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A PARAMETRIC STUDY FOR THE OPTIMUM DESIGN OF GUYED TOWERS

by

Subba Rao Venkata Majety

A Thesis
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of the Requirements for the degree of
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ABSTRACT

A parametric study of a guyed tower is presented. A computer program is developed to analyze, design, and estimate the mass and the total cost of a guyed tower. Matrix stiffness method is used for the static analysis of the mast. The non-linearity due to axial loads is accounted for by using the geometric stiffness matrix. The cable analysis including the change in sag of a guy cable due to change in tension is presented. An iterative procedure is adopted to satisfy compatibility conditions of guy attachment points. Four loading conditions are considered in the design.

Structural steel angles are considered for the design of members of the mast. Guy strands and bridge strands are considered for the guy cables and deadman anchorages are adopted for guy anchorages.

Variation in the mass and the total cost of a guyed tower due to the variation in (a) inclination of diagonal web members, (b) radial distance of guy anchor points, (c) height of guy connection points, and (d) size of the mast, are studied. An attempt is made to find an optimum design of a guyed tower.

DEDICATED TO

My Grandmothers

Kanakamma Majeti and Seethamma Nandula

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NOMENCLATURE

Notation	Meaning
A	Cross sectional area of member
A_c	Cross sectional area of guy cable
A_f	Face area of bare flat members for wind load calculations
A_g	Gross area of one face of the structure, i.e., the panel height multiplied by the face width of one panel.
A_i	Face area of radial ice
A_m	Cross sectional area of mast.
A_r	Face area of bare round members
A_s	Net projected area of members in one face of the structure, i.e., for one panel, the projected area of all members within that panel, including ice thickness and attachments.
A_{ne}	Net cross sectional area.
A'_{ne}	Modified cross sectional area with shear-lag effect.
a	Horizontal projection of cable chord
b	Vertical projection of cable chord
b_f	Breadth of guy anchorage block
C_a	Acceleration factor for wind load calculations
C_d	Drag factor for wind load calculations
C_{df}	Drag factor for flat members
C_{dr}	Drag factor for round members
C_e	Height factor for wind load calculations
C_g	Gust factor for wind load calculations
C_i	Compressive force in i^{th} leg member due to axial force and bending moments on the mast
C_r	Compressive resistance
D	Dead load
d	Size of the mast

Notation	Meaning
d_f	Depth of guy anchorage block
E	Modulus of Elasticity for mast members
E_c	Modulus of Elasticity for guy cables
F	Force in the members for square mast
$[F]$	Vector of element loads
F_i	Force in the members for triangular mast
F_x	Force acting on the mast in X-direction
F_y	Force acting on the mast in Y-direction
F_z	Force acting on the mast in Z-direction
H	Horizontal component of tension in a guy cable
H_o	Horizontal component of initial tension in a guy cable
H_x	Height of the structure for wind load calculations
H_y	Horizontal force in a cable in Y-direction
H_z	Horizontal force in a cable in Z-direction
h	height of soil above anchorage block
$h_{\min}$	Minimum height of soil required above guy anchorage block
I	Ice load
K	Effective length factor for compression member
$[K]$	Master stiffness matrix
K_d	Factor used to modify drag factor when wind load acts on the diagonal for towers with square mast.
K_1 and K_2	Factors used to determine K_d
$[K_e]$	Elastic stiffness matrix for an element
$[K_g]$	Geometric stiffness matrix for an element
K_y	Translational spring constant for all guy cables at one level in Y-direction

Notation	Meaning
K_z	Translational spring constant for all guy cables at one level in Z-direction
k_p	Passive earth pressure coefficient
L	Length of a member
l_f	Length of anchorage block
l, m, n	Direction cosines
M_x	Twisting moment acting on the mast section
M_y	Bending moment acting on the mast about Y-axis
M_z	Bending moment acting on the mast about Z-axis
P	Design wind pressure
$[P]$	Load vector
p	Uniformly distributed vertical load on guy cable per unit length
P_h	Horizontal component of force acting on guy anchorage block
p_o	Uniformly distributed initial vertical load on guy cable per unit length
P_p	Passive resistance from soil for guy anchorage block
P_v	Vertical component of force acting on guy anchorage block
p_1	Passive earth pressure at the top of guy anchorage block
p_2	Passive earth pressure at the bottom of guy anchorage block
q	Reference wind velocity pressure for wind load calculations
R	Wind load acting on the mast section in any arbitrary direction
$[R]$	Rotation matrix
R_s	Solidity ratio
R_y	Component of R in Y-direction
R_z	Component of R in Z-direction
r	Radius of gyration of member

Notation	Meaning
T	Tension in a guy cable
$[T]$	Transformation matrix
T_o	Initial tension in a guy cable
T_i	Tensile force in the i^{th} member
T_r	Tensile resistance in a member or guy cable
t	Resultant deflection of the mast
$[U]$	Displacement vector
U_p, V_p, W_p	Displacements of an element in X, Y, and Z directions at P-end
U_q, V_q, W_q	Displacements of an element in X, Y, and Z directions at Q-end
V	Vertical component of tension in a guy cable
W_b	Weight of guy anchorage block
W_o	Wind load without ice
W_i	Wind load with ice
W_s	Weight of soil above guy anchorage block
W_t	Total Weight $= W_b + W_s$
β_p, θ_p, λ_p	Element rotations about X, Y, and Z- directions respectively at P-end
β_q, θ_q, λ_q	Element rotations about X, Y, and Z- directions respectively at Q-end
γ_c	Density of concrete
γ_s	Density of soil
Δ_{ai}	Horizontal displacement in i^{th} cable at a level
ΔH_i	Change in horizontal force in i^{th} cable at a level
Δ_y	Mast deflection in Y-direction
Δ_z	Mast deflection in Z-direction
$[\delta]$	Element displacement vector
λ	Non-dimensional slenderness parameter

Notation	Meaning
θ	Inclination of diagonal web members of mast with horizontal
ϕ	Wind direction with respect to Y-axis
ψ	Direction of mast deflection with respect to Z-axis
σ_y	Yield strength of steel

CHAPTER I

INTRODUCTION

1.1 General

The demand for tall communication towers has increased in the recent past due to new developments in the telecommunications area. Self supporting towers prove to be satisfactory at low to medium (up to 60 metres) heights but, for tall towers (height more than 60 metres), guyed towers are the only alternative. Even for medium heights, guyed towers are sometimes preferred to self supporting towers based on economic considerations even though guyed towers require more land area. The increased demand for guyed towers necessitates the search for an optimal design criteria for given locations of antennas and load conditions.

A guyed tower consists of a mast, mostly of uniform cross section (circular, triangular or square), supported laterally by guy cables at different levels. Towers with circular masts are usually constructed with steel plate, and for masts with triangular and square cross sections structural steel angles are used. Depending on the height and loading conditions, solid steel rods may be used for mast legs.

The guys provide lateral support to the structure and are fastened to the anchorages at ground level. Zinc-coated steel wire strand is used for guys. Metric sizes are specified in CAN3-G12-M78, "Zinc-coated Steel Wire Strand" [5] but they are not yet readily available. Therefore, it is the common industry practice to specify the sizes in inch-units as given in the Wire Rope Industries Catalogue [35]. A seven wire galvanized guy strand is regularly manufactured in sizes from 3/16 in. through 5/8 in. in diameter. Galvanized guy strands are available in four different grades: Grade-50, Grade-110, Grade-160, and Grade-180. Grade-160 is not commonly available in the full range of sizes. Of these, only Grade-180 is readily available and is commonly specified in designs. For most installations outside of the utilities field, Grade-180 is satisfactory throughout the full size range. Bridge strands can be specified when a higher strength (greater than Grade-180) is required, or where pre-stretching is desirable. For the present study, guy strands of Grade-180 are used for cables of diameter 3/16 in. through 7/16 in. For guy cables of diameter greater than 7/16 in. galvanized bridge strands are used.

The compatibility equations of displacements at guy attachment points are very important in the guyed tower analysis. Since the guy cable is a flexible member, a direct, closed form solution for the structural analysis is not feasible. Consequently, an iterative procedure is adopted in the analysis to achieve compatibility conditions. The primary loads on the tower are the self-weight, ice, and wind loads. The ice and wind loads are taken as per *CAN/CSA-S37-M86*[7], "Antennas, Towers, and Antenna Supporting Structures."

Designing an optimum guyed tower is a difficult task because of the highly complex structural behaviour. The choice of design variables often influences the optimum design. For given loading conditions, previous experiments to design optimal towers focussed on member sizes and section properties. However, from a practical perspective, an optimal design has to consider other parameters, e.g., inclination of web members, and radial distance to guy anchor points, and cost of foundation for guy anchors.

1.2 Objectives

A survey of the literature from the year 1950 to 1990 revealed hundreds of papers on guyed towers. However, only two of these papers cover optimal guyed tower designs.

Bell[3] used a quadratic interpolation approach to minimize cable tensions but did not include geometric properties and cost of anchorage. *Greene*[18] studied the use of tubular steel and aluminum sections for members of triangular mast. His designs also did not include geometric parameters and the cost of tower. A generalization of these approaches to model total cost of the guyed tower which includes cost of anchorage was not possible.

Various optimization packages have been developed in the recent past in the area of optimal structural analysis and design. A survey of optimization models for generic structural analysis problems was undertaken to develop a strategy for optimal guyed tower design. A summary of key findings is presented here.

1.2.1 Models for Optimal Structural Analysis and Design

An Automated STructural Optimization System (ASTROS) and a STructural Analysis and Redesign System (STARS) are presented by *Duane, et al.*[13]. The STARS format is defined in three distinct parts: command procedure, analysis technique, and optimization technique. ASTROS is also defined in the same three-part procedures. Both ASTROS and STARS use a command procedure to govern the stuctural analysis and design stages. ASTROS uses a structured software that allows the user to modify the analysis and design routines whereas STARS command procedure is defined completely by the user. ASTROS requires an interface to NASTRAN library routines for structural analysis and design. STARS allows the user to adopt either NASTRAN or another analyzer developed by Royal Aircraft Establishment in England. Both ASTROS and STARS, unfortunately, do not use any other analysis program.

A general purpose design optimization package called IDESIGN was adopted by *David and Arora*[9]. Four principal subroutines are required to use IDESIGN. They are: USERMF, USERCF, USERMG, and USERCG. They return the cost function, constraint functions, cost function gradient, and constraint function gradients respectively at each iteration. These subroutines are developed using a modern data base management system called MIDAS/GR. An interactive pre-processing module was also developed to simplify data entry, model generation, and design variable initialization.

A non-linear programming based optimizer, CONMIN, was developed by *Vanderplaats*[33]. CONMIN is based on a feasible directions algorithm. Each time CONMIN makes a change to the set of design variables and requests constraint information, CONMIN also requires derivatives of the constraints with respect to the design variables.

A Modular In-core Nonlinear Optimization System *(MINOS)*[26], based on a generalized reduced gradient algorithm, was designed to solve large problems with non-linear objective functions and linear and/or non-linear constraints.

