RELIABILITY ANALYSIS OF PRESTRESSED ELECTRIC TRANSMISSION CONCRETE POLES

BY

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THESIS DECLARATION

I hereby declare that this Thesis has been written by me and that it is a record of my own research work. It has not been accepted in any previous publication for a higher degree. All sources of information are specifically acknowledged by means of references.

S. H AHMAD

DECEMBER, 1997

THESIS APPROVAL

This Thesis entitled "Reliability Analysis of Prestressed Electric Transmission Concrete Poles", by Mal. SHEHU HADI AHMAD meets the regulations governing the award of the degree of Master of Science (Structural Engineering) of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This work is dedicated to my children:

Naja'atu

Saliha

Sufyan

Kamaluddeen (Arafat).

ACKNOWLEDGEMENT

I am truly indebted to the kindness and generosity of my main supervisor DR. J.O. Afolayan for freely supplying the necessary data and Computer package which I used in the analysis of my work; more importantly, for taking his time patiently to go through the work and making useful suggestion where necessary. I have found my association with him invaluable and I can only pray that God bless him and his family.

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I pray for Allah's continued guidance!

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LIST OF SYMBOLS

b = minimum width of section/breadth of beam

d = effective depth of section

h = overall depth of section

A = gross area of section

A_c = gross area of concrete section

H = height of pole

f_{cu} = characteristic strength of concrete

f_{pu} = characteristic strength of prestressing tendons

A_{ps} = Area of prestressing steel

n_s = number of steel tendons

B = breadth of section

D = depth of section

t_f = thickness of flange

t_w = thickness of web

t_m = minimum thickness of section

E = elastic modulus of section

I = moment of inertia of section

J = polar moment of inertia of section

Z = section modulus

d_c = diameter of cables

N = number of cables

L_1 and L_2 = span of poles on either side

f_{pb} = stress in tendons at failure

f_{pe} = effective prestress after all losses

f_{pt} = stress due to prestress only

ρ_p = wind pressure on pole (N/m^2)

ρ_c = wind pressure on cables (N/m^2)

P_p = resultant wind load on pole

P_w = resultant wind load on cables

P = total concentrated load on pole

W = total uniform distributed load on pole

P_e = effective prestressing force

P_i = initial prestressing force

L_r = percentage loss ratio

X_i (i = 1,N) = Basic variables

R = Resistance effect

S = Load effect

M = safety margin

β = reliability or safety index

P_f = Probability of failure

E_c = expected total cost

E_i = initial cost

E_f = expected failure cost

E_{θ} = failure cost parameter

ϕ = phi function

E_v = cost function

E_{pi} = potential loss in respect of failure due cause 'i'

r = interest rate

q = inflation rate

T = design life

P_{fi} = probability of failure due to event associated with cause 'i'

P_{fl} = life time failure probability

μ_m = mean value of the safety margin

σ_m = standard deviation safety margin

μ_R = mean value of resistance effect

μ_s = mean value of load effect

σ_s = standard deviation of load effect

cov = coefficient of variation

F_R = probability distribution function of resistance effect

f_s = probability density function of load effect

f_{amin} = minimum allowable stress in section

f_{amax} = maximum allowable stress in section

f_{ctp} = permissible tensile stress in concrete

f_{ccp} = permissible compressive stress in concrete

f_{cw} = working stress in concrete

M_A = maximum moment applied

M_{ult} = ultimate moment

M_{UR} = ultimate moment of resistance

M_T = torsional or twisting moment

r_m = partial factor of safety for material

V_{co} = shear resistance for uncracked prestressed section

V_{CR} = shear resistance for cracked prestressed section

f_{cp} = longitudinal compressive stress due to prestressing force

f_t = principal tensile stress

M_o = moment required to produce zero stress

y = distance from the top fibre to the neutral axis

V_c = allowable shear stress in section

Q = total shear force on section

V = total vertical component of load on pole

W_D = self weight of pole, attachment etc

W_t = total vertical weight on pole excluding self weight

W_c = weight of conductor cables

P_1 = horizontal component of load due to sag on cables in span 1

P_2 = horizontal component of load due to sag in cables in span 2

S_1 = sag in cables in span 1

S_2 = sag in cables in span 2

l_e = effective height of member

L = clear height of pole

K = effective height factor

N_{cr} = critical buckling load

N_a = total axial load

δ_{max} = maximum deflection

δ_{perm} = permissible deflection

δ_T = total deflection on pole

δ_p = deflection due to load (P)

δ_w = deflection due to load (W)

δ_B = deflection coefficient

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ABSTRACT

Reliability and economy are inherently competing demands; the study of which is becoming an increasing concern for the designers of electric transmission line structures and more generally to the profession of Structural Engineering. The work covered in this study deals with the assessment of the reliability (safety) levels of the prestressed electric transmission concrete poles; as they are used for support of transmission line.

Safety levels at various heights of tappered I - section poles have been investigated for the serviceability limit states in various categories Viz:- Compressive and Tensile working stress conditions, deflection requirements and stability criteria. Several combinations of load and resistance effects are considered and implied safety levels for this class of poles are estimated.

A general purpose computer program package based on FORM5 (First Order Reliability Method; Version 5) was adopted for the estimation of the safety levels associated with the different limit states examined. As far as the serviceability limit states are concerned, it is found that the requirements for the working stresses in compressive and tensile and the criteria for deflection and stability are very conservative. In most cases, the critical points along the poles height fall within the solid rectangular section. It is also noted that the failure of the poles within the first 1.0 to 1.5m from the apex is quite alarming. The conditions governing the deflection requirements call for an immediate reappraisal since the associated safety levels are extremely very low.

The results have been plotted so that the safety level (0) can be read off for any given limit state condition for a pole at various heights for any load and resistance effects

desired. In addition, the cost-effectiveness of the design requirements for tensile criteria is also estimated.

CHAPTER ONE

1.0 INTRODUCTION

1.0.1 Historical B? Transmission Poles

Traditionally, p en manufactured of timber and steel before the development of reinforced concrete (Gerwick, 1971). In most countries of the world, reinforced concrete poles have been extensively used for railways, power and signals lines, lighting poles, antenna masts, telephone transmission, low and high voltage electric power transmission and as substation towers (Gerwick, 1971). This was necessary owing to the advantages of reinforced concrete poles over timber and steel poles. The main advantages have been enumerated (Essien, 1990; Gerwick, 1971; Krishna,1988), as:-

(a) resistance to bush fires near ground levels,

(b) resistance to freezing and thawing,

(c) durability against corrosion and termites or insects attack when embedded in the earth,

(d) ease of moulding and installation, and

(e) overall economic consideration when mass produced as in pre-cast techniques (Plate. 1).

One of the primary considerations in designing any structure or component is that, the strength must be sufficiently greater than the stresses to ensure an adequate margin of safety (Shigley, 1976). Structures used for supporting overhead transmission[1] lines

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[1]The term "transmission" has wrongly been applied in this sense (as quoted in most reference), for structures supporting low voltage lines. In the technical sense, the correct name for such structures supporting low voltage lines should be "Distribution"

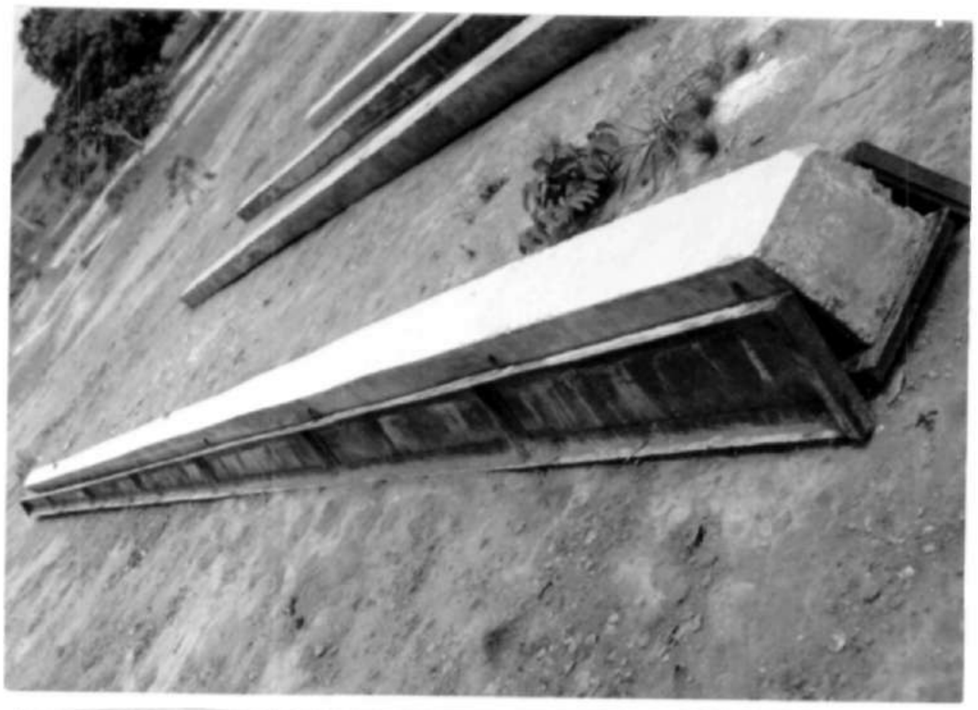


PLATE 1: MASS PRODUCTION OF R. C. POLES USING STEEL MOULDS;

recorded at a site in Gwagwalaada, Abuja

lines, whereas, "Transmission" is for high voltage lines where steel tower frames are used (NEPA, 1995).

network must be so designed to possess adequate strength and stability and must be reliable enough to support their own weights, weights of the transmission lines (cables), and any additional loads such as those due to wind pressure, weight of people carrying out repairs, etc (Gerwick, 1971).

1.1 PROBLEM STATEMENT

Recent studies (Essien, 1990) and survey conducted around Zaria, Kaduna and Abuja metropolises have indicated that, these structures are prone to failures and collapses; (Plate.2 and 3) notably at peak times of weather inclement such as during heavy rain storms, strong whirl winds , and sometimes owing to line snaps or as a result of damages due to impact or collision by vehicles etc.

From available data (NEPA, 1995), it is estimated that, no fewer than 741,555 kilometer length of distribution lines have so far been installed within the Federal Capital Territory (FCT), Abuja in which concrete poles of various sections, are used as the basic supporting structures. This figure is expected to rise even higher with time at an average rate of 15% of this total each year; given the pace of development in the FCT. According to another recent survey (Torru, 1995) conducted between the period of 1990 - 1995, the price of a pole has varied between the range of ₦3,000.00 in 1990 and ₦7,500.00 in 1995; representing about 250% increase in price in five (5) years. Now,

PLATE 2: FAILURE BY TILTING OF POLE;

recorded around F. C. D. A. Life - Camp, Abuja

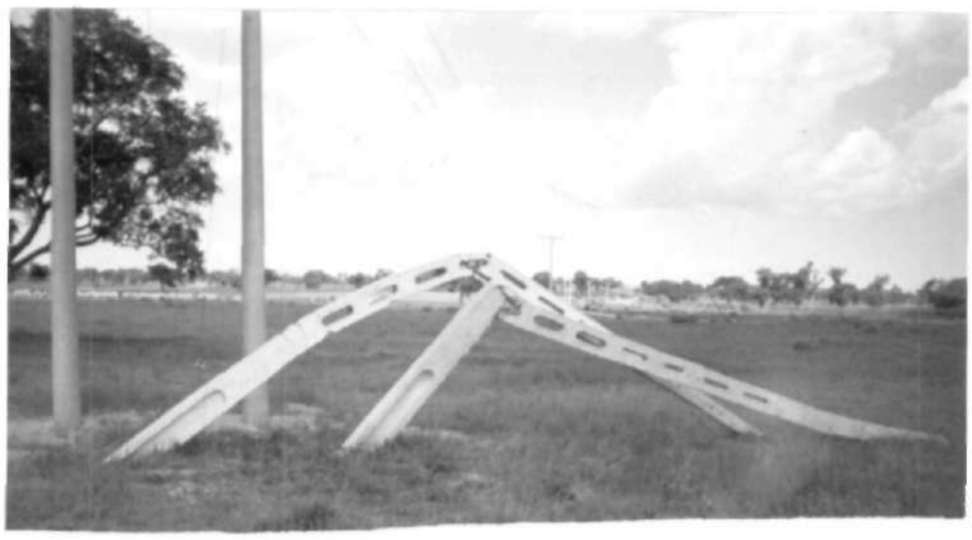


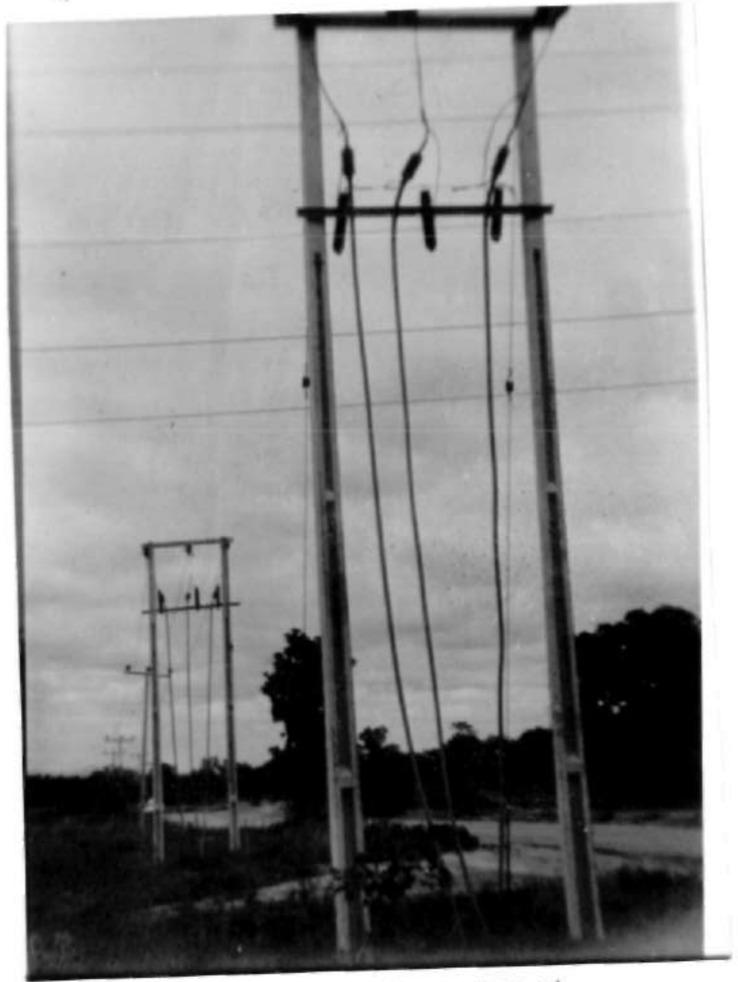
PLATE 3: FAILURE BY TOTAL COLLAPSE OF POLE;

recorded along Kaduna - Jos highway

in any 1km length of a distribution line, the poles are spaced at about 50m c/c (spans), i.e; about 20 units are used. Thus, over a total of 20 x 741,555 (14,831,000) units of concrete poles are currently in use within the FCT. This figure is merely a conservative estimate or empirically computed as no consideration is given for the double and multiple poles (Plate.4) sometimes introduced along the line network.

Torru (1995), estimated the percentage failure rate due to accidental causes and those resulting from damages during manufacture, transportation and erection difficulties to be about 0.05% of the total (14,831,000), or 0.05/100 x 14,831,000 = 7,415.5 poles are involved. The current unit price of poles (as at 1995) is ₦7,500. Thus, losses resulting from failures by 1995 estimates = ₦7,500 x 7,415.5 = ₦55,616,250.00. This amount (₦55.6m approx), represents the value of what is being lost within the FCT alone; which is not even yet as industrialised, congested or fully developed as places like Lagos, Ibadan, Kaduna, Kano etc. If such a projection were to be made for the whole country, the cost of these failures can be very enormous as it may be involving hundreds of millions of Naira.

The failures of these structures not only constitutes economic wastes, but often times are hazardous, and hence liable to cause severe injuries or even loss of lives; especially in the highly industrialised, congested urban centers. Besides, there is also the psychological and social effects these failures have on the consumers of the form of services they relay (electricity).



a - double poles system at

road crossing

b - multiple poles system

PLATE 4: USE OF DOUBLE AND MULTIPLE POLES SYSTEM ALONG A TRANSMISSION LINE NETWORK; recorded along the Outer Northern Expressway (ONEX), Abuja

1.3 JUSTIFICATION OF STUDY

In the light of the problems raised above and from the experiences highlighted by other researchers (Essien, 1990; Torru, 1995), it was considered necessary or at least desirable to carry out this study in order to improve on the strength of the pole so as to render it more reliable and thereby controlling the menace of these incidences. The tittle of this research work "RELIABILITY ANALYSIS OF PRESTRESSED ELECTRIC TRANSMISSION CONCRETE POLES", is so selected with a view to carry out the reliability study on such members and subsequently evaluate the parameters influencing their strength.

1.3 GENERAL AIM AND OBJECTIVES

1.3.1 Aim

To identify and analyse the effect of the various design parameters (variables) influencing the strength and reliability of electric transmission concrete poles.

1.3.2 Objectives

(a) To formulate and obtain solutions based on reliability analyses results for poles using probabilistic methods and to give recommendations for future design work.

(b) To compute safety levels (reliability indices) at various heights of the pole for the various limit states investigated.

1.4 METHODOLOGY

Most structural reliability procedures are iterative in nature, and hence require many design analyses,modifying member dimensions, compositions, etc. while considering the various constraints imposed on the design at all levels until the solution is improved. Because of the extensive iterations to be carried out in the treatment of the problems of

a - Artillery Barracks

b - C.B.N. Qtrs. Barnawa

c -Shopping Complex, Barnawa

PLATE 5: FLEXURAL FAILURES IN I-SECTION POLES (AROUND THE APEX)
recorded at various locations in Kaduna Metropolis

CHAPTER TWO

2.0 LITERATURE REVIEW

In recent times, the development of prestressed concrete poles has virtually replaced the use of the conventional reinforced concrete pole owing to the former's increased resistance to cracking, dynamic loadings, structural stability and the use of light weight sections, thereby making it easier to be transported over long haulages (Astill et al, 1975; Essien, 1990; Gerwick, 1971; Hulse et al, 1987; Krishna, 1988; Morrell, 1977).

Three (3) classes of the prestressed concrete poles are in use today namely:-

(a) I - section tappered poles,

(b) Rectangular/square hollow sections, and

(c) Circular hollow sections.

2.1 REVIEW OF FLEXURAL FAILURES IN TRANSMISSION POLES

A recent study (Essien,1990) highlighted some of the causes of failures of transmission concrete poles and also the various modes of failure encountered in them. The failures have been traced to a number of factors viz:-

(a) Inadequacy in the analysis of the critical loads and forces acting on the pole,

(b) Fatigue failure due to cyclic wind loading which causes resonance on the pole,

(c) Temperature stresses due to tension and lack of sag in the cables, and

(d) Torsional (twisting) moment from the cable due to skew snapping of wires etc.

Most electric poles in use to date are designed hollow and tappered (Gerwick, 1971). While the hollow core has been useful in reducing weight and providing raceway for electric wires etc, the tappering reduces bending moments arising from the cantilever action on the pole, and also wind and seismic loads over the top portion.

1. Fracture of the material of which the member is made, which is characteristic of brittle materials.

2. Initiation of inelastic (plastic) behaviour in the material; which is generally exhibited by ductile materials.

When a member is subjected to simple tension or compression, failure as defined above, occurs when the applied axial stress reaches the limiting value σ_0. If the member is ductile, σ_0 represent the yield point or the yield strength of the material. If the member is brittle, σ_0 represents the ultimate or the fracture strength. However, when a member is subjected to bi-axial or tri-axial state of stress, the cause of failure; whether by fracture or by inelastic action, is unknown and hence failure is predicted by one or several theories of failure (Shigley, 1976). Such a stress condition can result from the application of a torque and an axial

load, OR a torque, an axial load and a bending moment, to a member.

2.3 DESIGN METHOD FOR ELECTRIC POLES

According to BS 607,1960 - British Standard "Specification for Electric Transmission and Traction Lines", and the Indian Standard Code; IS : 785, 1964, poles are to be designed generally for the following critical loading conditions:

2.3.1 Loading Conditions

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

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

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

(iv) Production, handling and erection stresses - this being a major concern of the prestressed concrete designer.

The IS:785,(1964) classified all poles into eleven (11) categories depending on their heights and the transverse load carrying capacity. This classification is reproduced in Table 2.1

Table 2.1 Minimum transverse load(kN) for different classes of pole (IS: 785, 1964)

Class of poles	Maximum overall height/length (m)	Minimum ultimate transverse load applied at 600mm from top (kN)
1	17.0	30.0
2	17.0	23.0
3	17.0	20.0
4	17.0	14.0
5	16.0	11.0
6	12.5	10.0
7	12.0	8.0
8	12.0	7.0
9	11.0	5.0
10	9.0	3.0
11	7.5	2.0

Note: Minimum overall length of all classes shall be 6.0m

2.3.2 <u>Strength Requirements.</u>

With respect to strength of poles when in function, both codes (BS 607, 1960 and IS: 785,1964), require that:-

(i) a pole shall be designed such that when it is vertical , its strength in the transverse direction shall be sufficient to take an ultimate load equal to the horizontal wind load on wires and pole, multiplied by the load factor.

(ii) the design should ensure that at 40% of the ultimate load, the stresses and strains are such as not to cause any harmful cracking in the pole.

(iii) the minimum load factor for transverse strength of all classes of pole is prescribed as 2.5.

(iv) in the case of pole used for power transmission lines, the strength of the pole in the direction of the line should not be less than 1/4 of the strength required in the transverse direction.

(v) the torsional strength of the pole should not be less that 0.75KNm for classes 1 to 9 (with heights up to 10.0m), and not less than 0.5KNm for poles of other classes (with height below 10.0m).

2.4 FORCES ACTING ON A POLE

Transmission poles are normally erected and arranged in rows to provide supports for the conductor cables (Gerwick, 1971; Reynolds et al, 1976). Even though an infinite number of these poles may be involved (Fig.2.1), they can primarily be classified into two groups namely:-

(a) End poles

(b) Intermediate poles

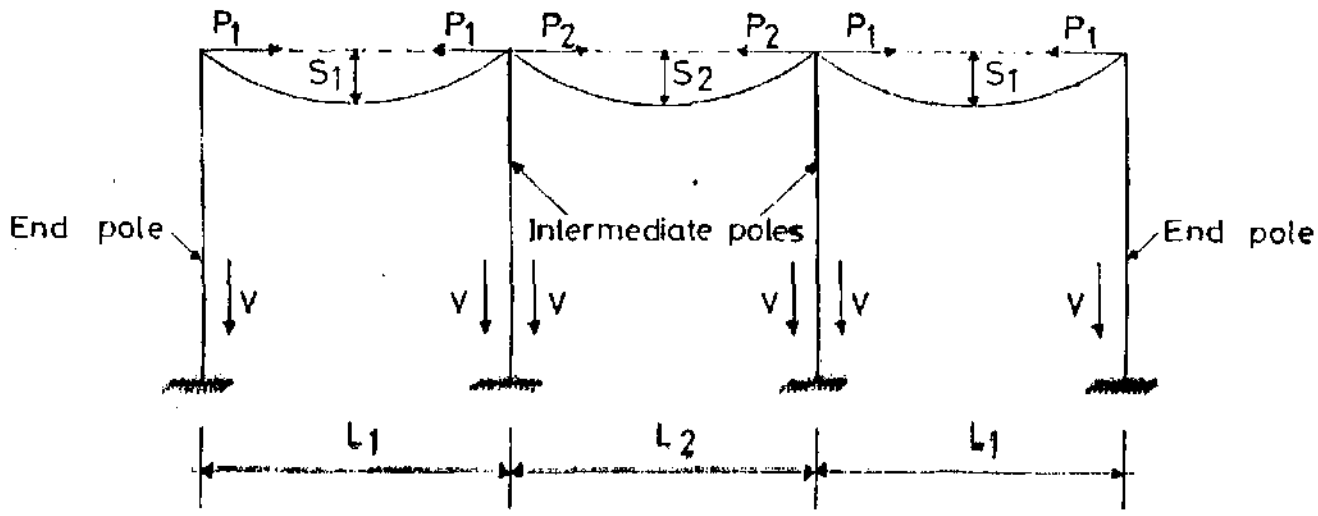


Fig 2.1; Forces acting in the direction of line.

Reynolds et al, (1976), computed the values of these forces for both the ends and intermediate poles as shown in Table 2.2

Table 2.2 Force due to wind and dead loads on pole

S/No	End Pole	Intermediate Pole	Description
1	$P_w = \frac{N\rho_c L_1 d_c}{24}$	$P_w = \frac{N\rho_c (L_1+L_2)\, d_c}{24}$	Transverse force due to wind on conductor.
2	$P_1 = \frac{NW_c L_1^2}{8S_1}$	$P_1 = \frac{NW_c L_1^2}{8S_1}$; $P_2 = \frac{NW_c L_1^2}{8S_2}$	Horizontal force in the direction of the line.
3	$V = \frac{NW_t L_1}{2} + W_D$	$V = \frac{NW_t (L_1+L_2)}{2} + W_D$	Total Vertical force
4	$P_p = \rho_p BH$	$P_p = \rho_p BH$	Resultant Horizontal force due to wind on the pole.

where,

ρ_c = Wind pressure on conductors

ρ_p = wind pressure on pole

d_c = norminal diameter of conductor

L_1 and L_2 = end and intermediate spans respectively

N = Number of conductors.

S_1 and S_2 = sags in the cables due to spans L_1 and L_2 respectively

P_1 and P_2 = horizontal forces in the direction of the line due to sags in the cable S_1 and S_2 respectively

W_c = Weight of Conductor

W_t = Self weight of pole

W_D = total weight of pole, attachment etc.

B = Breadth of the pole

H = total height of pole from ground level

P_w, P_p and V are as described in Table 2.2

It was also shown (Reynolds et al, 1976) that the maximum load that the cables transmit to the pole is when the wind is loaded on them and acts horizontally transverse to the direction of the line. The horizontal forces acting in the direction transverse to the cables line are shown in Fig. 2.2

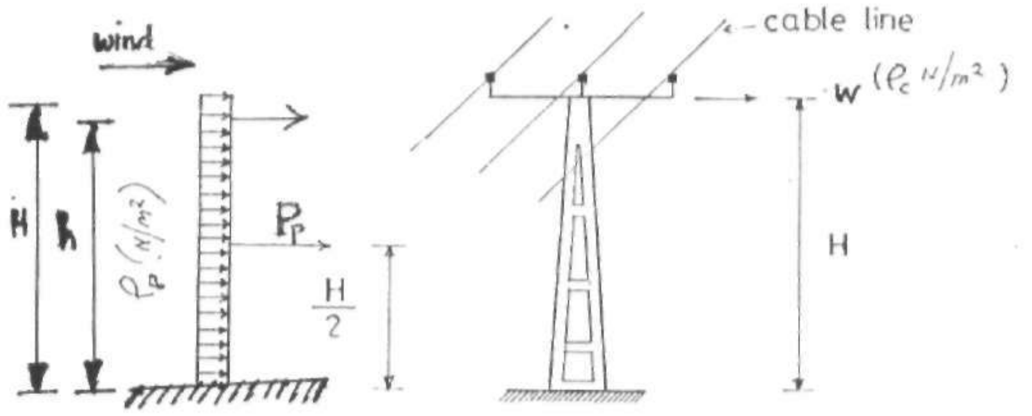


Fig. 2.2 Wind forces acting transverse to direction of line

2.5 SERVICEABILITY LIMIT STATE EQUATIONS FOR POLE DESIGN

The following serviceability checks are to be performed to examine the safety of the design.

