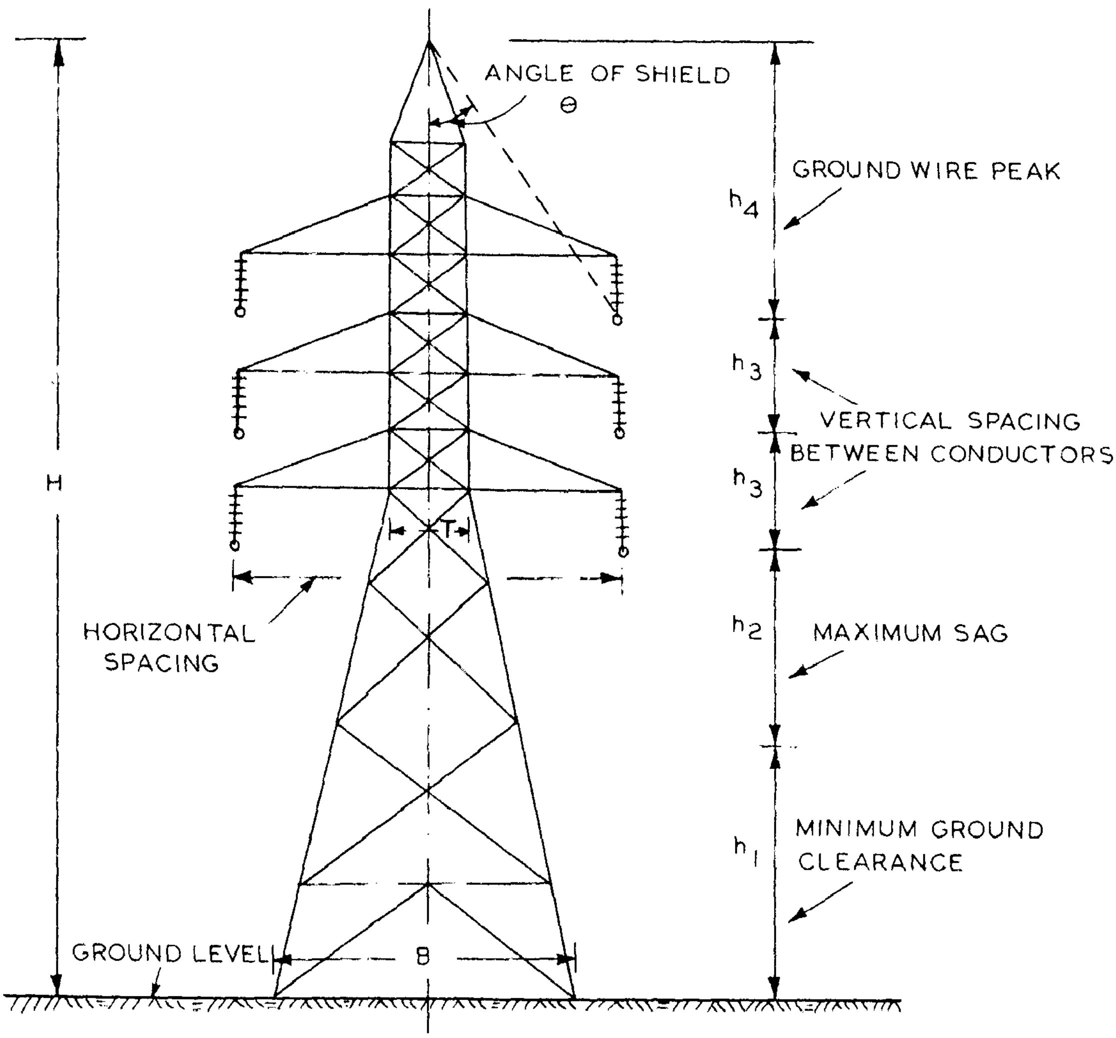
All the above dimensions are to be decided, keeping in view the various electrical and structural requirements listed earlier. The exact procedure followed in determining each of the dimension is explained as follows -

The Tower Height

The height of a tower (H) in level country comprises the permissible ground clearance of conductors required in accordance with overhead line regulations, (h_1), maximum sag for the lower most conductors (h_2), vertical spacing between conductors including maximum insulator string length (h_3), and height of the groundwire peak portion (h_4). These dimensions are shown in Fig.2.5. The permissible ground clearance (h_1) can be determined as explained earlier. The maximum sag for the lower most conductor is found out by the sag calculations. The sag at maximum temperature and still air condition is to be taken as (h_2). The procedure of sag calculations has been dealt with in detail in Chapter III.

For determining the vertical spacings between two cross-arms (h_3), the procedures are different for suspension towers and angles towers, and are reproduced here from the 'Manual on Transmission Line Towers'[5].

In the case of suspension towers the approximate value of the angle between the lower main members and the upper inclined members of the cross-arms is assumed.



CONSTITUENTS OF A TOWER OUTLINE DIAGRAM

FIG. 2·5

This also determines the heights of panels between two cross-arms in the main tower body. For E.H.V. lines, i.e. 220 kV and 400 kV lines, the angle between the lower main member and the upper inclined member (α) of the cross-arm generally lies between 20° to 25° (20° being the minimum preferable angle between any two members common to a joint of trussed frame in order to avoid any uncertainty of the amount of stress distribution between two closely spaced members) and that for 66 kV and 132 kV lines this angle lies between 20° to 30°. The distance between the conductor and the inclined member of the lower cross-arm is minimum when the line drawn from the conductor in its deflected position to the inclined member is perpendicular. This perpendicular makes an angle α with vertical, the angle (α) equals the angle between the inclined member and lower main member of the cross-arm. In case angle α (between lower main member and inclined member) is less than θ_1, the insulator string should deflect by an angle α, in order to have minimum vertical spacing between cross-arms. Allowance is made for flange width of plan redundants connected to the bottom of the lower main members of the cross-arm and at the top of tie members and also for the centre of gravity of lower main members and top inclined members of the cross-arm while computing vertical spacings.

As shown in Figure 2.6, the minimum vertical spacing between the two adjacent cross-arms(h_3) depends upon the magnitude of α with relation to θ_1 and θ_2.

Case I

When $\alpha < \theta_1 < \theta_2$

θ_1 = the insulator swing from vertical at reduced wind velocity

θ_2 = the insulator swing from vertical at full wind velocity

The minimum vertical spacing between two adjacent cross-arms, $(h_3) = H_g + b + h$ (2.6)

Where,

H_g = Height of hanger

$= (x_2 + B + C) - S.\ \text{Cos}\ \theta_2$ (2.7)

or $= (x_1 + B + C) - S.\ \text{Cos}\ \theta_1$ (2.8)

whichever is greater.

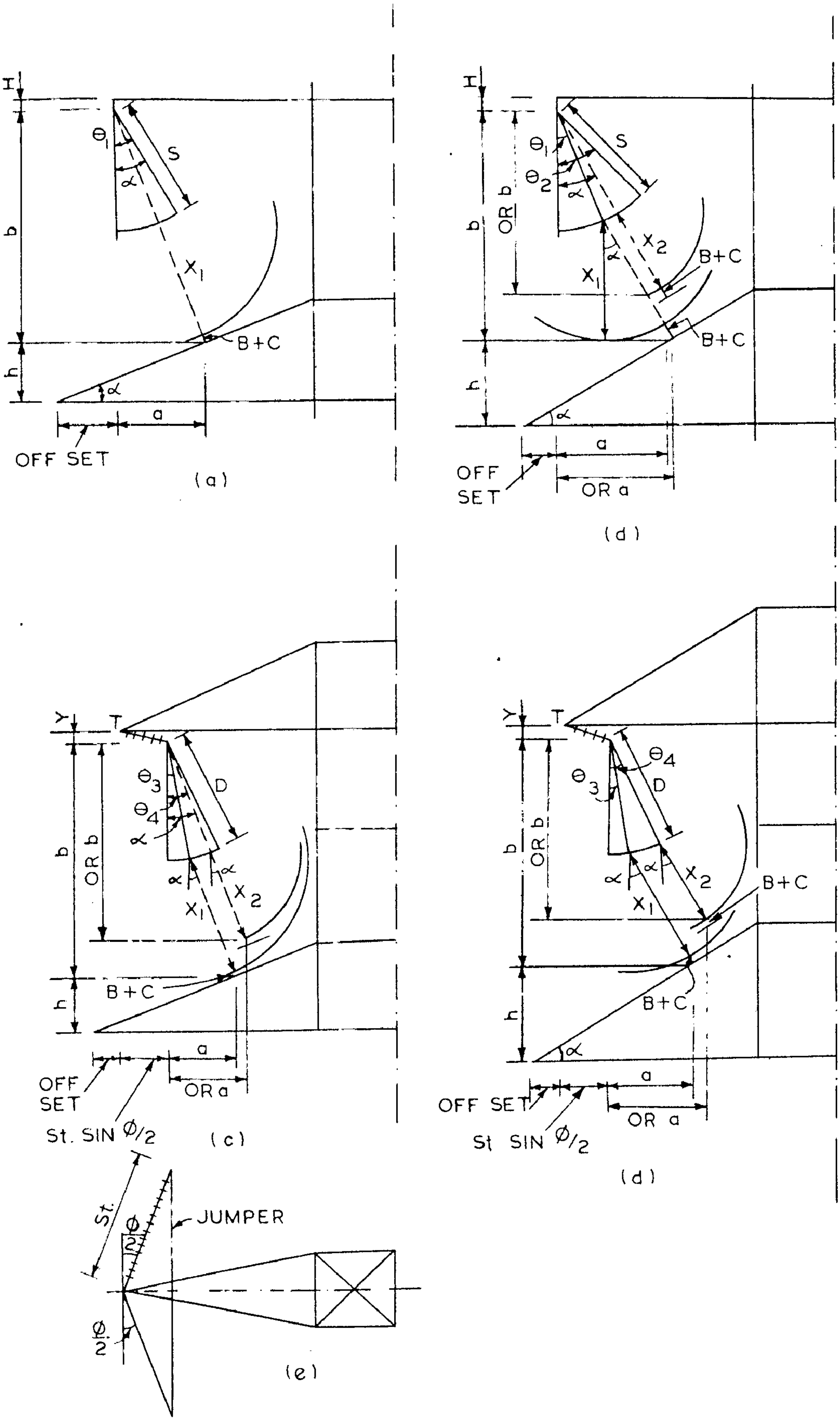
and x_1 = air clearance corresponding to θ_1

x_2 = air clearance corresponding to θ_2

B = Flange width of the nearest projecting angle section connected to the main angle members

C = Distance of centre of gravity of the main angle section from the heel of the angle

S = Length of the insulator string.



CROSS-ARM AND CONDUCTOR SPACINGS

FIG.2·6

Now, $b = (S + x_1 + B + C) \cos\alpha$ (2.9)

$h = (a + \text{offset between two cross arms}) \tan\alpha$ (2.10)

where $a = b \tan\alpha$ (2.11)

Substituting the value of b from equation 2.9 we get

$$a = (S + x_1 + B + C) \cos\alpha \cdot \tan\alpha$$

$$= (S + x_1 + B + C) \sin\alpha \qquad (2.12)$$

$$\therefore h = \left[(S + x_1 + B + C) \sin\alpha + \text{Offset} \right] \tan\alpha \quad (2.13)$$

Hence h_3 = equations 2.7 + 2.9 + 2.13

or = equations 2.8 + 2.9 + 2.13, whichever is greater.

<u>Case II</u>

When $\theta_1 < \alpha < \theta_2$

The value of h_3 can still be found out with the help of equation 2.6, where

$$b = S.\cos\theta_1 + (x_1 + B + C) \cos\alpha \qquad (2.14)$$

or $= (S + x_2 + B + C) \cos\alpha,$ (2.15)

whichever is greater

and, $h = (a + \text{offset}) \tan\alpha$, where

$$a = S.\sin\theta_1 + (x_1 + B + C) \sin\alpha \qquad (2.16)$$

or $= (S + x_2 + B + C) \sin\alpha$ (2.17)

The value of 'a' obtained from the equations 2.16 and 2.17 should be chosen, which corresponds to the greater value of 'b'.

In suspension towers, angle α is not generally greater than θ_2, the third case is therefore not considered.

In the case of angle towers, the angle between the cross-arm lower main member and upper inclined member is always more than the swing of the jumper from vertical under reduced wind velocity (θ_3).

Case I

When $\theta_3 < \alpha < \theta_4$

where θ_4 = the swing of the jumper under full wind velocity

$$h_3 = Y + b + h \qquad (2.18)$$

where Y = Depth of jumper terminal point below cross-arm level, and is given by the following relation.

$$Y = (\text{Max. sag of conductor x St.})/\text{Half span} \qquad (2.19)$$

Where, St = Length of the tension string

$$b = D.\ \text{Cos}\ \theta_3 + (x_1 + B + C)\ \text{Cos}\ \alpha \qquad (2.20)$$

$$\text{or} = (D + x_2 + B + C)\ \text{Cos}\ \alpha \qquad (2.21)$$

whichever is greater

here D = Depth of jumper.

The depth of jumper is selected in such manner that maximum specified clearance is maintained at all points along the jumper profile. This value is normally 10 percent more than the specified maximum clearance of conductor to earthed steel.

$$\therefore D = 1.10\, x_1 \tag{2.22}$$

$$h = (a + St.\ \mathrm{Sin}\ \frac{\phi}{2} + \text{offset})\ \tan\alpha \tag{2.23}$$

where

$$a = D.\ \mathrm{Sin}\ \theta_3 + (x_1\ B + C)\ \mathrm{Sin}\ \alpha \tag{2.24}$$

$$\text{or} = (D + x_2 + B + C)\ \mathrm{Sin}\ \alpha \tag{2.25}$$

The value of 'a' should correspond to the greater value of 'b'.

and ϕ = The angle of line deviation.

Case II

When $\theta_3 < \theta_4 < \alpha$

$$b = D.\ \mathrm{Cos}\ \theta_3 + (x_1 + B + C)\ \mathrm{Cos}\ \alpha \tag{2.26}$$

$$\text{or} = D.\ \mathrm{Cos}\ \theta_4 + (x_2 + B + C)\ \mathrm{Cos}\ \alpha \tag{2.27}$$

whichever is greater.

$$h = (a + St.\ \mathrm{Sin}\ \frac{\phi}{2} + \text{offset})\ \tan\alpha \tag{2.28}$$

$$a = D.\ \mathrm{Sin}\ \theta_3 + (x_1 + B + C)\ \mathrm{Sin}\ \alpha \tag{2.29}$$

$$\text{or} = D.\ \mathrm{Sin}\ \theta_4 + (x_2 + B + C)\ \mathrm{Sin}\ \alpha \tag{2.30}$$

The value of 'a' should correspond to the greater value of 'b'.

The spacings are determined both corresponding to single suspension/ double suspension string and single/ double tension string. It is preferable to make an allowance for additional distance of 50 mm while determining the final spacings to account for any slight deviations from the values of B and C assumed above. The spacings can also be determined graphically besides the analytical method explained above.

Now comes the last section, that is the groundwire peak (h_4) which has to be determined for arriving at the total height of the tower. Height of the groundwire peak is determined essentially for the protection from direct lightning strokes, and the criteria is therefore the shielding angle. The procedure for determining the shielding angle is discussed earlier.

Length of Cross-arms

The horizontal spacing of conductors depends on the length of cross-arms and the width of the tower at the shaft portion. Length of the cross-arms is determined as follows.

In case of suspension towers, the length of cross-arms equals -

$$S.\ \text{Sin}\ \theta_1 + x_1 + B + C \qquad (2.31)$$

or

$$S.\ \text{Sin}\ \theta_2 + x_2 + B + C \qquad (2.32)$$

whichever is greater.

In case arcing horns are used on the line sides (in a plane parallel to that of the conductor), the clearances are also checked from the tip of the arcing horn. Twice the length of cross-arm so determined may give the width of window in the case of tower with horizontal formation. Allowance is also made in the horizontal distance corresponding to the sloping leg members, if any.

The length of cross-arm in the case of angle towers is determined from the swing of jumper and the corresponding electrical clearance from the tower member. This length equals -

$$St.\ \operatorname{Sin}\frac{\phi}{2} + D.\operatorname{Sin}\theta_3 + x_1 + B + C \qquad (2.33)$$

or

$$St.\ \operatorname{Sin}\frac{\phi}{2} + D.\ \operatorname{Sin}\theta_4 + x_2 + B + C \qquad (2.34)$$

The greater of the two values gives the length of cross arm considered from the point of attachment (T) to the centroidal line of the main leg members.