1.2.2 Study Objectives

Based on the study of optimization models for generic structural analysis, it was determined that developing a model for optimal guyed tower design is beyond the scope of the present study. As a first step, a parametric study was undertaken to identify the effect of several key parameters on the total cost of a guyed tower. The study involved two guy towers of height 100 m and 350 m with a triangular mast. The experiments were conducted in two stages: in the first stage, the optimum number of guy levels was determined keeping other parameters constant, and in the second stage, an optimal design was attempted by varying the other parameters.

The study objectives are:

1. to develop a computer program to analyze, design, and estimate the mass and cost of a guyed tower,
2. to study the variation of mass and cost of a guyed tower with respect to various geometric parameters for different loading conditions, and
3. to study the feasibility of finding an optimum tower by considering a sample case of tower of 100 m height with two guy levels.

1.2.3 Optimizer MINOS

The present study adopted MINOS as the optimizer for the guyed tower analysis and design. For a given height and the number of guy levels, an iterative approach was used to finding an optimal design. In each iteration, key design parameters are passed to MINOS as decision variables, and the cost of guyed tower is estimated as the value of the objective function.

The MINOS optimizer requires two subroutines, viz., CALCFG and CALCON. Subroutine CALCFG is provided to calculate the objective function value. Computing the gradient of objective function is optional in the CALCFG routine. This subroutine uses a finite difference approximation to compute gradients. Subroutine CALCON is provided to compute non-linear constraint functions, if any. Subroutine CALCON is optional; if all constraints are linear, then CALCON is not required. The data for the optimization problem is provided through a number of input files. The order of input data is as follows:

— SPECS file: for the problem specification

— MPS file: to contain linear objective and constraint data in a format compatible with the IBM MPS structure, as well as bounds on variables, at the initial point.

— BASIS file (Optional): to provide an initial basic feasible solution, if available.

The computer program described in Chapter VI is supplied to MINOS as CALCFG, i.e., objective function.

1.2.4 Problem Formulation

A guyed tower of 100 m height with two guy levels (Figure 1.2) is taken as an example problem. The optimization problem is formulated as given below:

Objective Function = Cost of guyed tower

The cost of guyed tower is expressed as

(cost of mast + cost of guys + cost of anchorages)

where

Cost of mast = Total mass of 90^o angle members x Unit price + Total mass of 60^o angle members x Unit price

Cost of guys = Total mass of guys x Unit price + Number of connections at ends of guy cables x Unit price

Cost of anchorages = Mass of concrete x Unit price + Volume of earthwork excavation x Unit price

The following unit prices were obtained from one of the tower fabricators to estimate the cost of the structure.

Unit price for normal angles = \$ 337.23 /kN

Unit price for 60^o angles = \$ 449.64 /kN

Unit price for cable wires = \$ 674.46 /kN

Unit price for cable connections = \$ 600.00 (at each end)

Unit price for concrete block = \$ 392.40 $/m^3$

Unit price for earthwork excavation = \$ 65.40 $/m^3$

Design Variables:

x_1 = web inclination with respect to horizontal

x_2 = radial distance of guy anchor point for cable 1

x_3 = radial distance of guy anchor point for cable 2

x_4 = height of guy connection point for cable 1

x_5 = height of guy connection point for cable 2.

Size of the mast taken as 1.5 m from parametric study.

Constraints: Design constraints and compatibility constraints are built into the analysis, design, and CALCFG routines. Geometric constraints are listed below:

$$x_3 - x_2 \geq 0$$

$$x_5 - x_4 \geq 0$$

The output from MINOS is given in the floppy diskette enclosed. The results are discussed in Chapter VIII.

CHAPTER II

LITERATURE SURVEY ON STRUCTURAL OPTIMIZATION

2.1 General

A non-linear programming problem can be described as minimization or maximization of an objective function controlled directly or indirectly by a set of design variables, subjected to a set of equality and/or inequality constraints. It is commonly expressed as

$$Min\,/\,Max \quad F = f(x)$$

$$Subject\ to \quad g_i(x) \geq 0, \quad i = 1, 2, \ldots\ldots, m$$

$$Where \quad x = (x_1, x_2, \ldots\ldots, x_N)$$

"N" is called the dimension of the problem or design space. It is not possible to solve an optimization problem over different topologies. Hence the number of guy connections should be known for the formulation of an optimization problem.

2.1.1 Design Variables

The design variables may consist of member sizes (representing the cross sectional area, moment of inertia, thickness, etc.), geometric properties, and material parameters. In the present study, the computer program adopts a sequential search process to select a minimum mass/cost section for all members of the structure. The search is done on the member data supplied as input to the program. The analysis, design, and optimization process is described in Chapters III to V, and VIII.

2.1.2 Objective Function

The objective function constitutes a basis for the selection of one of several alternative acceptable designs. In the present study cost of the structure is adopted as the objective function.

2.1.3 Constraints

A constraint, in any structural optimization problem, is a restriction that must be satisfied for the design to be acceptable. Constraints can be classified as:

1. Design constraints: that define the relationship between actual forces in the members and member resistances.

2. Compatibility constraints: that define the compatibility equations, e.g., relationship between the deflection in the mast at guy connection points and elongation in the respective guys.

3. Geometric constraints: that define the relationship among various geometric parameters so that a distorted or infeasible geometry is not suggested by the optimal solution.

In the present study both design constraints and compatibility constraints are built into the structural analysis and design routines. These constraints are satisfied for every feasible design and, hence, are not included in the constraint set separately.

2.2 Models for Optimal Structural Analysis and Design

Mathematical programming was first applied to structural analysis problems in the 1960s to attempt optimum structural designs. The development of structural synthesis is presented in detail by *Schmit*[30]. Four major areas were identified in the development of optimum structural design technology. [14] They are (i) theory of layout, (ii) simultaneous mode of failure, (iii) optimality criteria, and (iv) mathematical modelling formulations.

Michell[25] presents an overview of the significance of theory of layout. However, these theorems, if applied without meaningful constraints on the geometric form of the structure, yield impractical solutions.

The simultaneous mode of failure approach assumes that optimality is achieved when each element of the complete structure is at its limit state. The term "simultaneous" means a single loading condition and this restriction governs most of the work during the 1950s. *Shanley*[30], *Gerard*[16], and *Cox*[4] present an overview of these ideas. The fact that there exist only a small number of simple solutions, together with limited applicability to practical design has resulted in very little new work in this area.

Prager and Marcal[28] and *Taylor*[32] have been instrumental in the development of a "criterion of optimality" approach to optimal structural designs. This approach derives from the extremum principles of structural mechanics and, for the most part, has been limited to simple structural forms and loading conditions. The procedures of *Venkayya*[34] and *Gellantly and Berke*[17] are also significant in the development of this approach.

The mathematical programming approach to optimal structural design is based on finding the minimum or maximum of a function of many variables subject to certain limitations. The limitations, expressed in the form of a constraint set, can be expressed as equalities or inequalities. The inequality constraints permit the design to be identified as one in which not all members are subject to limiting conditions under specified loads. Most of the techniques developed in mathematical programming require a clear expression for the objective function in terms of design variables. For most structural optimization problems no explicit expression for objective function is available.

2.3 Recent Applications

Gupta et al.[19] presented a geometric programming approach to an optimal bridge design. They presented an application of generalized geometric programming to the optimal design of prestressed concrete pedestrian bridge deck. The constraints are related to bending and shear stresses and minimum concrete cover.

Koyama and Kamiya[23] presented an application of fuzzy linear and non-linear programming to structural optimization. In his presentation, he discussed the multiobjective optimization approach. He considers two types of problems: one is non-fuzzy decision under fuzzy constraints, and another fuzzy decision under the same conditions.

Duane et al.[13] presented two structural optimization programs: ASTROS (Automated STRuctural Optimization System) and STARS (STructural Analysis and Redesign System). Both ASTROS and STARS are based on finite element analysis methods.

A two-level optimization of non-linear structural design problems is presented by *Ichiro et al.*[20] In their paper, they presented several optimization design problems and their applications to geometrical non-linear braced rib-arches and pin-jointed trusses.

A total optimum design method is developed for truss structures by *Sadaji et al.*[29] In this method, the shape of the structure, discrete material kinds, and cross sectional areas of member elements are optimized simultaneously. The primal design problem is formulated in terms of the shape, material, and size design variables, and approximated to a sequence of conservative, convex, and separable subproblems by using mixed, direct/reciprocal design variables. Each subproblem is solved by the dual method in which a two-stage minimization process is developed to optimize continuous as well as discrete variables by using Newtonian-type algorithm and discrete sensitivity analysis.

Balling and Fonseca[2] reported that a general discrete optimization procedure can be applied to the design of a realistic, three-dimensional steel building frame. *Ali*[1] used the approximate stiffness method for a procedure to evaluate the stiffness of various frames and the stiffness of the members within the frames to provide a drift-controlled building design with a minimum amount of steel.

David and Arora[9] formulated a non-linear optimization problem for the analysis and design of reinforced concrete framed tube buildings, and solved the problem using modern design optimization software. They used a design optimization called IDESIGN.

Dan and Merek[8] presented basic ideas of vector optimization of structural systems. In their study, both the weighing and constraint methods are applied to vector optimization of truss systems by minimizing the weight and displacements simultaneously.

Ju-ang and Cheng[22] present theoretical derivations and numerical procedures for a program called ODSEWS-2D-II for the analysis and optimum design of two-dimensional regular and irregular steel frames subjected to static, earthquake, and wind forces.

2.4 Optimum Guyed Towers

Optimization techniques are applied successfully to various structures as seen in Section 2.3. However, attempts to optimize guyed towers have been very few. Among hundreds of papers on guyed towers covering the period of 1950 to 1990, the author is able to identify only two attempts at finding optimum guyed towers. The first attempt was made by *Bell*[3]. Bell used a quadratic interpolation approach to minimize cable initial tensions. He adopted branch and bound technique to find cable areas and tower areas for the minimum cost. He did not include geometric parameters in his study. Also, his estimation of cost does not include the cost of anchorages which has a significant effect on the optimal design. Bell considered planar structures in which torsional effects could not be handled, which is also a drawback with the class of structures considered.

Greene[18] used thin tubular steel/aluminum sections for members of triangular mast. His design variables consist of outer and inner radii of leg and web members of mast, spacing of panels, cross sectional areas of the guys at each guy level and a variable defining the amount of pre-tension in the guys. He assumed that all the guy cable chords are at an inclination of 45^o. Greene did not study the effect of geometric parameters on the weight and cost of the structure. Also, he did not include design of anchorages in his study.

Marshall[24] carried out an extensive study of deflections and tensile forces of members of guyed masts subjected to direct torsional loads. He studied the effect of changes in the independent variables such as, the height of the mast, initial tensions, direction of the load, etc. His studies include experimental work involving prestressing of guys and measurement of deflections. However, his study objectives did not include the optimum design of a guyed tower.

CHAPTER III

THEORETICAL FORMULATION:

CABLE ANALYSIS

3.1 General

A systematic approach to guy cable analysis was first presented by *Dean*[10]. Dean developed exact theoretical equations for the behaviour of guy cables for static and dynamic conditions. *Selvappalam*[27] adopted a different approach. In his study, Selvappalam approximated the cable element as truss element with length equal to the chord length of the cable. The effect of sag has been incorporated by using an equivalent modulus of elasticity. *Ezra*[14] presented a detailed analysis of high guyed towers. He adopted Dean's approach for cable analysis and assumed that the cable geometry was a parabola.