(a) Working stress; this interacts the combined bending and torsional as well as the axial stress in the member.

(b) Lateral deflection

(c) Buckling or stability

2.5.1 Working Stress Condition

It is required that the actual applied stresses at working condition should be within the bounds specified by the Codes (BS8110, 1985) viz:-

$$f_{amin} \leq f_{cw} \leq f_{amax} \quad (2.1)$$

where;

f_{cw} = Stresses at working condition

f_{amin} = Minimum allowable stress in concrete

f_{amax} = Maximum allowable stress in concrete

For a prestressed section, these represent the permissible stresses on the top and bottom fibres of a typical stress block as shown in Fig. 2.3

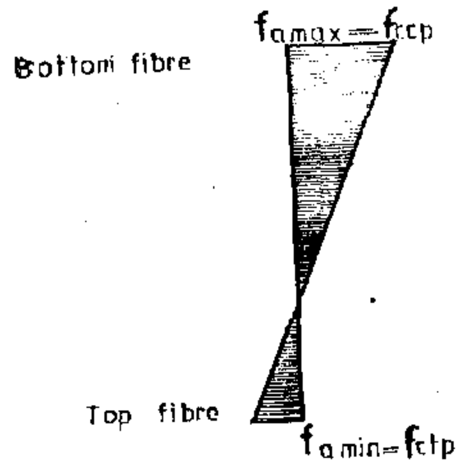


Fig. 2.3: Typical stress block for a prestressed section.

In fig. 2.3, f_{ccp} = Permissible compressive stresses in concrete.

f_{ctp} = Permissible tensile stresses in concrete.

Hence, following Equation (2.1), it implies that;

$$f_{ctp} \leq f_{cw} \leq f_{ccp} \quad (2.2)$$

For a class 1 member in which no tensile stresses are permitted,

CP110, 1972; specified the limiting value for these stresses which are given in Table (2.3).

Thus

$$f_{amin} = f_{ctp} = 0 \tag{2.3}$$

and

$$f_{amax} = f_{ccp} = 0.33 f_{cu} \tag{2.4}$$

Table 2.3: **Compressive Stresses in Concrete for Serviceability Limit State**

Nature of Loading	Allowable Compressive Stresses
Design load in bending	$0.33\ f_{cu}$ In continuous beams and other Statically Indeterminate structures this may be increased to $0.4\ f_{cu}$ within the range of support moments.
Design load in direct compression	$0.25\ f_{cu}$

It was shown (Essien, 1990) that the Stresses at working condition (f_{cw}) for a prestressed member in flexure; where class I member is considered, can be obtained from the expressions:-

(i) Bottom fibre (compression)

$$f_{cw} = \frac{L_r \cdot P_i}{A_c} + \frac{M_T t_m}{J} + \frac{M_A}{Z_T} \leq f_{amax} \tag{2.5}$$

and;

(ii) Top fibre (Tension)

$$f_{cw}=\frac{L_r \cdot P_i}{A_c}-\frac{M_T t_m}{J}-\frac{M_A}{Z_T}\geq f_{amin} \qquad (2.6)$$

In the above expressions, the sign conventions adopted are; compressive stresses are (+ve) while tensile stresses are (- ve)

From equations (2.3) to (2.6) we get:-

(a) for Compressive working stress condition;

$$f_{cw}=\frac{L_r \cdot P_i}{A_c}+\frac{M_T t_m}{J}+\frac{M_A}{Z_T}\leq 0.33 f_{cu} \qquad (2.7)$$

and

(b) for the tensile working stress condition;

$$f_{cu}=\frac{L_r \cdot P_i}{A_c}-\frac{M_T t_m}{J}-\frac{M_A}{Z_T}\geq 0 \qquad (2.8)$$

where;

f_{cu} = characteristic strength of concrete

L_r = loss percentage ratio

P_i = initial prestressing force applied

= f_{pu} x A_{ps} x n_s

f_{pu} = characteristic strength of the tendons

A_{ps} = area of steel tendons

A_c = gross area of concrete

M_T = torsional (twisting) moment

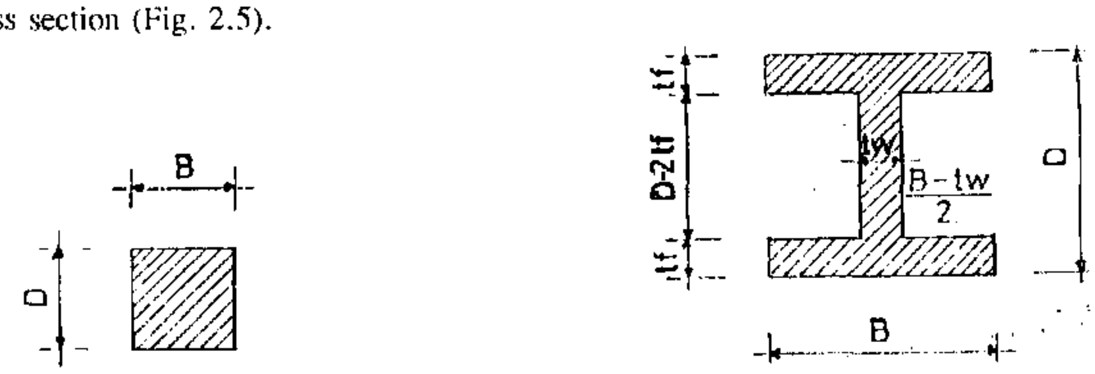
M_A total bending moment applied

J = polar moment of inertia of section

Z = modulus of section

t_m = minimum thickness of section

Noting that the pole is made up of a rectangular solid section portion i.e with x ≤ 2.5 for H > 7.0m and x ≤ 1.5m for H ≤ 7.0m and the remaining I- section, the following properties of section apply to the pole section as shown in Fig. 2.4 ; where x is the distant taken from the top end portion of the pole to the point of change in geometrical cross section (Fig. 2.5).



(a) Rectangular solid section (b) I - section

Fig. 2.4: Properties of section

From fig. 2.4

For the rectangular solid section portion;

$$A_c = B \times D \tag{2.9}$$

$$I_{xx} = \frac{BD^3}{12} \tag{2.10}$$

$$I_{yy} = \frac{DB^3}{12} \tag{2.11}$$

$$J = I_{xx} + I_{yy} \tag{2.12}$$

$$Z_T = \frac{BD^2}{6} \tag{2.13}$$

For the I - section portion of the pole i.e $x > 2.5$ for $H > 7.0$m or $x > 1.5$m for $H \leq 7.0$m

$$A_c = 2Bt_f + (D - 2t_f)\,t_w \qquad (2.14)$$

$$I_{xx} = \frac{1}{12}\left(BD^3 - (B - t_w)(D - 2t_f)^3\right) \qquad (2.15)$$

$$I_{yy} = \frac{1}{12}\left(DB^3 - (D - 2t_f)(B - t_w)^3\right) \qquad (2.16)$$

$$J = I_{xx} + I_{yy} \qquad (2.17)$$

$$Z_T = \frac{2I_{xx}}{D} \qquad (2.18)$$

where;

I_{xx} and I_{yy} are the moment of inertia of the section about the x - x and y - y axes respectively.

t_f and t_w are the thicknesses of the flange and web for the I - section respectively.

B and D are the overall breadth and depth of the section respectively.

To cater for the tappering of the section, it will be necessary to introduce a correction factor on the values of sectional properties calculated in equations (2.9) to (2.18) due to the variations in the cross - sectional dimensions; B and D, along the height of the pole, as shown in Fig. 2.5

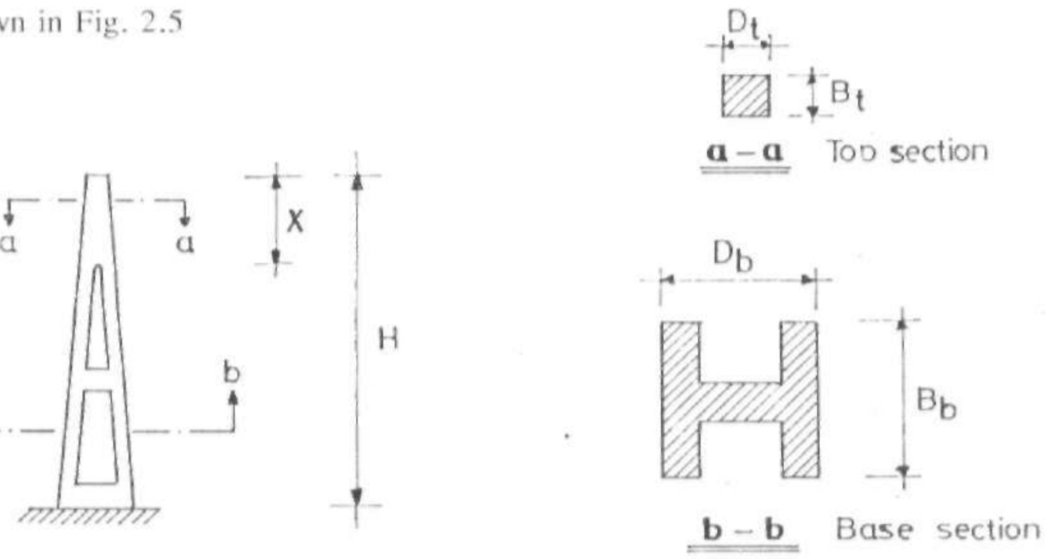


Fig. 2.5: Variation of cross - sectional dimensions with height for a tappered pole section

Gere et al, (1962), have shown that, adjustment in the values of B and D at any point x along the length of the pole; considered from the top end, can be obtained from the expressions as shown:-

$$D_x = D_t\left[1 + \left(\frac{D_b}{D_t} - 1\right)\frac{x}{H}\right] \tag{2.19}$$

similarly

$$B_x = B_t\left[1 + \left(\frac{B_b}{B_t} - 1\right)\frac{x}{H}\right. \tag{2.20}$$

where;

D_x and B_x are depth and breadth of section across any point x, as described before. D_t and B_t are depth and breadth of section at the top end. D_b and B_b are depth and breadth at the base. Previous researchers (Essien, 1990 and Wuyana, 1994) used the following dimensions viz:-

$D_t = B_t = 120$mm; $B_b = 252$mm

$D_b = 318$mm; $t_f = 70$mm and $t_w = 60$mm.

Referring to equations (2.19) and (2.20) we get:-

$$D_x = 120\left(1 + 1.65\frac{x}{H}\right) \tag{2.21}$$

and

$$B_x = 120\left(1 + 1.1\frac{x}{H}\right) \tag{2.22}$$

Equations (2.21) and (2.22) allow the depth and breadth of the section to be calculated across any section taken from the top end of the pole of any given height.

2.5.2 **Lateral Deflection**

The maximum load that may be applied to a member without causing it to cease to function properly may be limited by the permissible elastic strain or deflection of the member (Boresi et al, 1985). The formula for the computation of the maximum deflection of a simply supported prismatic beam section is given as:-

$$\delta_{max} = \delta_B \frac{WL^3}{EI} \tag{2.23}$$

In which; δ_B = deflection coefficient which depends on the type of loading and restraints at the ends.

W = total load on the member

L = total length/ height of member

I = moment of inertia

E = section modulus.

It was shown (Essien, 1990 and Wuyana, 1994) that two types of loading causing lateral displacement or deflection are encountered as shown in Fig. 2.6

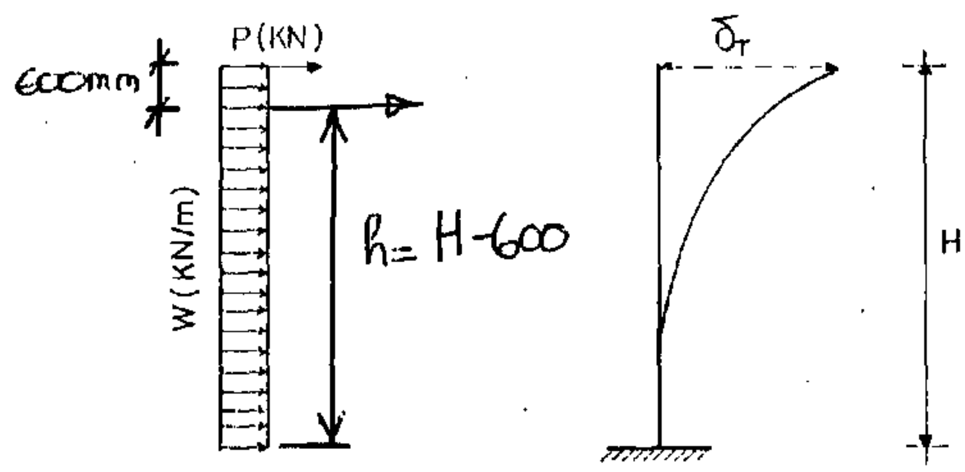


Fig. 2.6: Wind load causing lateral deflection.

of restraints at ends of the member are so selected to ensure that the tendency to buckle is minimised. The critical buckling load (Euler's critical load) for a pinned - ended strut is given as:-

$$N_{cr}=\frac{\pi^2 EI}{l_e^2} \tag{2.28}$$

where;

l_e = equivalent effective height for a pinned - ended strut

For the more general case; l_e = K.L which implies that:-

$$N_{cr}=\frac{\pi^2 EI}{(KL)^2} \tag{2.29}$$

where;

L = clear height of the member between supports

K = effective height factor which depends on the type of positional and directional restraint at the ends of a member

For the member with one end fixed, while the other end is free, K = 2

Hence

$$N_{cr}=\frac{\pi^2 EI}{4L^2} \tag{2.30}$$

Equation (2.30) allows for the critical buckling load on a member with one end fixed and the other end free to be calculated.

Now the total axial load applied on the member (N_a) is given as:-

$$N_a=P_e+V \tag{2.31}$$

where, P_e = effective prestressing force applied

= n_s x f_{pu} x A_{ps} x L_r

and V = total vertical component of load on pole (Table 2.2)

$$= \frac{NW_t\,(L_1 + L_2)}{2} + W_D$$

therefore, Equation (2.31) becomes;

$$N_a = (n_s \times f_{pu} \times A_{ps} \times L_r) + \frac{NW_t\,(L_1 + L_2)}{2} + W_D$$

2.6 ULTIMATE LIMIT STATE EQUATIONS FOR POLE DESIGN

From Section 2.3.1 (Loading conditions), considering case (i) i.e; bending due to wind load on the pole and cables on the transverse face, it was shown (Essien, 1990), that the total moment applied, M_A, due to above is:-

$$M_A = P_p H_3 + P_w H_2 \qquad (2.32)$$

where

$P_p = \rho_p$ Bh	=	resultant horizontal force due to wind pressure (ρ_p N/m^2) on the pole with contact surface B x H (See Table 2.2)
$P_w = N\rho_e(l_1 + l_2)d_e/24$	=	Transverse force due to wind pressure ρ_e N/m^2 on conductor of diameter d_e, consisting of N conductors of length $(L_1 + L_2)½$ in between poles.
$H_3 \approx h/2$ (approx)	=	distance/height from the base of pole to the point of application of the resultant force due to wind on pole (P_p).
$H_2 \approx$ h (approx)	=	distance/height from the base of the pole to the centroid of the cables lines.

Substituting the values of these quantities in Equation (2.32) we get

$$M_A = \frac{\rho_p B h^2}{2} + \frac{\rho_c N (L_1 + L_2)\, d_c h}{24} \quad (2.33)$$

Equation (2.33) above calculates the maximum applied moment on the pole in the transverse direction.

This is the value of the ultimate moment applied on the pole that must be resisted by the member.

2.6.1 Moment of Resistance of Section

The ultimate moment of resistance (M_{UR}) of a prestressed concrete section has been expressed in just the same way as for an ordinary reinforced concrete section (Astill et al, 1975; Hulse et al, 1987; Krishna, 1988; Martin et al, 1989; Morell, 1977; Moseley et al, 1982). It was shown that, for a prestressed section:-

$$M_{UR} = 0.4 f_{cu} b x (d - x/2) \quad (2.34)$$

and

$$M_{uR} = f_{pb} A_{ps} (d - x/2) \quad (2.35)$$

where

f_{pb} = stress in the tendons at failure

A_{ps} = Area of prestressed steel

x = depth to the neutral axis

b;d = width and effective depth of section respectively.

f_{cu} = Characteristic strength of concrete

CP110 (1972) allows the values of f_{pb} to be calculated from:-

$$f_{pb} = \frac{0.838 f_{pu}}{\gamma_m} \quad (2.36)$$

in which

f_{pu} = Characteristic of the steel tendons

γ_m = Partial factor of safety for materials = 1.15 for steel.

Thus substituting for f_{pb} in equation (2.35) above we have

$$M_{UR}=0.73 f_{pu}A_{ps}(d-x/2) \qquad (2.37)$$

2.6.2 Shear Resistance of Section

Both codes (CP110, 1972 and BS8110, 1985) provided the same expression for the computation of shear stress on the basis of cracked and uncracked section.

For sections uncracked in flexure, it was established that, shear cracks occur in the web (at the center of the beam) but do not extend to the tension face, and the expression for the calculation of the shear strength is given as:-

$$V_{co} = 0.67bh\sqrt{(f_t^2 + 0.8f_{cp}f_t)} \qquad (2.38)$$

It is assumed that, the concrete cracks when the maximum principal tensile stress (f_t) exceed the value of $0.24\sqrt{f_{cu}}$. Hence $f_t \leq 0.24\sqrt{f_{cu}}$ for uncracked section. Thus,

$$V_{co} = 0.67bh\sqrt{(0.058f_{cu} + 0.192.f_{cp}(f_{cu}))^{1/2}} \qquad (2.39)$$

with f_{cp} = longitudinal compressive stress due to the prestressing force.

For sections cracked in flexure, the ultimate shear resistance is given by

$$V_{cr} = (1 - 0.55f_{pe}/f_{pu})v_c bd + \frac{M_o Q}{M} \qquad (2.40)$$

where

d = effective depth of the section

M_o = moment necessary to produce zero stress at depth

= $0.8f_{pt}\, I/y$ wher f_{pt} is stress due to prestress only at

depth d and distance y from the centroid of the concrete section which has second moment of area I

f_{pe} = effective prestress after all losses have occurred and $\leq 0.6f_{pu}$(approx)

v_c = shear stress in section.

I = Second moment of area

Q and M = shear force and bending moment respectively at the section considered due to ultimate loads.

V_{cr} should be taken as not less than 0.1 $bd\sqrt{f_{cu}}$

Substituting for the quantities above in Equation (2.40) we get

$$V_{cr}=0.67v_c bd+0.8f_{pt}\frac{IQ}{yM}\nless 0.1bdf_{cu}^{\frac{1}{2}} \qquad (2.41)$$

Having examined both the cracked and the uncracked conditions, the critical shear force on the section is taken as V_c i.e, a value which is numerically equal to the lesser of V_{co} and V_{cr} from equations (2.39) and (2.41) above.

CHAPTER THREE

3.0 RELIABILITY - Theory and Applications

3.1 Definition:

Reliability has been defined in various ways (Ang et al, 1974; Blockley, 1980; Charles, 1976; Palle et al, 1982), but common to all is the general view that "Reliability is a probability associated with a no-failure performance of a device after an accumulated time in a specific environment with some desired level of confidence.

Reliability is however, a subjective concept (Palle et al, 1982), concerned with maintaining certainty only about what is known and being maximally vague or skeptical about what is unknown (Ang et al, 1974; Palle et al, 1982).

To a structural engineer, reliability can be considered as a development in the field of applied mathematics, treating the problems of probabilistic computations of failures of structural systems with respect to some predefined limit state (Blockley, 1980). The limit state for a structure includes all the various critical conditions which when the structure attains or exceeds during its life time, it becomes unfit for use (Astill et al, 1975; BS8110, 1985; CP110, 1972; Martins et al, 1989; Moseley et al, 1982; Reynolds et al,, 1976). Thus, directly or indirectly probability plays an important role in all engineering problems involving an element of uncertainty. The probability of failure (P_f) has an absolute meaning: the likelihood of occurrence of some specified unfavourable state (Ang et al, 1974).

3.2 Basic Concepts

The basic concept of the classical theory of structural reliability and probability based design have been the subject of many research efforts [Afolayan, 1994 (b)]. In all cases, however, the first step is to decide on the relevant load and the resistance effect, their parameters and functional relationship. Basically, structures and/or structural components fail when they encounter an extreme load, or when a combination of loads causes an extreme load effect of sufficient magnitude for the structure to attain a failure state; which may be an ultimate or serviceability condition (Abatan, 1990; Ang et al, 1974; Crisswell et al, 1986). Hence, the solution of most engineering problems involve partly the prediction of the magnitude of these extreme events or load effect (S) and or partly the prediction of the strength or load-deflection characteristics (R) of each structural component from the information available at the design stage (Blockley, 1980). Palle et al, (1982) have shown that, fundamentally, for a simple structural member selected at random from a population with known distribution function "F_R" of ultimate strength in some specified mode of failure, the probability of failure "P_f" under the action of a single known load effect S is given as:

$$P_f=P(R-S\leq 0)=F_R(S)=P(R/S\leq 1) \tag{3.1}$$

If the load effect S is also a random variable, with distribution function F_s Equation (3.1) is replaced by :-

$$P_f=P(R-S\leq 0)=\int_{-\infty}^{+\infty}F_R(X)\,f_s(x)\,dx \tag{3.2}$$

under the condition that R and S are statistically independent. Equation (3.2) gives the total probability of failure P_f as the product of the probabilities of two independent events, summed over all possible occurrences; namely, the probability P_1 that S lies in the range x, x + dx and the probability P_2 that R is less than or equal to x.

Note that R and S must necessarily have the same dimensions e.g load and load-carrying capacity or bending moment and flexural strength etc (Blockley, 1980; Palle et al, 1982).

$$\text{Thus} \quad P_1 = f_s(x)\,dx \qquad (3.3)$$

and

$$P_2 = F_R(X) \qquad (3.4)$$

Under these conditions, the reliability 'R' is the probability that the structure will survive when the load is applied and is given by:-

$$\Re = 1 - P_f = 1 - \int_{-\infty}^{+\infty} F_R(X)\, f_s(x)\,dx \qquad (3.5)$$

It has also been shown (Palle et al, 1982), that for normally distributed variables (R and S) P_f can be expressed in terms of the *safety margin* M as :-

$$P_f = P(M \leq 0) \qquad (3.6)$$

$$\text{where} \quad M = R - S \qquad (2.7)$$

Referring to Fig. 3.1 we find that,

$$\mu_M = E|M| = \mu_R - \mu_S \qquad (3.8)$$

$$\sigma_M^2 = Var|M| = (\sigma_R^2 + \sigma_S^2) \qquad (3.9)$$

from which;

$$\sigma_M = (\sigma_R^2 + \sigma_S^2)^{\frac{1}{2}} \qquad (3.10)$$

where μ_R, μ_s, σ_R and σ_s are respectively means and standard deviations of the random variables R and S. The reliability index β has been defined as the ratio μ_M/σ_M or the number of standard deviations by which μ_M exceeds zero (Palle et al, 1982; Crisswell et al, 1986).

Thus

$$P_f = \phi\left(-\frac{\mu_m}{\sigma_m}\right) = \phi(-\beta) \qquad (3.11)$$

where;

$$\beta = \frac{\mu_m}{\sigma_m} \qquad (2.12)$$

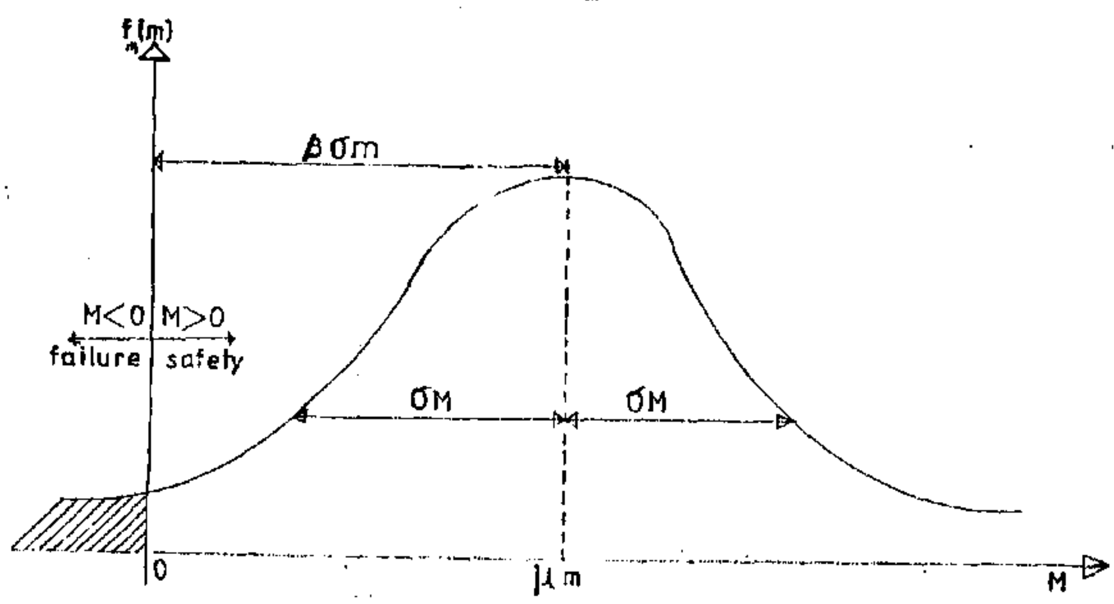


Fig.3.1 Reliability and Probabilistic (normal) Distribution function

3.3 Uncertainty

Classical reliability theory accepts that no structure can be absolutely safe (Ang et al, 1974). Reliability theory is based on probability theory, although it is only used today to indicate the possible values of the various safety factors in use (Ang et al, 1974; Blockley, 1980).

Structural reliability analysis is concerned with the rational treatment of uncertainties in structural engineering design and associated problems of rational decision making (Afolayan, 1993). For the purpose of structural reliability analysis, it is necessary to distinguish between at least three types of uncertainty (Palle et al, 1982) namely:-

(i) Physical uncertainty

(ii) Statistical uncertainty

(iii) Model uncertainty

3.3.1 Physical uncertainty

Whether or not a structure or structural element fails when loaded depends, in part, on the actual values of the relevant material properties that govern its strength. The analysis must therefore be concerned with the nature of the actual variability of physical quantities such as loads, material properties, and dimension which can be described in terms of probability distribution or stochastic processes. But, because physical variability can be quantified only by examining sample data; and since sample size are limited by practical and economic considerations, some uncertainty must remain. This practical limitation gives rise to the so-called statistical uncertainty.

3.3.2 Statistical uncertainty

Statistics as opposed to probability is concerned with inferences, and in particular with inferences that can be drawn from sample observations. Data may be collected for the purpose of building a probabilistic model of the physical variability of some quantity which will involve the selection of an appropriate probability distribution type and also the determination of numerical values of its parameters.