The horizontal spacing between conductors obtainable as above are adequate so as not to cause any arc over between them at the mid-span due to out of step swinging of conductors, if any, under dynamic conditions. In case of lines, running through snow bound areas, these spacings are checked from the consideration of galloping of conductors also, under specific condition of snow and wind. In case of barrel type of towers, where the conductors are hung one below the other in vertical plane, necessary off-setting to the extent of 600 mm between conductors is generally provided in order to avoid any possibility of conductor coming closer with each other under conditions of non-uniform shedding of ice from them and causing flashover.

Tower Widths at Base and Top Hamper

It is customary from the fabrication point of view, to have a constant batter from bottom up to the level of bottom most cross arm. The top widths of the tower remains the same as the width at the level of bottom most cross arms. The base width at the concrete level, depends upon the magnitude of the physical loads imposed upon the towers, and also depends upon the heights of application of external loads from ground level. Towers with large base width result in low footing costs and lighter main leg members at the expense of longer bracing members. This also decides the leg slope. The optimum slope generally lies between 1 on 5 to 1 on 12. The flatter slope is applicable for low dead end towers, while the steep slopes are applicable for light suspension towers or high river crossing towers[6]. Suspension towers with moderate loadings generally prove out to be economical with a leg slope of about 1 on 8.

Another formula gives the economical base width[5] of lattice tower proportional to the square root of over turning moment, that is

$$B = K\sqrt{M} \tag{2.35}$$

Where

B = Base width of tower at concrete level in centimetres.

M = The over turning moment in kg-m.

K = A constant

The value of K lies between 1.35 and 2.5 and 1.93 is a good average figure.

From the study of the total heights and base widths of several typical towers, it is observed in the ' Manual on Transmission Line Towers'[5], that the base width generally varies between 1/4 to 1/6 of the overall height of the tower up to concrete level, the values may be 1/6 for suspension tower, 1/5 for medium angle towers and 1/4 for heavy angle towers. It is suggested therein to use these simpler emperical relationship in working out the base width.

In medium and heavy angle towers for the bracings to carry minimum possible loads it is preferable to adjacent the leg slope in such manner that the legs when extended may meet at the line of action of the resultant loads.

Top hamper width is the width of the tower at the level of the lower cross arm in the case of barrel type of towers (in double-circuit towers, it is sometimes at the middle cross-arm level) and waist line in the case of Y towers. The width of top hamper is mainly decided by torsion loading. The torsional stresses are evenly distributed on the four faces of the square tower. The top hamper width is also decided in a manner that the angle between lower main member and the tie member of the same cross arm is not less than 20^{0}. The top hamper width is generally found to be about 1/3 of the base width for tangent and light angle towers and about 1/3.5 of the base width for medium and heavy angle towers[5]. For horizontal configurations, the width at the waist line is, however found to vary from 1/1.5 to 1/2.5 of the base width[5].

A variation of top and bottom widths usually results in variation of weight of the tower so that it is useful to try out a few combinations of top and bottom widths to obtain optimum weight of tower. Rest of the configurations and the bracings are established more or less in a conventional manner, and it is here that a designer can exercise his judgement to affect a reduction in weight of the tower. This is also one of areas in which computer can be effectively utilised to optimise the tower weight.

Bracing Pattern

Having determined all the basic dimensions of the tower it remains to pick a bracing system for the body. A suitable bracing system can be chosen from the different types of bracing systems already discussed.

CHAPTER-III

DETERMINING THE TOWER LOADS

3.1. Introduction

For analysis and design of an electrical transmission line tower, all external loads are to be worked out for the normal and various broken wire conditions. The IS: 802(Part I) 1973 specifies the basis for calculating various types of loads. The different broken wire conditions for which the towers are to be designed are also stipulated therein.

The data required by a tower designer for calculation of all the external loads are quite varied and they can be listed as follows -

i) The design spans - Normal span, Weight span and Wind span,

ii) The type of tower- Configuration and angle of deviation

iii) The climatic conditions - Maximum wind pressure, snow and ice conditions and range of temperatures,

iv) No., size and arrangement of conductors

v) No. and size of ground wires

vi) No. and type of insulators

3.2. Design Loads for Tower

The following types of loads are accounted for in the design of transmission line towers.

Transverse loads

The transverse loads on a tower are due to

i) Wind load on conductor and ground wires

ii) Wind load on supporting structure

iii) Transverse components of cable tension on angle towers, due to the line deviation.

Longitudinal Loads

The longitudinal loadson a tower are due to

i) Unbalanced pull due to broken conductor or ground wire

ii) Pull of all conductors and ground wires in the case of dead end towers

Vertical Loads

The vertical loads on a tower are due to -

i) Weight of the structure

ii) Weight of conductor and groundwire

iii) Weight of insulator and fittings

iv) Weight of ice coating if any

v) Weight of line-man with tools

Torsional Loads

Torsional loads are produced when there is unbalanced tension in the conductor on the two sides, for example, due to broken wire or due to dead ending of the conductor on single circuit lines and thus causing twisting moments about the central axis of the tower.

Eccentric Vertical Loads

The eccentric vertical loads arise in a tower under the following conditions -

i) Unequal loading in the case of arrangement of conductors on a single circuit tower

ii) Unbalanced vertical loads under broken wire conditions

Earthquake Forces

As transmission line towers are comparatively light structures and also that the maximum wind pressure is the chief criterion for their design, the earthquake forces can be neglected, considering that concurrence of earthquake and maximum wind pressure is unlikely to take place.

Temperature Stresses

Stresses in the structure due to variation of temperature need not be considered for structures of normal heights as stipulated in 'Manual on Transmission Line Towers'.

3.3. Calculation of Various Tower Loads

Wind Pressure Loads

On the basis of measured maximum wind velocities for different parts of the country, including winds of short duration as in squall, the country has been divided into three zones of low, medium and heavy wind pressures, in the IS : 802. The basic maximum wind pressure map of India, as given in the above I.S.S. has been reproduced here in

Fig.3.1. For each wind pressure zone the wind pressure stipulated for the structure is different from that of the conductors and earthwires.

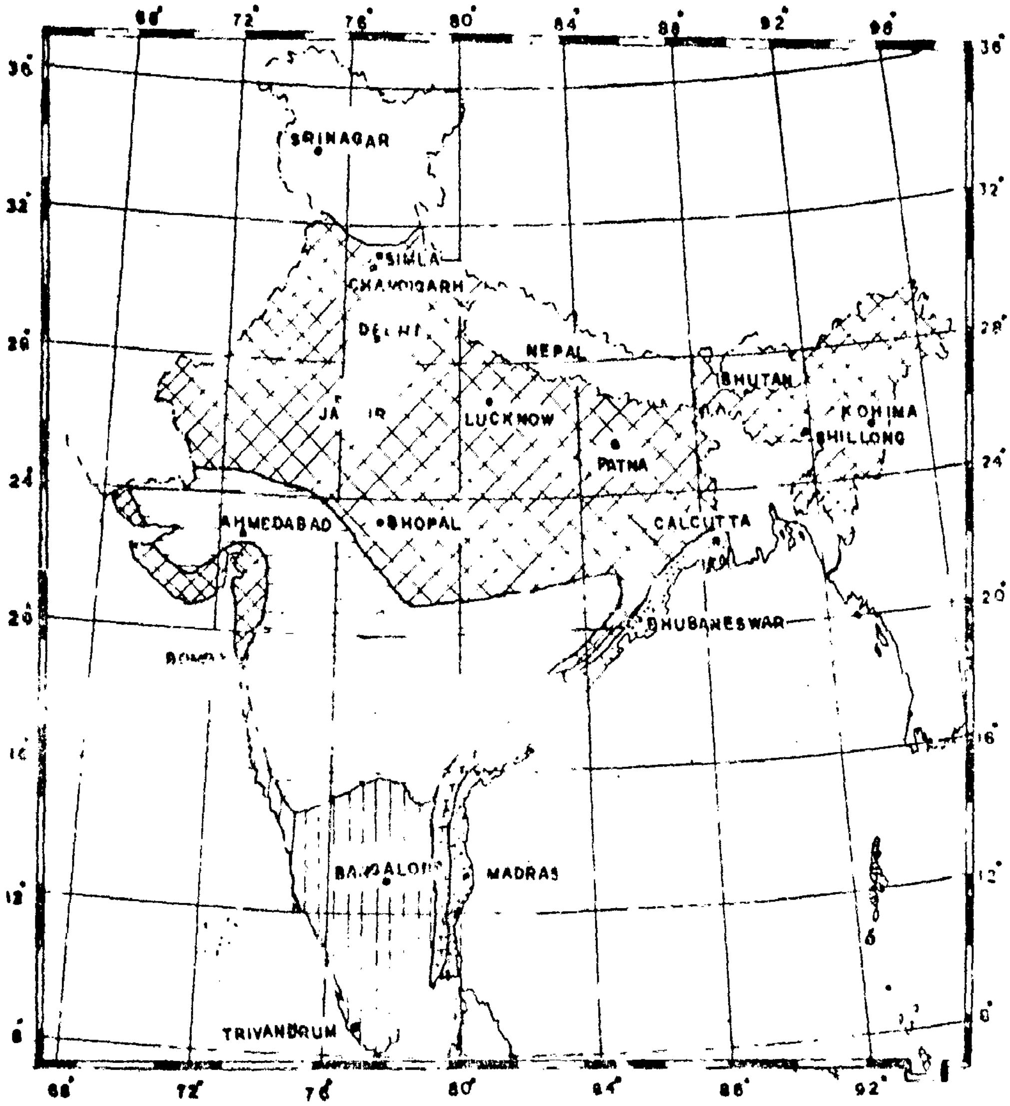
(a) Structure

In the case of towers the wind pressure is calculated on 1.5 times the projected area of the members on the wind-ward face. The calculation of loads due to wind on tower is one of the most time consuming and tedious jobs encountered in tower analysis. It is generally necessary to estimate the wind load with reference to old available design. In case the final windloads computed after the main member sizes and sail areas are fairly well established, are in much variance from the assumed loads, the design is recast by taking the correctly computed wind load on structures.

(b) Conductor and Ground Wires

In the case of conductors and groundwires the appropriate wind pressure is assumed as acting on the full projected area, since the wind pressures given for the conductors and groundwires is different from that for the structures, in each zone. In case of bundle conductors the wind pressure given is assumed as acting on full projected area of each conductor in a bundle.

MAP OF INDIA



LEGEND

WIND PRESSURE		MIN. TEMP	ZONE
TOWER *	130 KG/M²	0°C	
CONDUCTOR	43 KG/M²	10°C	
TOWER *	195 KG/M²	0°C	
CONDUCTOR	45 KG/M²	7.5°C	
TOWER *	260 KG/M²	5°C	
CONDUCTOR	52 KG/M²		

* APPLICABLE FOR HEIGHTS UPTO 30 METRES ABOVE THE MEAN RETARDING SURFACE. REFER IS 802 (PART-I) - 1973 FOR HEIGHTS ABOVE 30 METRES.

FIG. 3·1

Under normal conditions, the transverse load due to wind on conductors and groundwires is calculated on the wind span. Under broken wire conditions, 50 percent of the normal span and 10 percent of the broken span is assumed as wind span. The wind span is the sum of the two half spans adjacent to the support under consideration, and is usually taken same as the normal design span for the tower.

(c) Insulator Strings

For the purpose of computation of wind load on insulation strings, the effective projected area of the string is taken as 50 percent of the projected area of the cylinder with the diameter equal to that of the insulator skirt. The pressure is taken as that applicable to the structures.

Dead Weights

These are all vertical loads which are calculated separately for the structures, the conductors and groundwires and the insulators etc.

(a) Structure

Initially the dead weight of the structure can be assumed with reference to available structure designs. In case the weight of the designed structure is not in much variance from the assumed weight, the design need not be modified, but if there is considerable variation from the assumed weight, the design has to be recast by considering revised weights.

(b) Conductors and Groundwires

The conductor weight on any tower is equal to the weight of the conductor and ice, if any, between the adjacent low points of the catenary or the weight span. The weight span is the horizontal distance between the lowest points of the conductor, on the spans adjacent to the tower. The ratio of the weight span to normal span or wind span for any tower is a function of the terrain. For moderately rolling terrain, the vertical span capability is about 50 percent more than the wind span. In rough terrain, this is increased to 100 percent or more.

Under brokenwire conditions the weight of conductors or groundwire is taken as 60 percent of that under the normal condition.

(c) Weight of insulator and weight of Line-man with tools

For each point of conductor attachment the weight of insulator is calculated depending upon the number of discs per string and the number of strings . As regards the weight of Line-man with tools, a provision of 150 kg. is usually made. This figure increases to 500 kg. in the case of 400 kV. towers.

Loads due to conductor and Groundwire Tensions

For calculation of these loads the maximum working tensions for the conductor as well as the groundwire has to be determined first by carrying out necessary sag-tension calculation.

Sag-Tension Calculation and Determination of Maximum Working Tension -

The size and type of conductor, wind and climatic conditions of the region and span length determine the conductor sag and tensions. Span length is fixed from economic considerations. The maximum sag for conductor span occurs at the maximum temperature and still wind conditions. This maximum value of sag is taken into consideration in fixing the overall height of the steel structures. In snow regions, the maximum sag may occur even at 0°C with conductors loaded with ice in still wind conditions. While working out tension in arriving at maximum sag, the following stipulation laid down, in I.E. Rules (1956) are to be satisfied.

i) The minimum factor of safety for conductors shall be 2, based on their ultimate tensile strength

ii) The conductor tension at 32°C (90°F) without external load shall not exceed the following percentages of the ultimate tensile strength of the conductor.

Initial unloaded tension	-	35 percent
Final unloaded tension	-	25 percent

In accordance with this stipulation, the maximum working tension under stringet loading conditions shall not exceed 50 percent of the ultimate tensile strength of conductors. Sag-tension computations made for final stringing of the conductors, therefore, must ensure that factor of safety of 2 and 4 are obtainable under maximum loading condition and every day loading condition, respectively.

Standard sag-tension parabolic equation considering the combined effect of elasticity and temperature is given[5] as -

$$f^2 \left[f - (K - E\alpha t) \right] = \frac{l^2 . \delta^2 . E . q^2}{24} \qquad (3.1)$$

Where,

f	=	working tensile stress of conductor in kg/cm^2
K	=	constant computed from initial temperature and wind pressure condition assumed.
E	=	Final modulus of elasticity in kg/cm^2
α	=	coefficient of linear expansion of conductor per degree centigrade
t	=	change in temperature = final temperature minus initial temperature in degree centigrade.
l	=	span length in metres
δ	=	weight of conductor/m/cm^2 = W/A $kg/m/cm^2$

where,

A	=	Cross-sectional area of conductor in cm^2
q	=	loading factor $= \sqrt{\dfrac{W^2 + P^2}{W^2}}$ (3.2)

where,

W = Weight of conductor in kg/m length of conductor

P = Wind load on conductor in kg/m length of conductor

If suffix '1' corresponds to temperature condition at minimum temperature with maximum wind, '2' to every day temperature with still wind condition and '3' to maximum temperature in still wind condition, the above sag-tension formulae are written as follows -

(a) Every day temperature 32°C in still wind

(Assumed as starting initial condition -Suffix 2)

$$f_2^2 \left[f_2 - (K - E\alpha t) \right] = \frac{l^2 \cdot \delta^2 \cdot E \cdot q_2^2}{24} \qquad (3.3)$$

Assume

$$\frac{l^2 \cdot \delta^2 \cdot E}{24} = Z, \text{ and initial temperature } t_2 = 32^\circ C$$

$$t = 0$$

$\therefore E \alpha t = 0, \; q_2 = 1$

$$f_2 = \frac{\text{Ultimate strength of conductor (Kg)}}{4 \text{ x cross-sectional area in } cm^2 (A)}$$

Substituting the above values in the equation and solving we get

$$K = f_2 - Z / f_2^2 \qquad (3.4)$$

(b) Minimum temperature with full wind (suffix 1)

$$f_1^2 \left[f_1 - (K - E\,\alpha\,(t_1 - t_2)) \right] = Z \cdot q_1^2 \qquad (3.5)$$

Value of K computed from (a) is substitutted in equation (3.5) and the cubic equation is solved for f_1 - the maximum tensile stress of the conductor.

If T_1 is the maximum tension at the minimum temperature with full wind

$$T_1 = f_1 A$$

$$\text{F.O.S.} = \frac{\text{T (Ultimate strength)}}{T_1}$$

Factor of safety is to be equal to or greater than 2.