In the present study, Dean's approach is adopted for the analysis of guy cable. General guy cable geometry for a uniformly distributed load on the guy is developed as given in Section 3.2. When the loads are applied, the mast deflects and it results in a change in cable tensions. In Section 3.3 the relationship between change in length of guy cable and tension in guy cable is derived. The change in the guy cable tension causes a change in horizontal displacement of mast due to change in horizontal component of tension. Expressions for horizontal displacement at guy cable attachment points, in terms of horizontal component of tension at that level, are presented in Section 3.4.

Finally, equilibrium of external loads and guy cable tensions at guy cable attachment points must be ensured. The compatibility of displacements at attachment points for windward and leeward guy cables must be ensured such that all cables at a level are displaced as a unit along with the mast. In Section 3.5 equilibrium and compatibility at guy cable attachment point are discussed. Equivalent spring constants for guy cables are estimated (as shown in Section 3.6) and are used in forming the master stiffness matrix. The details of master stiffness matrix are explained in Chapter IV.

3.2 Formulation of Guy Cable Geometry

The geometry of a cable is as shown in Figure 3.1. Only vertical loads are shown in Figure 3.1 based on the assumption that the wind load on the guy is applied to the top and bottom guy attachment points.

Equilibrium of all forces in the vertical direction gives

$$V \;-\; (\,V - \Delta V\,) \;-\; p\;ds \;=\; 0$$

$$==> \quad \Delta V \;=\; p\;ds \qquad \text{(a)}$$

From the cable geometry,

$$\frac{V}{H} \;=\; \frac{dy}{dx}$$

$$==> \quad V \;=\; H\,\frac{dy}{dx}$$

$$==> \quad \Delta V \;=\; H\,\frac{d^2y}{dx^2}\,dx \qquad \text{(b)}$$

Equating (a) and (b) gives

$$\frac{d^2y}{dx^2} \;=\; \frac{p}{H} \times \frac{ds}{dx} \qquad (3.1)$$

Equation (3.1) is the basic equation of guy cable geometry. From elementary calculus it can be shown that

$$\frac{ds}{dx} \;=\; \sqrt{1 \;+\; \left(\frac{dy}{dx}\right)^2} \qquad \text{(c)}$$

Substituting (c) in Equation (3.1) gives

$$\frac{d^2y}{dx^2} = \frac{p}{H}\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{\frac{1}{2}}$$

Solving and simplifying this equation gives

$$y = \frac{2H}{p}\sinh\left(\frac{px}{2H} + c_1\right)\sinh\left(\frac{px}{2H}\right) \tag{3.2a}$$

where

$$c_1 = \sinh^{-1}\left[\frac{pb}{2H\sinh\left(\frac{pa}{2H}\right)}\right] - \frac{pa}{2H} \tag{3.2b}$$

The arc length of guy cable is derived from

$$L = \int_{curve} ds = \int_0^a \frac{ds}{dx}\,dx$$

$$= \left[\frac{4H^2}{p^2}\sinh^2\left(\frac{pa}{2H}\right) + b^2\right]^{\frac{1}{2}} \tag{3.3}$$

where

a = horizontal projection of guy cable

b = vertical projection of guy cable

3.3 Relationship Between Change in Length and Guy Cable Tensions

Tension in a guy cable at a point (x,y) is given by

$$T = H\left[\frac{ds}{dx}\right]$$

$$= H\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{\frac{1}{2}}$$

$$= H \cosh\left(\frac{px}{H} + c_1\right) \tag{3.4a}$$

and initial tension in guy cable is given by

$$T_o = H_o \cosh\left(\frac{p_o\, x}{H_o} + c_2\right) \tag{3.4b}$$

where

H_o = Horizontal component of initial tension,

p_o = initial vertical loading on the guy cable per unit length,

and

$$c_2 = \sinh^{-1}\left[\frac{p_o\, b}{2\, H_o \sinh\left(\frac{p_o\, a}{2\, H_o}\right)}\right] - \frac{p_o\, a}{2\, H_o}$$

The change in length of guy cable due to change in tension from T_0 to T is given by

$$\Delta L = \frac{1}{A_c E_c} \int_{curve} (T - T_0)\, ds \tag{3.5a}$$

where

A_c = *cross sectional area of guy cable*

E_c = *modulus of elasticity of guy cable*

Substituting for T and T_o from Equations (3.4 a) and (3.4 b) in Equation (3.5a) and simplifying,

$$\Delta L = \frac{L^2 \ \Delta H}{a \ A_c \ E_c} \left[1 - \frac{1}{6} c_3{}^2 \left(4 - \frac{a^2}{b^2} \right) \right] \tag{3.5b}$$

where,

$\Delta H = H - H_0$, *change in horizontal force*

and

$$c_3 = \frac{p_0 a}{2 H_0} \left(\frac{H_0}{H_0 + 0.5 \times \Delta H} \right)$$

3.4 Horizontal Displacement At Guy Cable Attachment Points

Horizontal displacement at guy cable attachment point(s) is expressed as a function of the change in the horizontal component of tension at that level as the mast is loaded or displaced.

$$dH = \frac{\partial H}{\partial a}\times da + \frac{\partial H}{\partial L}\times dL + \frac{\partial H}{\partial b}\times db$$

$\frac{\partial H}{\partial a}$, $\frac{\partial H}{\partial L}$, and $\frac{\partial H}{\partial b}$ can be obtained from Equation (3.3). After considerable manipulation, it can be shown that,

$$\begin{aligned} dH = {} & \frac{12H^3}{p^2a^3}\left(1 + 0.4\,c_1^2\right)\,da\ - \\ & \frac{12H^3}{p^2a^3}\left[\frac{L\times dL}{\sqrt{L^2 - b^2}\left(1 + 0.1\,c_1^2\right)}\right] + \\ & \frac{12H^3}{p^2a^3}\left[\frac{b\times db}{\sqrt{L^2 - b^2}\left(1 + 0.1\,c_1^2\right)}\right] \end{aligned}$$

Therefore,

$$\int_{H_o}^{H} \frac{dH}{H^3} = \frac{12}{p^2} \int_{h_o}^{h} \frac{\left[1 + 0.4\, c_1^2\right] da}{a^3} \quad -$$

$$\frac{12}{p^2 a^3} \int_{L_o}^{L} \frac{L \times dL}{\sqrt{L^2 - b^2}\left[1 + 0.1\, c_1^2\right]} \quad +$$

$$\frac{12}{p^2 a^3} \int_{v_o}^{v} \frac{b \times db}{\sqrt{L^2 - b^2}\left[1 + 0.1\, c_1^2\right]} \quad +$$

On simplification, we get

$$1 - \frac{H_o^2}{H^2} = \frac{6}{c_2^2\, a} \left[(1 + 0.1 c_3^2)\, \Delta a \right] \quad -$$

$$\frac{6}{c_2^2\, a} \left[\frac{L \times \Delta L \; - \; b \times \Delta b}{a \left[1 + \frac{1}{6}\, c_2^2 \right] \left[1 + 0.1 c_3^2 \right]} \right] \tag{3.6}$$

The relationship between Δa and Δb can be expressed as [10]

$$\Delta a = \frac{-b^2}{A_m E} \left[1 - \frac{1}{3}\, c_3^2 \right] \times \Delta b \tag{3.7}$$

where A_m and E are area and modulus of elasticity of the mast. Substituting (3.5) and (3.6) in Equation (3.7) gives

$$\Delta a = \frac{c_2^2 \, a}{6\left[1 + 0.4c_3^2\right]}\left[1 - \frac{H_o^2}{H^2}\right] +$$

$$\frac{\left[1 - \frac{1}{6}\left[4 - \frac{a^2}{L^2}\right]c_3^2\right] L^3 \, \Delta H \, (A_c E_c)}{a^2\left[1 + \frac{1}{6}c_2^2\right]\left[1 + 0.5c_3^2\right]} +$$

$$\frac{\left[1 - \frac{1}{3}c_3^2\right] b^3 \, \Delta H}{A_m \, E \, a^2\left[1 + \frac{1}{6}c_2^2\right]\left[1 + 0.5c_3^2\right]} \tag{3.8}$$

3.5 Equilibrium And Compatibility At Guy Cable Attachment Points

Equilibrium of external loads and guy cable tensions at guy cable attachment points must be ensured. Also, the compatibility of displacement at attachment points for windward and leeward guy cables must be ensured to confirm that all cables meeting at a level move as a unit along with the mast.

3.5.1 Triangular Mast Section

Figure 3.2 shows a triangular mast section subjected to an arbitrary wind load at any angle ϕ to an axis of symmetry. The equilibrium equations can be written as

$$\sum F_z = 0$$

$$R_z + \Delta H_3 \ cos30^o - \Delta H_2 \ cos30^o = 0$$

$$R_z + \frac{\sqrt{3}}{2} \Delta H_3 - \frac{\sqrt{3}}{2} \Delta H_2 = 0 \tag{a}$$

$$\sum F_y = 0$$

$$R_y - \Delta H_1 + \Delta H_2 \ cos60^o + \Delta H_3 \ cos60^o = 0$$

$$R_y - \Delta H_1 + \frac{\Delta H_2}{2} + \frac{\Delta H_3}{2} = 0 \tag{b}$$

Equilibrium equations (a) and (b) now become

$$\frac{2}{\sqrt{3}} R_z = \Delta H_2 - \Delta H_3 \tag{c}$$

$$2 R_y = 2 \Delta H_1 - \Delta H_2 - \Delta H_3 \tag{d}$$

The compatibility equations are:

$$\Delta a_1 = t \cos(90^o - \psi) = t \sin\psi$$

$$\Delta a_2 = t \cos(150^o - \psi) = -t \left[\frac{\sqrt{3}}{2} \cos\psi - \frac{1}{2} \sin\psi \right]$$

$$\Delta a_3 = -t \cos(\psi - 30^o) = -t \left[\frac{\sqrt{3}}{2} \cos\psi + \frac{1}{2} \sin\psi \right]$$

$$\Delta a_1 + \Delta a_2 + \Delta a_3 = 0$$

where Δa_i represents the horizontal displacement of i^{th} guy cable at the same guy connection point and 't' is the deflection of the mast at that level, and ψ is the direction of mast deflection with respect to Z-axis.

3.5.2 Square Mast Section

Figure 3.3 shows a square mast section subjected to an arbitrary wind load at any angle ϕ to an axis of symmetry. The equilibrium conditions are:

$$\sum F_z = 0$$

$$R_z - \Delta H_1 \cos 45^o + \Delta H_2 \cos 45^o - \Delta H_3 \cos 45^o + \Delta H_4 \cos 45^o = 0 \qquad \text{(e)}$$

$$\sum F_y = 0$$

$$R_y - \Delta H_1 \cos 45^o - \Delta H_2 \cos 45^o + \Delta H_3 \cos 45^o + \Delta H_4 \cos 45^o = 0 \qquad \text{(f)}$$

The compatibility equations are:

$$\Delta a_1 = t \cos(\psi - 45^o)$$

$$\Delta a_2 = t \cos(135^o - \psi)$$

$$\Delta a_3 = -t \cos(135^o - \psi)$$

$$\Delta a_4 = -t \cos(\psi - 45^o)$$

$$\Delta a_1 + \Delta a_2 + \Delta a_3 + \Delta a_4 = 0$$

where Δa_i and t are as defined in Section 3.5.1.