Common probability distributions have between one and four parameters which immediately place a lower bound on the sample size required (in practice, very large sample are required to establish reliable estimates of the numerical values of parameters). For a given set of data, therefore, the distribution parameter may themselves be considered to be random variables, the uncertainty in which is dependent on the amount of sample data, or in general on the amount of data and any prior knowledge. This uncertainty is termed as statistical uncertainty, and it arises solely as a result of lack of information.

3.3.3 Model uncertainty

Structural design and analysis make use of mathematical models relating desired output quantities (e.g the max. deflection at the midspan of a r.c. beam), to the values of a set of input quantities or basic variables (e.g; load intensities, modulus of elasticity, duration of loading, etc). These models are generally deterministic in form (e.g; linear elastic structural analysis), although they may be probabilistic (e.g; calculation of the stochastic waves loading). Furthermore, they may be based on an intimate understanding of the mechanics of the problem (e.g, plastic collapse analysis of a steel portal frame), or they may be highly classical/empirical (e.g punching shear at the tubular joint connection in offshore jacket structures). Thus, with very few exceptions, it is rarely possible to make highly accurate predictions about the magnitude of responses of typical civil engineering structures to loading even when the governing input quantities are known exactly. In other words, the responses of typical structures and structural components/elements contain a component of uncertainty in addition to those components arising from uncertainties in the values of the basic loading and strength variables (Afolayan, 1994b; Ang et al, 1974; Blockley, 1980; Charles, 1976; Palle et al, 1982).

This additional source of uncertainty is termed model uncertainty and it occurs as a result of simplifying assumptions, unknown boundary conditions, and as a result of the unknown effects of other variables and their interactions which are not included in the model. The model uncertainty associated with the particular mathematical model may be expressed in terms of the probability distribution of a variable X_m defined as :-

$$X_m = \frac{\text{Actual strength (response)}}{\text{Predicted strength (response) using model}}$$

In many components and structures, model uncertainty may have a large effect on structural reliability and should not be neglected.

3.4 Methods

The methods of structural reliability analysis (safety analysis) have been categorised (Blockley, 1980; Palle et al, 1982) into three levels as follows:-

LEVEL 1: Design method in which appropriate degrees of structural reliability are provided on structural element basis (occasionally on structural basis), by the use of factor or partial coefficient related to a predefined characteristic or nominal values of the major structural and loading variables.

LEVEL 2: Methods involving certain approximate iterative calculation procedures to obtain an approximation to the failure probability of a structure or structural system, generally requiring an idealisation of failure domain and often associated with simplified representation of the joint probability of the variables.

LEVEL 3: Methods in which calculations are made to determine the 'exact' probability of failure of a structure or structural component, making use of a full probabilistic description of the occurrences of the various quantities which affect the responses of structures and taking into account the nature of the failure domain.

A level 1 structural design, with the explicit consideration of a number of separate limit states, is what is now commonly called limit-state design, and is already in use for certain codes of practice (CP110, 1972; BS8110, 1985). In theory , both level 2 and 3 methods can be used for checking the safety of a design or directly in the design process, provided a target reliability or reliability index has been specified (Blockley, 1980). For practical purposes e.g., for direct use in design or for evaluating level 1 partial factors, it is necessary to have a method of a reliability analysis which is computationally fast and efficient and which produces result with the desired degree of accuracy. The only procedure which currently satisfies this requirement is the level 2 methods, although analysis by Monte - Carlo simulation is sometimes feasible (Blockley, 1980; Palle et al, 1982).

3.5 ECONOMIC VALUE ANALYSIS

3.5.1 Safety Analysis

Safety analysis has been simply defined as "the condition of being safe from any form of danger, harm or risk" (Blockley, 1980). The classical reliability theory accepts that no structure can be absolutely safe i.e; safety is not wholly attainable, but an acceptable danger or level of risk should be aimed at (Ang et al, 1974; Blockley, 1980; Palle et al, 1982).

The choice of an acceptable level will be affected by the special importance of structures in society, nature of the structure itself and the use to which it is put (Blockley, 1980). The design of safe and functionally reliable structures within the constraints of economy is a subject of continuing concern to the profession of structural engineering (Afolayan, 1994a). Often the structural engineer/designer has a major dilemma - how to balance the cost of structure with its safety (Blockley, 1980; Krishnasamy et al, 1986). In order to compare things satisfactorily, one has to be able to measure them. While economy could obviously be measured in money or by use of such concepts as "utility and value" etc, safety has no any explicit measure except through the use of 'Chance Probabilities' (Blockley 1980).

3.5.2 Decision Making

Decision theory was developed in order to assist business management with decision making. Managers are often faced with decision problems about which they have little information. They have to decide whether to seek new information (which may or may not involve extra costs), or whether to make their decision on the basis of available information. They may also be uncertain about the consequences of their decisions (Blockley, 1980, Palle et al, 1982).

Structural engineers face the same problem. If a particular structural solution is adopted, the consequences may depend upon some factors which are not known with certainty. These factors are called "the state of nature" and may be the settlement of soil below a footing or the deflection of a beam. A structure such as a building consists of a number of distinct parts, each of which may be dealt with by a different professional discipline. In comparing various solutions, the foundation, the structure itself, the heating, ventilation, and other services, the internal finishes, etc, are all cost interactive. A designer's primary criterion in making a decision is to meet a design objective. Naturally, the decision of a designer to meet the said objective are under uncertainty characterisation. Such uncertainty varies from modelling to human intervention (Afolayan, 1994b). In engineering, most applied systems are modelled in terms of a number of input variables. These variables are expected to stay within certain limit in order that the system behaves as they are planned to do. Practically, however, two sets of points are often encountered. One set contains the point which makes the system behaves in an orderly manner while the other set contains the point for which the system malfunctions i.e, non-orderable points in the total data space are being encountered (Afolayan, 1994b; Krishnasamy et al, 1986). Since we are forced to consider uncertain input variables, the data which provides such information must be under control to ensure a balance between safety and economy. Blockley (1980), described engineering design as "decision making under condition of extreme uncertainty", involving a lot of risk of varying degrees; some of which may be beyond the acceptable level. Structural design decisions affect the society as a whole in terms of their safety and the quality of their built environment. Hence, public safety protection from unusual structures such as special bridges, tunnels, dams, towers etc., depend on the professional responsibility and the reliability of a good and sound engineering judgement when taking decisions. Thus, conscious assessment of all condition of hazards that might arise must be given consideration in order to carry out a given design with precision and safety.

3.5.3 Engineering - Management Interference

Safe design must necessarily be directly related to costs savings as it lowers overall cost of failures (Krishnasamy et al, 1986). It then means that safety and economy have an integral relationship, since structural failures entail huge financial loss as well as cause discomfort and affect the survival of the occupants. This relationship between safety and economy can be extended to define the inter-relationship that exists between management and engineering (Afolayan, 1994a). The management consideration of failure occurrence is usually based on socio-economic criterion while engineering rigorously models failure cost and as well calculates failure levels. It was shown (Afolayan, 1994a) that within the framework of socio-economic criterion and with consideration given to associated risks and the selection of decision corresponding to economic principles of uncertainty, the consequence of failure may be modelled as:-

$$E_c = E_I + E_F \qquad (3.13)$$

where E_c = Expected total cost

E_I = Initial cost

E_F = Expected present value of potential loss.

Accounting for economic fluctuations, Equation (3.13) has been transformed (Afolayan, 1994a) to yield:-

$$E_c = E_I + E_v(r, q; T)\, P_{fl} \sum_{i=1}^{N} E_{pi} P_{fi} \qquad (3.14)$$

where E_v (r, q; T) is a cost function depending on interest rate, r; inflation rate, q; and the design life,T; P_{fl} is the life - time failure probability related to probability P_{fi} associated with the event due to cause 'i' resulting in failure and E_{pi} denotes the potential loss in respect of failure due to cause 'i'.

In Equation (3.14), engineering is responsible for the computation of P_{fl} or P_{fi}

Transformed in terms of a limit state equation;

$$P_f = P_i(M \le 0) \qquad (3.15)$$

where M = R_i - S_i; (i=1,2............,n)

Thus Equation (3.14) becomes

$$E_c = E_I + E_v(r, q; T)\, \Phi(-\beta_{fL}) \sum_{i=1}^{N} E_{pi} \Phi(-\beta_{fi}) \qquad (3.16)$$

where β_{fL} and β_{fi} correspond to life time and partial indices of survival respectively.

For a single structural member, with a singular potential loss characteristics,

it was shown (Afolayan, 1994b), that, a premium placed on structural failure by management and engineering safety consideration leads to the deduction that:-

$$\beta=\phi^{-1}(1-E_\theta) \tag{3.17}$$

where

$$E_\theta=[(E_c-E_i)/E_v(r,q;T)E_{pi}]^{\frac{1}{2}} \tag{3.18}$$

In the above expression, E_θ is a failure cost parameter.

It was recommended (Krishnasamy et al, 1986) that, the reliability index β must be defined for a design interval e.g for building structures, β is most often defined over an assumed 50 - years life and typical β - values may range between 2.0 to 5.0 corresponding to probability of failures P_f between 0.2275E-1 to 0.29E-6.

A designed interval of one - year is normally employed in utility line designs.

A Table of typical β values with their corresponding probability of failures is shown on Table 3.1.

Table 3.1: Values of β and Corresponding Probability of failures P_f

β	P_f	β	P_f
0.00	0.50000017	2.55	0.00538617
0.05	0.48006097	2.60	0.00466122
0.10	0.46817195	2.65	0.00402463
0.15	0.44038715	2.70	0.00346702
0.20	0.42074018	2.75	0.00297982
0.25	0.40129360	2.80	0.00255519
0.30	0.38208852	2.85	0.00218603
0.35	0.36316903	2.90	0.00186588
0.40	0.34457820	2.95	0.00158894
0.45	0.32635515	3.00	0.00134997
0.50	0.30853744	3.05	0.00140282
0.55	0.29115957	3.10	0.00096767
0.60	0.27425299	3.15	0.00061642
0.65	0.25784597	3.20	0.00068720
0.70	0.24196351	3.25	0.00057709
0.75	0.22662722	3.30	0.00048348
0.80	0.21185528	3.35	0.00040411
0.85	0.19766244	3.40	0.00033698
0.90	0.18406005	3.45	0.00028034
0.95	0.17105607	3.50	0.00023267
1.00	0.15065522	3.55	0.00019266
1.05	0.14685905	3.60	0.00016915
1.10	0.13566607	3.65	0.00013115
1.15	0.12507196	3.70	0.00107483
1.20	0.11506971	3.75	0.00008844
1.25	0.10564982	3.80	0.00077237
1.30	0.09680053	3.85	0.00005908
1.35	0.08850803	3.90	0.00004812
1.40	0.08075669	3.95	0.00003909
1.45	0.07152929	4.00	0.00003169
1.50	0.06680722	4.05	0.00002562
1.55	0.06057076	4.10	0.00002067
1.60	0.05479928	4.15	0.00001663
1.65	0.04947144	4.20	0.00001335
1.70	0.04456542	4.25	0.00001070
1.75	0.04005911	4.30	0.00000855
1.80	0.03593026	4.35	0.00000681
1.85	0.03215671	4.40	0.00000542
1.90	0.02871649	4.45	0.00000430
1.95	0.02558799	4.50	0.00000340
2.00	0.02275006	4.55	0.00000628
2.05	0.02018214	4.60	0.00000211
2.10	0.01786435	4.65	0.00000166
2.15	0.01577755	4.70	0.00000130
2.20	0.01390340	4.75	0.00000102
2.25	0.01222443	4.80	0.00000079
2.30	0.01072408	4.85	0.00000062
2.35	0.00838669	4.90	0.00000048
2.40	0.00819753	4.95	0.00000037
2.45	0.00714281	5.00	0.00000029
2.50	0.00620968		

3.6 RELIABILITY AND COST ESTIMATES

The main task to be considered here is the determination of the safety Indices (β) for the various limit states conditions and to estimate the failure costs associated with the computed β - values.

In chapter three, it was shown that, the expected total cost (E_e) is related to the initial cost (E_i) and the expected failure cost (E_f) from the relation.

$$E_e = E_i + E_f \cdot P_f$$

where; E_f is a function of 'P_f' which describes the probability of failure in a given limit state.

For the solution of the Structural problem under review, the probability of failure (P_f) is a function of the limit state equation considered.

i.e. $P_f(x) = P[\text{Limit State} \leq 0)$ (3.19)

where;

$P_f(x)$ defines the failure probability for a given limit state equation as a function of set of basic variables (x) governing the limit state condition, and is given as:-

$$P_f = \Phi(-\beta) \qquad (3.20)$$

3.7 DETERMINATION OF SAFETY INDICES

Basically two principal limit states are encountered i.e. the Ultimate Limit State (ULS) and Serviceability Limit State (SLS). The governing equations for design in this work are those for the Serviceability Limit State, since the member is required to remain serviceable during its working life. The conditions to be examined under the Serviceability Limit States are:-

(1) Compressive working stress condition

(2) Tensile working stress condition

(3) Deflection requirement

(4) Stability criteria

The failure of a member in any given limit state for specified conditions is given as:-

(1) Compression

$$P_{fwc}=P[g_1(x)\leq 0] \qquad (3.21)$$

2. Tension

$$P_{fwt}=P[g_2(x)\leq 0] \qquad (3.22)$$

3. Deflection

$$P_{fd}=P[g_3(x)\leq 0] \qquad (3.23)$$

4. Stability

$$P_{fs}=P[g_4(x)\leq 0] \qquad (3.24)$$

where P_f's are the failure probability levels for the four classes of limit state listed above, and $g_i(x)$ are the safety margin functions.

3.7.1 **Compressive working stress condition.**

The condition to be satisfied here is that the actual applied stresses at working condition should not exceed the maximum permissible value, so that

$$P_{fwc}=P[f_{amax}-f_{cw}\leq 0] \qquad (3.25)$$

in which, f_{amax} and f_{cw} are maximum allowable stress and working stress respectively.
For a prestress section, this condition can be expressed thus: -

$$P_{fwc}=P[(0.33f_{cu})-(\frac{L_I P_i}{A}+\frac{M_T t_m}{J}+\frac{M_A}{Z_T})\leq 0] \qquad (3.26)$$

but,

$$M_A=\frac{\rho_p B h^2}{2}+\frac{\rho_c N(L_1+L_2)\,d_c h}{24} \qquad (3.27)$$

The bending moment (M_A) at any point across the section along the length of the pole can be obtained by letting H = x in the above expression.
Thus:

$$M_x=\frac{\rho_p B x^2}{2}+\frac{\rho_c N(L_1+L_2)\,d_c x}{24} \qquad (3.28)$$

Equation (3.28) allows the moment to be computed in terms of the uncertain wind pressure on the pole (ρ_p) and cables (ρ_c), as well, it allows for the studying of the variation of the applied bending moment along the length of the pole.
It was reported (Essien, 1990) that the torsional moment (M_T) is constant along the height of the pole with the magnitude of 15KNm. This value itself can be considered as

uncertain since it can not be accurately predicted.

The initial prestressing force applied (P_i) has been expressed in terms of the characteristic strength of the prestressing tendons (f_{pu}), the area of prestressing tendons (A_{ps}) and the number of steel tendons used (n_s), Viz:-

$$P_i = n_s \times A_{ps} \times f_{pu} \qquad (3.29)$$

The quantity n_s can be fixed deterministically as follows:-

$n_s = 20$; for $x > 1.5m$ if $H \leq 7.0m$

$x > 2.5m$ if $H > 7.0m$

$n_s = 16$; for $x \leq 1.5m$ if $H \leq 7.0m$

$x \leq 2.5m$ if $H > 7.0m$

This is based on the information obtained from site conditions to the effect that 20 nos of wires are used in the lower part (I - section portion) of the pole, whereas 16Nos are used at the top end (solid rectangular portion).

The loss ratio (L_r) depends on a lot of other parameters e.g shrinkage, creep, elastic deformation, anchorage, friction etc, and hence can not be accurately predicted. Thus, it shall be treated as a random variable. The sectional properties A_c, J and Z_T have all been expressed in terms of the basic variables B, D, t_f and t_w in Chapter two in Equations (2.9) - (2.18) for various geometries.

Thus the basic variables for the design are D, B, t_f, t_w, f_{cu}, f_{pu}, A_{ps}, L_r, ρ_p, and ρ_c, which in row matrix can be expressed as:- 10 - variable problem.

$\{D, B, t_f, t_w, f_{cu}, f_{pu}, S_{ps}, L_r, \rho_p, \rho_c\} =$

$\{X_1, X_2, X_3, X_4, X_5, X_6, X_7, X_8, X_9, X_{10}\}$

3.7.2 Tensile working stress condition

A simple requirement to be investigated is written as:-

$$P_{fws}=P[f_{cw}-f_{amin}\leq 0] \qquad (3.30)$$

Substituting Equations (2.3) and (2.6) into Equation (3.30) we have:-

$$P_{fws}=P[\frac{L_r P_i}{A_c}-\frac{M_T t_m}{J}-\frac{M_A}{Z_T}-0\leq 0] \qquad (3.31)$$

By similar application, the basic variables for the design are D, B, t_f, t_w, f_{pu}, A_{ps}, L_r, ρ_p, and ρ_c, which in row matrix ca be expressed as a 9 - variable problem

$\{D, B, t_f, t_w, f_{pu}, A_{ps}, L_r, \rho_p, \rho_c\} =$

$\{X_1, X_2, X_3, X_4, X_5, X_6, X_7, X_8, X_9\}$

3.7.3 Deflection requirement

The condition to be examined here is that:-

$$P_{fd}=P[\delta_{perm}-\delta_{max}\leq 0] \qquad (3.32)$$

where; δ_{perm} = limiting or permissible deflection specified by the code (BS8110, 1985). This is given as L/350 i.e span/350; or 20mm whichever is smaller.

$\delta_{max} = \delta_T = \delta_p + \delta_w$

From Equation (2.27);

$$\delta_{max} = \frac{PL^3}{3EI} + \frac{WL^3}{8EI} \tag{3.33}$$

Substituting in Equation (3.32) we get:-

$$P_{fd} = P[\frac{1}{350} - (\frac{PL^3}{3EI} + \frac{WL^3}{8EI}) \leq 0] \tag{3.34}$$

But L = h

From Table (2.2)

$$P = P_w = \frac{N(\rho_c d_c (L_1 + L_2)}{24}$$

and

$W = \rho_p B$

I = moment of inertia; has been expressed before in Equations (2.10) to (2.16).

Thus, the basic variables for the design are D, B, t_f, t_w, f_{pu}, A_{ps}, L_r, ρ_p, and ρ_c, which can be expressed as arrow matrix of 7 - variable problem.

$\{D, B, t_f, t_w, f_{pu}, A_{ps}, L_r, \rho_p, \rho_c\} =$

$\{X_1, X_2, X_3, X_4, X_5, X_6, X_7\}$

In Equation (3.34), the following quantities are assumed to be fixed deterministically initially:-

N = number of cables = 3

d_c = diameter of cables = 13.7mm

$L_1 = L_2$ = spacing between poles = 50m

3.7.4 Stability criteria

The condition to be checked here is that:-

$$P_{fs}=P[N_{cr}-N_a \leq 0] \qquad (3.35)$$

Substituting Equation (2.30) and (2.31) into Equation (3.35) we have:-

$$P_{fs}=P[\frac{\pi^2 EI}{4L^2}-(P_e+V) \leq 0] \qquad (3.36)$$

In which;

L = h

I = moment of inertia; which has been explain earlier on and expressed in Equations (2.10) to (2.16).

P_e = effective prestressing force

= n_s x f_{pu} x A_{ps} x L_r

V = total vertical component of load on pole from cables

From Table (2.2)

$$= \frac{NW_t(L_1+L_2)}{2} + W_D$$

The value of V(KN) obtained from this expression can vary significantly since W_t and W_D are random. Hence V shall be treated as a random variable.

Thus the basic variables for the design are D, B, t_f, t_w, f_{pu}, A_{ps}, L_r, E, and V which in row matrix can be expressed as a 9 - variable problem.

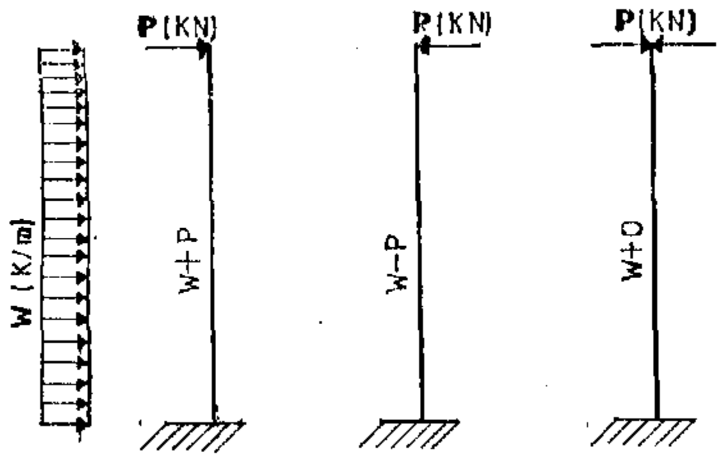
{D, B, t_f, t_w, f_{pu}, A_{ps}, L_r, E, and V} =

{X_1, X_2, X_3, X_4, X_5, X_6, X_7, X_8, X_9 }

3.8 LOAD CASES

The principal wind loadings on the pole can be grouped into two categories.

1. wind loading (uniform pressure) along the height of the pole (W)
2. wind loading (concentrated) at the level of the cables (P)



(a) End poles (b) End poles (c)Intermediate poles

Fig. 3.2 Load combinations on end and intermediate poles.

In Fig. 3.2(a), the load P can be considered to have a positive contribution effect on load W hence;

$$M_A = \frac{Wx^2}{2} + P_x \tag{3.37}$$

and

$$\delta_{max} = \frac{Wl^3}{8EI} + \frac{Pl^3}{3EI} \tag{3.38}$$

In Fig. 3.2(b), the load P is considered to have negative contribution effect on load W hence;

$$M_A = \frac{Wx^2}{2} - P_x \tag{3.39}$$

and

$$\delta_{max} = \frac{Wl^3}{8EI} - \frac{Pl^3}{3EI} \tag{3.40}$$

In Fig. 3.2(c), the load P is assumed to have neutralised its effect which is typical of intermediate poles condition

hence;

$$M_A = \frac{Wx^2}{2} \tag{3.41}$$

and

$$\delta_{max} = \frac{Wl^3}{8EI} \qquad (3.42)$$

These load cases shall be evaluated and sensitised in the analysis.

Transmission poles under review can be modelled by examining the state of stress in the tri-axial sense, and hence the failure in the member can then be predicted (Fig. 3.3).

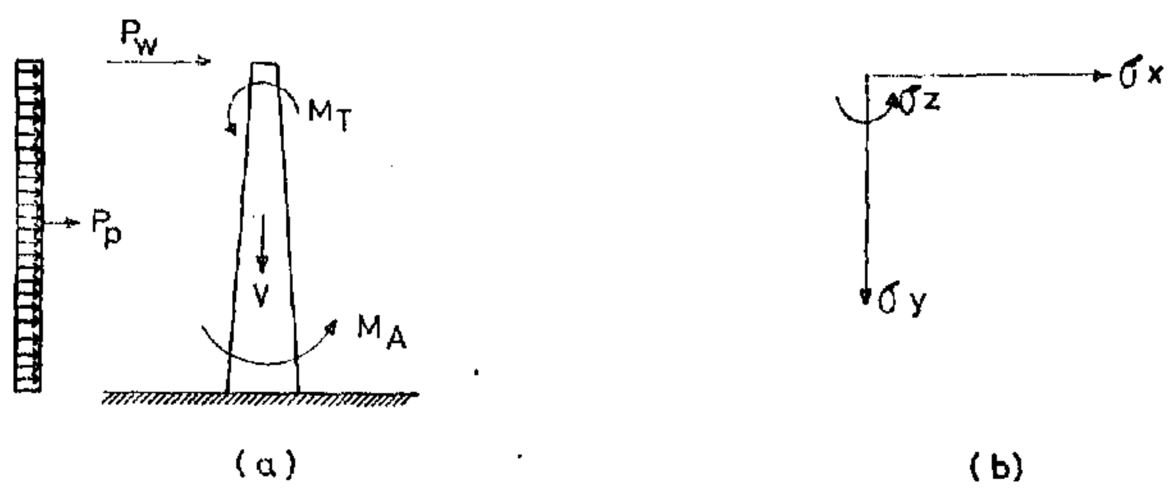


Fig. 3.3 Load effects and State of stress on a transmission pole

In Fig. 3.3(a) and (b) P_p is the resultant wind load on pole and P_w is the resultant wind load on cables; which causes the bending moment M_A; V is the total vertical load on pole due to self weight and weight of cables; M_T is the twisting (torsional) moment on the member; σ_x, σ_y and σ_z are the stress components due to the forces considered along the major axes.

3.9 COMPUTER PROGRAM CODING

A general - purpose computer program has been designed for the approximate computation of general probability integral over given domains with locally smooth boundary occurring in structural reliability. The program, First Order Reliability Method (FORM), is known to be efficient and generally user friendly (Goltwitzer et al, 1987), and thus has been adopted for the treatment of the transmission concrete pole problem.

3.9.1 GENERAL PROGRAM DESCRIPTION

The present version, FORM5, is an improvement and an extension of previous versions and has been adapted to the PC environment. It runs with very few modifications also on mainframe computers.

FORM5 can run either directly via a main program written by the user or via EFORM both of which call FORM5. The general structure and data flow for the program is shown in Fig. 3.4. Most users prefer to run it via EFORM.

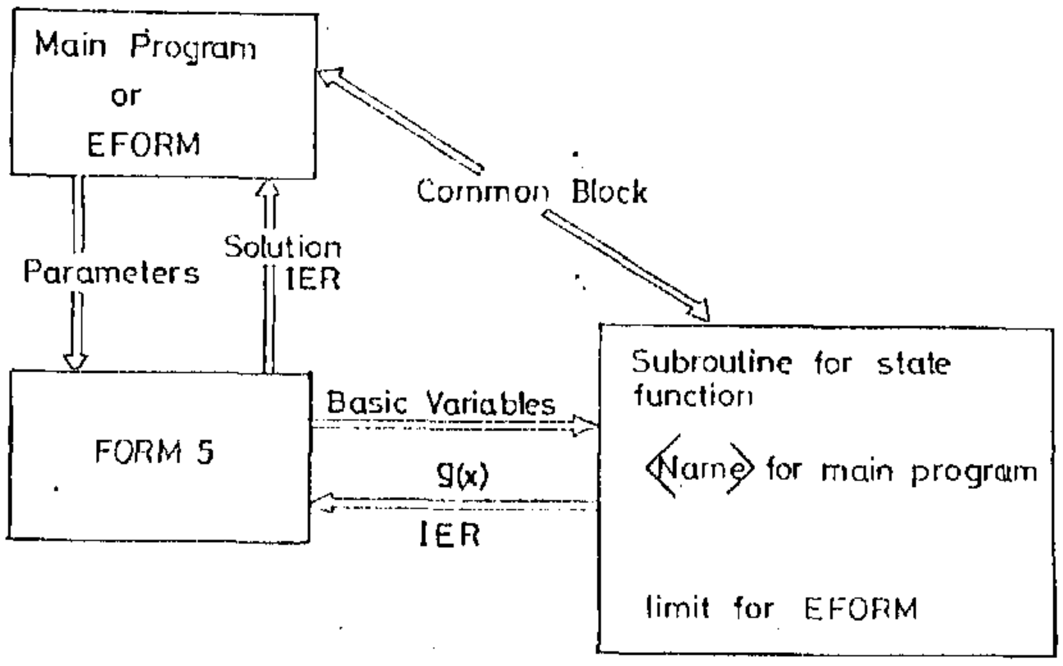


Fig. 3.4 Program Structure for FORM5

The user has to define the characteristic vector X(stochastic model) in the main program or in EFORM and the state function g(x) in a subroutine (state subroutine). EFORM specifies the stochastic model interactively and facilitates the input/output handling.