(c) Maximum temperature and still wind (suffix 3)

$$f_3^2 \left[f_3 - (K - E\,\alpha\,(t_3 - t_2)) \right] = Z \cdot q_3^2 \qquad (3.6)$$

Value of K computed from (a) and $q_3 = 1$ are substituted in (3.6) and the equation is solved for f_3, the stress of the conductor under maximum sag conditions. Having determined the conductor tensile stress (f_3) under maximum temperature in still wind

$$\text{Maximum Sag} = \frac{q_3\,\delta\,l^2}{8\,f_3} \text{ metres} \qquad (3.7)$$

Except in snowy zones, the sag-tension computations (for S.C.A. conductor) may be started from the condition of every day temperature (32°C) in still wind with a factor of safety of 4, assuming this to be an initial condition. Computations show that corresponding factor of safety under worst loading condition at 32°C with full wind pressure or minimum temperature with 2/3 full wind pressure is always more than 2. Maximum sag is determined from the working stress computed under maximum temperature and still wind conditions after the above two stipulations are satisfied. For lines running in snowy zones, the maximum tension occurs under snow conditions. The maximum sag may perhaps occur at 0°C with conductors loaded under snow. For S.C.A. conductors with higher aluminium contents normally used for 220 kV and 400 kV lines, an increased sag of about 2 percent to 4 percent of the maximum sag values is allowed to account for conductor creepage and the likely sag differential arising due to the assumption of parabolic shape of the conductor span instead of catenary.

While calculating the maximum working tension of groundwire, the governing criterion is the requisite minimum mid-span clearance between the conductor and groundwire. Hence the coordination of sag is essential, especially at the minimum temperature as already discussed.

After the maximum working tensions for conductor and groundwire are calculated the transverse and longitudinal loads due to conductor and groundwire under normal and broken wire conditions are determined as follows.

Power Conductor

The longitudinal load arising due to unbalanced pull under broken wire condition with suspension strings are assumed as equal to 50 percent of the maximum working tension of the conductor. Under normal condition however there is no longitudinal load on tower due to conductor tension. The transverse load under normal condition is given by the relation.

$$XNC = 2.0 \times \sin\frac{\phi}{2} \times \text{Maximum working tension}$$

where

XNC = the transverse load of conductor under normal condition (due to tension only)

ϕ = Angle of line deviation

The transverse load under broken wire condition, due to tension only in the case of suspension strings (XBC)

$$= 0.5 \times \sin\frac{\phi}{2} \times \text{maximum working tension} \quad (3.8)$$

$$XBC \text{ (with tension strings)} = \sin\frac{\phi}{2} \times \text{Max.working tension} \quad (3.9)$$

The expression for longitudinal load due to broken conductor with suspension, strings (YBC) = $0.5 \times \cos\frac{\phi}{2} \times$ Max.working tension

(3.10)

$$\text{YBC (with tension strings)} = \text{Cos}\,\frac{\emptyset}{2} \times \text{Max. working tension} \quad (3.11)$$

Groundwire

For the groundwire broken condition, 100 percent or such percent of the groundwire tension for which the ground wire clamp is proportioned and which ever is less has to be considered for the purpose of design of tower.

3.4. Points of Application of Loads

Transverse load

This load acts along the longitudinal axis of the cross-arms. It is assumed that the two faces parallel to the longitudinal axis of the cross-arms equally share the transverse load. The analysis of transverse load is most conveniently carried out by considering the reactions at the point of attachment of the cross-arm to the tower legs.

Longitudinal Load

The longitudinal load acts horizontally, along the direction of line and it is assumed that the longitudinal faces of the tower (faces along the directions of the line) equally share the load. Similar to the transverse loads, direct and torsional loads are assumed to act at the point of attachment of the cross-arm to the tower legs, and analysis is carried out in a similar manner, as in the case of transverse loads for transverse faces.

Torsional Load

Longitudinal loads applied at the end of the cross-arm, besides imposing direct loads which are shared by the two longitudinal faces of the tower, also include torsional loads. These are shared by all the four faces of the tower. The torsional loads are resisted by all the four faces in proportion to the lever arm of each face, as such in the case of a square based tower, all four faces will carry equal shears. As all four faces of a tower are subjected to twist shears, each leg member acts in conjunction with two adjacent system of bracings. In the Warren system of bracings, only very little load is taken by the leg members, in the case of square section tower. However, in the case of Pratt or Staggered system of bracings and rectangular towers the legs are also loaded by torsional loads.

Vertical and Eccentric Vertical Loads

The vertical loads are applied at the point of conductor supports. Vertical eccentric loads act in the case of single circuit tower, delta or right angle formation, but in the case of double circuit towers under normal conditions, there are no eccentric vertical loads, and they occur only under broken wire conditions or unequal loading of the two circuits under ice conditions.

Vertical and eccentric vertical loads induce forces in the web members of cross-arm panels, however, it has been found through manual analysis that eccentric vertical loads cause insignificant load in the web members, and their analysis is generally neglected. However, above the bend line the load in web members of the transverse faces due to vertical loads is quite significant as compared to the loads induced into these bracings of the transverse face due to other loads, e.g. transverse and torsional loads and, therefore, must be taken into account.

Dead Weight of Structure and Over load Allowed

The dead weight of the structure and the overload allowed for erection or maintenance usually constitute small percentage of the total load on the tower. It is assumed to act on the main legs. Based on the above assumption, the weight of the structure above the section under consideration is considered for analysis.

3.5. Load Combinations

The most safe tower would be that, which is designed to withstand simultaneous application of worst loadings. But, for economy combined with reliability, we must consider the probable combination of loads that are likely to occur. This will depend upon the importance of the line, type of tower, climatic conditions, terrain through which the line passes, alternative arrangements of supply to the receiving station and continuity of supply.

The following broken wire conditions have been laid down in the IS : 802 (Part I) for single circuit triangular and double circuit towers.

a) Single circuit towers	Any one power-conductor broken or one ground wire broken, whichever is more stringent for a particular member.
b) Double circuit towers	
1) Tangent tower with suspension string (0°) 2) Small angle towers with suspension string(0 to 5°)	Anyone power-conductor broken or one ground wire broken whichever is more stringent for a particular member.
3) Small angle tension towers (5 to 15°) 4) Medium angle tension towers (15 to 30°)	Any two of the power conductors broken on the same side and on the same span or any one of the power conductors and any one ground wire broken on the same span whichever combination is more stringent for a particular member.

5) Large angle (30 to 60^{o}) and dead end towers.	Three power conductors broken on the same side and on the same span or any two of the power conductors and any one ground wire broken on the same span, whichever combination constitutes the most stringent condition for a particular member.
6) Cross arms	In all types of towers the power conductor supports and ground wire supports shall be designed for the broken wire conditions also.

3.6. Factors of Safety and Overload Factors

The term factor of safety (f.o.s.) generally means a safety margin given to account for unknown factors contributing to weakness in material used. But the behavior of steel sections on load is rather well known and there is no unknown factor involved from this point of view. Hence, the term f.o.s. when applied to transmission towers which are made up of materials of well defined properties is unrealistic. It would therefore be proper to apply the term Over Load Factor (OLF) in place of factor of safety as is the practice indicated in the Report of the Task Committee on Tower Design[7].

The overload factor approach to design allows the variation of each item of strength (vertical, transverse and longitudinal) to be controlled according to its importance in the structure.

The overload factors for various types of loads recommended in the Report of the Task Committee on Tower Design[7] are as follows -

Table 3.1

Type of load	Over load factor
Wind loads - No ice	1.0
Wind loads occurring during ice condition	2.54
All vertical loads	1.27
All transverse loads caused by angular tension in the wires	1.65
All longitudinal loads resulting from wire tension when the tower is a one circuit dead end or a complete dead end	1.65
All longitudinal loads resulting from wire tension when less than one circuit is dead ended, or if a difference in tension exists in the wires in the adjacent spans	1.1

The current practice in this country is however to apply a factor of safety of 2.0 for normal condition and a factor of safety of 1.5 for the broken wire condition as stipulated in the Indian Electricity Rules, and the same criteria has therefore been adopted in the automated design procedure developed here. Modifications in the programme can easily be made if the overload factors are required to be used instead of the factors of safety.

3.7. Calculation of Design Loads by Digital Computer

Calculation of various design loads for the tower has been incorporated in the programme developed. The input data for this consists of the design spans, type of tower, the climatic conditions, size and properties of the power conductors and ground wire. The programme first determines the maximum working tension for the conductors and ground wire by making necessary sag tension computations and coordinating the sag between the conductor and groundwire. It then proceeds to compute the transverse, longitudinal and vertical loads for power conductor and groundwire under normal as well as broken wire conditions. The dead weight of the structure is calculated member by member and the load is distributed to the respective joints. The wind pressure is also calculated for each member, giving due consideration for the windward side and leeward side members. The wind load is also distributed to the respective joints of each member, instead of lumping them at selected points on the structure, as is done in the manual methods.

After all the loads are calculated, the various load combinations for normal as well as broken wire conditions depending upon the type of tower (as per IS : 802 - Part I) are automatically applied to the structure one after another for analysis.

CHAPTER-IV

EXISTING METHODS OF ANALYSIS AND THEIR LIMITATIONS

4.1. Introduction

The square and rectangular transmission line towers are highly redundant, and statically indeterminate space frames. The redundancy in transmission tower arises from -

i) Provision of double diagonals

ii) Horizontal bracings or cross frames

iii) Provision of extra members in the frame supporting the ground cables.

Because of the repetitive use of the tower in a transmission line, a designer has to carry out long and time consuming trial designs in order to prune the weight of the tower. This is essential for the reasons of economy and the necessity of having the tower members light in weight for convenience in erection and transportation. But, to attempt manually a classical structural analysis approach to determine the member forces will be a very tedious and time consuming task, and indeed impossible in certain cases. It has therefore been traditional to design the electric transmission towers using only static methods of analysis.

The conventional methods of analysis of transmission line tower require the structure to be resolved into separate determinate system of plane trusses together with the resolution of the vertical, transverse and longitudinal loads into the planes in which they act, ignoring entirely or approximating the effect of the redundant systems on the distribution of these forces. The assumption that these methods require, are too conservative in case of certain members, resulting in over estimation of sizes, and on the other hand, in some of the members, the forces are under estimated due to the inherent inaccuracy of these methods. Due to the doubt in the soundness of assumptions made in such methods of analysis, it is a common practice to carry out a full scale test of a prototype tower to verify the design.

4.2. The Basic Assumptions for Conventional Methods

The analysis of tower becomes complicated by conventional methods, if all the external loads are applied simultaneously and the analysis is carried out for the total load system. In order to simplify the analysis, certain basic assumptions are made, which are detailed out below -

a) All members of a tower frame work are pin-connected in such a manner that the members carry axial loads only.

b) The bolt slippages throughout the structure are such, to allow the use of the same modulus of elasticity for the entire structure, thus permitting the use of the principle of super position for stress analysis.

c) Shear is distributed equally between the two members of a double web system, i.e. Warren system

d). Shear is carried by the member under tension in a double web system with members designed for tension only, i.e. Pratt system.

e) Twist shears applied at cross-arm level are resisted by all the four tower faces in direct proportion to the distance from the centre of the tower to the tower face.

f) Cross frames at levels other than those at which external loads are applied are redundant, and serve only to facilitate erection.

g) Any face of the tower subjected to external loads lies in the same plane, so far as the analysis of the particular face is concerned.

h) The transverse loads are shared by the members on the transverse faces of the tower equally. Similarly, the longitudinal loads are shared equally by the two longitudinal faces.

i) Vertical loads and dead weight of the structure is shared equally by the four legs, except the cross-arm panel web members in certain cases.

j) The torsional loads are resisted by all the four faces in proportion to the lever arm of each face.

k) Loads coming on the common leg members from the longitudinal and transverse faces are added up.

4.3. Various Methods

The lattices are designed only for tension so that for transverse loads, only the lattice in tension is assumed to be active. This assumption renders the frame to be statically determinate. The forces in members of a plane truss may then be found out by one of the following methods-

a) Method of joints

b) Method of sections

c) The graphical method of analysis

d) The tension coefficient method

In the method of joints, since each joint is in equilibrium under the action of the member axial forces (and the applied forces and support reactions if any), the forces acting on any individual joint must form a closed polygon. If, at any joint, only two force components are unknown then these can be determined by drawing a force polygon. This method is however very tedious when applied to tower analysis. The method of sections, on the other hand, is useful when it is required only to determine the forces in few members of the truss, and therefore, is also not suitable. The tension coefficient method can be applied to two as well as three dimensional frames.

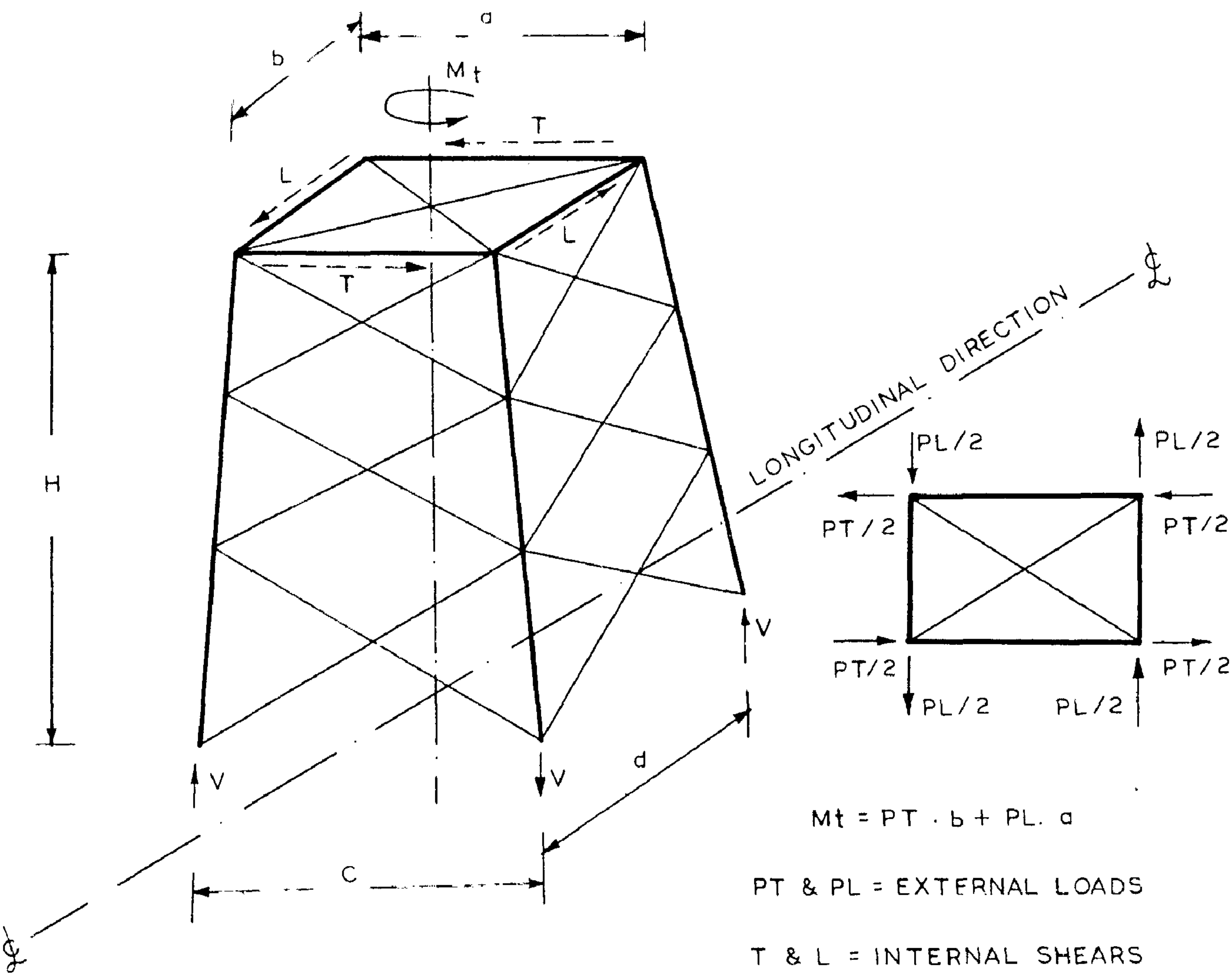
The graphical method of analysis is however easier than analytical methods, but the accuracy of the calculated stress by graphical method depends upon the accuracy of stress diagrams drawn and measurement of stress made on proportionate scale. The graphical analysis follows the procedure of the method of joints and a single farce diagram for the entire truss (the Maxwell diagram) is drawn which is called the unit stress diagram.

4.4. Limitations

One of the major concerns that results from the use of these conventional methods is the distribution of torsional shears in the structure caused by a broken conductor, ice shadding off ane span or pull imposed during stringing or maintenance operations. The conventional methods encounter major difficulties in assessing the design member forces under such unbalanced loads, which are eccentric with respect to the structure axis. These methods therefore seek simplified techniques to establish a distribution of the torsional moments in the form of equivalent shear forces applied to the structure faces. This distribution of torsional shears can be expressed in the following form,[18] referring to Fig.4.1.