3.6 Translational Spring Constants

A spring constant is defined as the external force which will cause a unit displacement at a guy attachment point. All guy cables at a level are treated together in computing the spring constants.

To compute the translational spring constants at an attachment point, the algebraic sum of the horizontal components of all guy cable tensions in the direction of Y-axis and Z-axis are taken. Spring constants at a guy attachment point are given by

$$K_y = \frac{\sum H_y}{\Delta y} \qquad \text{and} \qquad K_z = \frac{\sum H_z}{\Delta z} \tag{3.9}$$

where Δy and Δz are components of horizontal displacement of the mast in the Y- and Z-directions respectively. $\sum H_y$ and $\sum H_z$ are algebraic sum of horizontal components of all guy cable tensions at the same level.

CHAPTER IV

THEORETICAL FORMULATION: MAST ANALYSIS

4.1 General

Finite element method for a three-dimensional beam element is adopted for the analysis of mast. The support from cables to the mast is modelled as translational spring support (as shown in Figure 4.1) and the stiffness of springs is estimated from the compatibility and equilibrium at the guy attachment points. The analysis includes the effects of structural non-linearities due both to the stiffening of the guy cables with increasing tension (or increase in sag with decrease in tension) and decreasing stiffness of the mast due to the presence of axial force in the mast.

4.2 Element Stiffness Matrix

Mast is divided into number of beam-column elements. Each beam-column element has two nodes with six degrees of freedom at each node as shown in Figure 4.2. For the first element, however, the degrees of freedom are two at P-end and six at Q-end due to the restraint from hinge support.

The element stiffness matrix is formed for each element and then assembled to form the master stiffness matrix using integer variable correlation (IVC) table. Element elastic stiffness matrix is given in Equation 4.1. The effect of axial force on the flexural stiffnesses is taken into account by using geometric stiffness matrix. The geometric stiffness matrix depends on the geometry of the displaced element. A general geometric stiffness matrix is given in Equation 4.2. The relationship between the displacements and forces is given by

$$([\,K_e\,] + [\,K_g\,])\ \{\delta\} = \{F\} \tag{4.3}$$

where $[K_e]$ = Elastic stiffness matrix in local coordinate system,

$[K_g]$ = Geometric stiffness matrix in local coordinate system,

$\{\delta\}$ = A vector of generalized element displacements in local coordinate system, and

$\{F\}$ = A vector of geleralized element forces in local coordinate system.

$$
[K_e] = \begin{array}{c}
\begin{array}{cccccccccccc} u_p & v_p & w_p & \beta_p & \theta_p & \lambda_p & u_q & v_q & w_q & \beta_q & \theta_q & \lambda_q \end{array} \\
\begin{bmatrix}
\frac{EA}{L} & 0 & 0 & 0 & 0 & 0 & \frac{-EA}{L} & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{12E}{L^3}I_z & 0 & 0 & 0 & \frac{6E}{L^2}I_z & 0 & \frac{-12E}{L^3}I_z & 0 & 0 & 0 & \frac{6E}{L^2}I_z \\
0 & 0 & \frac{12E}{L^3}I_y & 0 & \frac{-6E}{L^2}I_y & 0 & 0 & 0 & \frac{-12E}{L^3}I_y & 0 & \frac{-6E}{L^2}I_y & 0 \\
0 & 0 & 0 & \frac{6GJ}{5L} & 0 & 0 & 0 & 0 & 0 & \frac{6GJ}{5L} & 0 & 0 \\
0 & 0 & \frac{-6E}{L^2}I_y & 0 & \frac{4E}{L}I_y & 0 & 0 & 0 & \frac{6E}{L^2}I_y & 0 & \frac{2E}{L}I_y & 0 \\
0 & \frac{6E}{L^2}I_z & 0 & 0 & 0 & \frac{4E}{L}I_z & 0 & \frac{-6E}{L^2}I_z & 0 & 0 & 0 & \frac{2E}{L}I_z \\
\frac{-EA}{L} & 0 & 0 & 0 & 0 & 0 & \frac{-EA}{L} & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{-12E}{L^3}I_z & 0 & 0 & 0 & \frac{-6E}{L^2}I_z & 0 & \frac{12E}{L^3}I_z & 0 & 0 & 0 & \frac{-6E}{L^2}I_z \\
0 & 0 & \frac{-12E}{L^3}I_y & 0 & \frac{6E}{L^2}I_y & 0 & 0 & 0 & \frac{12E}{L^3}I_y & 0 & \frac{6E}{L^2}I_y & 0 \\
0 & 0 & 0 & \frac{6GJ}{5L} & 0 & 0 & 0 & 0 & 0 & \frac{6GJ}{5L} & 0 & 0 \\
0 & 0 & \frac{-6E}{L^2}I_y & 0 & \frac{2E}{L}I_y & 0 & 0 & 0 & \frac{6E}{L^2}I_y & 0 & \frac{4E}{L}I_y & 0 \\
0 & \frac{6E}{L^2}I_z & 0 & 0 & 0 & \frac{2E}{L}I_z & 0 & \frac{-6E}{L^2}I_z & 0 & 0 & 0 & \frac{4E}{L}I_z
\end{bmatrix}
\end{array}
\qquad (4.1)
$$

$$[K_g] = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & a^l & 0 & e^l & 0 & b^lL & 0 & -a^l & 0 & -k^l & 0 & b^lL \\ 0 & 0 & a^l & e & -b^lL & o & 0 & 0 & -a^l & -k & -b^lL & 0 \\ 0 & e^l & e & m^l & iL & -i^lL & 0 & -e^l & -e & -m^l & -SL & S^lL \\ 0 & 0 & -b^l & iL & cL^2 & o & 0 & 0 & b^lL & -iL & d^lL^2 & 0 \\ 0 & b^lL & 0 & -i^lL & 0 & cL^2 & 0 & -b^lL & 0 & -i^lL & 0 & d^lL^2 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -a^l & 0 & -e^l & 0 & -b^lL & 0 & a^l & 0 & k^l & 0 & -b^lL \\ 0 & 0 & -a^l & -e & b^lL & 0 & 0 & 0 & a^l & k & b^lL & 0 \\ 0 & -k^l & -k & -m^l & -iL & -i^lL & 0 & k^l & k & m^l & -SL & S^lL \\ 0 & 0 & -b^lL & -SL & d^lL^2 & 0 & 0 & 0 & b^lL & -SL & cL^2 & 0 \\ 0 & b^lL & 0 & S^lL & 0 & d^lL^2 & 0 & -b^lL & 0 & S^lL & 0 & cL^2 \end{bmatrix} \tag{4.2}$$

where

$$a^l = \frac{36}{30}\ \frac{p}{L}$$

$$b^l = \frac{3}{30}\ \frac{p}{L}$$

$$c = \frac{4}{30}\ \frac{p}{L}$$

$$d^l = \frac{-1}{30}\ \frac{p}{L}$$

$$e = \frac{-36}{30L} M_{zp} - \frac{3}{30} Q_y$$

$$i = \frac{3}{30L} M_{zp} + \frac{6}{30} Q_y$$

$$k = \frac{-36}{30L} M_{zp} - \frac{33}{30} Q_y$$

$$m^l = \frac{-36}{30L}\ \frac{PJ}{A}$$

$$S = \frac{-3}{30L} M_{zp} + \frac{3}{30} Q_y$$

$$e^l = \frac{-36}{30L} M_{yp} - \frac{3}{30} Q_z$$

$$i^l = \frac{3}{30L} M_{yp} + \frac{6}{30} Q_z$$

$$k^l = \frac{-36}{30L} M_{yp} - \frac{33}{30} Q_z$$

$$S^l = \frac{-3}{30L} M_{yp} + \frac{3}{30} Q_z$$

L = Length of the member

p = axial load in the member

M_{zp}, M_{yp}, Q_y, *and* Q_z are bending moments and transverse shears in the member.

4.3 Master Stiffness Matrix

Previously all element stiffness matrices were defined in their respective local coordinate system. A transformation matrix, *[T]*, is required to transform the element stiffness matrices to a single, global coordinate system. For an element of the mast bending about one principal axis rotated an arbitrary angle β from the structure-oriented coordinate system, the transformation matrix, *[T]*, is obtained by first forming the [3x3] rotation matrix *[R]*.

$$[R] = \begin{bmatrix} l_1 & m_1 & 0 \\ l_2 & m_2 & 0 \\ 0 & 0 & n_3 \end{bmatrix}$$

where *l*, *m*, and *n* are direction cosines of the local coordinate system. In terms of the angle of inclination, β,

$$[R] = \begin{bmatrix} cos\beta & sin\beta & 0 \\ -sin\beta & cos\beta & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

The transformation matrix, *[T]*, for bi-axial bending

$$[T] = \begin{bmatrix} R & 0 & 0 & 0 \\ 0 & R & 0 & 0 \\ 0 & 0 & R & 0 \\ 0 & 0 & 0 & R \end{bmatrix}$$

In the present study, directions of local coordinate axes are identical to the direction of global coordinate axes and the transformation matrix, $[T]$, therefore becomes an identity matrix, $[I]$.

The master stiffness matrix is initialized as null matrix in the beginning and then systematically filled with elements from the contributing element stiffness matrices using the IVC table.

4.4 Computation of Nodal Displacements and Forces

The solution of displacements is arrived at by solving the equation

$$[\,K\,]\ \{\,U\,\}\ =\ \{\,P\,\} \tag{4.4}$$

where

$[\,K\,]$ = *Assembled master stiffness matrix*

$\{\,U\,\}$ = *Nodal displacement vector*

$\{\,P\,\}$ = *Load vector*

A direct solution of nodal displacements is impossible while statisfying the compatibility of guy connection points. An iterative procedure is adopted to achieve the solution.

4.5 Iterative Procedure

For the first iteration, the nodal displacements $\{U\}$ in Equation (4.4) are computed with arbitrarily assumed spring constants. The nodal displacements are used to compute the tensions in the guy cables and spring constants using Equations (3.5b) and (3.9). The load vector $\{P\}$ in Equation (4.4) is modified to take into account the change in the guy cable tensions. Geometric stiffness matrix $[K_g]$ in Equation (4.2) is formed to take into account the axial loads. The master stiffness matrix $[K]$ in Equation (4.4) is updated to reflect the modifications to load vector and geometric stiffness matrix. A new nodal displacement vector $\{U\}$ is computed based on the new load vector $\{P\}$ and the new master stiffness matrix $[K]$.

This procedure of forming modified load vector, geometric and master stiffness matrices, and solving for nodal displacements is repeated until the compatibility of guy connection points is achieved with the desired degree of accuracy (5%).

4.6 Determination of Forces in Members of the Mast

The mast of a guyed tower, typically with a triangular or square cross section, consists of two elements, viz., (i) leg members and (ii) web members. The leg members are the main elements and resist the axial forces and bending moments. The leg members are braced by diagonal and/or horizontal web members. The shear forces and twisting moments are resisted by the web members.