When running FORM5 via a main program, the user has to specify the stochastic model together with some other parameters. In any case, the user will be required to provide the state function as a subroutine in FORTRAN 77 (DOUBLE PRECESSION) which is called by FORM5. Input into the state subroutine are the values of the basic variables and possibly other deterministic parameters. The state subroutine returns the value of the state functions and possibly error messages to FORM5.

FORM5 then returns the solution and possibly error messages to the main program or EFORM. FORM5 can handle up to 60 uncertain variables (X - variables) and can perform up to 40 iterations to achieve convergence.

There are a few subroutines in FORM5 which can be called directly from the main program and help the user for input and output organisation.

(i) YINIT (N, IV, VP, IRHO, COV, NC)

This is an initialisation subroutine. It gives default values to the arrays VP, IV, COV:

VP(10, N): the first four (4) rows are set to zero (0) while the last six (6) rows are set to one (1)

IV(2, N): normal distribution for all variables (IV(1, i) = 2 and (IV(2, i) = 0; for i =1, N)

COV(NC, NC): COV is preset as a unit matrix if IRHO =1 and NC $\geq$ N

(ii) YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)

This subroutine produces an output for the stochastic model. it prints the variable number, the type of distribution, the mean value, the standard deviation, the coefficient of variation, the repetition number,the occurrence probability and the value of IRHO.

(iii) YFAUS (NAUS, N, F, "STRING")

This subroutine prints the vector F of dimension N in the output medium NAUS with the little "string". String is a CHARACTER - variable with 10 characters.

(iv) YMAUS (NAUS, N1, N2, F, "STRING")

This subroutine prints the square matrix F of dimension (N1, N2) to the output medium NAUS, N2 is the number of rows and columns to be printed. String is a CHARACTER - variable with 10 characters.

The parameter list of FORM5 which must be called by the main program or EFORM is given as:-

SUBROUTINE FORM5 (N, IV, EX, SX, VP, GG, IRHO, COV, NC, EIVEC, NE, V1, NAUS, BETA, X, U, ZES, IER).

These are now described as they are used in the program.

N: <INTEGER> number of uncertain variables and dimension of the vectors EX, SX, X and U. The maximum number N in this version of FORM5 is N = 60.

IV: <INTEGER> matrix of dimension (2,N) whose first element in the ith column assigns the probability distribution type J to the variable X(i) and whose second element defines the type of input for the distribution parameters.

EX: <REAL * 8> vector of mean values of uncertain variables

SX: <REAL * 8> vector of standard deviation of uncertain variables

VP: <REAL * 8> matrix with 10 rows and N column. The first two rows will be set by FORM5 except for distribution types 96 to 99 (user defined distributions). The third and fourth rows have to be specified by the user in some cases, while rows 5 to 8 must always be specified by the user.

Note: IV(2, N) and VP(10, N) may be preset using YINIT.

GG: name of the user defined subroutine containing the mechanical model (state function). The subroutine has to be declared as EXTERNAL in the main program calling FORM5 e.g GCOMP; GTENS; GDEFL and GSTAB as used in the transmission pole programs.

IRHO: <INTEGER> control parameter for correlation.

IRHO = 0; no correlation between X - variables.

IRHO = 1; correlation permitted

COV: <REAL * 8)> matrix with NC rows and NC columns.

For IRHO = 1, the user has to give the correlation coefficients in this matrix. Only the lower triangle is used.

For IRHO = 0; COV will not be used.

NC: <INTEGER>	dimension of the matrix COV. For IRHO = 0, NC must be greater or equal to one. For IRHO = 1, NC must be greater or equal to N.
EIVEC: <REAL * 8>	matrix with NE rows and NE columns, an auxiliary matrix to store the Eigenvectors which are EIVEC on output of FORM5 for IRHO = 1 (case of correlation).
NE: <INTEGER>	dimension of EIVEC. For IRHO = 0 NE must be $\geq$ 1. For IRHO = 1, NE must be $\geq$ N.
VI: <REAL * 8>	control parameter for iteration. Usually it suffices to set VI = 1.0. In case of non-convergence for VI = 1, try VI < 1.0.
NAUS: <INTEGER>	print control and unit number parameter. For NAUS $\geq$ 99 or NAUS < 0; all prints suppressed. For 0 $\leq$ NAUS $\leq$ 98; print to the unit NAUS
BETA: <REAL * 8>	on input: For BETA = 0; the starting solution on X is set by FORM5 to X(I) = EX(I)

For BETA > 0; a starting solution is expected on X(I), I= 1, N.

On output; safety index (=//u-*//)

X: <REAL * 8> vector of dimension N.

On input: possibly initial solution vector.

On output: coordinates of the checking point in X - space

U: <REAL * 8> vector of dimension N.

On output: coordinates of the checking point in U - space.

ZES: <REAL *> vector of dimension 3.

Contains the values; ZES(1) = value of the uncertain variable Z

ZES(2) = means of Z

ZES(3) = standard deviation of Z.

Z is the linearisation of g(X) at the checking point.

IER: <INTEGER> error indicator.

IER = 0; no error.

IER > 0; six digit number on output.

3.9.2 User Defined Main Programs And Subroutines.

As required by FORM5, the following main programs and accompanying subroutines are supplied for the solution of the transmission concrete problem. The detailed program listing for all the limit states considered are given in

Appendix I (A - D)

PROGRAM COMP:

Evaluates the reliability of the pole subjected to compressive working stress condition in the SLS using 10-variables. The main program calls subroutine GCOMP, which computes the limit state function and returns to the main program.

PROGRAM TENS:

Evaluates the reliability of the pole subjected to tensile working stress condition in the SLS using 9-variables. The main program calls subroutine GTENS, which computes the limit state function and returns to the main program.

PROGRAM DEFLEC:

Evaluates the reliability of the pole subjected to deflection in the SLS using 7-variables. The main program calls subroutine GDEFL, which computes the limit state function and returns the value to the main program.

PROGRAM STABIL:

Evaluates the reliability of the pole subjected to stability in the SLS using 9-variables. The main program calls subroutine GSTAB, which computes the limit state function and returns the value to the main program.

3.9.3: Basic Variable Statistical Data.

For the four principal cases being analysed, the basic variables and their statistical characters have been identified and listed in Tables (3.2) to (3.5). Most information regarding the variables listed were supplied by earlier researchers (Ellingwood et al, 1974; Theodore et al, 1986; Blockley, 1981; Sumbele, 1997).

Table 3.2 : Basic variable for SLS compressive working stress condition and their statistical characters.

S/No	Variable (x)	Distribution Types & Number	Mean (EX)	Standard Deviation (sx)	Coefficient of varation (cov)
1	D	normal=2	120mm	4.8mm	0.04
2	B	"	120mm	4.8mm	0.04
3	t_f	"	70mm	2.8mm	0.04
4	t_w	"	60mm	2.4mm	0.04
5	f_{cu}	"	$50N/mm^2$	$10N/mm^2$	0.30
6	f_{pu}	Lognormal	$1870N/mm^2$	$280.5N/mm^2$	0.15
7	A_{ps}	= 3	$4.91mm^2$	$0.177mm^2$	0.36
8	L_r	normal = 2	0.7(70%)	0.035	0.05
9	ρ_p	"	$520N/m^2$	$192.4N/m^2$	0.37
10	ρ_c	Gumbell =7 "	$101N/m^2$	$37.37N/m^2$	0.37

Table 3.3: Basic variables for SLS Tensile working stress condition and their statistical characters.

S/No	Variable (x)	Distribution Types & Numbers	Mean (x)	Standard deviation (sx)	Coeffecient of Variation (cov)
1	D	normal=2	120mm	4.8mm	0.04
2	B	"	120mm	4.8mm	0.04
3	t_f	"	70mm	2.8mm	0.04
4	t_w	"	60mm	2.4mm	0.04
5	f_{pu}	lognormal=2	1870N/mm^2	280.5Nlmm^2	0.15
6	A_{ps}	normal=2	4.91mm^2	0.177mm^2	0.036
7	L_r	"	0.7	0.0035	0.05
8	ρ_p	gumbell=7	520N/m^2	192.4N/m^2	0.37
9	ρ_c	"	101N/m^2	37.37N/mm^2	0.37

Table 3.4: Basic variables for SLS Deflection requirement and their statistical characters.

S/No	Variable (x)	Distribution Types & Numbers	Mean (EX)	Standard deviation (sx)	Coefficient of variation (cov)
1	D	normal=2	120mm	4.8mm	0.04
2	B	"	120mm	4.8mm	0.04
3	t_f	"	70mm	2.8mm	0.04
4	t_w	"	60mm	2.4mm	0.04
5	E	"	40,000N/mm^2	6,240N/mm^2	0.156
6	ρ_p	Gumbell=7	520N/m^2	192.4N/m^2	0.37
7	ρ_c	"	101N/m^2	37.37N/m^2	0.37

Table 3.5: Basic variables for SLS stability criteria and their statistical characters.

S/No	Variable (x)	Distribution Types & Numbers	Mean (EX)	Standard deviation (SX)	Coefficient of variation (cov)
1	D	normal=2	120mm	4.8mm	0.04
2	B	"	120mm	4.8mm	0.04
3	t_f	"	70mm	2.8mm	0.04
4	t_w	"	60mm	2.4mm	0.04
5	f_{pu}	lognormal=3	1870N/mm^2	280.5N/mm^2	o.15
6	A_{ps}	normal=2	4.91mm^2	0.177mm^2	0.036
7	L_r	"	0.7	0.035	0.05
8	E	"	40,000N/mm^2	6,240N/mm^2	0.156
9	V	"	10,000N	1,500N	0.15

CHAPTER FOUR

RESULTS ANALYSES AND DISCUSSIONS

4.1 DISCUSSION OF RESULTS

In assessing a particular limit state condition, the effect of changes in the values of the design variables on the β - values reliability indices along the length of the pole was observed so as to appreciate the failure characteristic as well as the influence of the variable in each limit state on the reliability function.

Typical results obtained for the various limit states are as shown in

Appendices II(A-D).

More than 80 different cases were examined although only 54 of these are plotted as 18 sets per chart. These are shown on the graphs in Fig. 4(a) to 4(r).

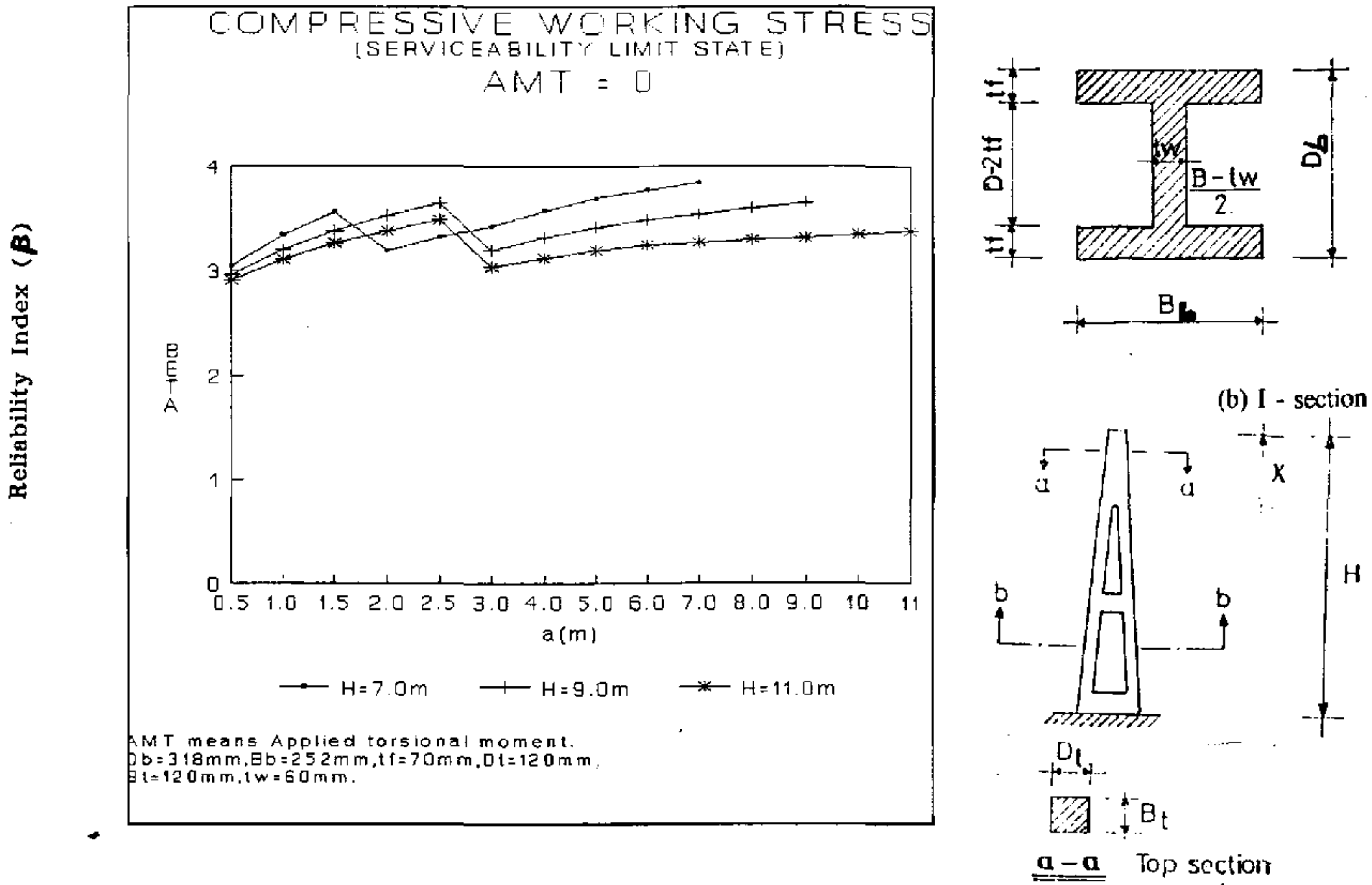
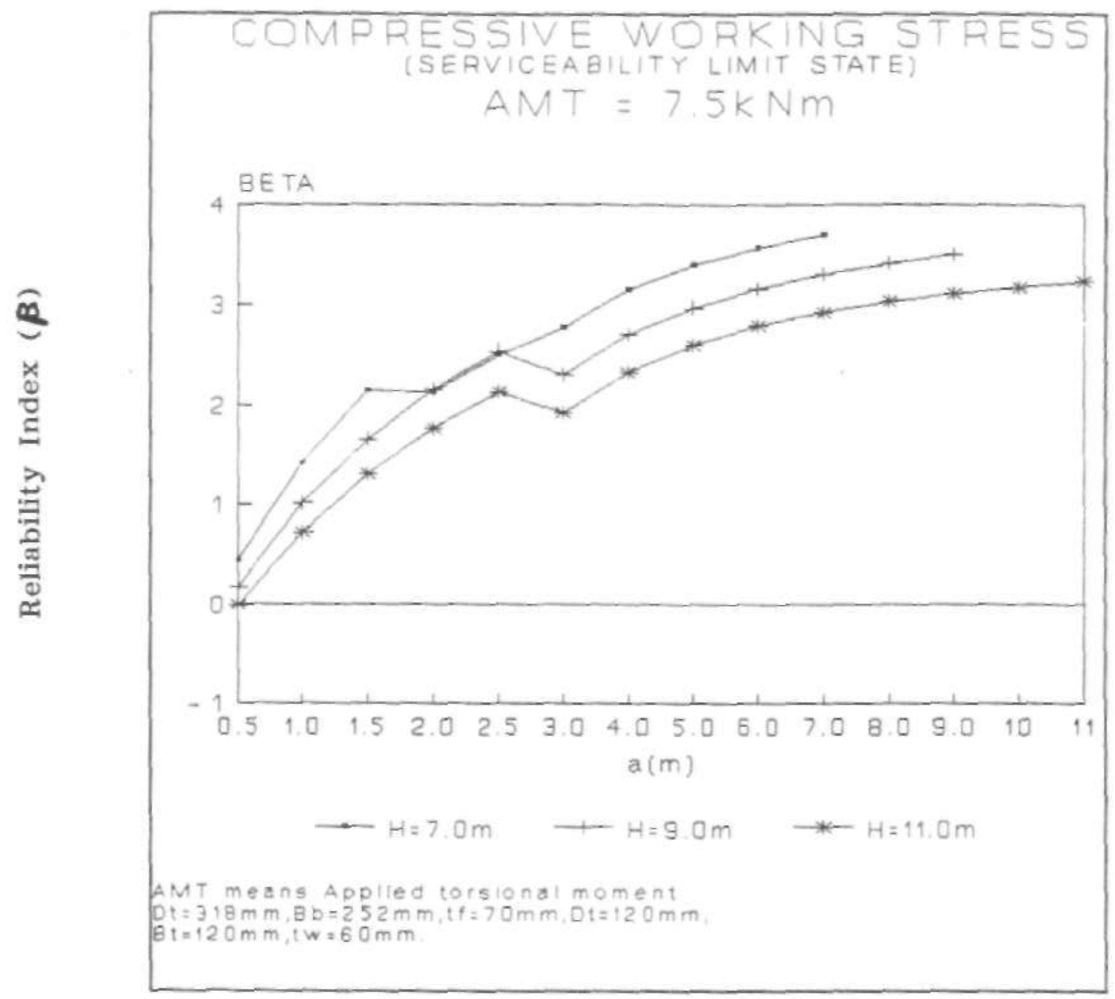


FIG. 4.1(a): Variation of reliability of the pole in the compressive working stress condition with torsion moment and height.



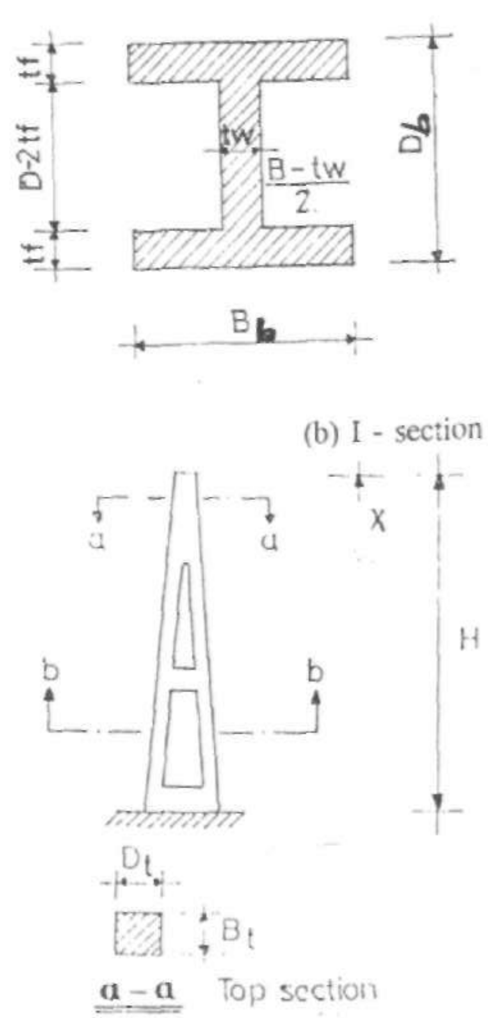
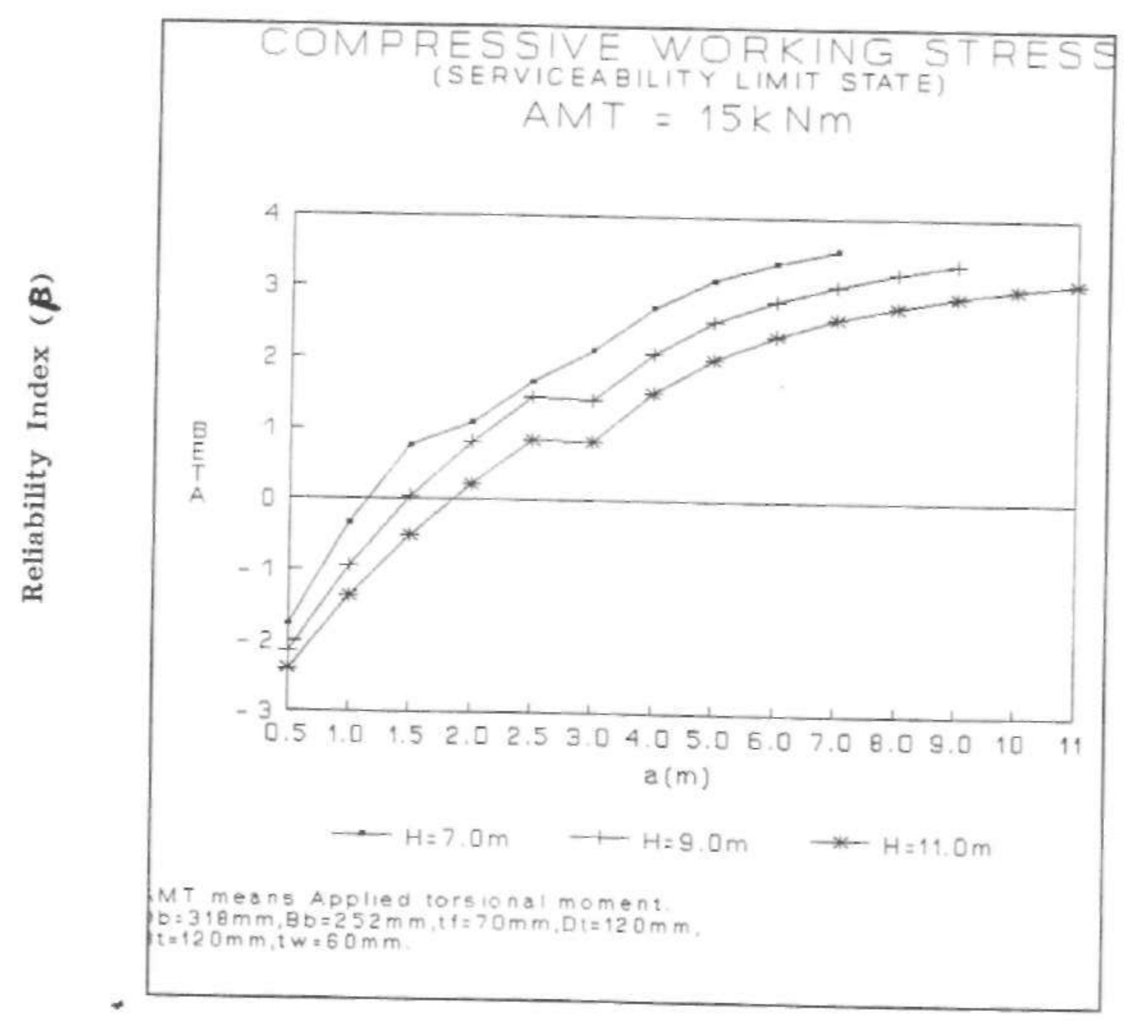


FIG. 4.1(b): Variation of reliability of the pole in the compressive working stress condition with torsion moment and height.



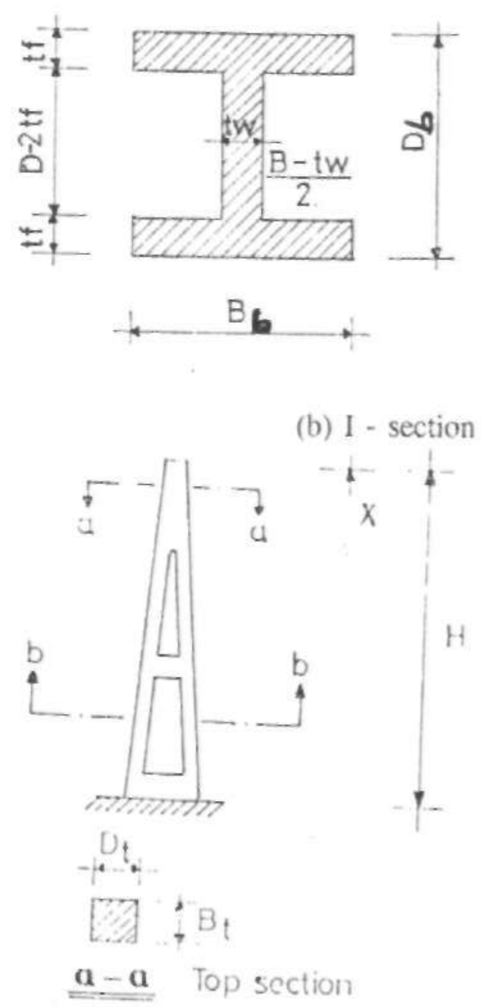
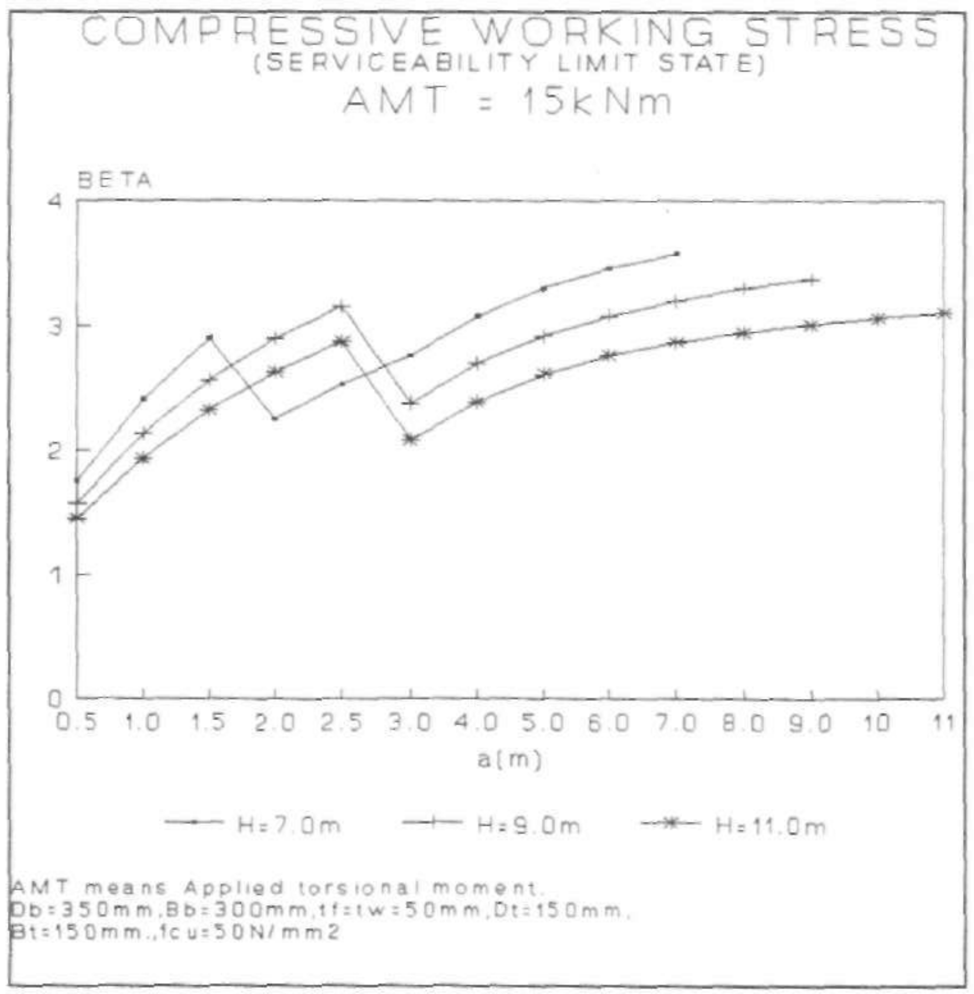
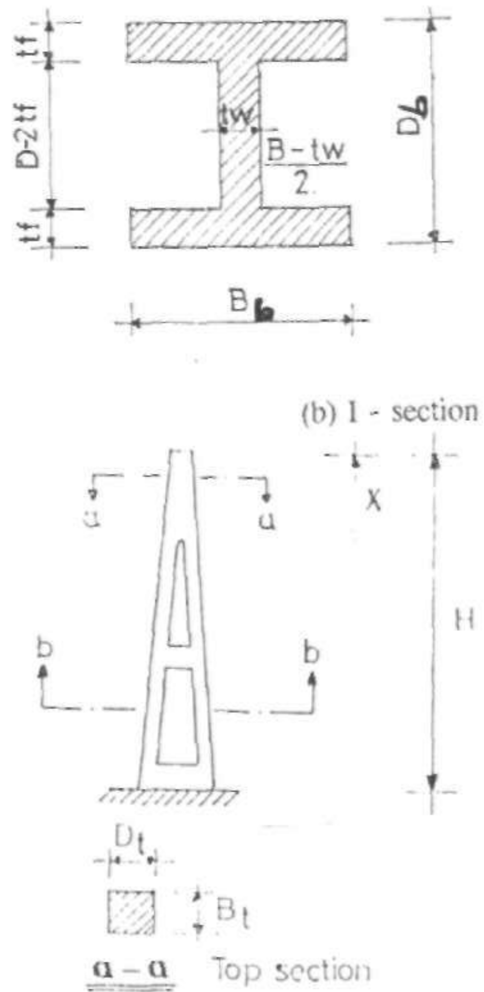


FIG. 4.1(c): Variation of reliability of the pole in the compressive working stress condition with torsion moment and height.