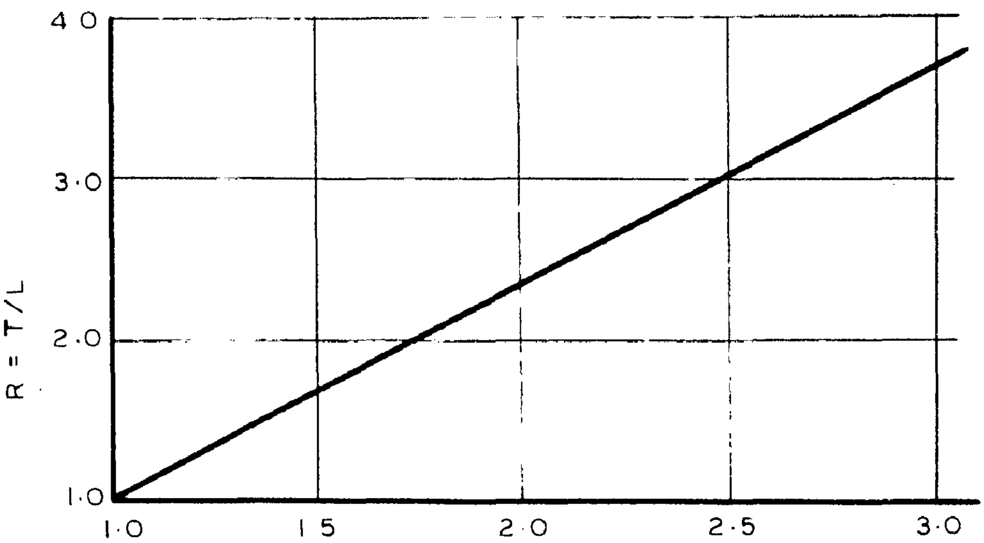
$$T.b + L.\ a = Mt \qquad (4.1)$$

All the methods attempted an evaluation for 'T' the internal transverse shear, and 'L', the internal longitudinal shear, through relating the ratio T/L, denoted here by 'R' to the main outline of the structure, ignoring totally or partially, the elastic properties of the members. The most widely used emperical relation is perhaps the one given[11] in the design standard prepared by the United States Department of the Interior Bureau of Reclamation, (U.S.B.R.), and for convenience is shown in Fig.4.2. The USBR formula represents the average ratio 'R' obtained from thesolution of a few actual structures. This formula ignores completely the structure characteristics such as member sizes, number of panels, support conditions etc. and only uses the main dimensions of the structure. Thus for any structure designed on this basis or on much simpler assumptions than this(as in section 4.2) little correlation with the actual structural behaviour can be expected. This becomes very much evident when a comparative study of the results of a tower analysed by manual and automated methods are made in the next chapter. For carrying out an exact analysis of trasmission towers leading to an economical design it is therefore, necessary to resort to three dimensional analysis.



LATTICE BODY SUBJECT TO TORQUE

FIG. 4·1



$\frac{a+c}{b+d}$ OR $\frac{d+b}{a+c}$ WHICHEVER IS GREATER THAN UNITY

U. S. B. R. FORMULA

FIG. 4·2

CHAPTER-V

THREE DIMENSIONAL ANALYSIS

5.1. Introduction

A transmission line tower is considered to be a space frame, which is essentially a space cantilevered truss with its major loads acting at the extreme ends and intermittent loadings at all the joints. As the towers are generally built of bolted angle sections they are ideal to be treated as pin-jointed space trusses for the purpose of analysis. The entire structure consists of principal bars, which function as the load carrying members, and unloaded secondary bars that stabilize the joints and provide lateral support to the principal members. As already discussed in chapter-III it can be seen that the towers are subjected to a multiplicity of loading systems that are all reversible except for gravity loads.

5.2. Theory

The Finite Element Method can conveniently be applied to carry out three dimensional analysis for this type of structures. First of all, it is not necessary to descretise the structure since it is already made up of discrete one dimensional elements. These one dimensional pin-jointed truss members can sustain a single type of loading only, namely a uniform axial force.

The truss member in three dimensional space is shown in Fig.5.1. It will be assumed to have a length L and an uniform axial stiffness factor AE. Let x, y, z represent reference axes and $\bar{x}, \bar{y}, \bar{z}$ represent local axes. The local axis $\bar{x}$ points along the member from 1 to 2. The local and reference axes systems are related through angles defined as follows -

$$\begin{aligned} \theta x\bar{x} &= \text{angle between } x \text{ and } \bar{x} \\ \theta x\bar{y} &= \text{angle between } x \text{ and } \bar{y} \\ &\cdots\cdots\cdots\cdots\cdots\cdots \\ \theta z\bar{y} &= \text{angle between } z \text{ and } \bar{y} \\ &\cdots\cdots\cdots\cdots\cdots\cdots \end{aligned} \qquad (5.1)$$

The local stiffness matrix of the truss member ($\bar{K}$) is of the following form

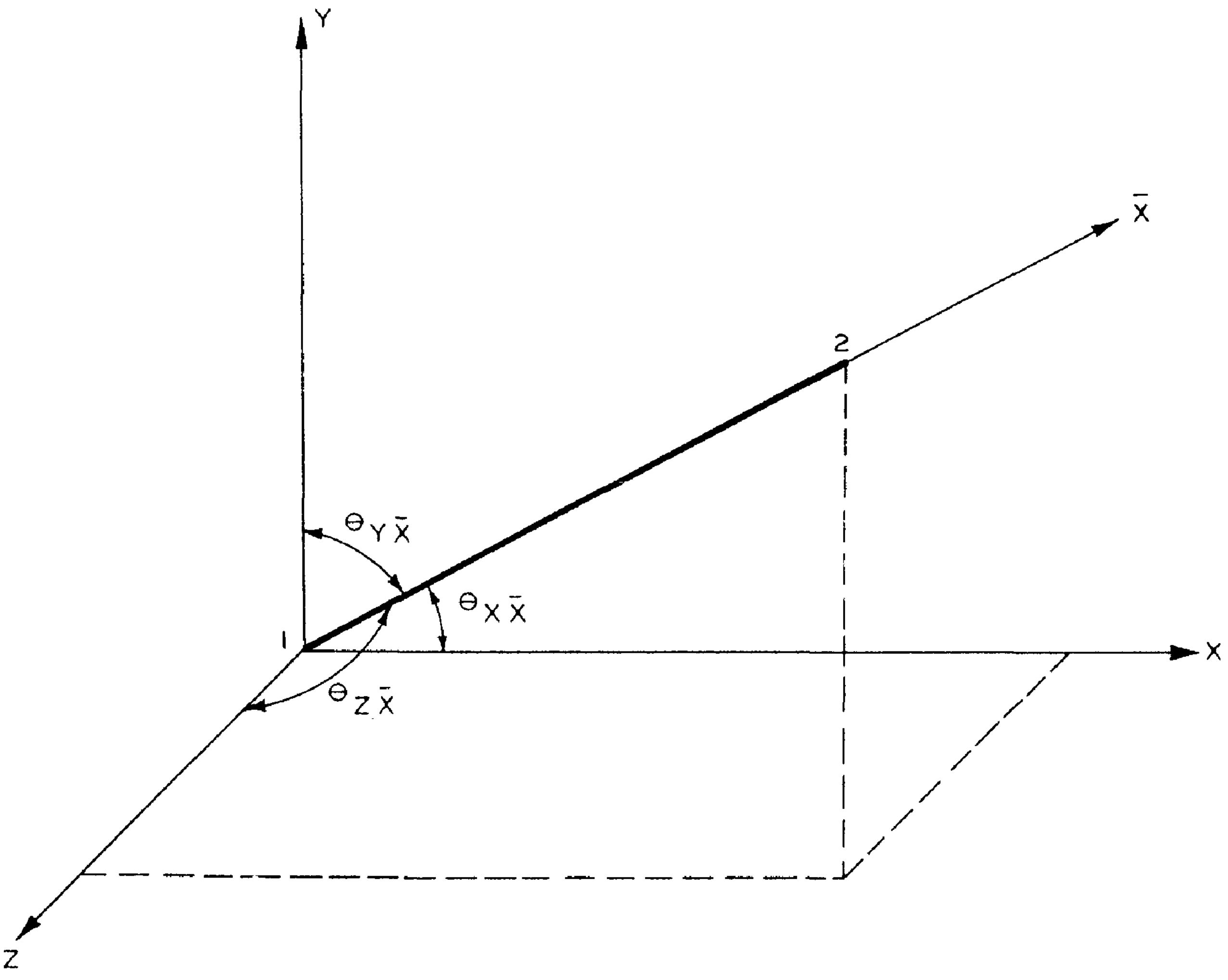
$$\bar{K} = \frac{AE}{L} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \qquad (5.2)$$

Transformation of the local stiffness matrix of the truss member into the stiffness matrix with reference to the reference axes or the global coordinates (K). can be carried out according to the following relationship

$$K = T' \bar{K} T \qquad (5.3)$$

Where T = Transformation or the rotation matrix

and T' = Transpose of the rotation matrix



TRUSS MEMBER IN THREE-DIMENSIONAL SPACE

FIG. 5.1

The form of rotation matrix is given[3] as follows -

$$T = \begin{bmatrix} \lambda\bar{x} & \mu\bar{x} & \nu\bar{x} & 0 & 0 & 0 \\ \lambda\bar{y} & \mu\bar{y} & \nu\bar{y} & 0 & 0 & 0 \\ \lambda\bar{z} & \mu\bar{z} & \nu\bar{z} & 0 & 0 & 0 \\ 0 & 0 & 0 & \lambda\bar{x} & \mu\bar{x} & \nu\bar{x} \\ 0 & 0 & 0 & \lambda\bar{y} & \mu\bar{y} & \nu\bar{y} \\ 0 & 0 & 0 & \lambda\bar{z} & \mu\bar{z} & \nu\bar{z} \end{bmatrix} \qquad (5.4)$$

Where,

$$\begin{array}{lll} \lambda\bar{x} = \cos\theta x\bar{x} & \lambda\bar{y} = \cos\theta x\bar{y} & \lambda\bar{z} = \cos\theta\, x\bar{z} \\ \mu\bar{x} = \cos\theta y\bar{x} & \mu\bar{y} = \cos\theta y\bar{y} & \mu\bar{z} = \cos\theta\, y\bar{z} \\ \nu\bar{x} = \cos\theta z\bar{x} & \nu\bar{y} = \cos\theta z\bar{y} & \nu\bar{z} = \cos\theta z\bar{z} \end{array} \qquad (5.5)$$

Now, on carrying out the transformation according to equation 5.3, the stiffness matrix for the truss member referred to reference axes (K) is obtained in the following form, as shown in equation 5.6.

This matrix represents influence coefficients for the actions due to unit displacement at the joints parallel to each one of the reference axes.

$$K = \frac{AE}{L} \begin{bmatrix} \lambda_{\bar{x}}^2 & & & & & \\ \lambda_{\bar{x}}\,\mu_{\bar{x}} & \mu_{\bar{x}}^2 & & & \text{SYM} & \\ \lambda_{\bar{x}}\,\nu_{\bar{x}} & \mu_{\bar{x}}\,\nu_{\bar{x}} & \nu_{\bar{x}}^2 & & & \\ -\lambda_{\bar{x}}^2 & -\lambda_{\bar{x}}\,\mu_{\bar{x}} & -\lambda_{\bar{x}}\,\nu_{\bar{x}} & \lambda_{\bar{x}}^2 & & \\ -\lambda_{\bar{x}}\,\mu_{\bar{x}} & -\mu_{\bar{x}}^2 & -\mu_{\bar{x}}\,\nu_{\bar{x}} & \lambda_{\bar{x}}\,\mu_{\bar{x}} & \mu_{\bar{x}}^2 & \\ -\lambda_{\bar{x}}\,\nu_{\bar{x}} & -\mu_{\bar{x}}\,\nu_{\bar{x}} & -\nu_{\bar{x}}^2 & \mu_{\bar{x}}\,\nu_{\bar{x}} & \mu_{\bar{x}}\,\nu_{\bar{x}} & \nu_{\bar{x}}^2 \end{bmatrix} \tag{5.6}$$

The next step is to form the global stiffness matrix$^{(S)}$ for the entire structure by assembling the element stiffness matrices. This global stiffness matrix is the mathematical model of the complete structure. On applying necessary boundary conditions the reduced stiffness matrix for the complete structure $(\bar{S})$ is obtained. The displacement of each joint with reference to the global coordinates are determined on application of the following force-displacement relations.

$$F = \bar{S}.\,u \tag{5.7}$$

where, F is the force vector

and u is the displacement vector

Having known the joint displacement (u) the member forces ($\bar{X}$) can easily be determined as follows.

$$\bar{X} = \bar{K} . \bar{u} \qquad (5.8)$$

where

$\bar{X}$ is the member force with reference to local axis

$\bar{K}$ is the element stiffness matrix with reference to local axis

and $\bar{u}$ is the joint displacements with reference to local axis

The equation for obtaining the member force therefore reduces to

$$\bar{X} = \lambda\bar{x} (u_1 x - u_2 x) + \mu\bar{x} (u_1 y - u_2 y) + \nu\bar{x} (u_1 z - u_2 z) \qquad (5.9)$$

Where, u_1x, u_1y and u_1z are the displacements of the joint No.1 in the x,y and z axes respectively.

and u_2x, u_2y and u_2z are the displacements of the joint No.2 in the x,y and z axes respectively.

Simplification of force displacement relationship to the above equation (5.9) avoids any matrix operation in determining the member forces from the joint displacements obtained with reference to the global ordinates.

5.3. Computer Application

The key to an efficient computer analysis however lies in the technique used to store the global stiffness matrix, and solve the simultaneous equations. Taking advantage of the inherent symmetry of the stiffness matrix, only half the band matrix is stored in a single subscripted array form, in this programme as shown in Fig.5.2. The storage requirement is therefore greatly reduced.

The storage space required for the stiffness matrix in this programme, can be calculated from the relation -

$$NS = (N-NR)\,NR + NR\,(NR+1)/2 \qquad (5.10)$$

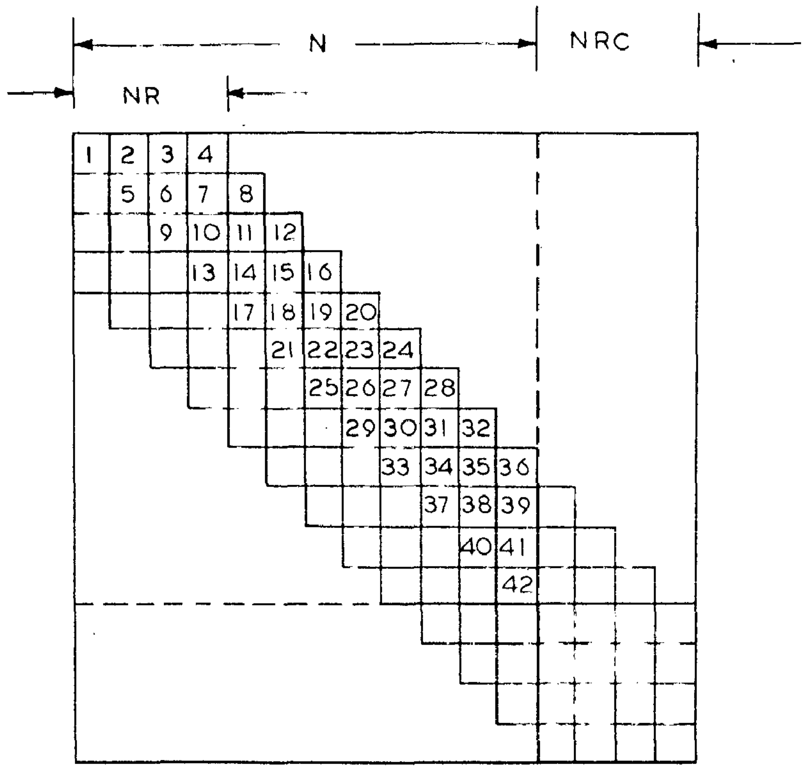
Where N is the dimension of the reduced global stiffness matrix

and NR is the half band width

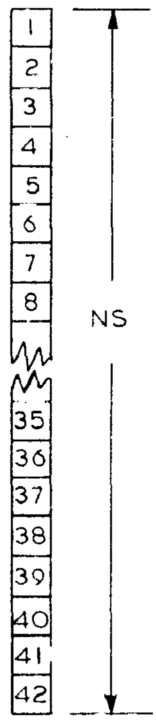
It may be seen from the above that the storage space depends directly on the half band width. The half band width of th stiffness matrix for a space truss is given by

$$NR = 3\,(MDF + 1) \qquad (5.11)$$

where, MDF is the maximum arithmetic difference between the nodal points or joints numbers in any element



BANDED GLOBAL STIFFNESS MATRIX



STORED FORM

FIG. 5·2

Efforts to minimise the arithmetic difference between the joint numbers of each element tends to reduce the band width and thus minimise the storage space.