It is generally observed that towers with triangular cross section weigh less and are economical when compared to towers with square cross section. Typical tower sections are shown in Figure 4.3. The axial force, shear forces, bending moments, and twisting moments at different sections obtained from the matrix stiffness method of analysis are used to compute the forces in leg and web members at different sections of the mast, based on the following assumptions:

a. The axial force is equally resisted by the leg members.

b. The bending moments induce compressive and tensile forces in the leg members only.

c. The shear is resisted only by the web members acting as a tension-compression system.

d. The twisting moment at any section is resisted by the web members only.

These assumptions have been used by all antenna tower designs and, therefore, are also used in the present study.

4.7 Mast With Triangular Cross Section

4.7.1 Leg Members

A mast with triangular cross section is shown in Figure 4.4 with all forces acting in the positive sense. Axial load F_x is acting perpendecular to the plane of paper in Figure 4.4 as mentioned in Section 4.6. As listed in the assumptions, the axial load is equally resisted by the leg members and axial force in each leg member due to F_x will be

$$C_1 = C_2 = C_3 = \frac{F_x}{3} \tag{4.5}$$

Considering the symmetry and equilibrium of the section (see Figure 4.5), the axial force in each member induced by the moment M_z is determined as

$$T_1 \left(\frac{2}{3} d \sin 60^o \right) + [C_2 + C_3] \left(\frac{1}{3} d \sin 60^o \right) = M_z$$

From symmetry, $C_2 = C_3$; also $C_2 + C_3 = T_1$. Therefore,

$$C_2 = C_3 = \frac{1}{2} T_1$$

$$T_1 = \frac{M_z}{d \sin 60^o} \tag{4.6a}$$

$$C_2 = \frac{M_z}{2d \sin 60^o} \tag{4.6b}$$

$$C_3 = \frac{M_z}{2d \sin 60^o} \tag{4.6c}$$

Similarly, axial forces in leg members due to M_y are derived from the equation

$$C_2 \left[\frac{d}{2}\right] + T_3 \left[\frac{d}{2}\right] = M_y$$

$$C_2 = T_3$$

$$M_y = C_2 d = T_3 d$$

$$C_2 = \frac{M_y}{d} \tag{4.7a}$$

$$T_3 = \frac{M_y}{d} \tag{4.7b}$$

Summing up all forces from Equations (4.5), (4.6), and (4.7) gives the total axial force in each member.

$$F_1 = \frac{M_z}{d \sin 60^o} - \frac{F_x}{3} \tag{4.8}$$

$$F_2 = -\frac{M_z}{2d \sin 60^o} - \frac{M_y}{d} - \frac{F_x}{3} \tag{4.9}$$

$$F_3 = -\frac{M_z}{2d \sin 60^o} + \frac{M_y}{d} - \frac{F_x}{3} \tag{4.10}$$

A change in the direction of wind will affect the sign of moments M_y and M_z and hence there will be six possibilities as given below.

$$F_1 = \frac{M_z}{d\ sin60^o} - \frac{F_x}{3} \tag{4.11a}$$

$$F_2 = -\frac{M_z}{d\ sin60^o} - \frac{F_x}{3} \tag{4.11b}$$

$$F_3 = \frac{M_z}{2d\ sin60^o} + \frac{M_y}{d} - \frac{F_x}{3} \tag{4.11c}$$

$$F_4 = \frac{M_z}{2d\ sin60^o} - \frac{M_y}{d} - \frac{F_x}{3} \tag{4.11d}$$

$$F_5 = -\frac{M_z}{2d\ sin60^o} + \frac{M_y}{d} - \frac{F_x}{3} \tag{4.11e}$$

$$F_6 = -\frac{M_z}{2d\ sin60^o} - \frac{M_y}{d} - \frac{F_x}{3} \tag{4.11f}$$

Leg members are designed for the largest compressive and tensile forces obtained from Equations (4.11a) to (4.11f).

4.7.2 Web Members

Axial forces in web members induced by the twisting moment and shear are shown in Figure 4.6. Considering the equilibrium of the section,

$$\sum M = 0 \implies (F_1 + F_2 + F_3) \times \frac{d}{2\sqrt{3}} = M_x$$

$$(or) \quad F_1 + F_2 + F_3 = \frac{2\sqrt{3}\ M_x}{d} \tag{a}$$

$$\sum F_z = 0 \implies F_1 - F_3 sin30^o - F_2 sin30^o = F_z \tag{b}$$

$$\sum F_y = 0 \implies (F_3 - F_2)\ cos30^o = F_y \tag{c}$$

Solving (a), (b), and (c) above for F_1, F_2, and F_3 gives

$$F_1 = \frac{2M_x}{\sqrt{3}\ d} + \frac{2}{3}F_z$$

$$F_2 = \frac{2M_x}{\sqrt{3}\ d} - \frac{F_z}{3} - \frac{F_y}{2sin60^o}$$

$$F_3 = \frac{2M_x}{\sqrt{3}\ d} - \frac{F_z}{3} + \frac{F_y}{2sin60^o}$$

The axial force in the web member will be the larger of

$$F = \frac{F_1}{\cos\theta} \ or \ \frac{F_2}{\cos\theta} \ or \ \frac{F_3}{\cos\theta} \qquad (4.12)$$

where θ is the web inclination with the horizontal.

4.8 Mast With A Square Cross Section

4.8.1 Leg Members

A mast with a square cross section is shown in Figure 4.7 with all forces acting in the positive sense. Axial load F_x is acting perpendicular to the plane of paper in Figure 4.7. As stated in Section 4.6, the axial load is equally resisted by the leg members and the axial force in each leg member due to F_x will be

$$C_1 = C_2 = C_3 = C_4 = \frac{F_x}{4} \qquad (4.13)$$

Considering the symmetry and equilibrium of the section (see Figure 4.8) the force in each leg member induced by moment M_y is determined. Leg members 2 and 4 are in compression and members 1 and 3 are in tension.

$$C_2 \left(\frac{d}{2}\right) + C_4 \left(\frac{d}{2}\right) + T_1 \left(\frac{d}{2}\right) + T_3 \left(\frac{d}{2}\right) = M_y$$

$$T_1 = T_3 = C_2 = C_4 = \frac{M_y}{2d} \tag{4.14}$$

Similarly, the force in each leg induced by moment M_z is given by

$$T_1 \left(\frac{d}{2}\right) + T_2 \left(\frac{d}{2}\right) + C_3 \left(\frac{d}{2}\right) + C_4 \left(\frac{d}{2}\right) = M_z$$

$$T_1 = T_2 = C_3 = C_4 = \frac{M_z}{2d} \tag{4.15}$$

Summing all forces from Equations (4.13), (4.14), and (4.15) gives the total axial force in each leg as

$$LEG\ 1 : F = \frac{M_y}{2d} + \frac{M_z}{2d} - \frac{F_x}{4} \tag{4.16a}$$

$$LEG\ 2 : F = -\frac{M_y}{2d} + \frac{M_z}{2d} - \frac{F_x}{4} \tag{4.16b}$$

$$LEG\ 3 : F = \frac{M_y}{2d} - \frac{M_z}{2d} - \frac{F_x}{4} \tag{4.16c}$$

$$LEG\ 4 : F = -\frac{M_y}{2d} - \frac{M_z}{2d} - \frac{F_x}{4} \tag{4.16d}$$

A change in the wind load direction will only alter the leg number choices but force combinations will not be altered. Hence each leg member will be designed to the maximum compression and maximum tension from the above equations.

4.8.2 Web Members

Following the assumptions stated in Section 4.6, web members are designed to resist the shear and twisting moment. The forces acting on the web members are shown in Figure 4.9. Forces acting on the members due to the twisting moment are calculated as:

$$F\frac{d}{2} + F\frac{d}{2} + F\frac{d}{2} + F\frac{d}{2} = M_x$$

$$\text{or} \quad F = \frac{M_x}{2d} \tag{4.17}$$

Including the contribution from forces F_y and F_z, the total force will be

Face shear on face parallel to Z-axis $$F_1 = \frac{F_z}{2} + \frac{M_x}{2d} \tag{4.18}$$

Face shear on face parallel to Y-Axis $$F_2 = \frac{F_y}{2} + \frac{M_x}{2d} \tag{4.19}$$

For compression-tension system, the axial force in the web member will be the larger of

$$F = \frac{F_1}{2\cos\theta} \quad \text{or} \quad F = \frac{F_2}{2\cos\theta} \tag{4.20}$$

where θ indicates web inclination with respect to horizontal plane.

CHAPTER V

DESIGN PROCEDURE

5.1 General

All members in the mast are assumed as axially loaded and are designed as per CAN/CSA-S16.1-M89. The members are designed for compression and checked for tension.

5.2 Details of Loading According to CAN/CSA-S37-M86

The tower is designed for the loads and load combinations as specified in CAN/CSA-S37-M86.

5.2.1 Loads

The following loads are considered in the design of the structure

a. Dead load, D,

b. Ice load, I, produced by ice formed radially on all exposed surfaces of all members of the structure including guys. Ice density is taken as 900 kg/m^3. There are four classes of loading determined by local topography or other factors for each particular site. Figure 5.1 gives the minimum requirements for ice thickness corresponding to each class.

c. Wind load, W.

5.2.2 Design Wind Pressure

The design wind pressure is given by [7]

$$P = q\, C_e\, C_g\, C_a$$

where P = *the design wind pressure*

q = *the reference velocity pressure.* It is the one in 30 year return hourly wind pressure for the site, or it may be obtained from Figure 5.2 and modified with consideration given to local topography, weather, or other considerations.

Ce = *the height factor*

$= \left(\frac{H_x}{10}\right)^{0.2}$ Where H_x is the height, in metres, above grade of the portion of the structure under consideration, and where $1.0 \leq C_e \leq 2.0$

C_g = the gust factor, taken as 2.0

C_a = the acceleration factor, taken as 1.0

5.2.3 Wind Load

The wind pressure is non-uniform and increases with height. However, in the present study, the wind pressure corresponding to the mid height of each element is assumed constant for design simplicity.

The wind load, W, on the structure, including antennas, is determined by using the design wind pressure, P, times the drag factor, C_d, applied to the net projected area, A_s. Where there is a combination of round or flat members, or for iced members, the wind load is calculated using the following formulas:

a. Wind load for bare (uniced) condition

$$W_o = P \left[C_{df} Af + C_{dr} A_r \right]$$

b. Wind load for iced condition

$$W_i = 0.5\,P\left[C_{df}\,Af + C_{dr}\,A_r + C_{dr}\,A_i\right]$$

where

C_{df} = *drag factor for flat members*

C_{dr} = *drag factor for round members*

A_f = *face area of bare flat members*

A_r = *face area of bare round members*

A_i = *face area of radial ice*

Face area of radial ice is shown in Figure 5.3.