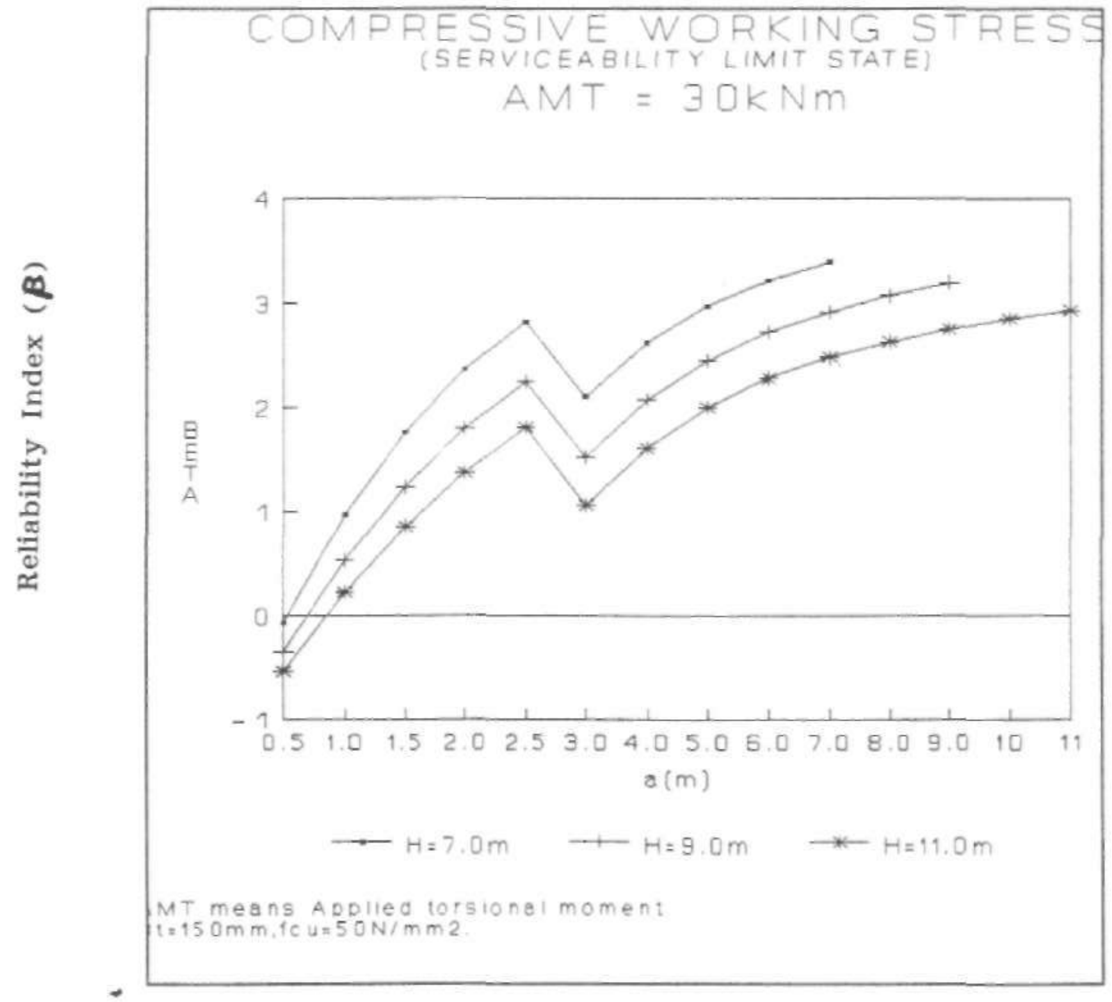
Reliability Index (β)





ment and

height.



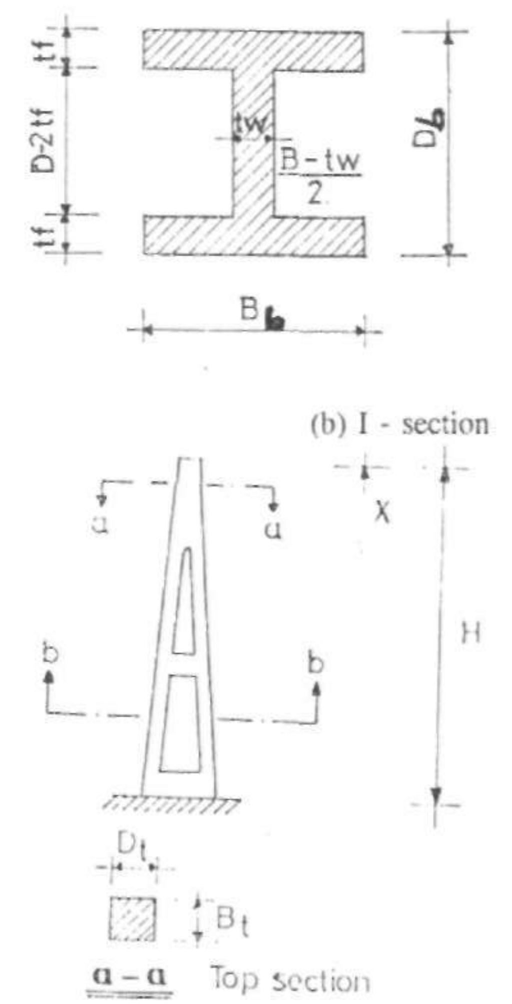
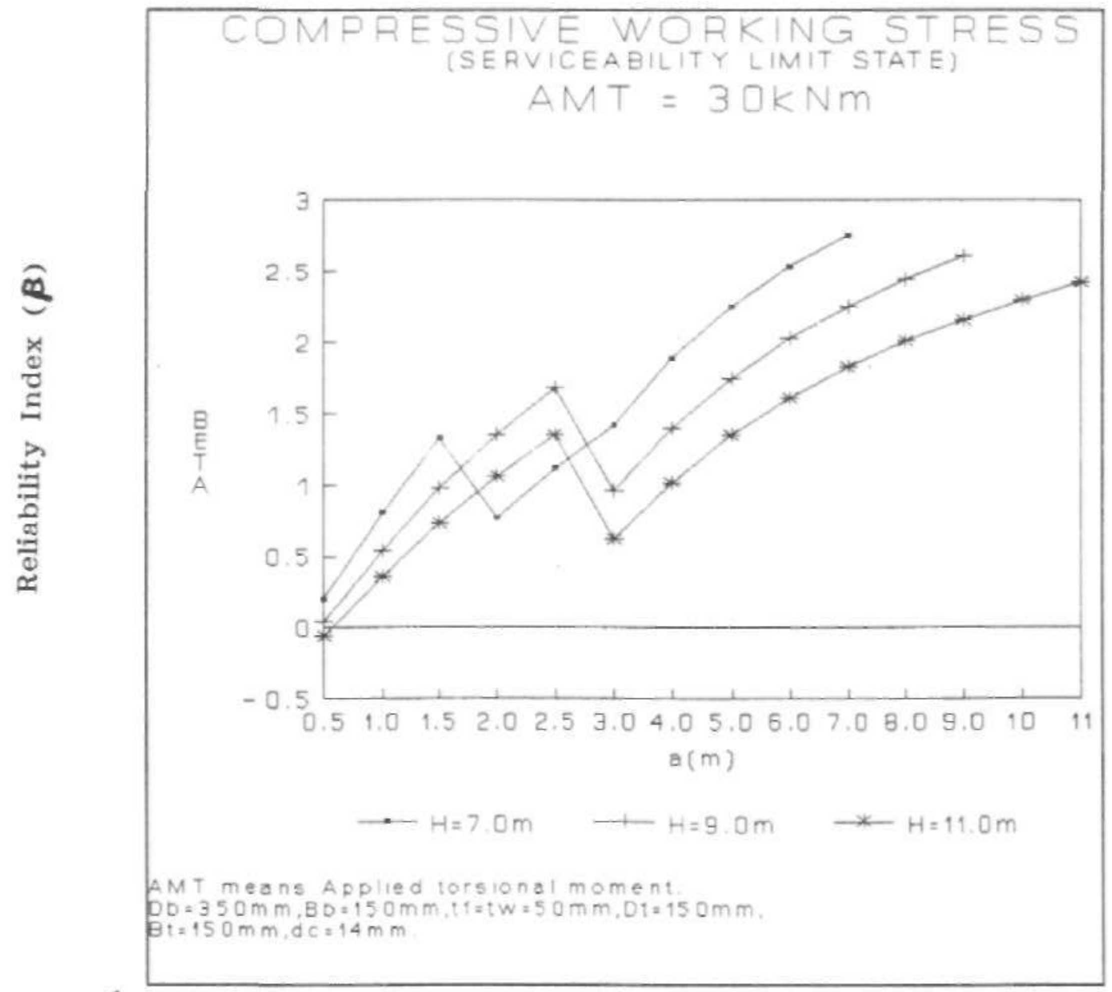


FIG. 4.1(e): Variation of reliability of the pole in the compressive working stress condition with torsion moment and height.



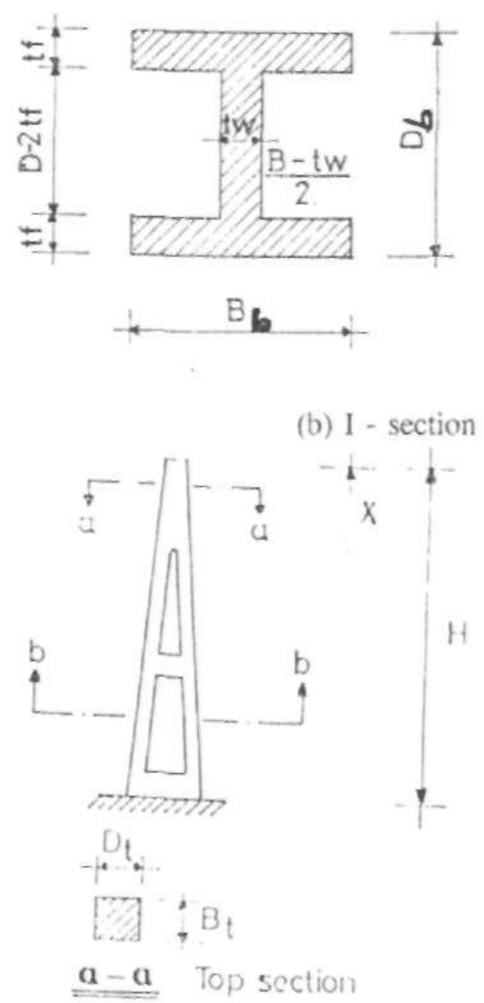


FIG. 4.1(f): Variation of reliability of the pole in the compressive working stress condition with torsion moment and height.

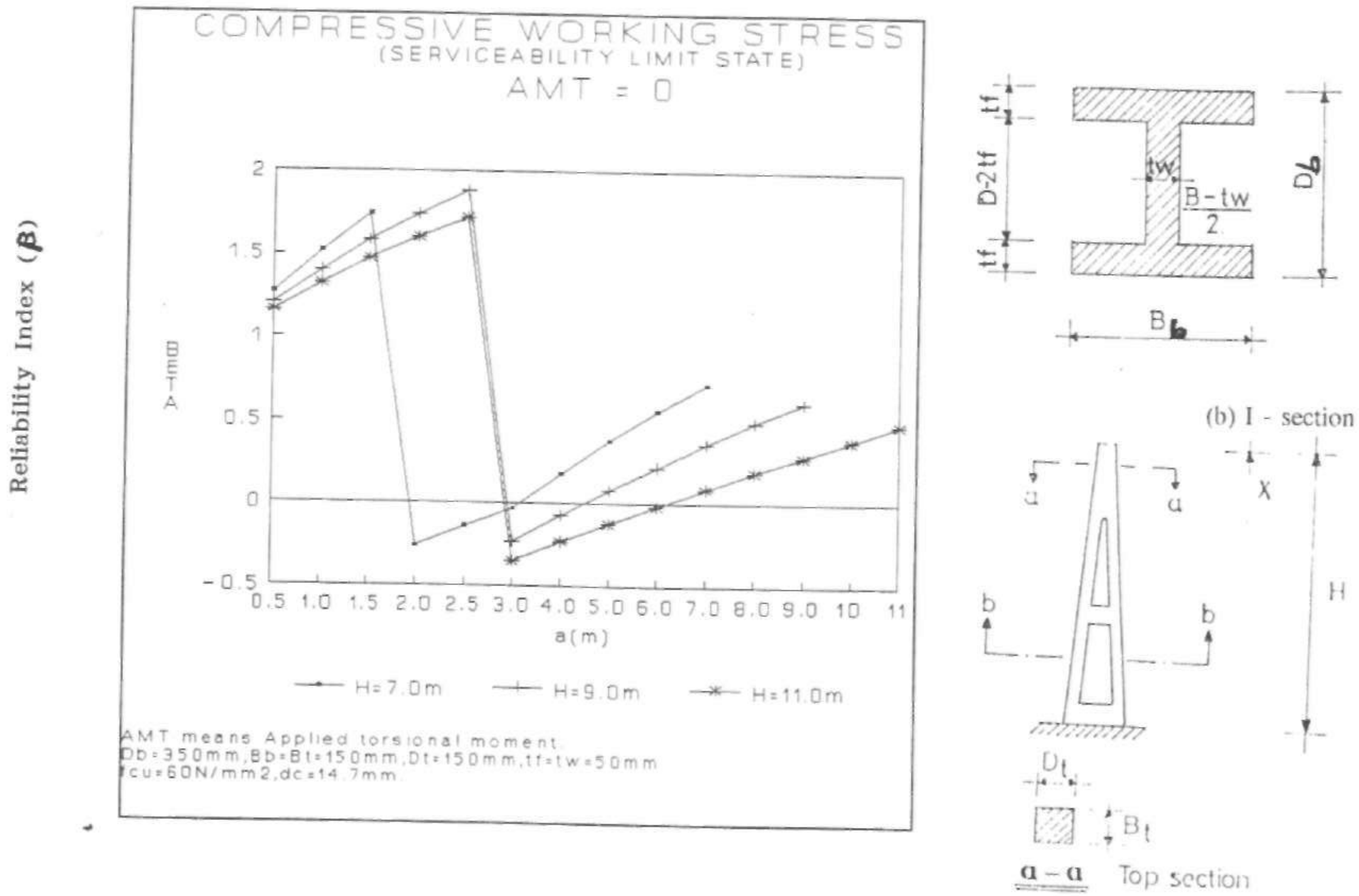
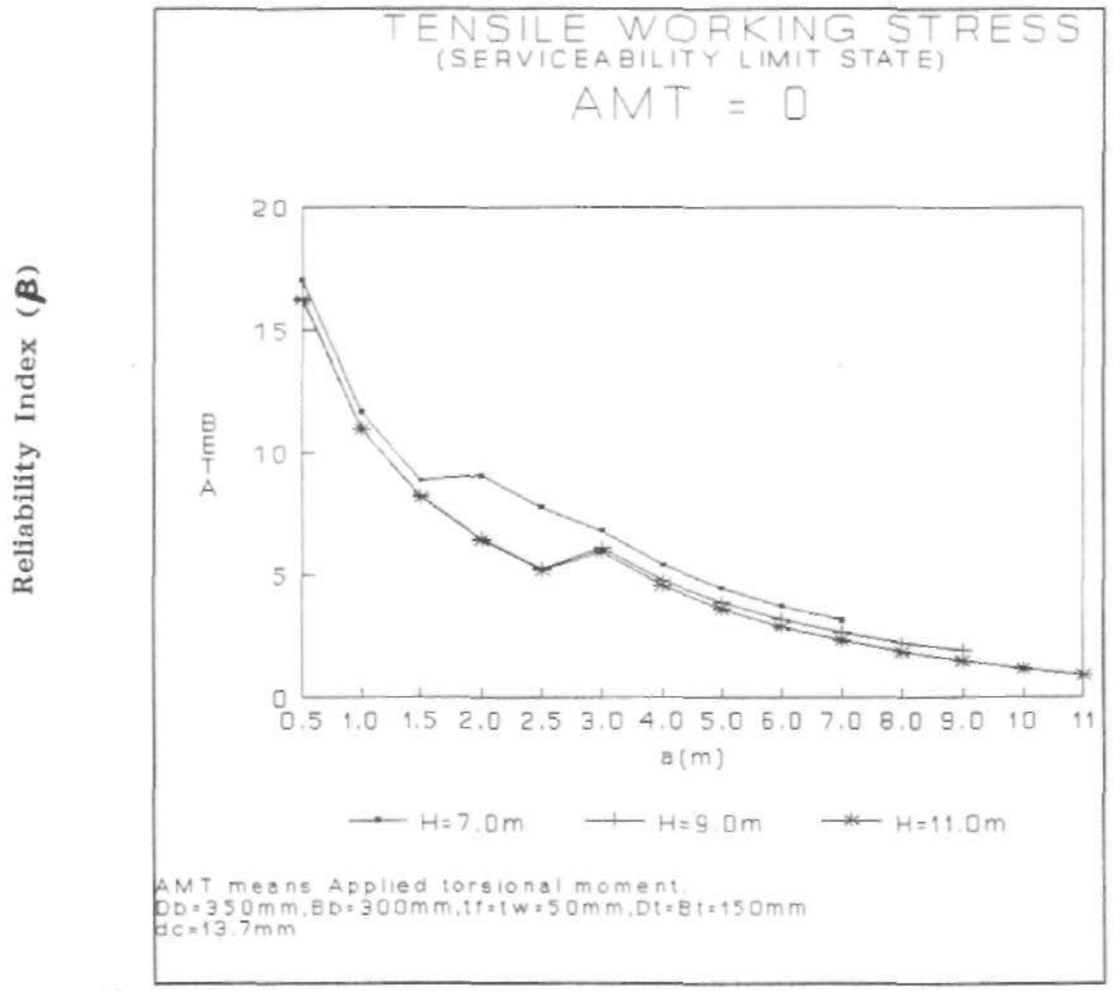


FIG 4(g): Variation of reliability of the pole in the compressive working stress condition with torsion moment and height.



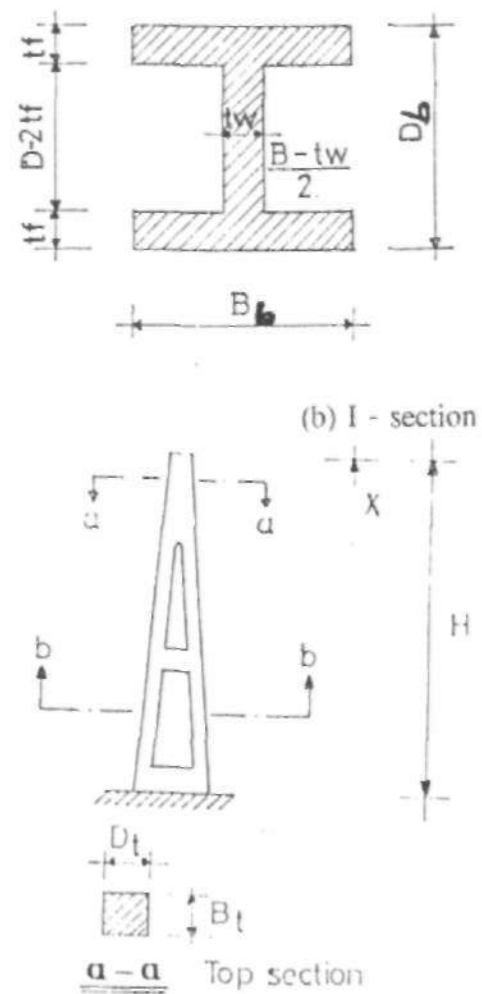


FIG 4(h): Variation of reliability of the pole for the compressive working stress condition with torsion moment and height.

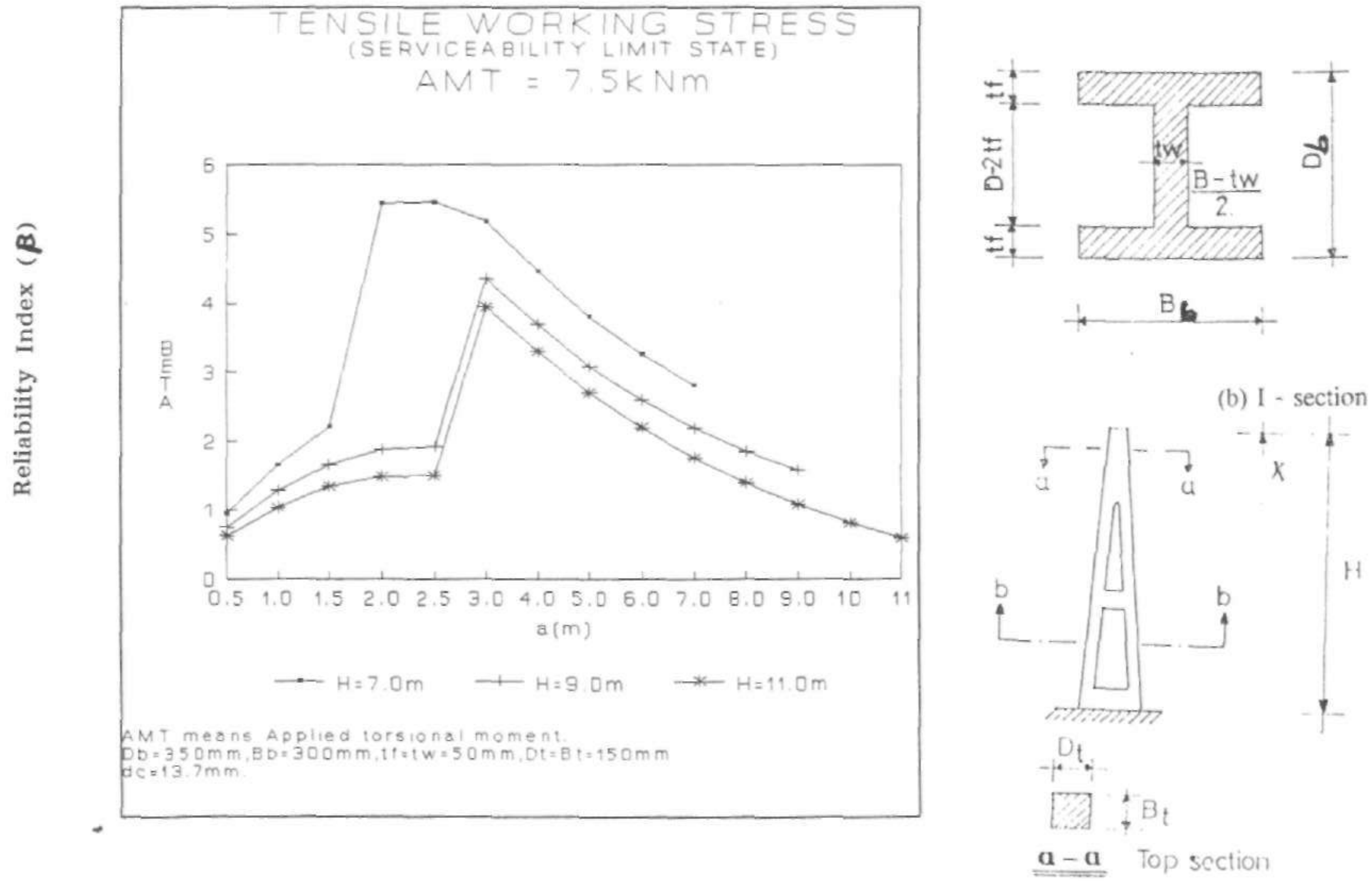


FIG 4(i): Variation of reliability of the pole for the compressive working stress condition with torsion moment and height.