For solving the simultaneous equations a modified Crout's procedure has been used for decomposition of the stiffness matrix and the solution is obtained by forward and backward substitution. This procedure of analysis, besides its efficient storage of the stiffness matrix, can, for the type and size of structures considered herein, perform all the operations in the core of the computer without resorting to the use of auxiliary storage units for which the access time is usually more.

5.4. Idealisation of the Structure

It quite often happens that joints are placed within the length of a member to connect bracing systems that lie only in one plane, thus subdividing the member in that plane into a number of shorter members. If no loads are expected to act on the member in a plane perpendicular to the bracing system, then no bracings are provided to support it in that plane, and the member is physically a continuous member. However, in the mathematical model of three dimensional pin jointed structures, this situation is regarded as instability of the member and it reflects in producing a singular stiffness matrix. It is therefore, necessary to introduce flexible fictitious, secondary members to provide the needed stability. This would suggest that the secondary members, which have to be included for the sole purpose of stabilizing the joints, need not be the members that will be actually provided, but may be modified in such a manner so as to minimise their number, and achieve savings of storage and computational effort.

As regards all the secondary members, that are used to provide lateral support to the active members are excluded and their effect is simulated by the effective length factors or the fixity factors. By doing this, majority of the secondary members are excluded from the input data.

5.5. Comparison of Manual and Automated Analysis

For the purpose of study here, an existing 132 kV single circuit, 30^{0} angle tower, shown in Fig.6.7 was chosen. The various details in respect of this tower are as follows -

Tower type- Single circuit, triangular formation
30^{0} angle tower

Total number of joints	-	56
Total number of members	-	191
Normal span	-	320 m
Wind span	-	320 m
Weight span	=	500 m
Wind pressure	-	100 kg/sq.m.
Minimum conductor temperature		$4^{o}C$
Maximum conductor temperature	-	$67^{o}C$

Power conductor - 30/ 3 mm st. + 7/3mm Al.

PANTHER ACSR

Groundwire - 7/4 mm G.I.

Three dimensional analysis was carried out on this tower for the following types of loads, as specified in the IS : 802 - 1973.

(1) Transverse loads

(a) Wind pressure on the structure

(b) Wind pressure on the conductors and groundwire

(c) Component of the conductor and groundwire tension due to line deviation

(d) Wind pressure on the insulators

(2) Longitudinal loads

(a) Component of the conductor and groundwire tension. This exists only under broken wire conditions.

(3) Vertical loads

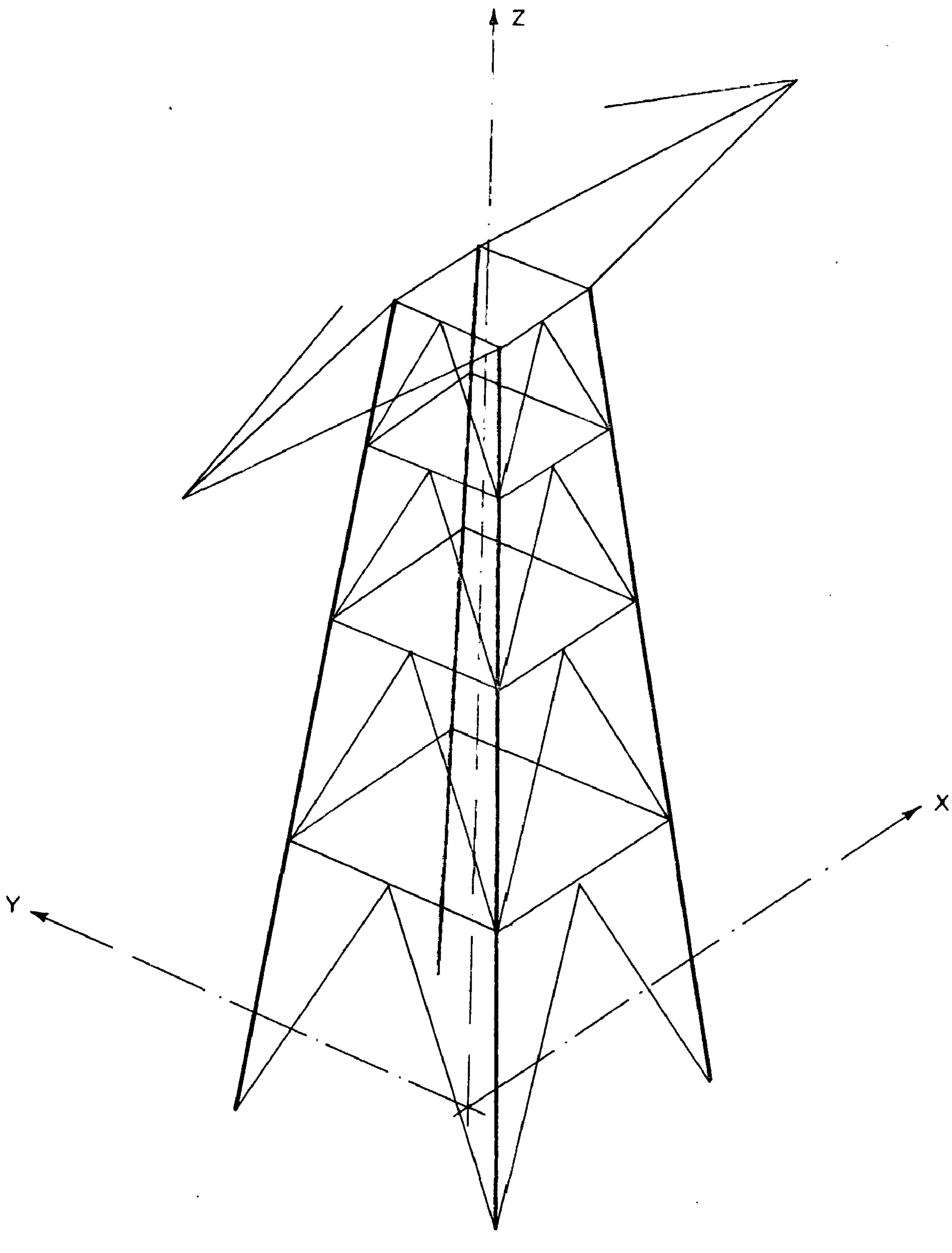
(a) Weight of the structure

(b) Weight of conductor and groundwire

(c) Weight of insulators and fittings

(d) Weight of line-man with tools

There are five load cases for this tower as listed below and the tower was analysed automatically for all these load cases one after another-



TRANSMISSION TOWER AS A SPACE TRUSS

FIG. 5·3

(1) Normal condition, with all conductors and groundwire intact.

(2) Broken wire condition with groundwire broken

(3) Broken wire condition with top conductor broken

(4) Broken wire condition with middle conductor broken

(5) Broken wire condition with bottom conductor broken

The member forces which are available through manual design has been compared with that obtained through the automated design. Table 5.1 shows the comparison. The percentage Difference shown in column 10 has been calculated as follows -

$$\text{Percentage Difference} = \frac{(\text{Automated design} - \text{Manual design})}{\text{Manual design}}$$

Following observations are apparent from the tabulated results -

1. Leg Members -

i) Groundwire Peak - It is noticed that the forces in the leg members of this section are slightly over estimated in the manual design by about 1 percent to 2 percent, which is quite negligible.

ii) Shaft portion - In this portion, the member forces are found to be over estimated in the manual design by about 15 percent.

iii) Pedestal portion - In this section the forces in the leg members are found to be under estimated in the manual design by about 10 percent.

2. Lattices

i) Longitudinal lattice - It is observed in this case that the longitudinal bracings are loaded under normal condition upto about 600 kgf, although the manual design assumes the load on longitudinal bracings under normal condition to be nil.

ii) Transverse lattice - As regards the member forces in the transverse lattice, there is a wide variation between the two results. Specially in the section near the top cross arm where the difference is maximum, being about 118 percent. This difference gradually reduces to about 20 percent at the base level.

The variation in the forces of lattice members appear to be due to the conservative approach the designer takes in proportioning such highly internally indeterminate arrangements when subjected to torsional moments.

3. Cross-arms

i) Top inclined members - The conventional method of transmission tower design assumes that these members do not resist conductor tensions in the longitudinal directions and only support vertical and transverse loads. This assumption is incompatible with the structural arrangements and the automated design procedure shows a load of 230 percent to 260 percent higher than the manual procedure under broken wire conditions.

ii) Lower Main members - These members have been found to be over estimated in the manual procedur by as much as 50 percent to 100 percent. This appears to be as a consequence to the assumptions made in the design of the top inclined members of the cross-arms.

In general, it can be said that a three dimensional analysis by computer gives better insight regarding the actual stress distribution in the various members due to various external loads, which can help in making safer and more economical design possible.

COMPARISON OF MANUAL AND AUTOMATED METHODS OF DESIGN

FORCES ARE IN KGS AND THE NEGATIVE SIGN INDICATES TENSION

N.C. - Normal Condition, G.W.B. = Groundwire Broken, T.C.B = Top conductor broken
M.C.B. = Middle conductor broken, B.C.B. = Bottom conductor broken,
B.W.C. = Brokenwire condition

Member No.	Joint Nos.	Description	MANUAL DESIGN			AUTOMATED DESIGN			Percentage difference (Relative to manual
			Load case	Member force	Size	Load case	Member force	Size	
1	2	3	4	5	6	7	8	9	10
4	1-5	Groundwire peak	NC	2735	65x65x6	NC	2683	65x65x6	-1.90
1	1-2	-do-	NC	-2434	-do-	NC	-2414	-do-	-0.82
3	1-4	-do-	GWB	5330	-do-	GWB	5227	-do-	-1.93
1	1-2	-do-	GWB	-5070	-do-	GWB	5007	-do-	-1.24
40	12-16	Leg Members	NC	4245	-do-	NC	3561	-do-	-16.11
36	11-15	-do-	NC	-3305	-do-	NC	-2831	-do-	-14.34
40	12-16	-do-	GWB	7110	-do-	GWB	6105	-do-	-14.13
31	10-14	-do-	GWB	-6240	-do-	GWB	-5448	-do-	-12.69
116	30-34	-do-	NC	11020	100x100x6	NC	10494	80x80x8	-4.77
107	28-32	-do-	NC	-8820	-do-	NC	-8648	-do-	-1.95
116	30-34	-do-	GWB	16145	-do-	GWB	15189	-do-	-5.92
107	28-32	-do-	GWB	-14015	-do-	GWB	-13406	-do-	-4.34
150	39-43	-do-	NC	13205	100x100x8	NC	13860	100x100x8	4.96
143	37-41	-do-	NC	-15250	-do-	GWB	-16209	-do-	6.29
162	43-47	-do-	NC	13725	-do-	NC	14703	110x110x8	7.16
155	41-45	-do-	NC	-11352	-do-	NC	-12419	-do-	9.39

Contd.

1	2	3	4	5	6	7	8	9	10
162	43-47	Leg Members	GWB	17500	100x100x8	GWB	18740	110x110x8	5.54
155	41-45	-do-	-do-	-15169	-do-	-do-	-16262	-do-	7.20
175	47-55	-do-	N C	14275	-do-	N C	14952	-do-	4.74
169	45-53	-do-	-do-	-11537	-do-	-do-	-12506	-do-	8.39
175	76-55	-do-	GWB	17635	-do-	GWB	18300	-do-	3.77
169	45-53	-do-	-do-	-14939	-do-	-do-	-15921	-do-	6.57
37	11-16	Transverse Lattice	N C	1076	45x45x5	N C	1132	45x45x5	5.20
43	13-14	-do-	-do-	-1076	-do-	-do-	-1177	-do-	9.38
33	10-18	-do-	MCB	1076	-do-	TCB	1574	-do-	46.28
43	13-14	-do-	-do-	-1076	-do-	-do-	-2352	-do-	118.58
42	12-18	Longitudinal Lattice	N C	0	-do-	N C	637	-do-	+
32	10-15	-do-	-do-	0	-do-	-do-	-454	-do-	+
44	13-16	-do-	GWB	750	-do-	TCB	1285	-do-	71.33
35	11-15	-do-	-do-	-750	-do-	GWB	-989	-do-	31.86
52	14-22	Transverse Lattice	N C	1147	-do-	N C	1131	-do-	-1.39
64	18-19	-do-	-do-	-1147	-do-	-do-	-1323	-do-	15.34
52	14.22	-do-	TCB	3077	-do-	TCB	2636	-do-	-14.33
64	18-19	-do-	-do-	-3077	-do-	-do-	-3781	-do-	22.88
62	16-22	Longitudinal Lattice	N C	0	-do-	N C	620	-do-	+
55	15-19	-do-	-do-	0	-do-	-do-	-430	-do-	+
65	18-21	-do-	TCB	3320	-do-	TCB	3685	-do-	10.99
62	16-22	-do-	-do-	-3320	-do-	-do-	-2581	-do-	-22.26

1	2	3	4	5	6	7	8	9	10
72	19-27	Transverse Lattice	N C	1811	45x45x5	N C	725	45x45x5	-59.97
82	22-23	-do-	-do-	-1811	-do-	-do-	-402	-do-	-77.80
77	20-26	-do-	TCB	3742	-do-	GWB	1038	-do-	-72.26
82	22-23	-do-	-do-	-3742	-do-	TCB	-1946	-do-	-47.99
81	21-27	Longitudinal Lattice	N C	0	-do-	N C	563	-do-	+
71	19-25	-do-	-do-	0	-do-	-do-	-554	-do-	+
83	22-26	-do-	TCB	3320	-do-	TCB	3620	-do-	9.04
81	21-27	-do-	-do-	-3320	-do-	-do-	-2645	-do-	-20.33
91	23-31	Transverse lattice	N C	1830	-do-	N C	2110	-do-	15.30
102	27-28	-do-	-do-	-1830	-do-	-do-	-1818	-do-	-0.66
97	25-30	-do-	TCB	3761	-do-	M CB	4516	-do-	20.07
102	27-28	-do-	-do-	-3761	-do-	TCB	-3843	-do-	2.18
101	26-31	Longitudinal lattice	N C	0	-do-	N C	592	-do-	+
90	23-29	-do-	-do-	0	-do-	-do-	-608	-do-	+
103	27-30	-do-	TCB	3320	-do-	TCB	3614	-do-	8.85
95	25-28	-do-	-do-	-3320	-do-	M CB	-3462	-do-	4.28
109	28-36	Transverse lattice	N C	2495	50x50x5	N C	2611	-do-	4.65
119	31-32	-do-	-do-	-2495	-do-	-do-	-2726	45x45x5	9.26
113	29-34	-do-	TCB	4426	-do-	M CB	4288	-do-	-3.12
119	31-32	-do-	-do-	-4426	-do-	TCB	-4648	-do-	5.02
118	30-36	Longitudinal lattice	N C	0	45x45x5	N C	1185	-do-	+
108	28-33	-do-	-do-	0	-do-	-do-	-929	-do	+
120	31-34	-do-	B CB	3320	-do-	TCB	3857	-do-	16.17
111	29-32	-do-	-do-	-3320	-do-	M CB	-3921	-do-	18.10
123	32-40	Transverse lattice	N C	636	50x50x5	N C	657	-do-	3.30

1	2	3	4	5	6	7	8	9	10
136	34-38	Transverse lattice	N C	-636	50x50x5	N C	-642	45x45x5	0.94
128	32-40	-do-	TCB	2799	-do-	TCB	2882	-do-	2.96
140	36-37	-do-	TCB	-2799	-do-	TCB	-2836	-do-	1.32
131	33-37	Longitudinal lattice	N C	0	-do-	N C	182	-do-	+
138	34-40	-do-	-do-	0	-do-	-do-	-213	-do-	+
141	36-39	-do-	B CB	3260	-do-	B CB	3087	-do-	-5.31
138	34-40	-do-	-do-	-3260	-do-	-do-	-3426	-do-	5.09
145	-37-44	Transverse lattice	N C	472	45x45x5	N C	296	50x50x5	-37.29
149	39-42	-do-	-do-	-472	-do-	-do-	-225	-do-	-52.33
148	38-43	-do-	TCB	2076	-do-	M CB	1975	-do-	-4.86
152	40-41	-do-	-do-	-2076	-do-	TCB	-2048	-do-	-1.35
151	39-44	Longitudinal lattice	N C	0	50x50x5	N C	460	55x55x5	+
144	37-42	-do-	-do-	0	-do-	-do-	-377	-do-	+
153	40-43	-do-	B CB	2418	-do-	B CB	2850	-do-	17.87
146	38-41	-do-	-do-	-2418	-do-	M CB	-2430	-do-	0.49
157	41-48	Transverse lattice	N C	341	-do-	N C	119	60x60x5	-65.10
161	43-46	-do-	-do-	-341	-do-	-do-	-85	-do-	-75.07
157	41-48	-do-	TCB	1499	-do-	TCB	1374	-do-	-8.34
164	44-45	-do-	-do-	-1499	-do-	-do-	-1283	-do-	-14.41
58	16-17	Top X-arm (lower)	N C	2115	65x65x6	N C	-221	60x60x5	-
63	17-18	-do-	-do-	-832	-do-	-do-	-222	-do-	-
58	16-17	-do-	B WC	7295	-do-	B WC	4357	-do-	-40.27
63	17-18	-do-	-do-	-6270	-do-	-do-	-4126	-do-	-34.19
41	12-18	Top X-arm (Upper)	N C	0	45x45x6	N C	-1055	55x55x5	+
45	13-17	-do-	-do-	-10 25	-do-	-do-	-1055	-do-	2.93