5.2.4 Drag Factor

For latticed towers and masts, the value of C_d is determined as follows:

I. For flat members

For square towers: $C_{df} = 4.0(R_s)^2 - 5.9(R_s) + 4.0$

For triangular towers: $C_{df} = 3.4(R_s)^2 - 4.7(R_s) + 3.4$

II. For round members

$C_{dr} \;=\; C_{df}\,[\;0.51(R_s)^2 \;+\; 0.57\;]$ where C_{df} is the appropriate value from (I) above and $C_{dr} \leq C_{df}$.

III. For square towers with wind on the diagonal, the drag factor is to be modified by the factor

$$K_d \;=\; 1 \;+\; K_1 K_2,$$

where

$K_1 \;=\; 0.55$ *for flat members*

$\;\;\;\;\;\;=\; 0.80$ *for round members*

K_2 is determined from Table 5.1.

$$R_s \;=\; \frac{A_s}{A_g} \;=\; \text{solidity ratio}$$

$A_s \;=\;$ net projected area of members in one face of the structure. That is, for one panel, the projected area of all members within that panel, including ice thickness and attachments.

A_g = gross area of one face of the structure, that is, the panel height times the face width of one panel.

The value of C_d may also be taken from Table 5.2.

IV. For guy cables with or without ice, the drag factor is 1.2.

Diehl[12] studied the effect of various factors on the drag factor for cables and he suggested an average drag factor of 1.2.

5.2.5 Loading Combinations

The following loading conditions are to be considered:

1. $D + W_o$

2. $D + I + W_i$

3. $0.7D + W_o$

4. $0.7(D + I) + W_i$

where D, I, W_o, and W_i are as defined in Sections 5.2.1 through 5.2.3.

Wind load is considered at 30^o, 60^o, and 90^o to the face for triangular towers, and 45^o, and 90^o for square towers, in order to cause the maximum force in any member.

5.3 Design of Members

The leg and web members are designed for compression and checked for tension. The factored axial compressive resistance is given in clause 13.3.1 of CAN/CSA-S16.1-M89 as

1. $0 \le \lambda \le 0.15, \quad C_r = \phi A \sigma_y$

2. $0.15 < \lambda \le 1.0, \quad C_r = \phi A \sigma_y \left[1.035 - 0.202\lambda - 0.222\lambda^2 \right]$

3. $1.0 < \lambda \le 2.0, \quad C_r = \phi A \sigma_y \left[-0.111 + 0.636\lambda^{-1} + 0.087\lambda^{-2} \right]$

4. $2.0 < \lambda \le 3.6, \quad C_r = \phi A \sigma_y \left[0.009 + 0.877\lambda^{-2} \right]$

5. $3.6 < \lambda \quad C_r = \phi A \sigma_y \lambda^{-2}$

where

$$\lambda = \frac{KL}{r}\sqrt{\frac{\sigma_y}{\pi^2 E}}$$

and

ϕ = *0.9 for leg members*

= *0.72 for web members*

σ_y = *yield strength of steel*

K = *effective length factor*

L = *unsupported length of the member as shown in Figure 4.3*

r = *minimum radius of gyration of the member*

A = *cross-sectional area of the member*

The factored axial resistance is given by clause 5.5 of CAN/CSA-S37-M86 as

$$\phi R \geq (\alpha) \times \text{Member force}$$

where, R = resistance of the structure's members and components, including foundations, and $\alpha = $ load factor, 1.5 ($\alpha = 2.0$ for cantilever position of mast).

In the design, equal leg angles are used for the leg members, and equal leg or unequal leg angles are used for the web members. All angles are hot-rolled and are available from Canadian mills. Since the leg angles for triangular guyed towers are 60^o schifflerized angles, the properties of such angles are determined from fundamental principles. Properties for 60^o schifflerized equal leg angles are given in Appendix B.

In the design of the leg members, the force in each member is calculated according to Equation (4.11) for a triangular tower, or according to Equation (4.16) for a square tower. The largest negative value, or compressive force, and the largest positive value, or tensile force, is determined.

The axial force in each web member is determined according to Equation (4.12) for triangular towers or Equation (4.20) for square towers.

The compressive resistance, C_r, is then calculated for all angle sizes with the angle size of least mass. A selection is made when the following two requirements are met:

1. The compressive resistance of the member is greater than the largest applied factored compressive force.

2. The slenderness ratio of the member is less than the maximum slenderness ratio as given by CAN/CSA-S37-M86, clause 6.2.2. For

   a. leg members, $\frac{KL}{r} < 120$

   b. web members, $\frac{KL}{r} < 200$

The member selected is then checked in tension. The tensile resistance of the member is given in clause 13.2 of CAN/CSA-S16.1-M89. It is taken as the least of

a. $T_r = \phi A F_y$

b. $T_r = 0.85 \phi A_{ne} F_u$

c. $T_r = 0.85 \phi A'_{ne} F_u$

where A = Gross cross sectional area

A_{ne} = Net cross sectional area

A'_{ne} = Net cross sectional area with shear lag effect included

A'_{ne} = $0.90 A_{ne}$ for WWF, W, M, or S shapes with flange widths not less than two-thirds the depth and for structural tees cut from these shapes, when only the flanges are connected and there are 3 or more transverse lines of bolts.

A'_{ne} = $0.85 A_{ne}$ for all other structural shapes connected by 3 or more transverse lines of bolts.

A'_{ne} = $0.75 A_{ne}$ for all shapes connected by one or two transverse lines of bolts.

If the member size already selected for the compressive force is able to resist the applied factored tensile force, and also meet the slenderness ratio requirements given in clause 10.2.2 of CAN/CSA-S16.1-M89, ($\frac{KL}{r}$ < 300), then the member is selected in the design. Otherwise, the selection procedure is repeated until the requirements for tension are met.

5.4 Design of Guy Cables

A load factor of 1.5 and a resistance factor of 0.9 are considered in the design. The tensile resistance in the guy cable is calculated as

T_r = 0.9 x Breaking strength of guy cable

Design Force ≤ T_r

where the Design Force = 1.5 x Maximum Tension

The set of guy cables used in the design is listed in Table 5.3. In the design, maximum tension in each guy is computed from all the loading conditions mentioned in Section 5.2.5. From this, the design force is computed and the value of design force is compared to the tensile resistance, T_r, of guy cables in a sequential order. A selection of minimum weight guy cable is made when T_r is greater than the corresponding design force.

5.5 Design of Guy Anchorages

Design of guy anchorages is based on the guidelines provided in "Design Manual: Soil Mechanics, Foundations, and Earth Structures," *NAVFAC, DM-7*[11]. Only the deadman anchorages are considered in the present study.

5.5.1 Deadman Anchorages

Figure 5.4 indicates the various forces acting on an anchorage block. The analysis and design of concrete are carried out on the following lines:

P_v = total vertical component of tensile forces from guy cables anchored to the block.

P_h = total horizontal component of tensile forces from guy cables anchored to the block.

W_b = weight of concrete block.

γ_c = density of concrete

γ_s = density of soil

l_f = length of anchorage block

b_f = width of anchorage block

d_f = depth of anchorage block

h = height of soil above the anchorage block

P_p = total passive resistance from soil

p_1 = passive pressure at the top of concrete block

p_2 = passive pressure at the bottom of concrete block

Weight of soil above the anchorage, W_s is given by

$$W_s = \gamma_s \times l_f \times b_f \times h \tag{5.2a}$$

The total weight is given by

$$W_t = W_b + W_s \tag{5.2b}$$

The total passive resistance from soil is given by

$$P_p = \left[\frac{p_1 + p_2}{2} \right] \times d_f \times l_f$$

$$= k_p \times \gamma_s \times (h + d_f)$$

$$==> \quad = \frac{1}{2} k_p \gamma_s \left[d_f^2 l_f + 2h d_f l_f \right] \tag{5.3}$$

5.5.2 Resistance to Vertical and Horizontal Forces

Safety factors in the vertical direction are:

$$(i) \quad \frac{W_t}{P_v} \geq 1.5 \tag{5.4}$$

$$(ii) \quad \frac{W_b}{P_v} \geq 1.0 \tag{5.5}$$

Safety factor in the horizontal direction is:

$$\frac{P_p}{P_h} \geq 1.5, and \tag{5.6}$$

$$h \geq h_{\min} \tag{5.7}$$

where $h_{\min}$ represents the minimum height of soil required above the anchorage block, taken as 1.0 m in the present study.

Assuming equality sign for Equations (5.4) through (5.7), from (5.2b),

$$W_t = 1.5 W_b \quad \text{and} \quad W_t = W_b + W_s$$

$$==> \quad W_s = 0.5 W_b$$

From Equations (5.1) and (5.2a), we get

$$\frac{W_b}{W_s} = \frac{\gamma_c \, l_f \, b_f \, d_f}{\gamma_s \, l_f \, b_f \, h} = \frac{d \, \gamma_c}{h \, \gamma_s} = 2$$

$$==> \quad d_f = 2 \times h \times \frac{\gamma_s}{\gamma_c} \tag{5.8}$$

Substituting $P_p = 1.5 \times P_h$ in equation (5.3), we get

$$P_p = \frac{1}{2} \, k_p \, \gamma_s \left[d_f^2 \, l_f + 2h \, d_f \, l_f \right] = 1.5 \, P_h$$

Substituting for h from Equation (5.8), and solving for l_f gives

$$l_f = \frac{3.0 \times P_h}{k_p \, d_f^2 \, (\gamma_s + \gamma_c)} \tag{5.9}$$

Substituting $W_b = P_v$ in Equation (5.1) and solving for b_f gives

$$b_f = \frac{P_v}{d_f \, l_f \, \gamma_c} \tag{5.10}$$

The solution gives the design of anchor block with minimum volume of concrete and minimum earth work excavation satisfying the safety criteria.

CHAPTER VI

DEVELOPMENT OF COMPUTER PROGRAM

6.1 General

The analysis of an indeterminate structure depends on the section properties of members, and structures with cable elements pose more problems as compatibility is difficult to arrive at. For the present problem, an iterative procedure is used to analyse and design the guyed tower. The iterative procedure is a modification of the "CABLE" subroutine developed by *Isheke*[20].

The computer program developed is capable of designing the guyed tower by selecting leg and web members in a sequential order. It also selects guy cables for the calculated design tension forces. Finally the mass and cost of the tower are estimated.

6.2 Main Program

The data required for the program and the flow chart of various steps in the design process are listed in Figures 6.1 through 6.5. The computer program and the sample outputs are given in the floppy diskette enclosed.

The main program generates the nodal connectivity table and integer variable correlation (IVC) table. It also computes cross sectional properties for schifflerized angles used in triangular masts and stores the data in ascending order of mass. Regular equal leg angles are arranged in ascending order separately, and are also combined with regular unequal leg angles and together they are arranged in ascending order of mass.

Loads on the mast, different from self weight and wind loads, are read from the data file(s) if they are present and equivalent fixed end forces at nodes are calculated. These loads are then added to the nodal loads calculated from self weight and wind loads before forming the final load vector.

Cable tensions, nodal displacements, and element displacements are calculated and, upon achieving the compatibility of guy connection points the element forces and axial forces in the leg members and web members are computed. The computational procedure is described in Chapter IV. The analysis is carried out for different wind directions and also for different loading conditions as mentioned in Section 5.2.5. The analysis and the design procedure is repeated until the cross sectional area for each segment obtained in one iteration does not differ significantly from a previous iteration.