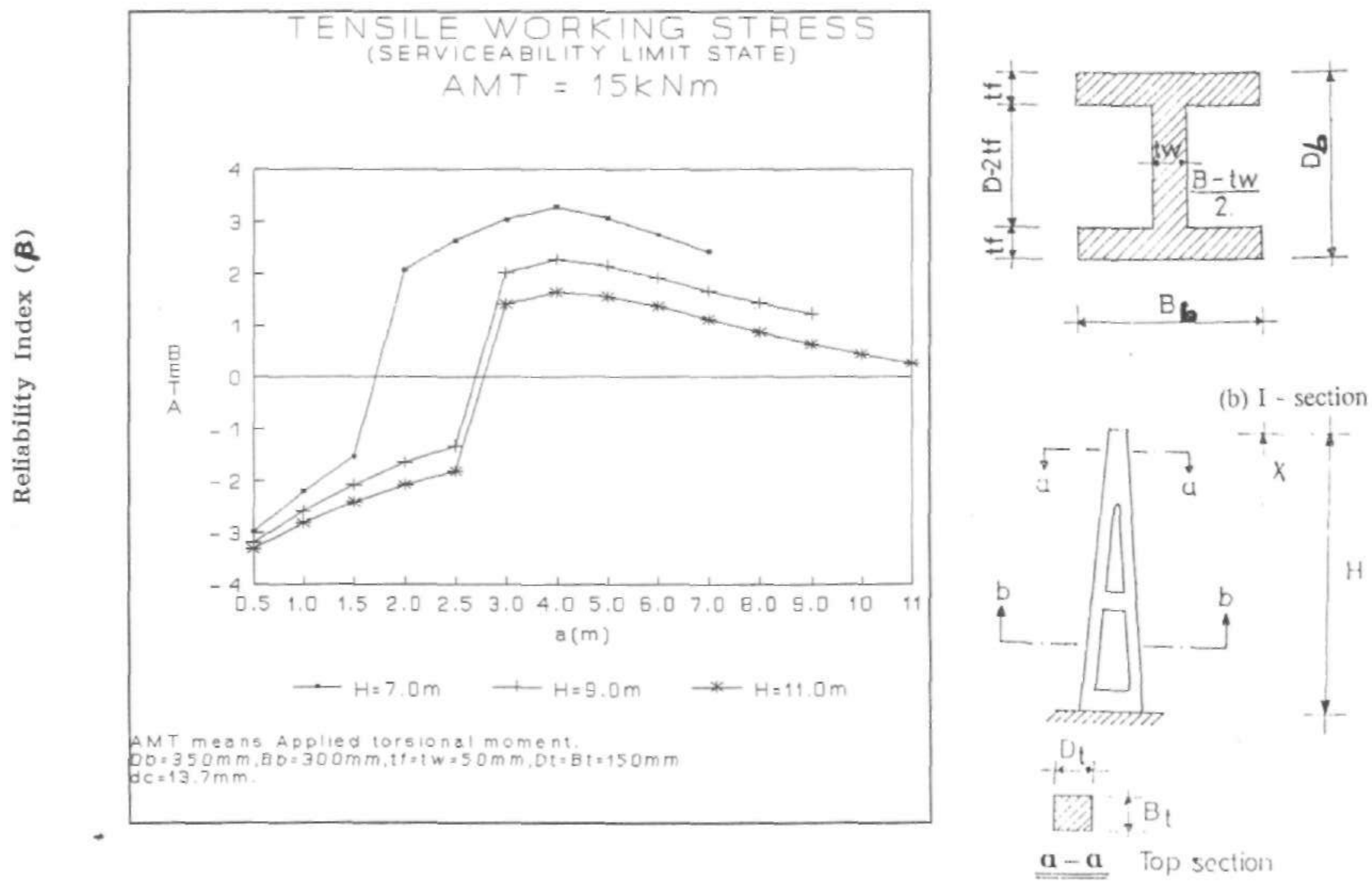
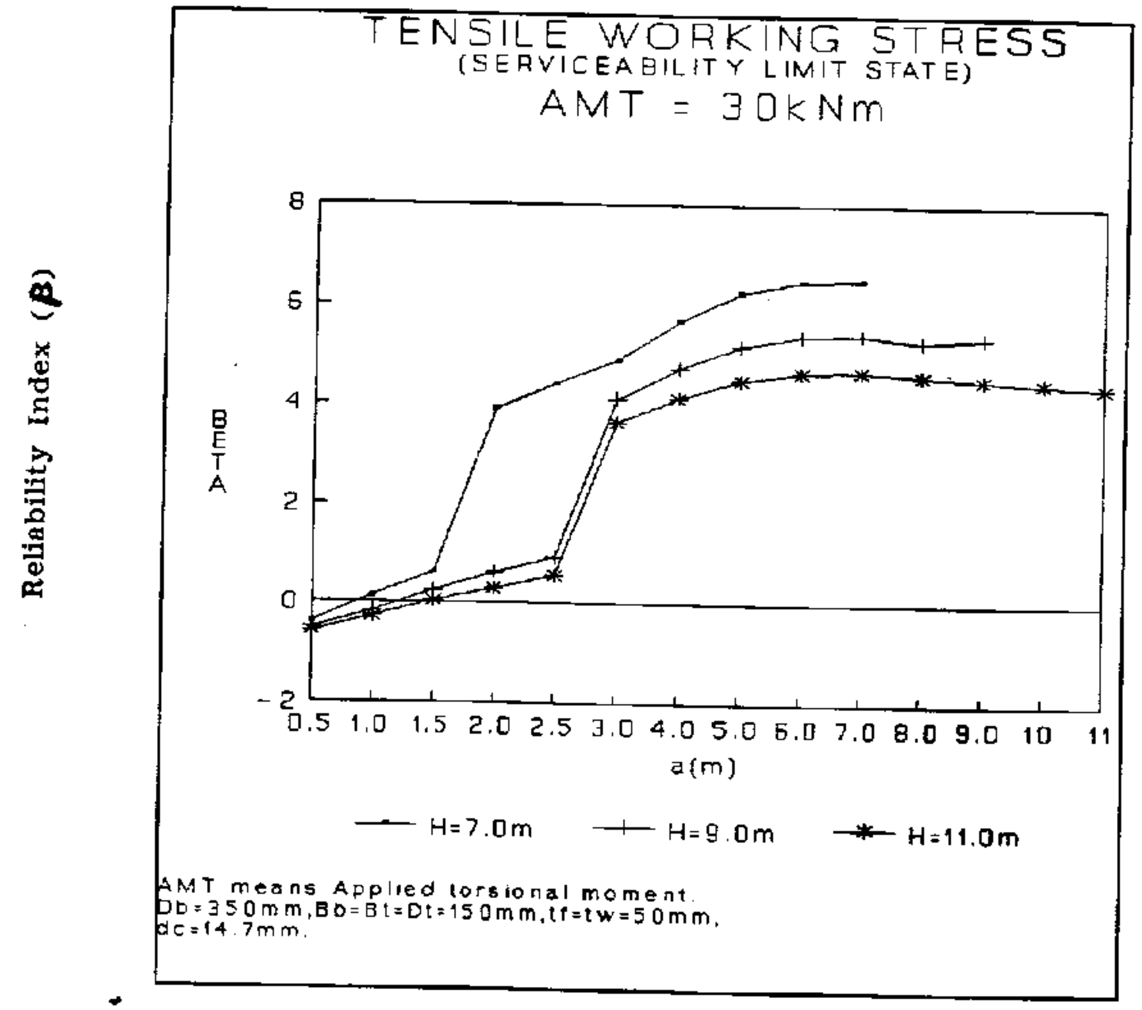


FIG 4(j): Variation of reliability of the pole for the compressive working stress condition with torsion moment and height.



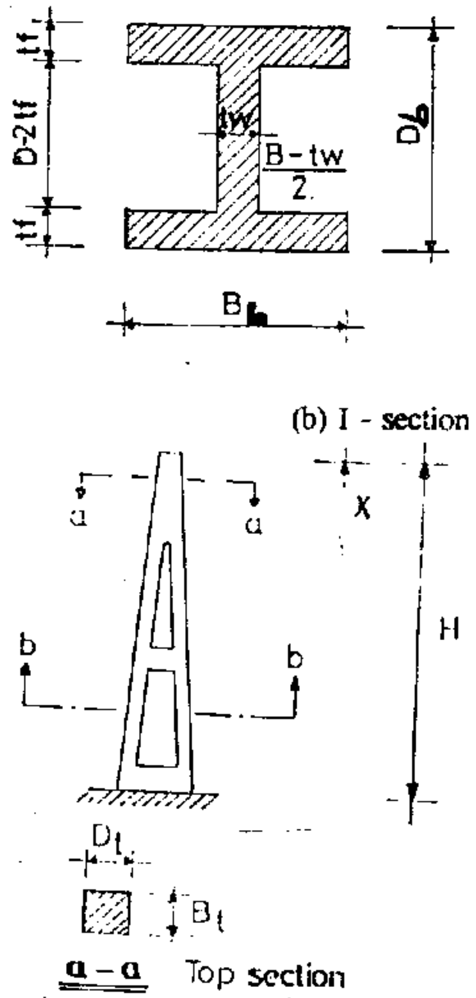
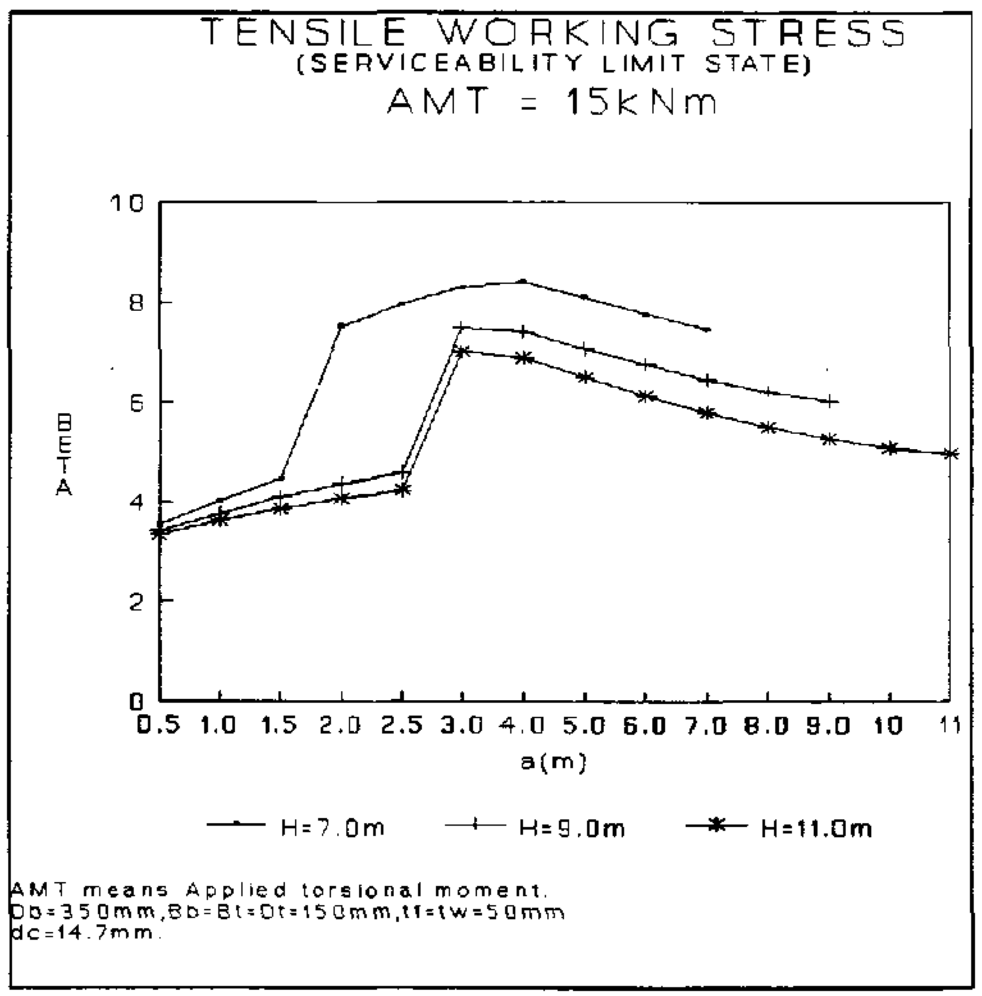


FIG 4(k): Variation of reliability of the pole in the tensile working stress condition with torsion moment and height.

Reliability Index (β)



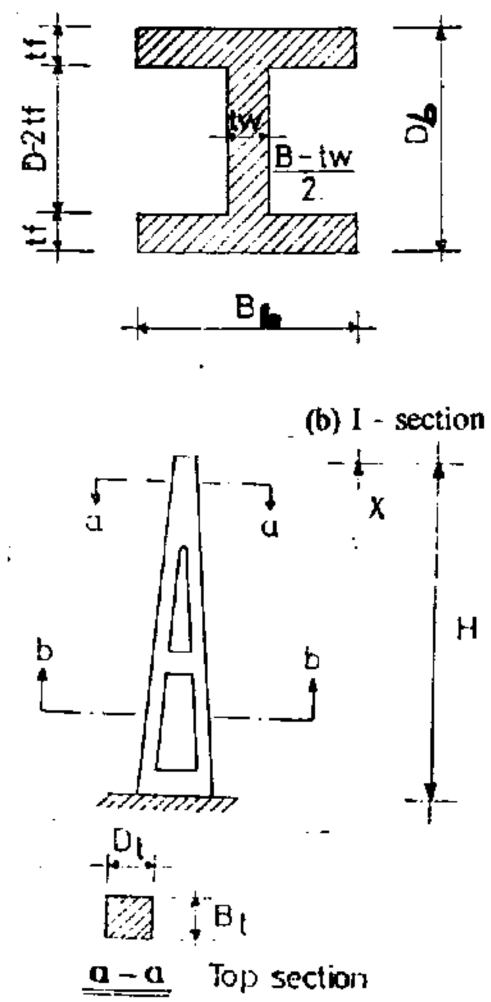
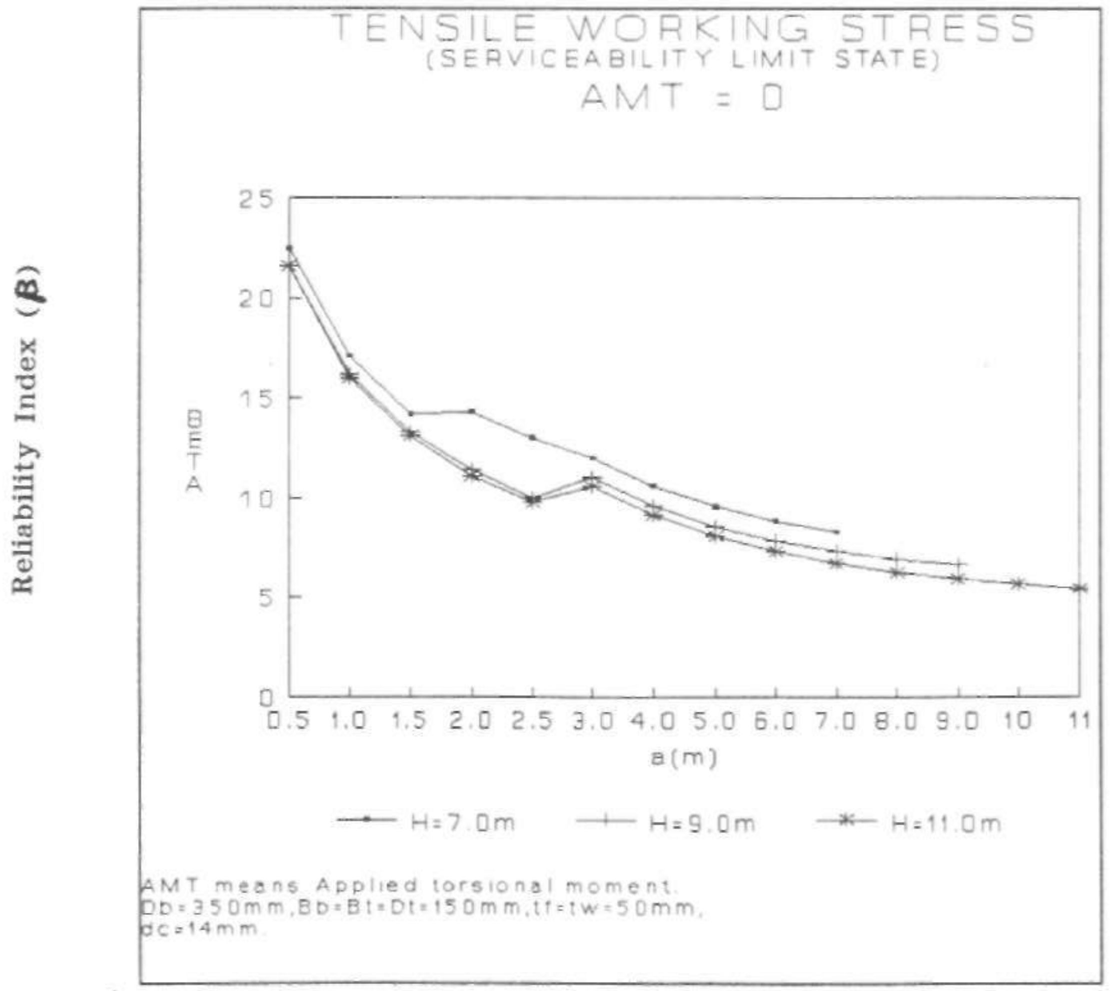


FIG 4(i): **Variation of reliability of the pole in the tensile working stress condition with torsion moment and height.**



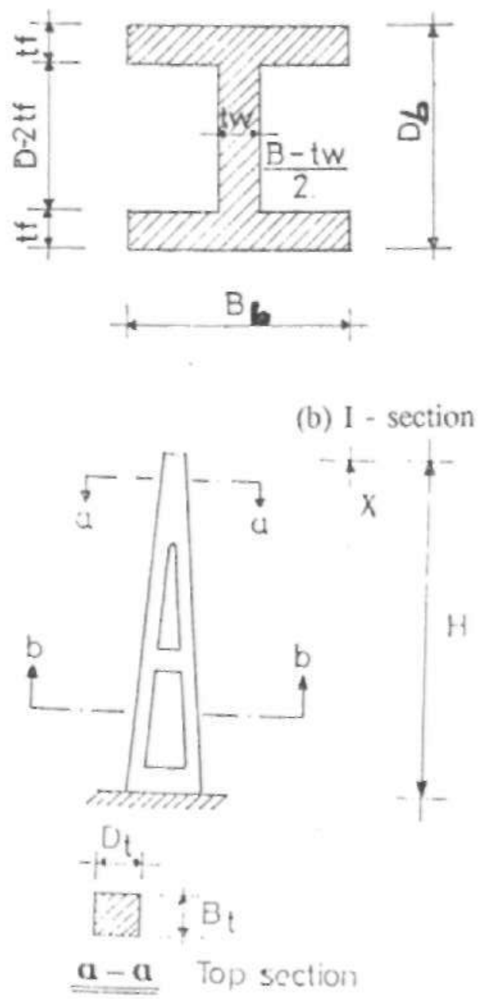


FIG. (4m): Variation of reliability of lattice pole with tensile stress and height.

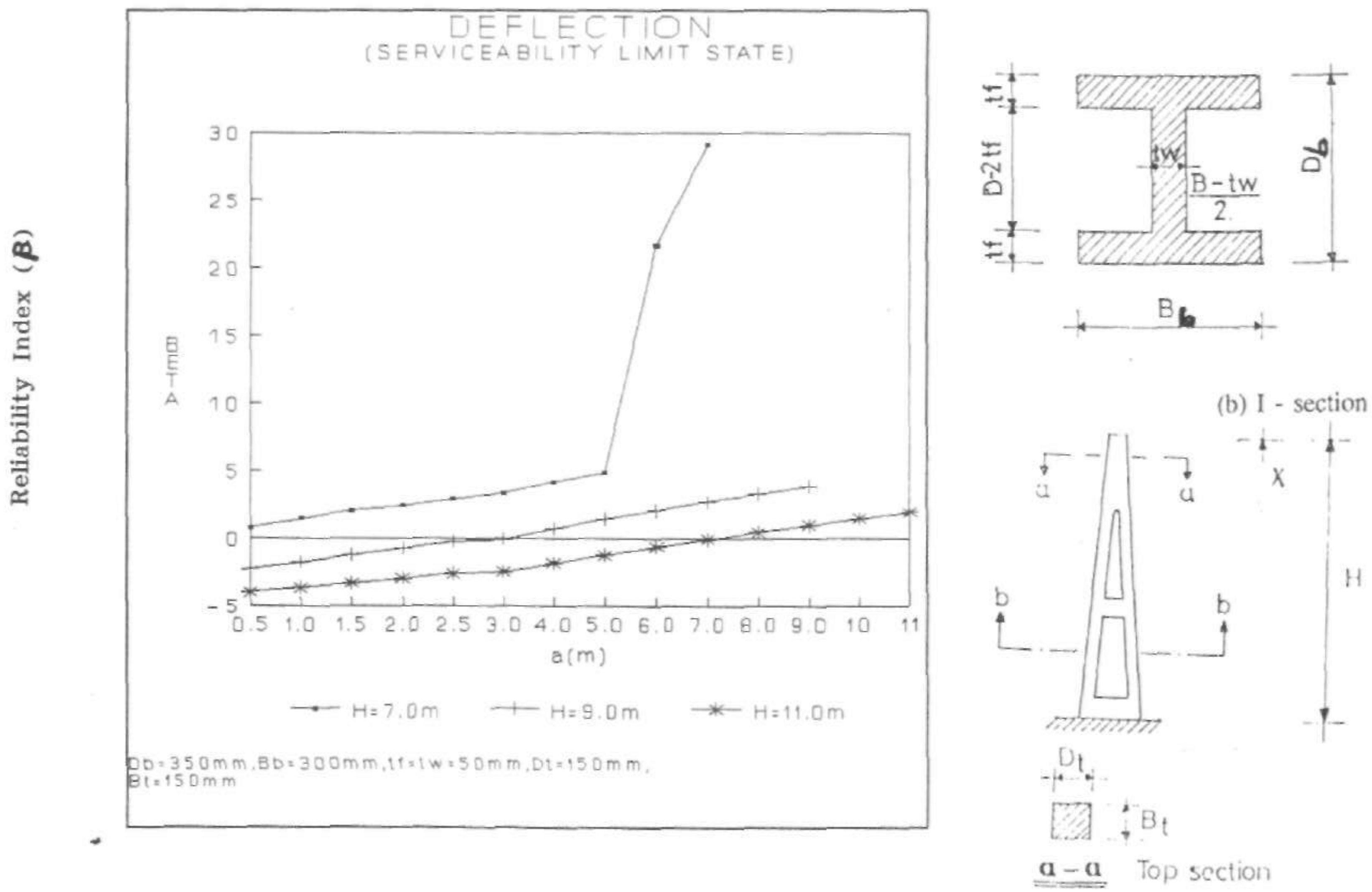


FIG. 4(o): Variation of reliability of the pole in deflection with height and geometry.

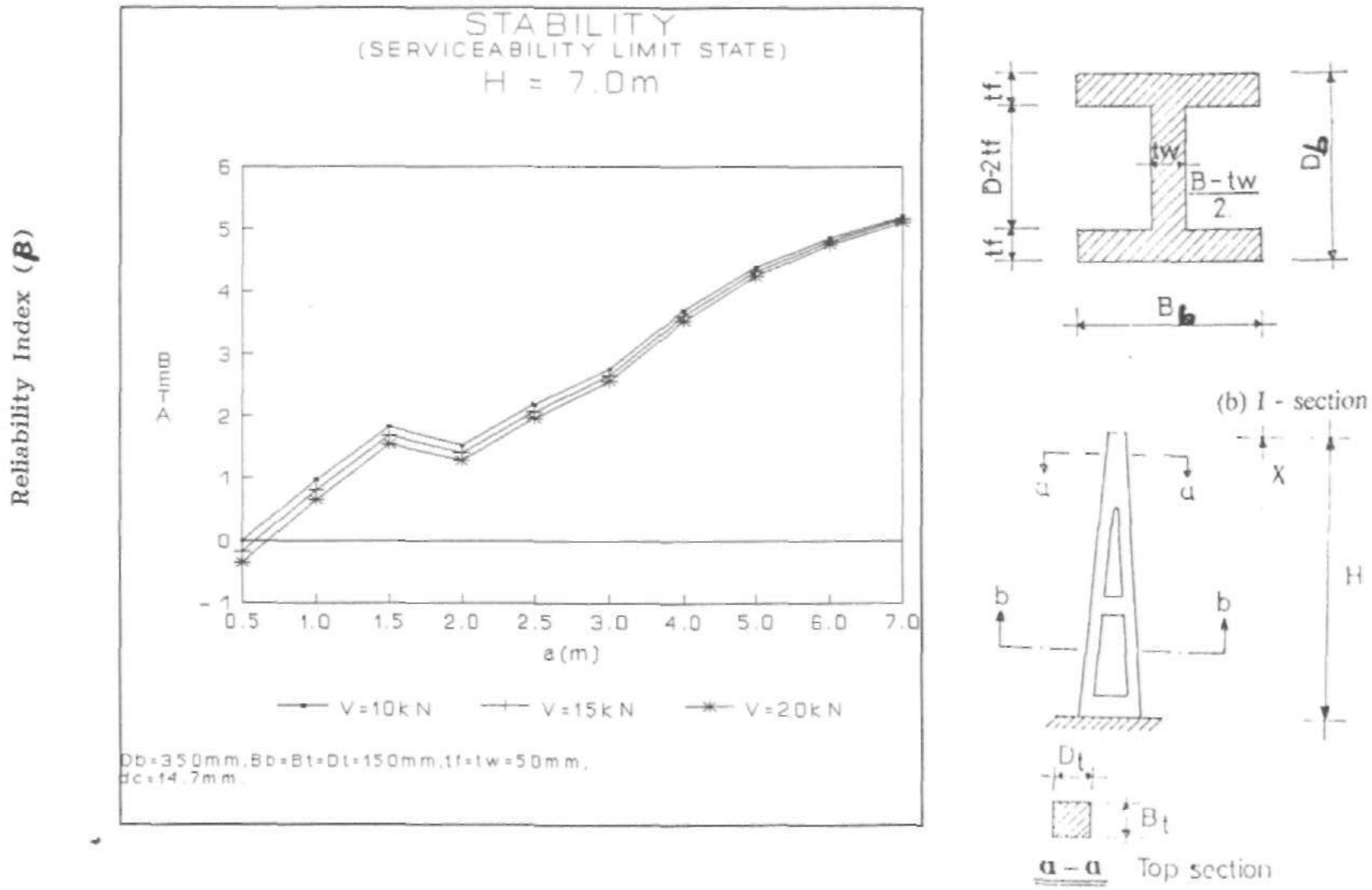
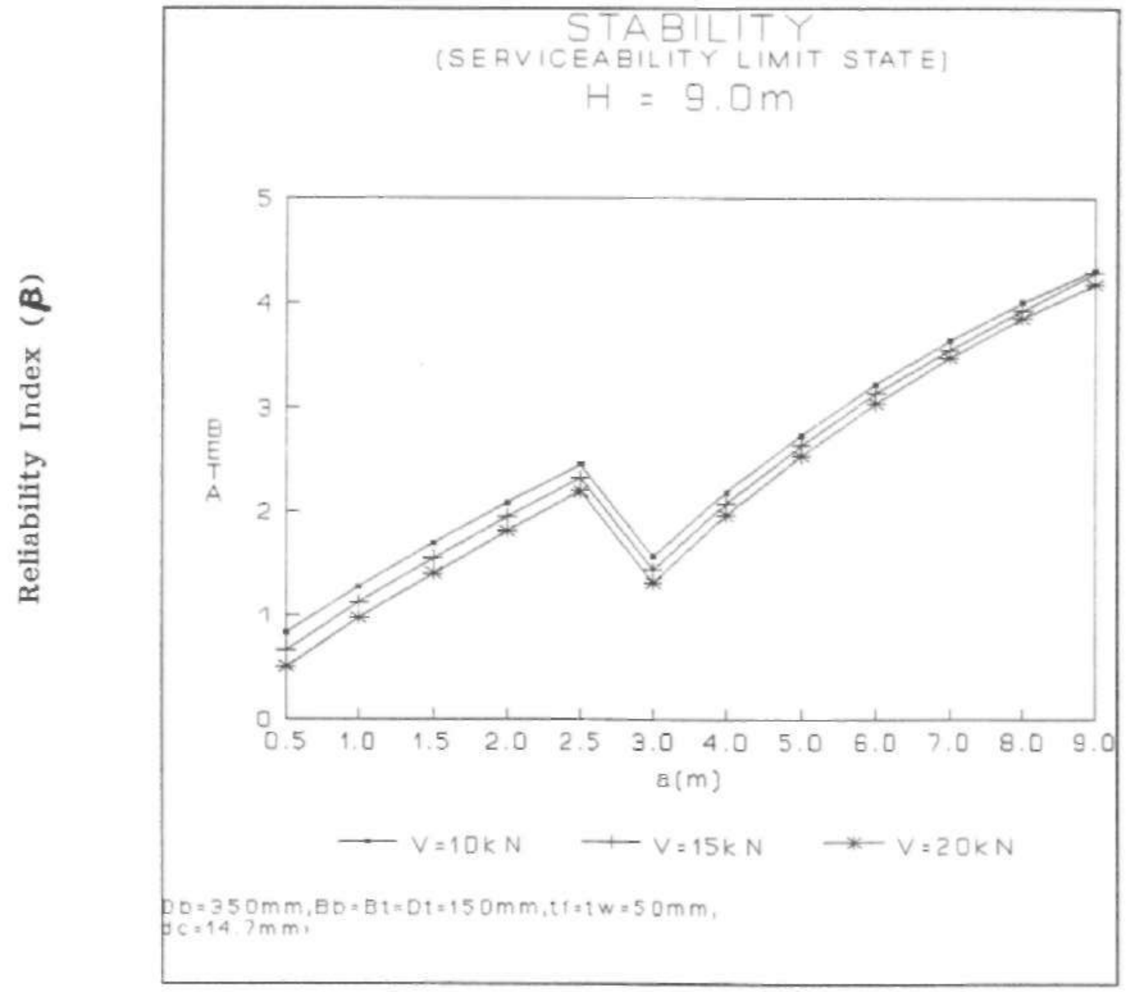


FIG. 4(p): Variation of reliability of the pole in deflection with height and geometry.