1	2	3	4	5	6	7	8	9	10
75	20-24	Mid X-arm (upper)	NC	0	45x45x6	NC	-1240	55x55x5	+
70	19-24	-do-	-do-	-1208	-do-	-do-	-1240	-do-	2.65
75	20-24	-do-	BWC	0	-do-	BWC	1236	-do-	+
70	19-24	-do-	-do-	-935	-do-	-do-	-3168	-do-	238.82
85	23-24	Mid X-arm (lower)	NC	2290	65x65x6	NC	2340	60x60x5	2.18
92	24-25	-do-	-do-	-739	-do-	-do-	2340	-do-	-
85	23-24	-do-	BWC	7430	-do-	BWC	5387	-do-	-27.49
92	24-25	-do-	-do-	-6185	-do-	-do-	-2316	-do-	-62.55
134	34-35	Bottom X-arm (lower)	NC	2470	75x75x6	NC	-20	60x60x5	-
139	35-36	-do-	-do-	-919	-do-	-do-	-20	-do-	-
134	34-35	-do-	BWC	7613	-do-	BWC	4059	-do-	-46.68
139	35-36	-do-	-do-	-6368	-do-	-do-	-3516	-do-	-4479
117	30-35	Bottom X-arm (upper)	NC	0	45x45x5	NC	-1239	55x55x5	+
121	31-35	-do-	-do-	-1208	-do-	-do-	-1239	-do-	2.57
117	30-35	-do-	BWC	0	-do-	BWC	1305	-do-	+
121	31-35	-do-	-do-	-935	-do-	-do-	-3238	-do-	246.31

+ Results not possible by manual method

CHAPTER-VI

AUTOMATED DESIGN OF TOWER WITH DIGITAL COMPUTER

6.1. Introduction

The distinction between analysis and design is often not clearly drawn in the mind of a practicing engineer, whose scope of responsibility includes both, design and analysis. This is particularly true when a high level of interaction is maintained between design and analysis, such as in a hyperstatic structure like that of a transmission line tower. By the term ' design ' we mean the selection, within specified constraints, of the geometric and material configuration of a structure. 'Analysis', on the other hand, refers to a prediction of the behaviour of the structure under a specified set of conditions.

In the case of statically determinate structures, designed to satisfy a given set of permissible stresses, it may be sufficient to carry out one analysis. This is because the forces in the members of these structures are independent of their cross-sectional properties. The analysis of hyperstatic structures, however, requires a knowledge of the member properties such as the area or the modulus of elasticity of the sections. It is therefore necessary to assume these sectional properties, analyse the structure and use the results to select a new set of properties. Repeating this cycle of operations one can arrive at a feasible design. Therefore the problem of design often appears to be that of repeated analysis. The main drawback of this approach is that it is not always possible to try a number of alternatives manually and select the most suitable. However, once a computerised method for analysis is available, design of a hyperstatic structure becomes possible.

In the above iterative method the member forces are first determined by making an initial assumption as regards the member sizes. The member forces thus determined will not likely be compatible with the desired performance of the structure and modifications will be necessary. Attempt to improve the initial assumptions on member sizes is then made by adjusting those members, which violate the set design criteria or are considered excessively safe by the same standard. The process is then repeated until a satisfactory design is reached. It is, therefore, clear that this design procedure consists of two parts, judgement and logical operations. The first stage in the development of computerised design process is that the designer passes on to the computer the logical operation portion of the design process, and intervenes after each cycle by inspecting the results and altering the parameters according to his judgement. This operation is said to be ' computer-aided design '. The next stage of the development suggests that the designer would pass on to the computer the judgement part, as well as the logical operations of the design process, such operation is termed as ' automated design '. Judgement in engineering design is a hierarchy of decisions, some quantitative and other qualitative, and the degree to which automated design can advance will depend on the ability to turn the qualitative decisions into quantitative ones.

6.2. Method of Design

The automated design procedure developed here, starts first by analysing the preliminary structure for all design loading conditions and the maximum compressive and tensile forces are determined for each member. Estimation of the member size is then made for each member group, basing on the most critical loadings with appropriate factor of safety. The factor of safety adopted in the programme is 2.0 for normal condition and 1.5 for the broken wire conditions, as per the IS : 802 (Part I). The estimated member sizes are not allowed to fall below certain minimum area and exceed certain maximum slenderness ratio limits.

This estimating cycle is repeated as specified by the designer. The number of cycles this operation may need will vary depending on the structure complexity. It is seen from experience through this study that for a common tower three or four estimating cycles are normally sufficient to stabilise the estimated design. Following the completion of the estimation cycles, a search through a list of available angle sizes commences, and suitable sizes are assigned to each member groups. The structure is again analysed, and cyclic modifications continue until there is no change in the member sizes. The design is then finalised and the final size of each member is printed. All the analyses done throughout the process are three dimensional.

The main components of the procedure are -

a) Calculation of loads

b) Analysis

c) Member area Estimation

d) Member Size Selection

Fig.6.1. shows a macro-flow chart of this procedure. The call upon the area estimation routine or the size selection routine is at the discretion of the designer, who has to dictate the number of cycles each of these routines shall be used. There is no way to determine before hand how many size selection cycles will be needed, but specifiying a relatively large number of cycles, five to ten, will ensure that a final design can be reached. This does not imply that unnecessary computation will be made, since the programme terminates automatically when the design is completed.

MACRO-FLOW CHART OF AUTOMATED TOWER DESIGN PROCEDURE

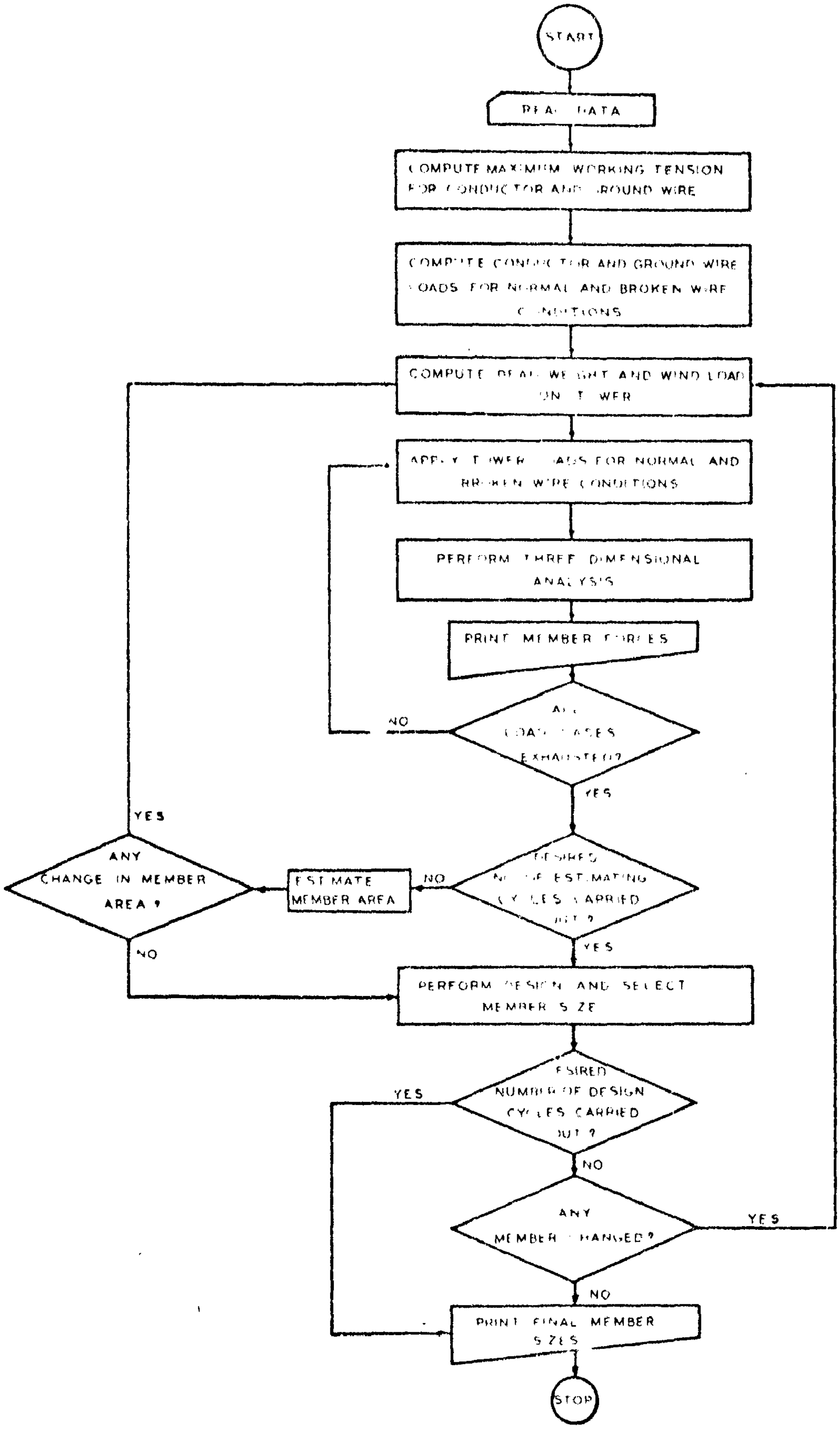


FIG. 6·1

6.3. Types of Structural Members

In the procedure used here all the tower members have been classified under four dinstinct types, so that specific design criteria, in consistent with the design practice of transmission line tower can be applied in proportioning the different types of members, as will be explained later.

Type 1 - Main, concentrically loaded, exterior members, such as tower legs and cross arm chords.

Type 2 - Bracings and all principal members which are designed for both, compression and tension

Type 3 - Primarily tension members, such as cross-arm hanger bars.

Type 4 - All secondary and fictitious members for which no design is required and members for which the configuration is determined by other than structural requirements.

6.4. Minimum Sizes and Limiting Slenderness Ratios

Depending on the member type, a minimum permissible area, leg width and thickness should be specified in the input data. For tension members, the restrictions on leg width and thickness are meaningless and are omitted.

The slenderness ratios of the members are limited to stipulted maximum values to ensure a minimum level of stiffness and to reduce vibrations. There are wide differences in assessing the effective length of the compression members. An effective length factor has therefore been incorporated in the input data. By doing so the majority of the secondary members used only to support the active members can be excluded from the input data, thereby reducing computational effort.

The limiting slenderness ratios used this work for different types of members are as recommended in the relevant I.S. Specifications and are given below -

Member type 1	-	$(KL/r) \leq$	150
Member type 2	-	$(KL/r) \leq$	200
Member type 3	-	$(KL/r) \leq$	250
Member type 4	-	$(KL/r) \leq$	500

6.5. Stress Criteria

(a) Tension Members

The determination of the net effective area of a tension angle member connected by one leg only is the subject of major controversy. In this case, axial stresses are accompanied with bending stresses due to the eccentricity of the loading, however, empirical rules are often used to simplify the design of such members by neglecting the bending stresses. One empirical approach, specified in the IS : 802 (Part I), suggests that a portion of the outstanding leg be deducted along with bolt holes, as follows -

$$\text{Net effective area} = a + b.k \qquad (6.1)$$

Where

a	=	Net sectional area of the connected leg,
b	=	area of the outstanding leg = $(l-t)t$,
l	=	length of outstanding leg
t	=	thickness of the leg, and
k	=	$k/(1 + 0.35\ b/a)$ (6.2)

Another empirical rule is given in the British Standard Code B.S.S. 449, ' The Use of Structural Steel in Building '. A third rule tends to benefit from the fact that the yield stress of thin angle sections exceeds its minimum guaranted value, and specify only the deduction of the bolt holes from the gross area of the section, as given in the ' Report of the Task Committee on Tower Design, Electrical Transmission Line and Tower Design Guide '[7]. For the purpose of this study, the latter rule is adopted with the modification that the net area of the primary tension members should not exceed 85 percent of the gross area.

b) Compression Members

The design criteria given in the I.S. Code 802 (Part I) which are in accordance with the Task Committee Report of A.S.C.E. and the recommendations of the Column Research Council, are implemented in this study.

The slenderness ratio of a member is based on the effective length of that member and the applicable radius of gyration. For the member shown in Fig.6.2., the effective length factors are $Kx = 1.0$ and $Kz = 0.5$. The slenderness ratio (SL) is the greatest of (KxL/ rx) and (KzL / rz). It is assumed that local instability of the angle section would be avoided if the leg width to thickness ratio (b/t), Fig. No.6.2., is

$$(b/t)_{limit} \leq \frac{662.5}{\sqrt{Fy}} \qquad (6.3)$$

Where Fy = guaranteed yield strength of material in kgf/cm^2.

In the inelastic range, defined by

$$SL \leq Cc = \pi\sqrt{2E/Fy} \qquad (6.4)$$

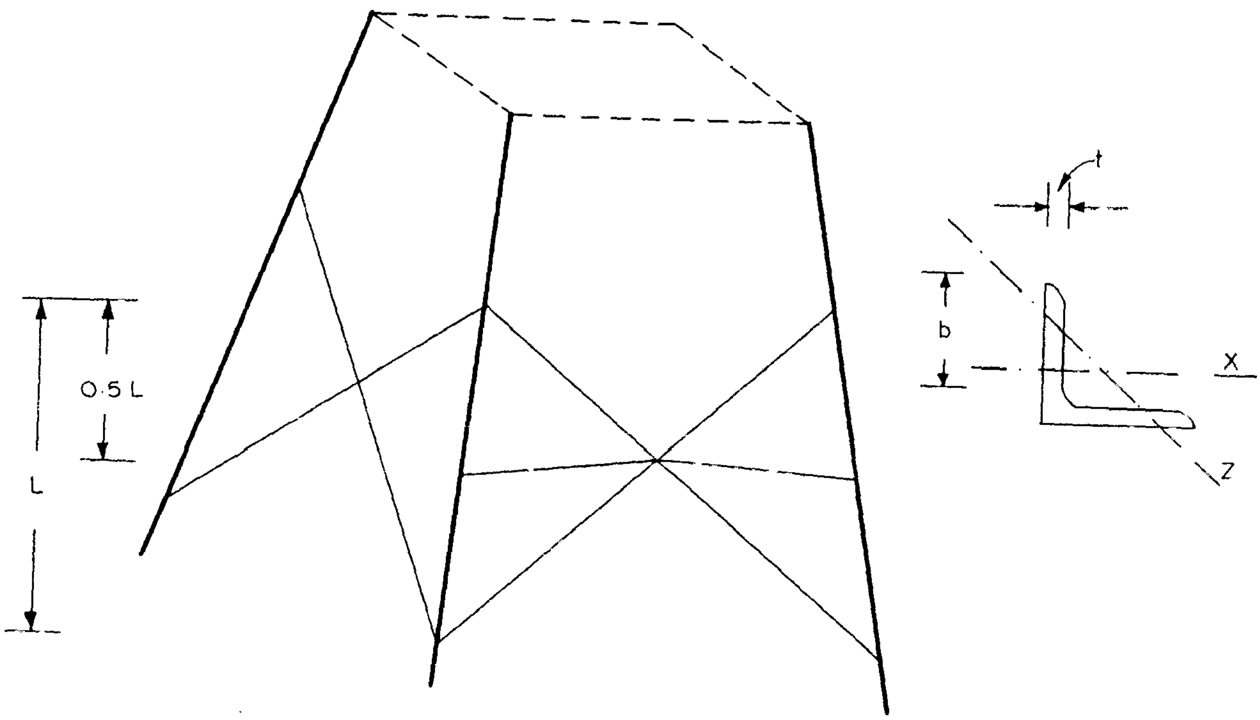
where E = modulus of elasticity of the material in kgf/cm^2

The Crippling stress is

$$Fcr = Fy \left(1 - \frac{1}{2} \left(\frac{SL}{Cc} \right)^2 \right) \qquad (6.5)$$

Provided that the (b/t) ratio is within i'ts limiting value. When the (b/t) ratio exceeds the limiting value of equation 6.3, the yield stress, Fy, is replaced by F'y where,

$$F'y = \left[Fy\ (1.8 - 0.8(b/t)/(b/t)\ limit \right] \qquad (6.6)$$



EFFECTIVE LENGTHS OF COMPRESSION MEMBERS

FIG. 6·2

In the elastic range,

$SL > Cc$

The crippling stress, Fcr is governed by the overall buckling of the strut and not by local buckling, if (b/t) is less than 20. In this region, the Euler buckling curve is adopted.