The subroutines used in the program are discussed in the following sections.

6.3 SUBROUTINES

6.3.1 Subroutine EXLOAD

This subroutine collects the input of forces acting on segments other than self weight, wind and/or ice loads from the main program and then calculates equivalent fixed end forces at nodes. Loads contributed by antennas are supplied as input data. The fixed end loads calculated in the subroutine are added to those from self weight, wind, and/or ice loads in the main program before the final load vector is formed.

6.3.2 Subroutines FORMSTIF, GEOSTIF, and MASTSTIF

Subroutine FORMSTIF is used to form elastic element stiffness matrices for all the elements and then MASTSTIF is used to form the master stiffness matrix by combining all element stiffness matrices using IVC table. In the initial process of analysis the effect of axial forces on flexural stiffness is not considered but, after the first iteration, all forces in the element are computed and geometric stiffness matrix is formed for each element using GEOMSTIF. Again, MASTSTIF is used to assemble all the element elastic and geometric stiffness matrices into master stiffness matrix.

6.3.3 Subroutine LOADVECT

Subroutine LOADVECT is used to form the final load vector. It also adds the forces from cables at different levels to the forces at corresponding levels. The load vector needs to be modified after each iteration and, hence, this subroutine is called by the main program after each iteration to modify the load vector.

6.3.4 Subroutine CABLE

Subroutine CABLE computes initial and final cable tensions and estimates the stiffness coefficients for each cable level from the computed displacements. These stiffness coefficients are then added to the corresponding elements in the master stiffness matrix of the mast in the MASTSTIF subroutine. This subroutine is called by the main program for each cable set at different levels and also for each iteration of analysis. Iterations are continued until the compatibility of all guy attachment points is satisfied.

6.3.5 Subroutines SEGDISP and SEGFRCE

Subroutine SEGDISP is used to compute the displacement vector from master stiffness matrix and load vector. The segment displacements are computed from this displacement vector and then subroutine SEGFRCE is called by the main program to compute end forces in each segment. Geometric stiffness matrix is modified from these forces and, hence, these subroutines are called by the main program in each iteration.

6.3.6 Subroutines DESIGN, AXIL, AXIW, COMP, and TENS

Subroutine DESIGN, with the help of other subroutines, calculates axial forces in each member and selects a section from the available data. Properties of the section for each segment are computed and supplied to the main program for the next iteration.

Subroutine AXIL computes the maximum compressive force and maximum tensile force in each leg member as described in Sections 4.7.1 and 4.8.1. The forces are used in the subroutines COMP and TENS for selection of members. Subroutine AXIW computes the maximum compressive force and tensile force in each web member. As in the case of leg members these forces are supplied to DESIGN. Eventually COMP and TENS use the forces to select a section for web member.

The subroutine COMP, called by DESIGN, selects a section for a given load. For towers with a triangular mast, a section for leg member is chosen from the set of schifflerized 60^o equal leg angles. For towers with square mast, a section for leg member is chosen from the set of regular 90^o equal leg angles. For web members, a section is chosen from the set of regular equal and unequal angles. Selection of the section is based on the design process described in Section 5.3. The selection process of any member begins with the lightest member in the corresponding set. The subroutine TENS compares the tensile strength of the section chosen by COMP with the maximum tensile force calculated. If the tensile strength is not sufficient and/or slenderness ratio exceeds the permissible limit, then the search process is repeated until a suitable section is chosen.

6.3.7 Subroutine ANCHOR

Subroutine ANCHOR is used to compute forces on each deadman anchorage and design deadman anchorage for the computed forces. The subroutine is also used to estimate the cost of concrete block and earthwork excavation. This data is supplied to the main program before the total cost of the structure is computed.

CHAPTER VII

PARAMETRIC STUDY

7.1 General

Parametric study is very important in many optimization problems, particularly when the relationship between the objective function and design variables is highly implicit. As in most structural optimization problems, it is very difficult to form an expression explicitly for the cost of a guyed tower in terms of the design variables. Parametric study provides a better understanding of the nature of objective function. In parametric study one design variable is chosen at a time while other design variables are fixed tentatively. The variations in the objective function (i.e., cost of guyed tower for the present study) are studied with respect to the chosen design variable. The process is repeated with all variables to identify key design parameters that influence the optimal value of the objective function.

7.2 Example I: A 100 m High Guyed Tower With Two Guy Levels

A 100 m high guyed tower with two guy levels (Figure 7.1) is studied to understand the key parameters contributing to the total cost. The design variables considered in the parametric study are:

1. inclination of web to the horizontal,
2. radial distance of guy anchor point for cable 1,
3. radial distance of guy anchor point for cable 2,
4. height of guy connection point for cable 1,
5. height of guy connection point for cable 2,
6. size of the mast, and
7. number of guy levels.

Mass and cost are estimated in the design process and the variations of both mass and cost are studied with respect to the design variables. Finding the minimum cost design was given higher priority than minimum weight. The study is carried out with four loading conditions:

1. Class 'C' Wind + Class IV Ice loads,
2. Class 'C' Wind + Class I Ice loads,
3. Class 'A' Wind + Class I Ice loads, and
4. Class 'A' Wind + Class IV Ice loads.

The variations of mass and cost with respect to each design variable are presented in Figures 7.2 through 7.15. The results are analyzed in Chapter VIII.

For a 100 m high guyed tower, four mast sizes are considered for parametric study. For mast sizes 1.25 m, 1.5 m, and 2.5 m, parameters 1 through 5 are studied. These observations are shown in Figures A.1 through A.30 in Appendix-A. For a mast size of 2.0 m, parameters 1 through 7 are studied.

7.3 Example II: A 350 m High Guyed Tower With Seven Guy Levels

A 350 m high guyed tower with seven guy levels is studied (Figure 7.16). The design variables considered in the parametric study are:

1. inclination of web to the horizontal,

2. radial distance of guy anchor point for cable 1,
3. radial distance of guy anchor point for cable 2,
4. radial distance of guy anchor point for cable 3,
5. radial distance of guy anchor point for cable 4,
6. radial distance of guy anchor point for cable 5,
7. radial distance of guy anchor point for cable 6,
8. radial distance of guy anchor point for cable 7,
9. height of guy connection point for cable 1,
10. height of guy connection point for cable 2,
11. height of guy connection point for cable 3,
12. height of guy connection point for cable 4,
13. height of guy connection point for cable 5,
14. height of guy connection point for cable 6,

15. height of guy connection point for cable 7,

16. size of the mast, and

17. number of guy levels.

Mass and cost are estimated in each iteration and the variations of both mass and cost are plotted with respect to the design variables in Figures 7.17 through 7.50. Four loading conditions similar to the 100 m tower described in Section 7.2 are considered for this experiment. The results are analyzed in Chapter VIII.

CHAPTER VIII

ANALYSIS AND DISCUSSION OF RESULTS

8.1 General

To understand the impact of various parameters on the mass and the total cost of a guyed tower, a parametric study of the guyed tower analysis was undertaken in two stages. In the first stage, the optimum number of guy levels is determined for a given guyed tower. In the second stage, the effect of various parameters on a guy tower mass and total cost are observed. The design parameters are as described in Chapter VII. The study can be broadly categorized as:

- study for optimum number of guy connections,
- study of inclination of diagonal web member,
- study of radial distance of a guy anchor point,

- study of the height of a guy connection point, and
- study of the size of the mast.

8.2 Effect of Number of Guy Connections

The objective of the study of number of guy connections was to understand how the number of guy levels influences the total cost of a guyed tower design. In order to determine the optimum number of guy levels it was necessary to fix other parameters such as mast size, web inclination with the horizontal, and location of guy anchor points. It was assumed that all cable connections are equally spaced on the mast, all cable chords are at 45^o inclination, and that the mast size is 2.0 metres.

The output of simulation experiments for the 100 m guyed tower is shown in Figure 7.2 for mass and Figure 7.3 for the total cost respectively. In both cases, a steady decrease in mass is observed with the increase in number of guy levels. The cost decreases initially for high wind conditions and increases steadily for low wind conditions. The minimum cost is observed at five guy connections for high wind conditions and at two guy connections for low wind conditions.

From this experiment, two guy connections are adopted for uniformity to study the effect of other parameters on the mass and total cost of a guyed tower.

The ourput for a 350 m high guyed tower are shown in Figure 7.17 for mass and Figure 7.18 for the total cost respectively. The cost decreases steadily as the number of guy connections is increased from six to fourteen. However, the reduction in cost is not significant when guy connections are more than seven for low wind conditions.

From this experiment, the number of guy connections is taken as seven for uniformity to study the effect of other parameters on the mass and total cost of a guyed tower.

8.3 Effect of Inclination of Diagonal Web Member

Variation in the mass and the total cost of a guyed tower due to changes in web inclination for a 100 m tower are shown in Figures 7.4 and 7.5 respectively. Figures 7.19 and 7.20 present similar results for a 350 m high guyed tower.

The variation in the mass and the total cost for both experiments are similar. The minimum cost is observed for web inclinations in the range 35^o to 50^o. The effect of web inclination is very significant on the mass and the total cost of a guyed tower for all loading conditions.

As the inclination of web increases, the following effects are observed:

1. Initially, the number of panels in the mast reduce and, thus, the dead load from web members reduces.

2. The effective lengths of web members and leg members increase which may result in the selection of heavier sections for leg and web members for design.

3. The weight component from web members reduce initially because the number of panels are reduced but, as heavier sections are chosen from the design stage, the weight component increases.

4. As a result of decrease in dead loads, the weight components of leg members decrease at low web inclinations.

8.4 Effect of Radial Distance of a Guy Anchor Point

The effect of variation in radial distance for a 100 m guyed tower for the two guy anchor points are shown in Figures 7.6 and 7.7 for the mass and the total cost respectively, for cable 1. The results for cable 2 are shown in Figures 7.8 and 7.9.

The effect of variation in guy anchor point for cable 1 is very significant. The variation in the mass and the total cost for cable 2 is not that significant. This indicates that, for a 100 m high guyed tower, the cable connected at the lower level (cable 1) provides most of the lateral support and controls the deflections in the mast.

The effect of radial distance for a 350 m guyed tower are shown in Figures 7.21 through 7.34 for cables 1 through 7 respectively. It is observed that, for all cables, the variation in mass and cost is very significant under high wind conditions. For low wind conditions, the variation in the mass and the total cost is not significant. The high wind conditions result in higher deflections in the mast.

The maximum difference in mass for high and low wind conditions is 60 tonnes and 3.5 tonnes, respectively.

A joint observation of variation in radial distance for guy anchor points for cables 1, 2, and 3 show that:

- the minimum mass and the minimum cost are observed for cable 1 chord inclination at 30^o,
- the minimum mass and the minumum cost are observed for cable 2 chord inclination at 45^o, and

- the minimum mass and the minimum cost are observed for cable 3 chord inclination at 40^o.

Note: all guy connection points are fixed for the above observations.

At minimum mass and the minimum cost for cable 1, anchor point moves closer to that of cable 2. Similarly, cable 3 anchor point can be moved closer to that of cable 2. This promotes grouping of cables at anchor points for an economical design of a tower with many guy levels.