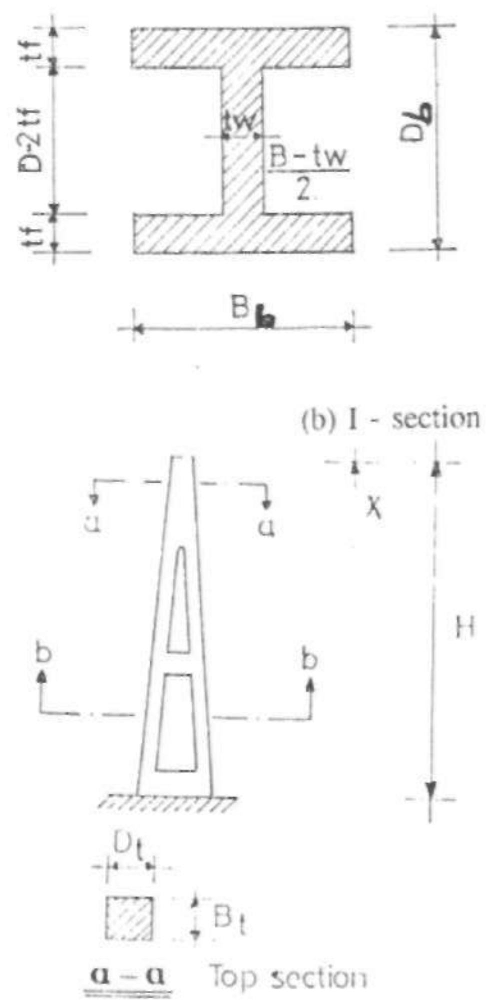


FIG. 4(q): Variation of reliability of the pole in stability with load and height.

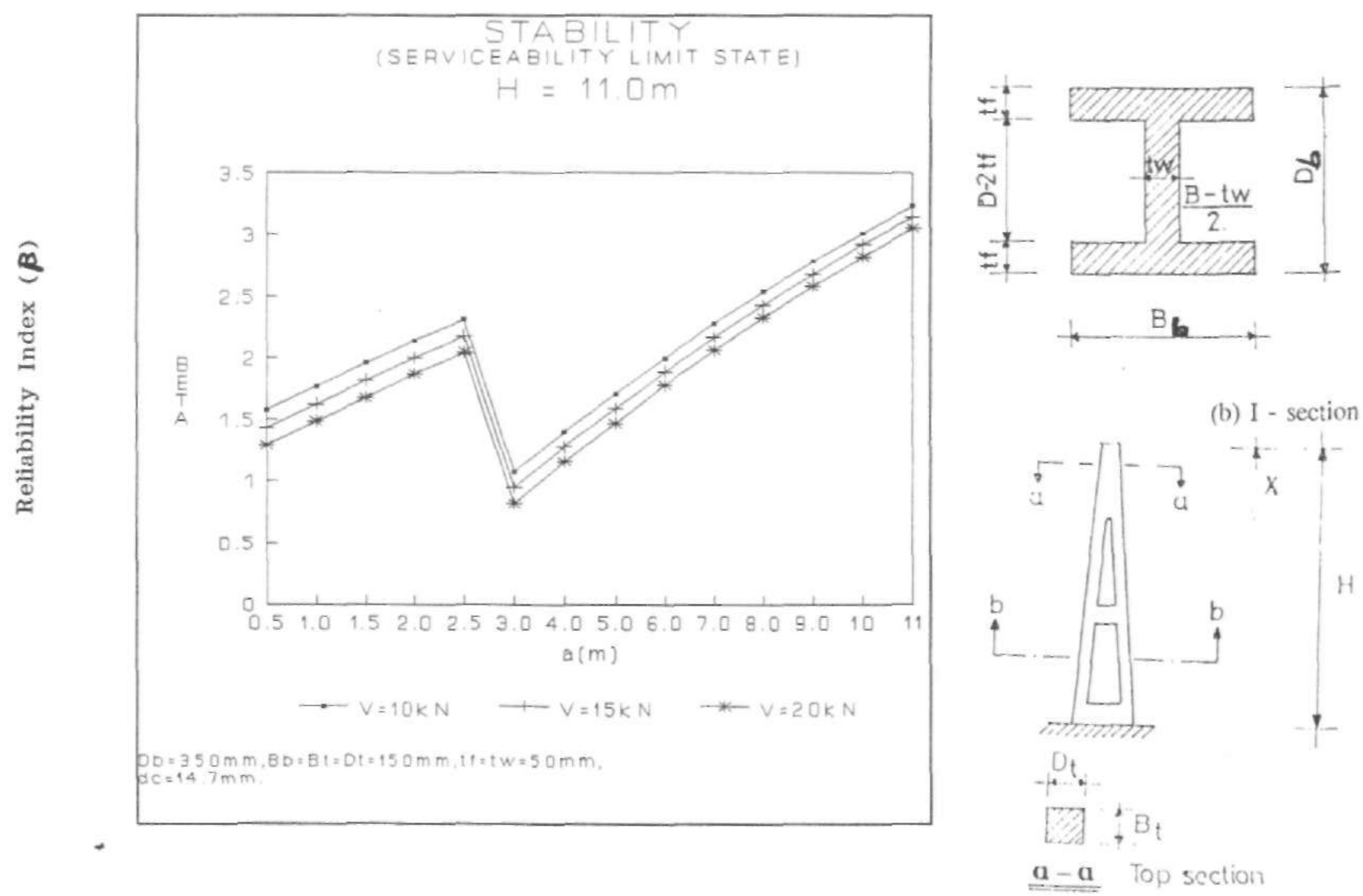


FIG. 4.(r): Variation of reliability of the pole in stability with load and height.

4.1.1 <u>Height of pole</u>

All the results obtained (Fig. 4 (a) to 4 (b), show that the reliability is highly sensitive to changes in the height; wherein, for a given set of load effects, the reliability decreases with increased height and vice versa. This is true for all the limit state conditions.

It is also observed that, the height is even more sensitive to changes in the load effect especially for the stability, deflection and the tensile working stress conditions. A slight variation in the load, causes a major instability; which increases with increased height. For instance, consider any point x (say 4.0m), distance taken from the apex along the length of the pole, the reliability decreases with increase in the magnitude of the load effect, e.g. torsion moment (for compression and tension), see Fig 4.1 to 4.2 and vertical load (for stability) see Fig 4.4(a - c).

Generally, the behaviour of the pole with changes in its height can best be explained in relation to changes in values of other design variables which shall be discussed in turn.

4.1.2 <u>Critical Region</u>

All the results examined suggest the existence of a region defined along the length of the pole in which transitional changes in the reliability takes place. This is true for all the limit state conditions, although the pattern of the changes as well as the interval within which the change takes place vary with the height, type and magnitude of the load effect considered. In all cases however, it falls within the range 1.5m - 4.0m (approx) from the apex; which almost coincides with the critical region described elsewhere (Essien, 1990) for the same member. Incidentally, this portion makes up the solid section portion of the pole which otherwise would have been expected to offer better resistance with higher reliability. It is suspected however, that, reversal of stresses takes place as a result of changes in the geometry which could have introduced some imperfections and perhaps other variabilities in the structural dimensions which could have led to stress concentration around such areas. (Palle et al, 1982; Boresi et al, 1976)

4.1.3 **Loading Conditions**

The major load effects encountered are torsion moment, bending moment, and the vertical component of the load supported by the pole, including self weight.

The primary source of application of these load effects is by way of wind pressure on pole (ρ_p) and cables (ρ_c) and perhaps of lesser impact, that due to self weight of pole (W_p) and cable (W_c).

The results (Fig 4.1 to 4.2) show that the reliability is highly sensitive to torsion moment with respect to both the compression and tension modes. It was found that, the application of torsion moment has more severe effect on the tension than on the compression face. From analogy, this can be attributed to concrete's better performance in compression than tension and especially a prestressed member in which the entire volume has been transformed into a compressive block.

The reliability levels implicit in the four limit states are discussed with respect to the various load effects.

(a) Effect of torsion moment on the Compressive working stress.

The variation in the reliability function observed here,

Fig. 4.1(a - g) can be summarised as follows:-

(i) for a given height, the reliability decrease with increase in load and vice versa.

(ii) the reliability decreases for a given load with increase in the height.

(iii) the reliability increases with increase in the structural dimensions.

(iv) the reliability increases with increase characteristic strength of concrete but decreases as the ratio of the steel tendons is increased in the section.

(v) generally, the reliability along the length of the pole from the apex increases up to a point (the critical point discussed in 4.1.2), then there is a slight drop, before it again continues to rise sharply down to the base of the pole.

(b) Effect of torsion moment on the tensile working stress

The severest effect of torsion moment can be more noticeable here, Fig. 4.2(a - f). As can be seen from the graphs, the reliability for a given pole height subjected to torsion moment increases from the apex until it attains a peak value at a point within the critical region described earlier. From there it begins to drop.

However, for a pole on which torsion is not applied (i.e, $M_T = 0$), it can be seen that the reliability becomes conservatively high around the apex ($\beta \geq 16.0$). It drops sharply along the length of the pole and then increases before it begins to decrease again down to the base. These results indicate that there are regions on the pole in which the reliability is highest, average and lowest respectively, with respect to applied torsion moment. These regions have been identified for the various heights of pole considered as follows:-

Table 4.1: Critical region for various pole heights in tension.

Height of pole (m)	Critical Region on Pole			
	Higher β -value		Lower β - value	
	Range	β - value	Range	β - value
7.0	2.0 - 5.0	4.89 to 6.51	0.5 - 1.5m	-0.384 to 0.612
9.0	3.0 - 6.0m	4.12 to 5.36	0.5 - 2.50m	-0.505 to 0.920
11.0	3.0 - 7.0m	4.13 to 4.40	0.5 - 2.50m	-0.582 to 0.556

The condition in which no torsion is applied (i.e. $M_T = 0$), corresponds to a free - standing pole on to which the runs of cables have not been mounted or possibly, an intermediate pole which is assumed will not experience line snaps at all. All the two possibilities above are impracticable and in another respect meaningless.

Since the results have shown that torsion moment is a critical load effect on the pole, then some form of regulatory measures are necessary to ensure the control of its effect so as to improve on the reliability of the pole.

(c) <u>The influence of wind load on deflection</u>

Excessive deflection of the member occurs around the apex but the member becomes more rigid down at the base. The β - values around the base for a pole of 7.0m is highly conservative,

Fig. 4.3(a - b).

The influence of the wind load on cables was found to be quite insignificant compared to that on the pole. The major regard here is the structural dimension and the height. Pole of 9.0 and 11.0m high all failed when tested for the trial section utilised by Essien (1990). It was found that heavy deep sections running from the top to the base will be favoured to ensure a robust pole section.

(d) The influence of vertical load on stability

As with the deflection limit state, the stability of the pole is only slightly affected by small changes in the vertical load but highly sensitive to changes in the height as well as structural dimensions, Fig. 4.4(a - c). The pole of height 11.0m has failed completely for the profile suggested by Essien (1990).

4.1.4 Spacing of poles.

The influence of the spacing between the poles on the reliability was also investigated for various possible combinations. Practical spacings for end poles and intermediate poles were considered e.g. 50m, 75m, 100m and 150m. In general, it was found that the variability of the spacing does not seem to have much effect on the reliability function and in some respect almost insensitive. Thus, the normal spacing of 50m and or 100m between poles was adopted and used throughout the analyses.

4.1.5 Characteristic strengths of materials.

The analysis also examined changes in the characteristic strength values of concrete (f_{cu}) and the prestressing tendons (f_{pu}) and area of steel tendons (A_{ps}). It was found that an increase in the value of f_{cu} causes a slight increase in reliability for the compressive working stress whereas changes in f_{pu} and A_{ps} have negative effect for the same mode. Conversely, an increase in f_{pu} and A_{ps} causes an increase in the reliability in the tensile working stress but lowers the reliability for stability.

4.1.6 Dimensional and geometrical property of section.

As can be inferred, the reliability is highly sensitive to variability in the structural dimensions and other geometrical imperfections. In all the limit states analysed, it was found necessary to adjust the value of 'D' as well as 'B' slightly.

The contributions of the flange thickness (t_f) as well as the web thickness (t_w) were found to be quite insignificant. The major structural dimension is the depth and then perhaps the tappering behaviour of the section.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Reliability based design procedures are increasingly being used in the structural engineering community. This reflects in part, an increasing appreciation that many load and resistance quantities are inherently random variable. The designer can only predict the future loads and even the as built characteristics of the structure, in a statistical sense (Kirshnasamy, et al, 1986).

The basic goal of reliability based methods is to better control the amount of unsatisfactory performance (failure) permitted in a design. Failure occurs when the demand (load effect) exceeds the supply (structural resistance).

Precisely, a transmission line designer should aim to build an efficient system that conveys energy between any two locations. During its operating life, the line will be subjected to an almost infinite variety of climatic events that cannot be predicted with certainty, but can only be approximated by their probability laws. For instance, a windstorm event can be characterised by the largest velocity, direction and spatial event of its gusts together with ambient temperature. The parameters often used are the mean, coefficient of variation and a T - year return period value. The T - year return period is a value that has 1/T probability of being exceeded in one year. (Peyrot, et al, 1986).

Thus, many quantities often assumed to be deterministic in design approaches are actually random variables which can be modelled by statistical distinctions. Variability often arises owing to incomplete control on materials quality, fabrication and construction, incomplete data, inexact information, approximations knowingly made, incomplete behavioural models and many other sources including those due to human errors.

It has been shown from the foregoing analysis that economy and reliability are inherently competing demands; since one of the obvious ways to increase reliability is to increase structural strength at an increased cost. A second approach; which often increases economy, is to improve the structural design process by better allocating the material within a structure and among the structures in a system so that the loads can be more efficiently resisted. Whatever the method adopted or choosen, it can lead to difficult managerial decisions at both planning and operational phases.

Since transmission lines are highly sensitive to wind directions as well as its magnitude and considering also that wind loadings are time variant and therefore not amenable to effective control, it will be necessary to device other means of improving the reliability of the pole. The value of the wind speed utilised by Essien (1990) which is 54m/s for Zaira and Kaduna environment is highly conservative. A value of 35m/s based on the improved wind velocity data (Afolayan et al, 1996) has been utilised (Appendix II).

In conclusion, an attempt has been made to present the method of reliability analysis for solving the problem of failures of transmission line structures. Four (4) serviceability limit state conditions were examined and a sensitivity analysis for some basic variables was carried out to investigate the influence of these variables on the reliability of the pole. All the results obtained indicate that reliability is highly sensitive to torsion moment with respect to both the compression and tension modes. The reliability of the pole is also affected by other load effects, height of the pole, structural dimension and to a lesser extent, the characteristic strength of the materials.

Of concern, was the existence of a critical region along the length of the pole (for all classes) in which transitional changes in the β - value occur. From the sensitivity analyses, a profile is recommended alongside the corresponding measures to ensure the regulation of the load effect on the member.

5.3 Recommendations

So far, attempt has been made to justify the need to establish balance design between safety and economy in relation to design decisions. From the summary of the results discussed, the following recommendations can be made for future designers of transmission concrete poles, with reliability consciousness.

(a) Height of pole

The height of pole for use around this location with average wind velocity of 35m/s should be limited to between 7.0 - 9.0m from the ground level.

(b) Spacing of pole

The spacing between poles along a transmission line network should be taken as 100m as much as practicable; other than due to variabilities necessitated by some king of obstacles or other form of restrictions along the line , in which case a variation of ± 25m can be admissible.

(c) Structural dimensions

The following dimensions are recommended for poles with I - section :

D_b = 350mm

D_t = 150mm

$B_b = B_t$ = 150mm

$t_f = t_w$ = 50mm

(d) Geometrical property of section

The use of dual section does not seem to be favoured by all the limit states conditions. Some researchers (e.g Essien, 1990 and Wuyana, 1994) have confirmed the suitability of other geometrical profiles such as circular and rectangular hollow sections over the I - section.

(e) Characteristic strength of materials.

It is recommended that the quality and grade of concrete should be well controlled to ensure satisfactory performance and the attainment of the specified characteristic value of f_{cu} = 50N/m^2 or over. Similarly, a value of f_{pu} = 1870 N/m^2 as well as A_{ps} = 4.91mm^2 is considered adequate.

Alternatively, where in - situ conditions will not ensure the attainment of the desired concrete strength, then the possibility of using structural steel sections can be considered (as encountered in some places like Bauchi and Gombe states). These sections can withstand higher loads overtime, although may be relatively more expensive.

(f) Loading conditions

In order to reduce the magnitude of the torsion moment as well as the vertical load applied on the pole, aluminum conductor cables are preferred over copper conductors. The former is much lighter in weight (0.35kg/m^3) as against 1.04kg/m^3 for copper thereby reducing the total weight to be supported by pole and also transmit less wind load on to the pole compared to that from copper.

Additionally, while the breaking load for the copper conductor is 44.1KN, that for aluminum is only about 20.3KN. The practical implication is that, less torsion will be applied on the pole with aluminum conductors than with copper conductors whenever the line snaps.

Another advantage is that there is an increase in the diameter of the cables from 13.7mm for copper to 14.7mm for aluminum thereby reducing the tendency of line snaps.

It is worth noting that, the use of prestressed concrete poles developed fairly recently compared to steel and wood poles. However, while failures have become very common with the former, it is not the case in the latter especially steel poles. It is doubtful if actually prestressed concrete poles are used in their true specifications. Perhaps ordinary reinforced concrete poles are achieved at the manufacturers yard and are then made to function as prestressed sections.

Finally, the need for the understanding of the implication of failure of a structure must be appreciated. Most people have limited the understanding of failure to simply mean "observed collapse" (Plate 3). In reality, this is not so and entails much more than that. An examination of most structures or structural systems will confirm the fact that they have failed in service. For instances, most poles are out of alignment along a line network, some poles have clearly tilted (Plate 2), others are founded on weak foundations etc.

Therefore, there is the urgent need to be more concious about service failures than "ultimate (collapes) failures to ensure a balance between safety and economy in the performance of structural systems. The structural reliability analysis approach is a good beginning.

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APPENDIX 1A

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                 PROGRAM COMP
C
C This program evaluate the reliabilty of a Prestressed concrete
C transmission pole subjected to compressive working stress
C constraints
c
       Implicit Double Precision (A - H, O - Z)
C
C The Subroutine with the limit state function is
C declared external.
C
       External GCOMP
C
C The basic variable used are arranged in the following
C order:- D, B, tf, tw, fcu, fpu, Aps, Lr, pp, pc
C
       Dimension X(10), EX(10), SX(10), VP(10,10), COV(10,10),
     +          ZES(3), UU(10), EIVEC(10,10), IV(2,10)
C
       Character * 10 PRT
C
C Any deterministic parameter is set in this common block
C e.g.no. of steel tendons (s), the height of the pole from
C the ground  level (H), the diameter of the cables (diam),
C the no. of  cables (c) and the spacing between poles (Space)
C
       COMMON /GCOMP/s, a, amax, AMT, H, diam, c, Space
C
C The mean and standard deviation of the variables as
C well as the parameters of FORM5 are given in this
C data block.
C
       Data EX/1.2D2, 1.2D2, 5.D1, 5.D1, 6.D1, 1.87D3, 4.91D0,
     +     7.D-1, 5.2D-4, 1.01D-4/,
     +     SX/4.8D0, 4.8D0, 2.D0, 2.D0, 1.2D1, 2.805D2,
     +     1.77D-1, 3.5D-2, 1.924D-4, 3.74D-5/,
     +     N/10/, NC/10/, NE/10/, IRHO/1/
C
          s = 2.D1
          AMT = 3.D7
           H = 7.D3
           diam = 1.47D1
            c = 3.D0
             Space = 1.D5
C
       Write(*,*) 'Enter the Value of   a .......>'
       Read(*,*) a
C
       IF(H . GT . 7.D3)THEN
         amax  =  2500.
```

```
      ELSE
        amax  =  1500.
      ENDIF
C
C To carter for the tappering of the member, an adjustment is made
C on the values of D, B and pp initiated above.
C
      X(1) = 150. * (1. + 1.33 * a/H)
      EX(1) = X(1)
      SX(1) = EX(1) * 4.D-2
      X(2) = 150.
      Ex(2) = X(2)
      Sx(2) = Ex(2) * 4.D-2
C
C Adjustment is made for the wind pressure on the pole based on the
C height of pole considered.
C
      IF(H . EQ . 7.D3)THEN
       X(9) =  5.2D-4 + 1.D-4 * 2.3 * (H-a)/H
       Ex(9) = X(9)
       Sx(9) =  Ex(9) * 3.7D-1
      ELSE
       X(9)  =  5.2D-4 + 1.D-4 * 3.8 * (H-a)/H
       EX(9)  =  X(9)
       SX(9)  =  EX(9) * 3.7D-1
      ENDIF
C
C Adjustment is made for the number of prestressing steel (s)
C at the top portion of the pole.
C
      IF(a . LE . amax)s =  1.6D1
C
C The result are written in file No. 7
C
      NAUS = 7
C
C  Print to Screen
C
      ICRT = 0
      Open (7, File = 'COMP.RES', STATUS = 'OLD', ERR = 10)
      Go to 20
 10   Open (7, File = 'COMP.RES', STATUS = 'NEW')
C
C The variables VP, Cov and IV are Preset using YINIT
C
 20   Call YINIT (N, IV, VP, IRHO, COV, NC)
      IV (1,6) = 3
      IV (1,9) = 7
      IV (1,10) = 7
      DO 100 I = 1, N
 100  X(I) = Ex(I)
      V1 = 1.D0
```

```
      BETA = 1.D0
C
      WRITE(NAUS,500)
 500  FORMAT(///,5X,70('*'),/,30X, 'FORM5',/,5X,70('*'),/,
     + 'SLS FOR COMPRESSIVE WORKING STRESS CONSTRAINTS, 10
     + VARIABLES :')
c
C The Stochastic model is printed using 'YKOPF'
C
      Call YKOPF(NAUS, N, IV, EX, SX, VP, IRHO)
C
C Print also to screen
      WRITE(ICRT,*) 'START OF FORM5'
      WRITE(ICRT,*) 'STOCHASTIC MODEL:'
C
      CALL YKOPF(ICRT, N, IV, EX, SX, VP, IRHO)
C
C The matrix COV is printed using 'YMAUS'
C
      CALL YMAUS(NAUS, NC, N, Cov, PRT)
C
C The main Subroutine in FORM is called
C
      CALL FORM5(N, IV, EX, SX, VP, GCOMP, IRHO, COV, NC, EIVEC, NE,
     +           V1, NAUS, BETA, X, UU, ZES, IER)
C
C The Coordinate of the beta - point are printed with the vector
C titled UU
C
       PRT = '  UU  '
C
C
       CALL YFAUS(NAUS, N, UU, PRT)
C
C The vector ZES is printed with the  title vector ZES
C
       PRT = '  ZES  '
       CALL YFAUS(NAUS, 3, ZES, PRT)
       WRITE(ICRT,*) 'END OF FORM5'
       WRITE(ICRT,*) 'FOR RESULTS SEE FILE COMP.RES'
       WRITE(ICRT,*) 'a..........=',a
       WRITE(ICRT,*) 'X(1)........=',X(1)
       WRITE(ICRT,*) 'X(2)........=',X(2)
       WRITE(ICRT,*) 'X(9)........=',X(9)
       STOP
       END
C
C
C
             SUBROUTINE GCOMP(N,X,FX,IER)
C
      Implicit Double Precision (A - H, O - Z)
```

```
      Dimension X(N)
      COMMON /GCOMP/ s, a, amax, AMT, H, diam, c, Space
C
      W = X(9) * X(2)
      P = X(10) * diam * c * Space/24.
      AMA = W * a**2/2. + P * a
C
C Let AC, AIX, AIY, PJ and ZT represent the cross sectional area,
C moments of inertia across the x-x and the y-y axes, polar moment
C of inertia and the modulus of section across any section
C distant 'a' from the top end of the pole respectively.
C
      IF(a.LE.amax)THEN
        AC = X(1) * X(2)
        AIX = X(2) * X(1)**3/12.
        AIY = X(1) * X(2)**3/12.
        PJ = AIX + AIY
        ZT = X(2) * X(1)**2/6.
      ELSE
        AC = 2.* X(2) * X(3) + (X(1) - 2.* X(3)) * X(4)
        AIX = (X(2)*X(1)**3 - (X(2)-X(4))*(X(1)-2.*X(3))**3)/12.
        AIY = (X(1)*X(2)**3 - (X(1)-2.*X(3))*(X(2)-X(4))**3)/12.
        PJ = AIX + AIY
        ZT = 2.* AIX/X(1)
      ENDIF
C
C
      IF(AC .GT. 0. .AND. PJ .GT. 0. .AND. ZT .GT. 0.) THEN
          FX = .33 * X(5) - (s * X(6) * X(7) * X(8)/AC + AMA/ZT
     +         + AMT * X(4)/PJ)
          IER = 0
      WRITE(*,*)'FX..........=',FX
      ELSE
          FX = 1.D20
          IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX 1B