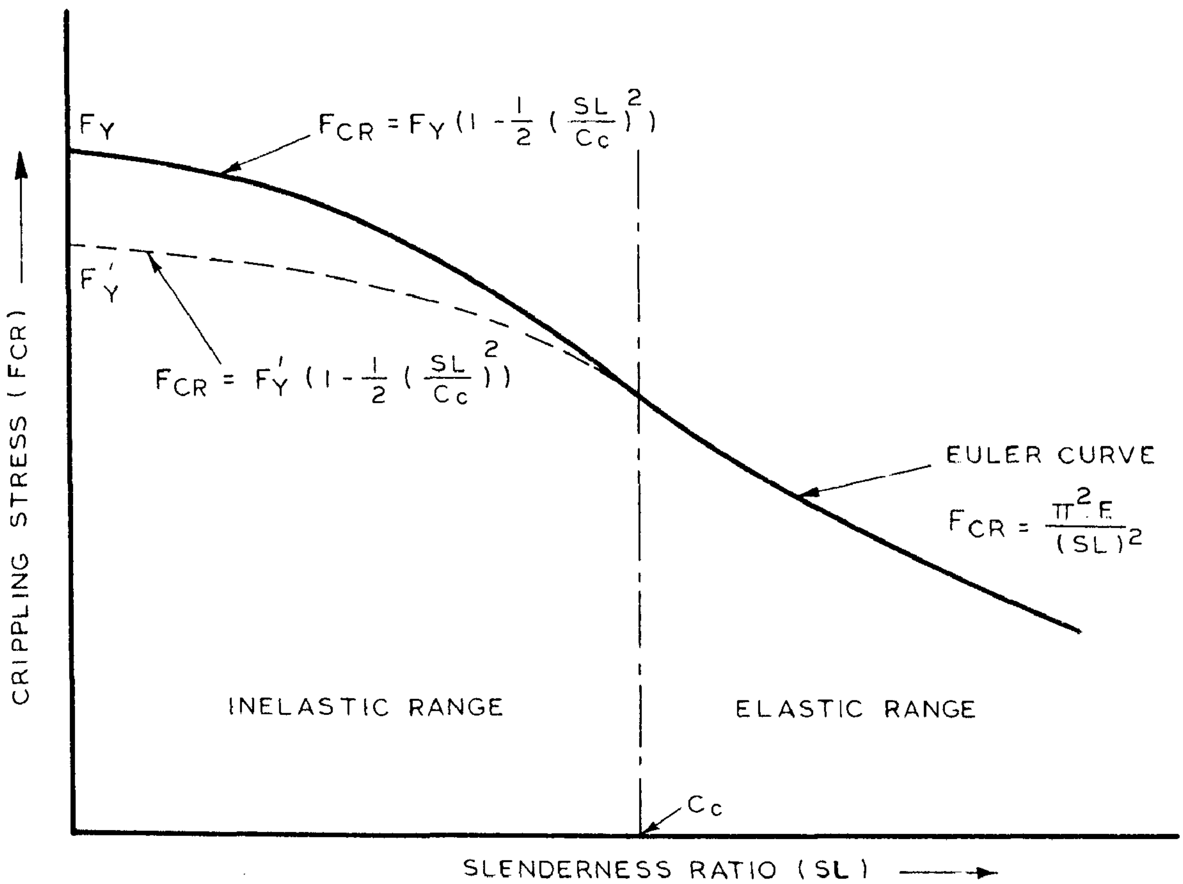
$$Fcr = \frac{\pi^2 E}{(SL)^2} \tag{6.7}$$

These criteria are shown in Fig.6.3.

6.6. Member Design Groups

In the practical design of structure which are comprised of a large number of members, it is always desirable to aim at keeping the number of different member sizes at a minimum, consistent with reasonable economy. This is accomplished here by prescribing that a certain number of members should have identical cross-sectional shape, and each member is assigned a code to indicate to which group of members it belongs.

It is also frequent that members of different design lengths are to be assigned the same size, and since the critical compressive stresses depend on the member length, this would result in increased computational effort. To minimise these efforts, subgroups of members having identical configuration are formed within the group. An array of length codes is given as a part of the input data.



STRUT FORMULAE

FIG. 6·3

A length code 1 is given to the leading member of each group, and a length code of -1 is given to the leading member of each sub-group. No length code is given to the rest of the members.

Upon the completion of the analysis and the determination of the controlling compressive and tensile forces for each member, search for the controlling tensile and compressive forces for each sub group is made. A suitable structural size is determined for each subgroup, and the largest of these size is assigned to the entire group. In this way, fewer designs are made, since otherwise a design for each member of the group would be needed.

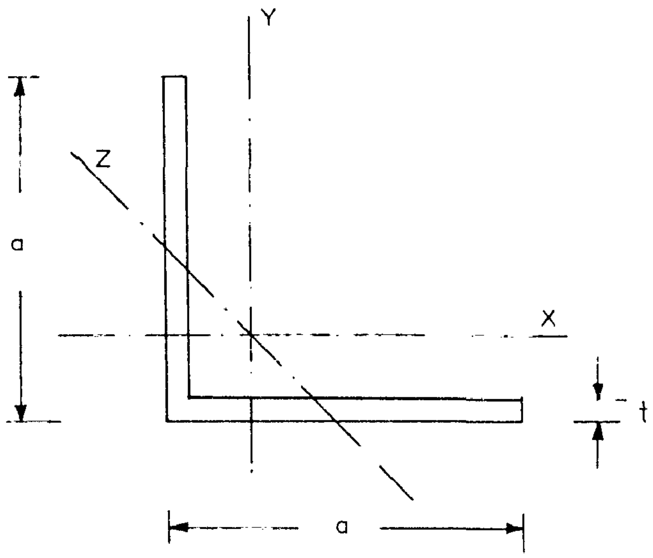
6.7. Member Area Estimation

The purpose of this procedure is to provide a tool for quick estimation and to accelerate convergence to the final design. It is assumed in this section that a continuous range of possible member sizes are available, and as such the computation time required to size the members is less than that required to search a list of available sizes.

To simplify the procedure, only equal-legged angle sections are considered, since this types of section is the predominant structural member used in tower construction. For an equal legged angle section, approximate relations between its geometry and the properties can be easily derived. The following relations between the leg width, 'a' and the radii of gyration are observed for the range of available equal-legged angles (Fig.6.4.).

SECTION AREA $\simeq t(2a-t)$

FIG. 6.4



$$rx \simeq 0.31\ a \qquad (6.8)$$

$$rz \simeq 0.198\ a \qquad (6.9)$$

In order to further simplify the estimation procedures, only the overall buckling failure mode is considered, and a restriction on the section geometry is therefore imposed. This restriction takes the following form.

$$\frac{a}{t} = \frac{662.5}{\sqrt{Fy}} \qquad (6.10)$$

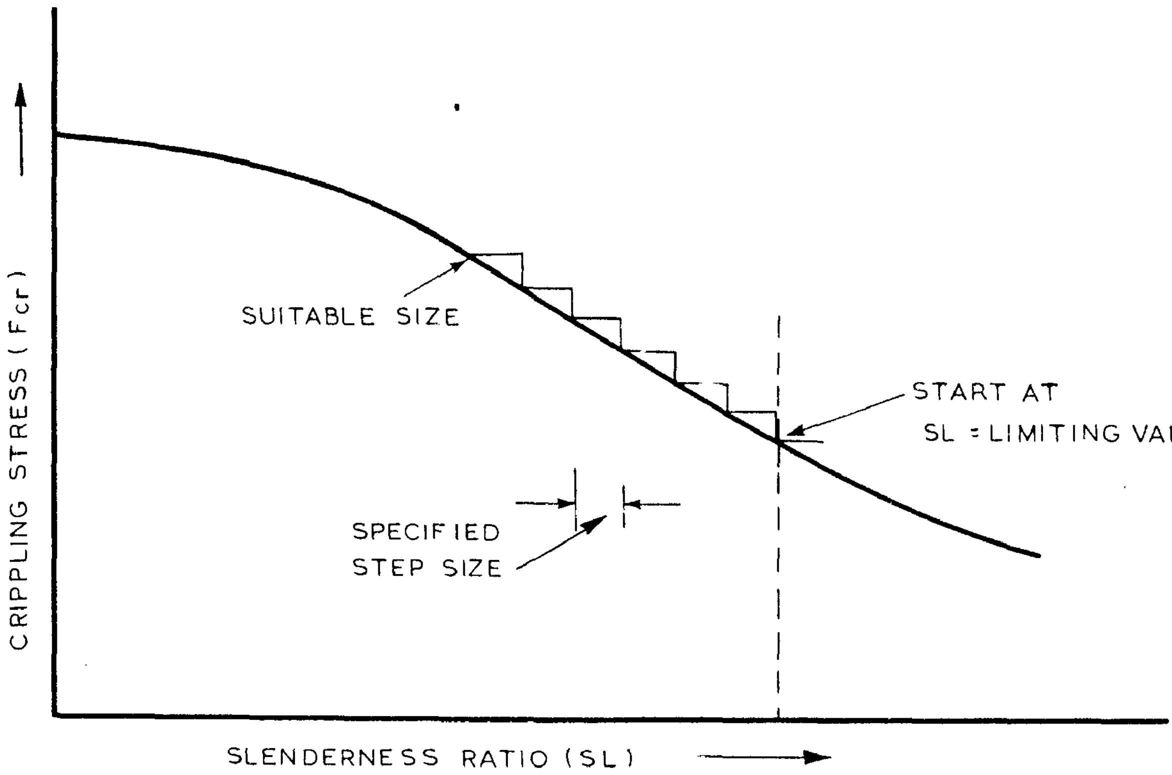
thus the section area can now be expressed in terms of only one variable, the leg width 'a'.

$$\therefore \text{ section area} = a^2 \left[\frac{\sqrt{Fy}}{662.5} \left(2 - \frac{\sqrt{Fy}}{662.5} \right) \right] \qquad (6.11)$$

$$\text{or} = a^2 . F(Fy) \qquad (6.12)$$

where F (Fy) is a function of the yield stress, and constant for a particular material.

To estimate an area to satisfy a compressive load in a member, the limitation and requirements of the member type, as described in sub-sections 6.3 and 6.4, are taken into consideration. By starting at the maximum allowable slenderness ratio for the member, and with a specified step size, an estimated area to satisfy the load requirement is located. This is shown diagramatically in Fig.6.5.



AREA ESTIMATION PROCESS FOR COMPRESSION MEMBER

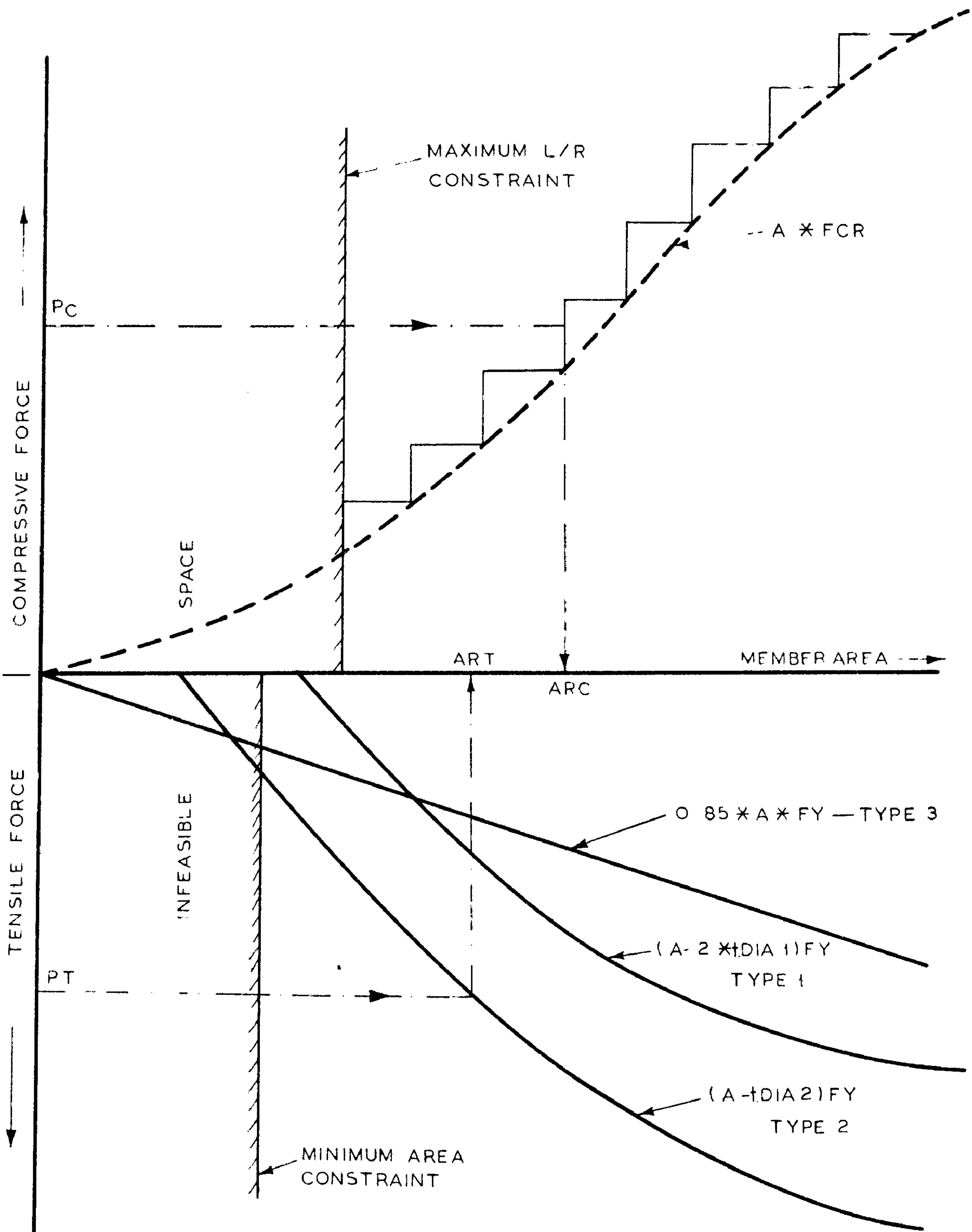
FIG. 6·5

To estimate a member area to satisfy a tensile load requirement, two bolt holes of a specified size are deducted from the gross area of type 1 members, one bolt hole for type 2 members and 15 percent of the gross area for type 3 members. Fig.6.6 presents schematically the area estimation process.

6.8. Size Selection

The size selection procedure uses the information gained from the member area estimation procedure and selects suitable available sizes. In view of the simplified assumption made in the area estimation routine, it is necessary to start the selection with a section lighter than the estimated one. In this work the selection starts with a section having an area of at least 75 percent of that estimated. This would also ensure that the most economical sections will be used. The section is then tested with regard to all the specified requirements, and alterations are made accordingly.

The make up of this procedure implies that a list of all the available sizes and their properties are stored in the memory of the computer. These are, in this case, 32 equal legged angles normally rolled in this country. Properties of these angle sections are given in Table 6.1. Any additional shapes can also be included in the input data which should include their designation and cross-sectional properties.