PROGRAM TENS

```
C
C This program evaluate the reliabilty of a Prestressd concrete
C transmission pole subjected to  tensile  working stress
C constraints
c
      Implicit Double Precision (A - H, O - Z)
C
C The Subroutine with the limit state function is
C declared external.
C
      External GTENS
C
C The basic variables used are arranged in the following
C order:- D, B, tf, tw, fpu, Aps, Lr, pp, pc
C
      Dimension X(9), EX(9), SX(9), VP(10,9), COV(9,9),
     +          ZES(3), UU(9), EIVEC(9,9), IV(2,9)
C
      Character * 10 PRT
C
C Any deterministic parameter is set in this common  block
C e.g.no. of steel tendons (s), the distance to any point
C along the length of the pole from the top end (a), the
C applied torsional moment on the pole (AMT), the
C height of the pole from the ground  level (H),
C the diameter of the cables (diam), the no. of cables (c)
C and the spacing between poles (Space)
C
      COMMON /CTENS/s, a, amax, AMT, H, diam, c, Space
C
C The mean and standard deviation of the variables as
C well as the parameters of FORM5 are given in this
C data block.
C
      Data EX/1.2D2, 1.2D2, 5.D1, 5.D1,1.57D3, 1.96D1,
     +     7.D-1, 8.03D-4, 1.01D-4/,
     +     SX/4.8D0, 4.8D0, 2.D0, 2.D0, 2.355D2, 7.056D-1,
     +     3.5D-2, 2.97D-4, 3.74D-5/,
     +     N/9/,NC/9/,NE/9/,IRHO/1/
C
         s = 2.D1
          AMT = 3.D7
          H = 1.1D4
          diam = 1.47D1
            c = 3.D0
             Space = 1.D5
C
      Write(*,*) 'Enter the Value of   a .......>'
      Read(*,*)a
```

```
C
      IF(H . GT . 7.D3)THEN
        amax  =  2500.
      ELSE
        amax  =  1500.
      ENDIF
C
       X(1) = 120. * (1. + 1.65 * a/H)
       EX(1) = X(1)
       SX(1) = EX(1) * 4.D-2
       X(2) = 120. * (1. + 1.1 * a/H)
       EX(2) = X(2)
       SX(2) = EX(2) * 4.D-2
C
C Adjustment is made for the value of the wind pressure on the pole
C depending on the height of the pole considered.
C
      IF(H . EQ . 7.D3)THEN
       X(8) = 5.2D-4 + 1.D-4 * 2.3 * (H-a)/H
       EX(8) = X(8)
       SX(8) = EX(8) * 3.7D-1
      ELSE
       X(8) = 5.2D-4 + 1.D-4 * 3.8 * (H-a)/H
       EX(8) = X(8)
       SX(8) = EX(8) * 3.7D-1
      ENDIF
C
C Adjustment is made for the number of prestressing steel (s)
C at the top portion of the pole
C
      IF(a . LE . amax)s  =  1.6D1
C
C The result are written in file No. 7
C
      NAUS = 7
C
C  Print to Screen
C
      ICRT = 0
      Open (7, File = 'TENS.RES', STATUS = 'old', ERR = 10)
      Go to 20
 10   Open (7, File = 'TENS.RES', STATUS = 'NEW')
C
C The variables VP, COV and IV are Preset using YINIT
C
 20   Call YINIT (N, IV, VP, IRHO, COV, NC)
      IV (1,5) = 3
      IV (1,8) = 7
      IV (1,9)  = 7
      DO 100 I = 1, N
 100  X(I) = EX(I)
      V1 = .5D0
```

```
      BETA = 1.D0
C
      WRITE(NAUS,500)
 500  FORMAT(////,5X,70('*'),/,30X, 'FORM5',/,5X,70('*'),/,
     + 'SLS FOR   TENSILE   WORKING STRESS CONSTRAINTS, 10
     + VARIABLES :')
C
C The Stochastic model is printed using 'YKOPF'
C
      Call YKOPF(NAUS, N, IV, EX, SX, VP, IRHO)
C
C Print also to screen
C
      WRITE(ICRT,*) 'START OF FORM5'
      WRITE(ICRT,*) 'STOCHASTIC MODEL:'
      CALL YKOPF(ICRT, N, IV, EX, SX, VP, IRHO)
C
C The matrix COV is printed using 'YMAUS'
C
      CALL YMAUS(NAUS, NC, N, COV, PRT)
C
C The main Subroutine in FORM is called
C
      CALL FORM5(N, IV, EX, SX,VP, GTENS, IRHO, COV, NC, EIVEC, NE,
     +           V1, NAUS, BETA, X, UU, ZES, IER)
C
C The Coordinate of the beta - point are printed with the vector
C titled UU
C
       PRT = '  UU  '
C
C
       CALL YFAUS(NAUS, N, UU, PRT)
C
C The vector ZES is printed with the  title vector ZES
C
       PRT = '  ZES  '
       CALL YFAUS(NAUS, 3, ZES, PRT)
       WRITE(ICRT,*) 'END OF FORM5'
       WRITE(ICRT,*) 'FOR RESULTS SEE FILE TENS.RES'
       WRITE(ICRT,*) 'a........=',a
       WRITE(ICRT,*) 'X(1).....=',X(1)
       WRITE(ICRT,*) 'X(2).....=',X(2)
       WRITE(ICRT,*) 'X(8).....=',X(8)
       STOP
       END
C
C
C
            SUBROUTINE GTENS(N,X,FX,IER)
C
      Implicit Double Precision (A - H, O - Z)
```

```
      Dimension X(N)
      COMMON/CTENS/s, a, amax, AMT, H, diam, c, Space
C
      W = X(8) * X(2)
      P = X(9) * diam * c * Space/24.
      AMA = W * a**2/2. + P*a
C
C Let AC, AIX, AIY, PJ and ZT represent the cross-sectional area,
C moments of inertia about the x-x and the y-y axes, polar moment of
C inertia  and the modulus of section for the pole across any section
C distant 'a' from the top end of the pole respectively.
C
      IF(a . LE . amax)THEN
        AC = X(1) * X(2)
        AIX = X(2) * X(1)**3/12.
        AIY = X(1) * X(2)**3/12.
        PJ = AIX + AIY
        ZT = 2. * AIX/X(1)
      ELSE
        AC = 2.*X(2)*X(3) + (X(1) - 2.*X(3)) * X(4)
        AIX = (X(2)*X(1)**3 - (X(2) - X(4))*(X(1) - 2.*X(3))**3)/12.
        AIY = (X(1)*X(2)**3 - (X(1) - 2.*X(3))*(X(2) - X(4))**3)/12.
        PJ = AIX + AIY
        ZT = 2. * AIX/X(1)
      ENDIF
      IF(AC .GT. 0. .AND. PJ .GT. 0. .AND. ZT .GT. 0.) THEN
        FX = s * X(5)*X(6)*X(7)/AC - AMT*X(4)/PJ - AMA/ZT
          IER = 0
      WRITE(*,*) 'FX......=',FX
      ELSE
          FX = 1.D20
          IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX 1C

```
                PROGRAM DEFLEC
C
C This program evaluate the reliabilty of a Prestressed concrete
C transmission pole subjected to  deflection constraints.
c
      Implicit Double Precision (A - H, O - Z)
C
C The Subroutine with the limit state function is
C declared external.
C
      External GDEFL
C
C The basic variables used are arranged in the following
C order:- D, B, tf, tw, E, pp, pc
C
      Dimension X(7), EX(7), SX(7), VP(10,7), COV(7,7),
     +          ZES(3), UU(7), EIVEC(7,7), IV(2,7)
C
      Character * 10 PRT
C
C Any deterministic parameter is set in this common  block
C e.g.no. of cables (c), the distant from the top of pole to any
C point alonng the length of the pole (a), the height of the pole
C (H), the diameter of the cables (diam), the spacing between
C the poles (Space), deflection coefficient due to load P(ALPHA)
C
      COMMON/CDEFL/c, a, amax, H, diam, Space
C
C The mean and standard deviation of the variables as
C well as the parameters of FORM5 are given in this
C data block.
C
      Data EX/1.2D2, 1.2D2, 5.D1, 5.D1, 4.D4, 1.133D-3, 1.01D-4/,
     +     SX/4.8D0, 4.8D0, 2.D0, 2.D0, 6.24D3, 4.2D-4,3.74D-5/,
     +     N/7/,NC/7/,NE/7/,IRHO/1/
C
         c = 3.D0
         H = 1.1D4
         diam = 1.37D1
         Space = 1.D5
C
      WRITE(*,*) 'Input the value of  a........>'
      READ(*,*)a
C
      IF(H . GT . 7.D3)THEN
        amax  =  2500.
      ELSE
        amax  =  1500.
      ENDIF
C
```

```
      X(1) = 250. * (1. + 4.D-1 * a/H)
      EX(1) = X(1)
      SX(1) = EX(1) * 4.D-2
      X(2) = 150.
      EX(2) = X(2)
      SX(2) = EX(2) * 4.D-2
C
C Adjustment is made for the value of the wind pressure on the pole
C depending on the height of the considered.
C
      IF(H . EQ . 7.D3)THEN
        X(6) =  5.2D-4 + 1.D-4 * 2.3 * (H-a)/H
        EX(6) = X(6)
        SX(6) = EX(6) * 3.7D-1
      ELSE
        X(6)  = 5.2D-4 + 1.D-4 * 3.8 * (H-a)/H
        EX(6) = X(6)
        SX(6) = EX(6) * 3.7D-1
      ENDIF
C
C The result are written in file No. 7
      NAUS = 7
C
C  Print to Screen
C
      ICRT = 0
      Open (7, File = 'DEFL.RES', STATUS = 'OLD', ERR = 10)
      Go to 20
 10   Open (7, File = 'DEFL.RES', STATUS = 'NEW')
C
C The variables VP, COV and IV are Preset using YINIT
C
 20   Call YINIT (N, IV, VP, IRHO, COV, NC)
      IV (1,6) = 7
      IV (1,7) = 7
      DO 100 I = 1, N
 100  X(I) = EX(I)
      VI = 1.D0
      BETA = 1.D0
C
      WRITE(NAUS,500)
 500  FORMAT(////,5X,70('*'),/,30X, 'FORM5',/,5X,70('*'),/,
     + 'SLS FOR DEFLECTION CONSTRAINTS, 7 VARIABLES:')
C
C The Stochastic model is printed using 'YKOPF'
C
      Call YKOPF(NAUS, N, IV, EX, SX, VP, IRHO)
C
C Print also to screen
C
      WRITE(ICRT,*) 'START OF FORM5'
      WRITE(ICRT,*) 'STOCHASTIC MODEL:'
```

```
      CALL YKOPF(ICRT, N, IV, EX, SX, VP, IRHO)
C
C The matrix COV is printed using 'YMAUS'
C
      CALL YMAUS(NAUS, NC, N, COV, PRT)
C
C The main Subroutine in FORM is called
C
      CALL FORM5(N, IV, EX, SX, VP,GDEFL, IRHO, COV, NC, EIVEC, NE,
     +          V1, NAUS, BETA, X, UU, ZES, IER)
C
C The Coordinate of the beta - point are printed with the vector
C titled UU
C
      PRT = '  UU  '
C
C
      CALL YFAUS(NAUS, N, UU, PRT)
C
C The vector ZES is printed with the  title vector ZES
C
      PRT = '  ZES  '
      CALL YFAUS(NAUS, 3, ZES, PRT)
      WRITE(ICRT,*) 'END OF FORM5'
      WRITE(ICRT,*) 'FOR RESULTS SEE FILE DEFL.RES'
      WRITE(ICRT,*) 'a.........=',a
      WRITE(ICRT,*) 'X(1).......=',X(1)
      WRITE(ICRT,*) 'X(2).......=',X(2)
      WRITE(ICRT,*) 'X(6).......=',X(6)
      STOP
      END
C
C
            SUBROUTINE GDEFL(N,X,FX,IER)
C
      Implicit Double Precision (A - H, O - Z)
      Dimension X(N)
      COMMON/CDEFL/c, a, amax, H, diam, Space
C
C Let AIX represents the moment of inertia  of the section across the
C x-x axis, P and W are resultant wind loads on cables and poles
C respectively.
C
C       P = X(7) * c * diam * Space/24.
       W = X(6) * X(2)
      IF(a . LE . amax)THEN
       AIX = X(2) * X(1)**3/12.
      ELSE
       AIX = (X(2)*X(1)**3 - (X(2) - X(4))*(X(1) - 2.*X(3))**3)/12.
      ENDIF
       AIE = AIX * X(5)
C        DELTAP = P * H**3/(3.* AIE)
```

APPENDIX 1D

PROGRAM STABIL

```
C
C This program evaluate the reliabilty of a Prestressed concrete
C transmission pole subjected to stability/buckling constraints.
c
       Implicit Double Precision (A - H, O - Z)
C
C The Subroutine with the limit state function is
C declared external.
C
      External GSTAB
C
C The basic variables used are arranged in the following
C order:- D, B, tf, tw, fpu, Aps, Lr, E, V
C
      Dimension X(9), EX(9), SX(9), VP(10,9), COV(9,9),
     +           ZES(3), UU(9), EIVEC(9,9), IV(2,9)
      Character * 10 PRT
C
C Any deterministic parameter is set in this common  block
C e.g.no. of steel tendons (s), the distance to any point
C along the length of the pole from the top end (a), the
C height of the pole from the graound level (H), a constant (PHI)
C
      COMMON/CSTAB/s, a, H, PHI, amax
C
C The mean and standard deviation of the variables as
C well as the parameters of FORM5 are given in this
C data block.
C
      Data EX/1.2D2, 1.2D2, 5.D1, 5.D1, 1.57D3, 1.96D1, 7.D-1,
     +     4.D4, 2.D4/,
     +     SX/4.8D0, 4.8D0, 2.D0, 2.D0, 2.355D2, 7.056D-1, 3.5D-2,
     +     6.24D3, 2.25D3/,
     +     N/9/, NC/9/, NE/9/, IRHO/1/
C
         s = 2.D1
          H = 1.1D4
            PHI = 3.142D0
C
      Write(*,*) 'Enter the Value of   a .......> '
      Read(*,*) a
C
      IF(H . GT . 7.D3)THEN
        amax  =  2500.
      ELSE
        amax  =  1500.
      ENDIF
C
       X(1) = 150. * (1. + 1.33 * a/H)
```

```
      EX(1) = X(1)
      SX(1) = EX(1) * 4.D-2
      X(2)  = 150.
      Ex(2)  =  X(2)
      Sx(2)   =  Ex(2) * 4.D-2
      X(9)  =  2.D4
      EX(9)  = X(9)
      SX(9)  = EX(9) * 1.5D-1
C
C Adjustment is made for the number of prestressing steel
C at the top portion of the pole.
C
      IF(a . LE . amax)s  =  1.6D1
C
C
C The result are written in file No. 7
C
      NAUS = 7
C
C  Print to Screen
C
      ICRT = 0
      Open (7, File = 'STAB.RES', STATUS = 'OLD', ERR = 10)
      Go to 20
 10   Open (7, File = 'STAB.RES', STATUS = 'NEW')
C
C The variables VP, COV and IV are Preset using YINIT
C
 20   Call YINIT (N, IV, VP, IRHO, COV, NC)
      IV (1,5) = 3
      DO 100 I = 1, N
 100  X(I) = EX(I)
      VI = 1.D0
      BETA = 1.D0
C
      WRITE(NAUS,500)
 500  FORMAT(////,5X,70('*'),/,30X, 'FORM5',/,5X,70('*'),/,
     + 'SLS FOR STABILITY BUCKLING CONSTRAINTS, 9 VARIABLES:')
c
C The Stochastic model is printed using 'YKOPF'
C
      Call YKOPF(NAUS, N, IV, EX, SX, VP, IRHO)
C
C Print also to screen
C
      WRITE(ICRT,*) 'START OF FORM5'
      WRITE(ICRT,*) 'STOCHASTIC MODEL:'
      CALL YKOPF(ICRT, N, IV, EX, SX, VP, IRHO)
C
C The matrix COV is printed using 'YMAUS'
C
      CALL YMAUS(NAUS, NC, N, COV, PRT)
```

APPENDIX IIA
COMPRESSIVE WORKING STRESS CONDITION
PROBABILITY ANALYSIS FOR PRESTRESSED ELECTRIC TRANSMISSION CONCRETE POLE
FORM5 RESULT SHEET

SECTION NO......		SPECIFICATION DATA: AMT = 0 H = 9.0m. Space = 100m			SPECIFICATION DATA: AMT = 7.5KNm H = 9.0m. Space = 100m			SPECIFICATION DATA: AMT = 15KNm H = 9.0M. Space = 100m		
		Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (Pf)	Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)
LOAD EFFECT	$\frac{wa^2}{2} + p_a$	0.50	2.963	0.152E-2	0.50	0.172	0.432	0.50	-2.152	0.984
		1.00	3.200	0.688E-3	1.00	1.007	0.157	1.00	-0.943	0.827
		1.50	3.383	0.359E-3	1.50	1.653	0.492E-1	1.50	0.037	0.485
		2.00	3.526	0.211E-3	2.00	2.151	0.157E-1	2.00	0.824	0.205
		2.50	3.639	0.137E-3	2.50	2.537	0.559E-2	2.50	1.450	0.735E-1
		3.00	3.190	0.711E-3	3.00	2.298	0.108E-1	3.00	1.410	0.793E-1
SERVICEABILITY LIMIT STATE CONSIDERED	COMPRESSIVE STRESS WORKING CONDITION	4.00	3.317	0.455E-3	4.00	2.701	0.346E-2	4.00	2.083	0.186E-1
		5.00	3.410	0.325E-3	5.00	2.970	0.149E-2	5.00	2.527	0.575E-2
		6.00	3.483	0.248E-3	6.00	3.158	0.793E-3	6.00	2.832	0.231E-2
		7.00	3.543	0.198E-3	7.00	3.298	0.487E-3	7.00	3.051	0.114E-2
		8.00	3.597	0.161E-3	8.00	3.407	0.328E-3	8.00	3.217	0.649E-3
		9.00	3.648	0.132E-3	9.00	3.499	0.234E-3	9.00	3.348	0.407E-3

APPENDIX IIB
TENSILE WORKING STRESS CONDITION
PROBABILITY ANALYSIS FOR PRESTRESSED ELECTRIC TRANSMISSION CONCRETE POLE
FORM5 RESULT SHEET

SECTION NO......		SPECIFICATION DATA: AMT = 0 H = 9.0m. Space = 100m			SPECIFICATION DATA: AMT = 15 KNm H = 9.0m. Space = 100m			SPECIFICATION DATA: AMT = 30KNm H = 9.0M. Space = 100m		
		Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)
LOAD EFFECT		0.50	16.25+	0.113E-58	0.50	0.752	0.226	0.50	-3.177	0.999
		1.00	10.97	0.278E-27	1.00	1.278	0.101	1.00	-2.579	0.995
		1.50	8.24	0.875E-16	1.50	1.661	0.484E-1	1.50	-2.070	0.981
		2.00	6.51	0.390E-10	2.00	1.878	0.302E-1	2.00	-1.650	0.950
		2.50	5.28	0.639E-7	2.50	1.914	0.278E-1	2.50	-1.315	0.906
		3.00	6.13	0.432E-9	3.00	4.350	0.682E-5	3.00	+2.022	0.216E-1
SERVICEABILITY LIMIT STATE CONSIDERED	TENSILE STRESS WORKING CONDITION	4.00	4.79	0.847E-6	4.00	3.689	0.113E-3	4.00	2.263	0.118E-1
		5.00	3.86	0.569E-4	5.00	3.092	0.993E-3	5.00	2.148	0.158E-1
		6.00	3.17	0.753E-3	6.00	2.596	0.471E-2	6.00	1.913	0.279E-1
		7.00	2.65	0.409E-2	7.00	2.187	0.114E-1	7.00	1.663	0.482E-1
		8.00	2.23	0.130E-1	8.00	1.852	0.320E-1	8.00	1.432	0.761E-1
		9.00	1.90	0.290E-1	9.00	1.580	0.570E-1	9.00	1.233	0.109

APPENDIX IIC
DEFLECTION REQUIREMENT
PROBABILITY ANALYSIS FOR PRESTRESSED ELECTRIC TRANSMISSION CONCRETE POLE
FORM5 RESULT SHEET

		SPECIFICATION DATA: H = 7.0m, Space = 100m			SPECIFICATION DATA: H = 9.0m, Space = 100m			SPECIFICATION DATA: H = 11.0M, Space = 100m		
		Distance along length of pole (x) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (x) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (x) m	Reliability Index (β)	Probability of failure (P_f)
SECTION NO......		0.50	0.769	0.221	0.50	2.318	0.990	0.50	-4.000	1.000
LOAD EFFECT		1.00	1.450	0.735E-1	1.00	-1.770	0.962	1.00	-3.676	1.000
		1.50	2.070	0.192E-1	1.50	-1.236	0.892	1.50	-3.334	1.000
		2.00	2.404	0.811E-2	2.00	-0.722	0.765	2.00	-2.980	0.999
		2.50	2.887	0.194E-2	2.50	0.229	0.591	2.50	-2.617	0.996
		3.00	3.337	0.435E-3	3.00	0.076	0.530	3.00	-2.509	0.994
SERVICEABILITY LIMIT STATE CONSIDERED	DEFLECTION REQUIREMENT	4.00	4.151	0.166E-4	4.00	0.711	0.238	4.00	-1.870	0.969
		5.00	4.849	0.671E-6	5.00	1.424	0.773E-1	5.00	-1.249	0.894
		6.00	21.7	+∞	6.00	2.078	0.188E-1	6.00	-0.653	0.743
		7.00	29.1	+∞	7.00	2.692	0.356E-2	7.00	-0.086	0.534
					8.00	3.276	0.527E-3	8.00	0.451	0.326
					9.00	3.837	0.624E-4	9.00	0.962	0.168
								10.00	1.450	0.736E-1
								11.00	1.920	0.274E-1

APPENDIX IID

STABILITY CRITERIA

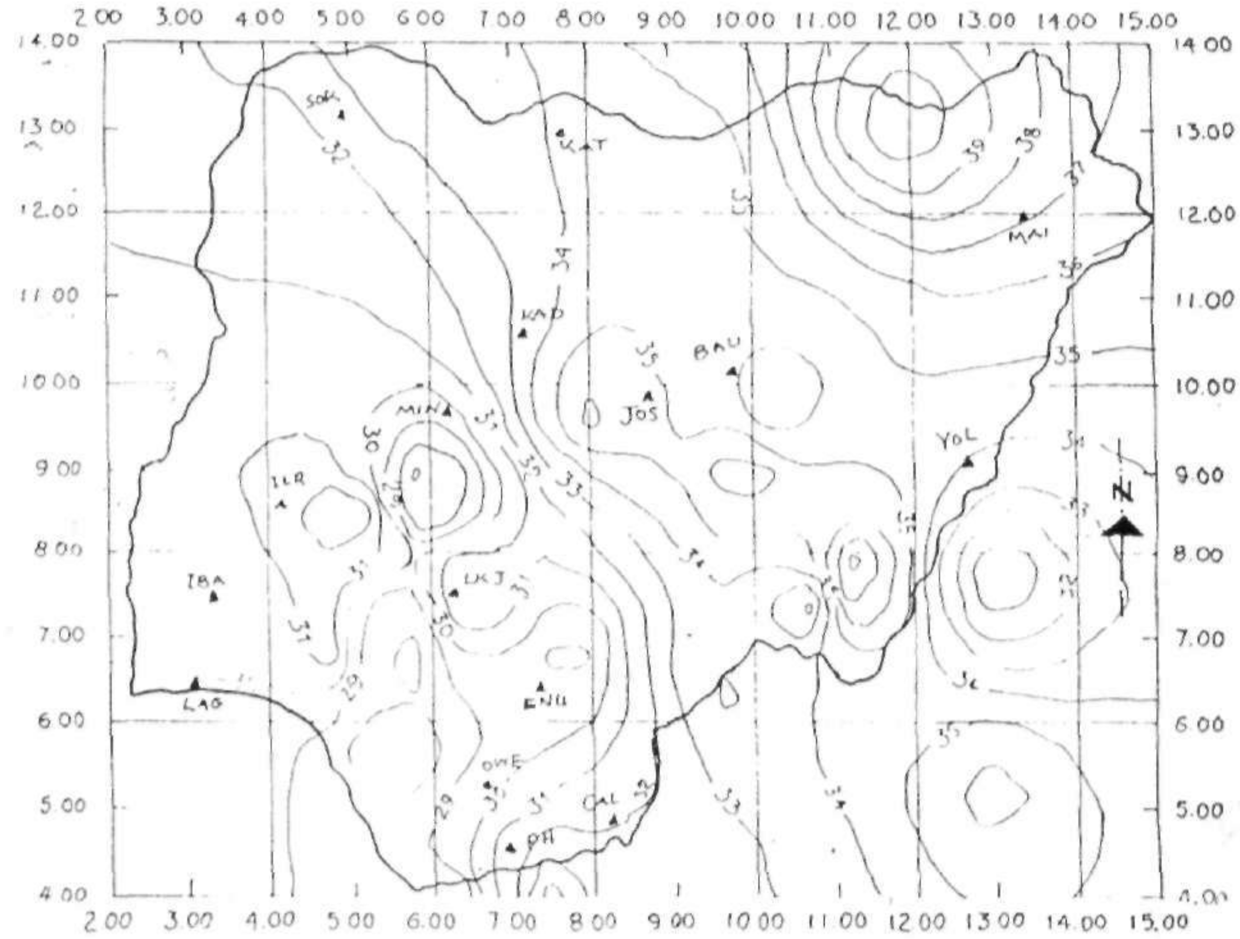
PROBABILITY ANALYSIS FOR PRESTRESSED ELECTRIC TRANSMISSION CONCRETE POLE

FORM5 RESULT SHEET

		SPECIFICATION DATA: H = 9.0m, V = 10KN			SPECIFICATION DATA: H = 9.0m, V = 15KN			SPECIFICATION DATA: H = 9.0M, V = 20KN		
SECTION NO......		Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)	Distance along length of pole (a) m	Reliability Index (β)	Probability of failure (P_f)
LOAD EFFECT		0.50	0.825	0.205	0.50	0.663	0.254	0.50	0.503	0.307
		1.00	1.272	0.102	1.00	1.120	0.131	1.00	0.968	0.166
		1.50	1.690	0.455E-1	1.50	1.546	0.610E-1	1.50	1.403	0.803E-1
		2.00	2.080	0.187E-1	2.00	1.945	0.259E-1	2.00	1.809	0.352E-1
		2.50	2.445	0.725E-2	2.50	2.316	0.103E-1	2.50	2.188	0.143E-1
		3.00	1.554	0.601E-1	3.00	1.433	0.759E-1	3.00	1.313	0.946E-1
SERVICEABILITY LIMIT STATE CONSIDERED	STABILITY CRITERIA	4.00	2.179	0.147E-1	4.00	2.070	0.192E-1	4.00	1.960	0.250E-1
		5.00	2.731	0.315E-2	5.00	2.632	0.425E-2	5.00	2.532	0.568E-2
		6.00	3.215	0.652E-3	6.00	3.124	0.891E-3	6.00	3.033	0.121E-2
		7.00	3.636	0.139E-3	7.00	3.553	0.190E-3	7.00	3.470	0.260E-3
		8.00	3.998	0.319E-4	8.00	3.923	0.437E-4	8.00	3.848	0.595E-4
		9.00	4.307	0.827E-5	9.00	4.240	0.112E-4	9.00	4.172	0.151E-4

APPENDIX III

Isopleths of the Improved Basic Wind Speed for a 50-year Return period in Nigeria.



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