ESTIMATED AREA = THE GREATER OF 'ART' AND 'ARC'

AREA ESTIMATION PROCESS

FIG. 6·6

Table - 6.1

Properties of Available Angle Sections

Sl. No.	Area sq.cm.	Leg Width cm	Thickness cm	R_{min} cm	R_{sq} cm
1	3.78	4.0	0.5	0.77	1.20
2	4.28	4.5	0.5	0.87	1.36
3	4.79	5.0	0.5	0.97	1.52
4	5.07	4.5	0.6	0.87	1.35
5	5.27	5.5	0.5	1.06	1.67
6	5.68	5.0	0.6	0.96	1.51
7	5.75	6.0	0.5	1.16	1.82
8	6.25	6.5	0.5	1.26	1.99
9	6.26	5.5	0.6	1.06	1.66
10	6.84	6.0	0.6	1.15	1.82
11	7.44	6.5	0.6	1.26	1.98
12	8.66	7.5	0.6	1.46	2.30
13	8.96	6.0	0.8	1.15	1.80
14	9.29	8.0	0.6	1.56	2.46
15	9.76	6.5	0.8	1.25	1.96
16	10.47	9.0	0.6	1.75	2.77
17	11.38	7.5	0.8	1.45	2.28
18	11.67	10.0	0.6	1.95	3.09
19	12.00	6.5	1.0	1.25	1.94
20	12.21	8.0	0.8	1.55	2.44
21	13.79	9.0	0.8	1.75	2.75
22	14.02	7.5	1.0	1.45	2.26
23	15.39	10.0	0.8	1.95	3.07
24	17.08	11.0	0.8	2.18	3.40
25	19.03	10.0	1.0	1.94	3.05
26	21.12	11.0	1.0	2.16	3.37
27	25.12	13.0	1.0	2.57	4.02
28	29.88	13.0	1.2	2.56	3.99
29	34.77	15.0	1.2	2.97	4.61
30	45.65	15.0	1.6	2.94	4.58
31	50.79	15.0	1.8	2.91	4.54
32	57.80	20.0	1.5	3.91	6.17

6.9. Design of a Single Circuit Tower

The 132 kV, 30^{o}, single circuit tower shown in Fig.6.7 is an already existing structure. This structure was designed based on a manual method of analysis and could be considered as representative of current practice for the design of this type of structures. This tower has been redesigned here by means of the procedure developed. In the first case the tower was redesigned with 12 member groups. The results of this design is shown in Fig.6.8. as that of tower 1. Here, the initial weight of the tower is the actual weight of the existing tower. The results in this case, indicate no saving in steel, since there is practically no difference in the initial weight and final weight of the tower 1. In the second case, the tower was redesigned with the same configuration and also same number of member groups, but with a higher initial weight. This was done to show that this automated design is capable of producing the same final design irrespective of the initial member sizes. It can be seen from the results of this trial design shown in Fig.6.8 (Tower 2), that in the first estimation cycle itself the weight of the structure dropped from 5209 kg to 2088 kg. From the second estimation cycle onwards the structure weight at each stage of estimation and design cycle coincide, till the final design is reached. This indicates the tendency of the results of automated design to converge fast to the same final structure,

132 KV S C TOWER FOR 30° DEVIATION
(100 KG WIND PRESSURE ZONE)
SCALE : 1 100

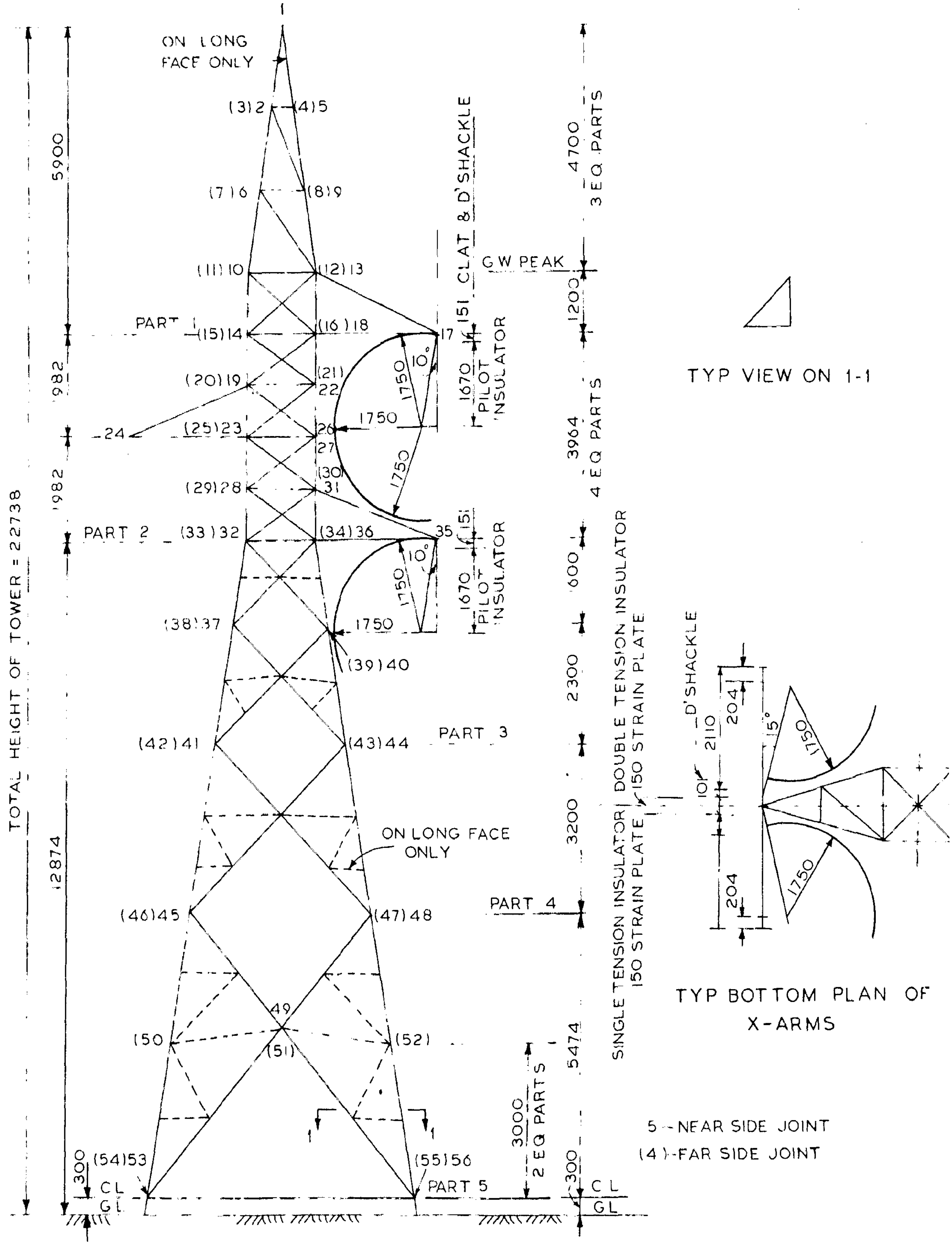
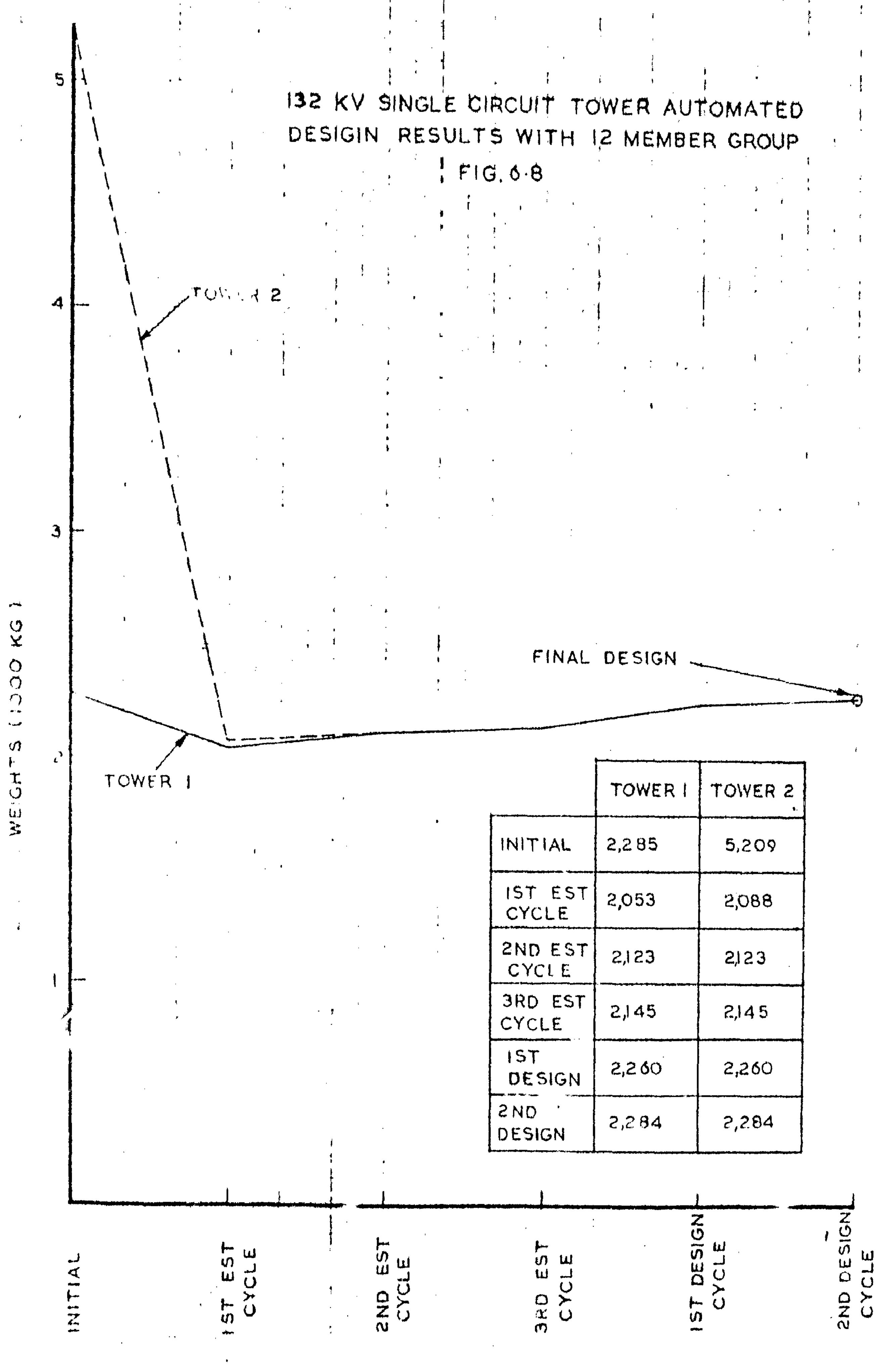


FIG. 6 7

132 KV SINGLE CIRCUIT TOWER AUTOMATED DESIGIN RESULTS WITH 12 MEMBER GROUP

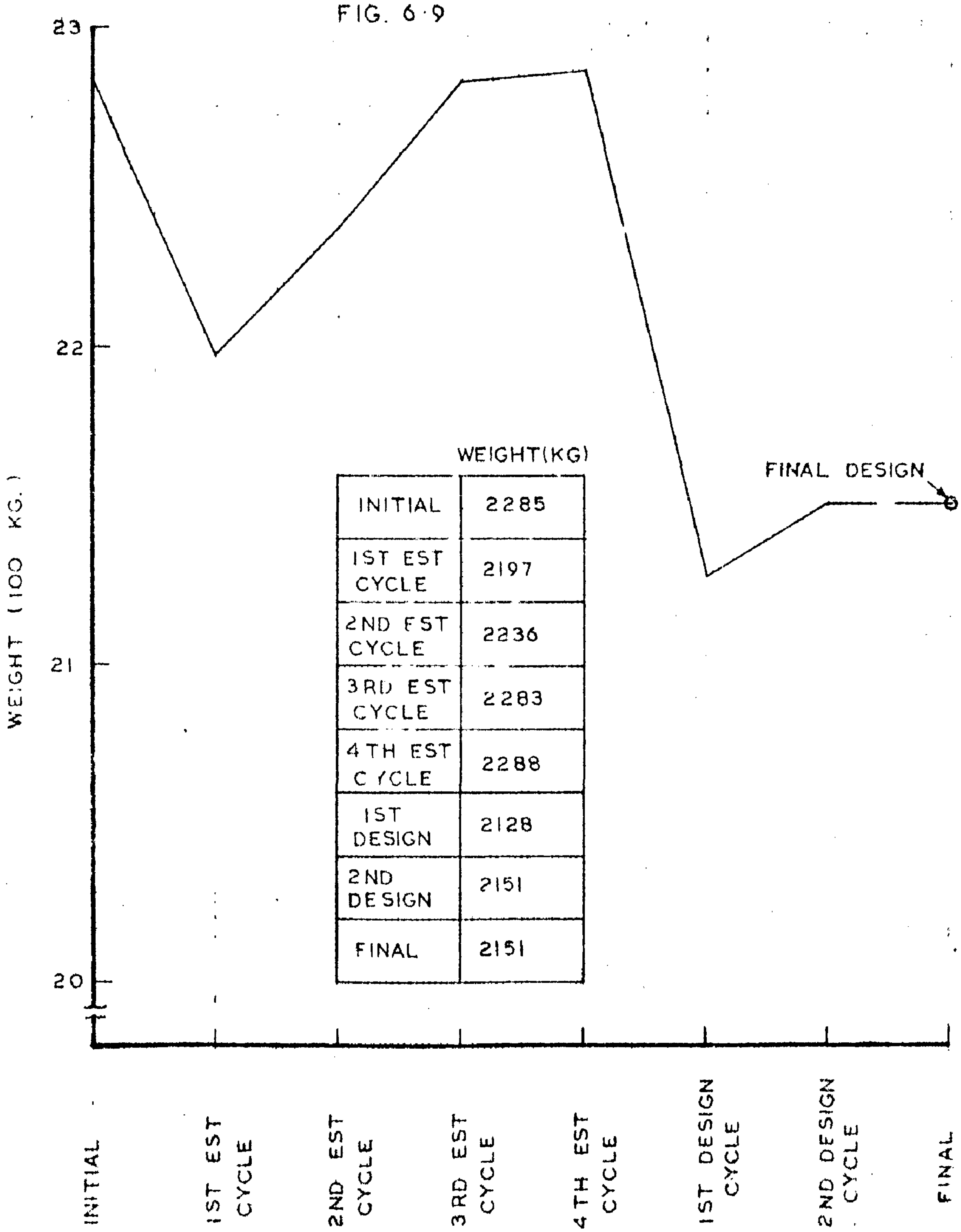
FIG. 6.8



	TOWER 1	TOWER 2
INITIAL	2,285	5,209
1ST EST CYCLE	2,053	2,088
2ND EST CYCLE	2,123	2,123
3RD EST CYCLE	2,145	2,145
1ST DESIGN	2,260	2,260
2ND DESIGN	2,284	2,284

AUTOMATED DESIGN RESULTS OF 132KV SINGLE CIRCUIT 30° TOWER WITH 31 MEMBER GROUPS

FIG. 6·9



	WEIGHT(KG)
INITIAL	2285
1ST EST CYCLE	2197
2ND EST CYCLE	2236
3RD EST CYCLE	2283
4TH EST CYCLE	2288
1ST DESIGN	2128
2ND DESIGN	2151
FINAL	2151

irrespective of the initial member sizes. The computation time on IBM 370 computer for the tower 1 and tower 2 were 3 min. 5.1 seconds and 3 min. 12.32 seconds respectively.

In the third case, when the same tower was redesigned with 31 member groups, the automated design results show (Fig.6.9), a saving in the tower weight by as much as 5.8 percent. The computation time in this case was 3 minute 19.09 seconds on IBM 370 computer.

CHAPTER - VII

CONCLUSIONS

An efficient automated design procedure for the design of self supported transmission line towers has been presented herein. The programme performs all operations within the computer core. It has demonstrated its superiority over the conventional design, in the case of the single circuit tower discussed, where a saving in the structure weight would be achieved by its use, besides producing a more reliable design.

The following specific conclusions can be drawn from the study in this dissertation.

(1) A three dimensional analysis takes into consideration the entire structure as it is, with all the loads acting at their true points of application. The results from such an analysis is accurate and any design based on these results is more reliable. Besides, a three dimensional analysis is essential in certain cases like when all the tower footings are not at the same level. The automated design procedure developed will be specially advantageous in such cases to make a more safe and economical design.

(2) From the trial runs made in the case of a 132 kV single circuit tower, it is observed that the automated design programme developed has a faster convergence to the

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final design, irrespective of the initial member sizes assumed. The time taken to design the single circuit tower with 56 joints, 191 members and 12 member groups was 3 minutes 5.1 seconds and for the same tower with 31 member groups the time taken was 3 minutes 19.1 seconds on IBM 370 computer. This time saving design procedure will not only free the engineer from long hours of hand calculation, but will also be able to provide quick and reliable answers to certain problems faced during advanced stage of construction, such as to check whether certain increased weight or wind span for an angle tower is within safe limits in special locations, where the angle of deviation utilised is less than the designed value. In the absence of such facilities the decision has often to be taken, based on experience and engineering judgement, rather than reliable analytical data.

(3) In the design of transmission line towers it is usually aimed to keep the number of member groups as few as possible, for the reasons of convenience in detailing, fabrication and handling. Automated design was therefore, carried out for the single circuit tower with only 12 member groups. Although there was no saving in the total weight, it resulted in a more reliable design for the same weight as that of the conventional design. The automated design therefore gives a safer tower for the same weight

as that of conventional design, with comparatively lesser number of member groups.

(4) A saving in steel of the order of 6% over that of the conventional design was found in the case of the single circuit tower discussed, when designed with 31 member groups. This shows that the automated design procedure can achieve a considerable saving in the weight of the tower, while improving its safety.

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