The minimum mass and cost for cables 4 through 7 are observed for cable chord inclination of 45^o.

The effect of cable chord inclination on the mass and total cost can be summarized as:

1. At higher inclinations, the cable stands steeper which results in increased tension; however, the length of cable, the mass and the total cost of guyed tower decrease initially.

2. The increase in cable tension results in more axial thrust on the mast. This will

result in the selection of heavier sections for leg members, thus increasing the mass and total cost.

3. The forces on the anchor block vary significantly as the cable chord inclination varies. When the cable is steeper, the vertical component of the force on anchor block increases and it results in the increase in weight (cost) of concrete block.

4. An increase in chord inclination results in reduction of axial thrust on the mast. Consequently, the weight of mast members can be reduced.

8.5 Effect of Height of a Guy Connection Point

The effect of height of guy connection points for a 100 m high guyed tower for cable 1 are shown in Figures 7.10 and 7.11. Similar results for cable 2 are shown in Figures 7.12 and 7.13.

The variation in the mass and the total cost is not significant for low wind conditions, and is very significant for high wind conditions. The minimum cost is observed when cable 1 is at 30^o and 42^o for low ice and high ice conditions respectively. The minimum cost is observed when cable 2 is at 30^o for all loading conditions.

The effect of height of guy connection points for cables 1 through 7 for the 350 m high guyed tower are shown in Figures 7.35 through 7.48 respectively.

The variation in mass and cost are significant for all loading conditions. The minimum mass and the minimum cost are observed at cable chord inclinations 35^o to 45^o for all cases. The span arrangement varies with the guy connection points. Also, the variation in span arrangement is not comparable for different guy levels. Hence the observed effect on the mass and the cost is different as the heights of different guy connection points are varied.

The effect on the mass and the total cost due to variation in cable chord inclination can be summarized as follows:

1. As the height of a cable connection is increased, the increase in cable length results in design of heavier cables.

2. The axial thrust on the mast increases as cable stands steeper and it results in the design of heavier sections for leg members.

3. On the other hand, the deflections in the mast are reduced and this results in the design of mast members with less weight.

4. The forces on the anchor block also vary. The vertical compc:ient of force increases since cable becomes steeper and it results in the increase of weight of concrete block.

8.6 Effect of Size of The Mast

The effect of mast size on the mass and total cost of a 100 m high guyed tower are studied with four mast sizes (1.25 m, 1.5 m, 2.0 m, and 2.5 m). As the mast size is increased from 1.25 m to 1.5 m both the mass and the total cost of a guyed tower decresed and a steady increase is observed as mast size increases beyond 1.5 m. The variations are shown in Figure 7.14 and 7.15.

For the optimization experiment the size of mast is taken as 1.5 m.

The effect of mast size for a 350 m high guyed tower is presented in Figures 7.49 and 7.50. Both the mass and the total cost decrease initially as mast size is increased to 2.0 m and a steady increase is observed for mast sizes beyond 2.0 m to 2.5 m. The variation in the mass and the cost is not significant for low wind conditions.

The effect of mast size on the mass and total cost of a guyed tower is very significant. As the mast size is increased, the following observations can be made:

1. The sectional properties of the mast such as moment of inertia increase resulting in improved stiffness of the structure and, hence, in the design of mast members with less weight.

2. With the increase in stiffness of the structure, deflections in the mast are reduced, resulting in cables of less weight.

3. The lengths of web members increase and the weight component of web members increases.

8.7 The Optimization Experiment

A 100 m high guyed tower with two guy anchor points is selected for an optimization experiment. The optimization problem formulation is as described in Section 1.2.4. After nearly 48 hours of CPU time, the optimization experiment was considered highly expensive and was beyond the scope of present study. The result of optimization experiment after five iterations are given below:

Inclination of web member = 33.37^o

Radial distance of anchor point for cable 1 = 79.34 m

Radial distance of anchor point for cable 2 = 100.18 m

Height of guy connection point for cable 1 = 44.22 m

Height of guy connection point for cable 2 = 63.32 m

Cost of the structure = $ 33,569.60

CHAPTER IX

CONCLUSIONS AND RECOMMENDATIONS

9.1 General

The computer program developed is capable of finding an optimal design of a guyed tower for a given height and geometric properties. The parametric study carried out in the present study covers all design parameters of a guyed tower.

9.2 Conclusions

From the limited analytical investigations carried out in the present study the conclusions can be summarized as:

1. For a guyed tower of 100 m height, the optimal design is observed at a mast size of 1.5 m. Also, for a guyed tower of 350 m height, the optimal mast size is 2.0 m.

2. The optimal design is observed when inclination of web members to horizontal is in the range of 35^o to 50^o.

3. Grouping of cable anchorages for multi-level guyed towers is observed to be economical for deadman anchorages.

4. The literature survey suggests use of generic optimizers such as MINOS for optimum guyed tower design. However, the present study concludes that generic optimizers such as MINOS are highly CPU intensive and, thus, are not economical. Study of a tall guyed tower with many guy levels may not be feasible.

9.3 Recommendations

1. It is recommended that outriggers be provided and considered as torsional springs when large torsional moments are present in the structure. Each outrigger is usually provided with six guy cables for a mast with triangular cross-section to supply the necessary torsional rigidity. The torsional spring constants can be estimated and added to the corresponding elements of the stiffness matrix.

2. The present study covers static analysis of the structure. Usually wind loads are dynamic in nature. Hence it is recommended tht dynamic analysis may be incorporated in the analysis of the structure.

3. Extensive study is carried out in the present investigation for an optimum design of a guyed tower. The study recommends that partial load factors for dead load, wind load, and ice load should be developed for a truly limit state design.

4. The data available from extensive parametric study may be stored in an expert system data base so that users in coming times can benefit from the author's experience.

5. The present study suggests that regression analysis may be carried out to arrive at a mathematical relationship between the cost of a guyed tower and various parameters of a guyed tower.

TABLE 5.1 - DETERMINATION OF K_2

R_s	K_2
0.0 to 0.2	0.2
0.2 to 0.5	R_s
0.5 to 0.8	$1 - R_s$
0.8 to 1.0	0.2

Source

CAN/CSA-S37-M86
September 1986

TABLE 5.2 - DRAG FACTOR, C_d

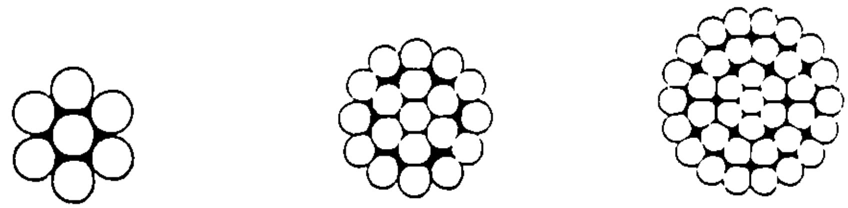
Solidity Ratio R_s	Triangular Towers All Wind Directions		Square Towers Wind on Face		Square Towers Wind on Diagonal	
	Flat Members	Round Members	Flat Members	Round Members	Flat Members	Round Members
0.0	3.4	1.9	4.0	2.3	4.4	2.7
0.1	3.0	1.7	3.5	2.0	3.9	2.3
0.2	2.0	1.9	0.9	1.8	3.3	2.1
0.3	2.3	1.4	2.6	1.6	3.0	2.0
0.4	2.1	1.4	2.3	1.5	2.8	2.0
0.5	1.9	1.3	2.1	1.4	2.7	2.0
0.6	1.8	1.4	1.9	1.4	2.3	1.8
0.7	1.8	1.5	1.8	1.5	2.1	1.9
0.8	1.8	1.6	1.8	1.7	2.0	2.0
0.9	1.9	1.9	1.9	1.9	2.1	2.2
1.0	2.1	2.1	2.1	2.1	2.3	2.4

Source

CAN/CSA-S37-M86
September 1986

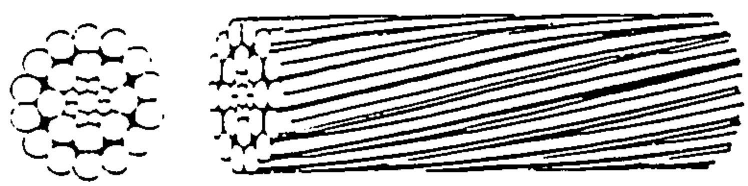
TABLE 5.3 - Properties of Guy Cables

Guy Diameter		Guy Weight		Guy Breaking Load	
in.	mm	lb/1000 ft.	N/m	kips	kN
3/16	4.76	79	1.15	4.00	17.8
1/4	6.35	129	1.88	6.45	28.7
9/32	7.14	169	2.47	8.45	37.6
5/16	7.94	222	3.24	11.15	49.6
3/8	9.53	210	3.94	13.5	60.0
7/16	11.1	388	5.66	19.5	86.7
1/2	12.7	520	7.59	30.0	133
9/16	14.3	660	9.63	38.0	169
5/8	15.9	820	12.0	48.0	214
11/16	17.5	990	14.5	58.0	258
3/4	19.1	1180	17.2	68.0	502
13/16	20.6	1390	20.3	80.0	356
7/8	22.2	1610	23.5	92.0	409
15/16	23.8	1850	27.0	108	480
1	25.4	2000	29.2	122	543
1-1/16	27.0	2300	33.6	138	614
1-1/8	28.6	2610	38.1	156	694
1-3/16	30.2	2920	42.6	172	765
1-1/4	31.8	3220	47.0	192	854
1-5/16	33.3	3580	52.2	212	943
1-3/8	34.9	3890	56.8	232	1030
1-7/16	36.5	4090	62.8	252	1130
1-1/2	38.1	4700	68.6	276	1230
1-9/16	39.7	5110	74.6	300	1330
1-5/8	41.3	5520	80.6	324	1440
1-11/16	42.9	5980	87.3	352	1570
1-3/4	44.5	6450	99.1	375	1670
1-13/16	46.0	6920	101	404	1800
1-7/8	47.6	7420	108	432	1920
1-15/16	49.2	7950	116	460	2050
2	50.0	8480	124	490	2180
2-1/16	52.4	8980	131	522	2320
2-1/8	54.0	9470	198	554	2460
2-3/16	55.6	10130	148	586	2610
2-1/4	57.2	10830	158	620	2760
2-5/16	58.7	11290	165	654	2910
2-3/8	60.3	11810	192	688	3060
2-7/16	61.9	12250	179	720	3200
2-1/2	63.5	13250	193	752	3340
2-9/16	65.1	13800	201	784	3490

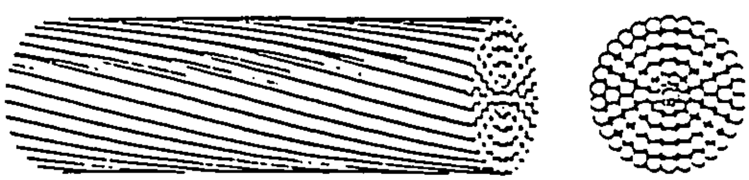


7 Wire 19 Wire 37 Wire

GUY STRAND



31 Wire Galvanized Bridge Strand



103 Wire Galvanized Bridge Strand

FIG 1.1 : GUY STRAND AND BRIDGE STRAND

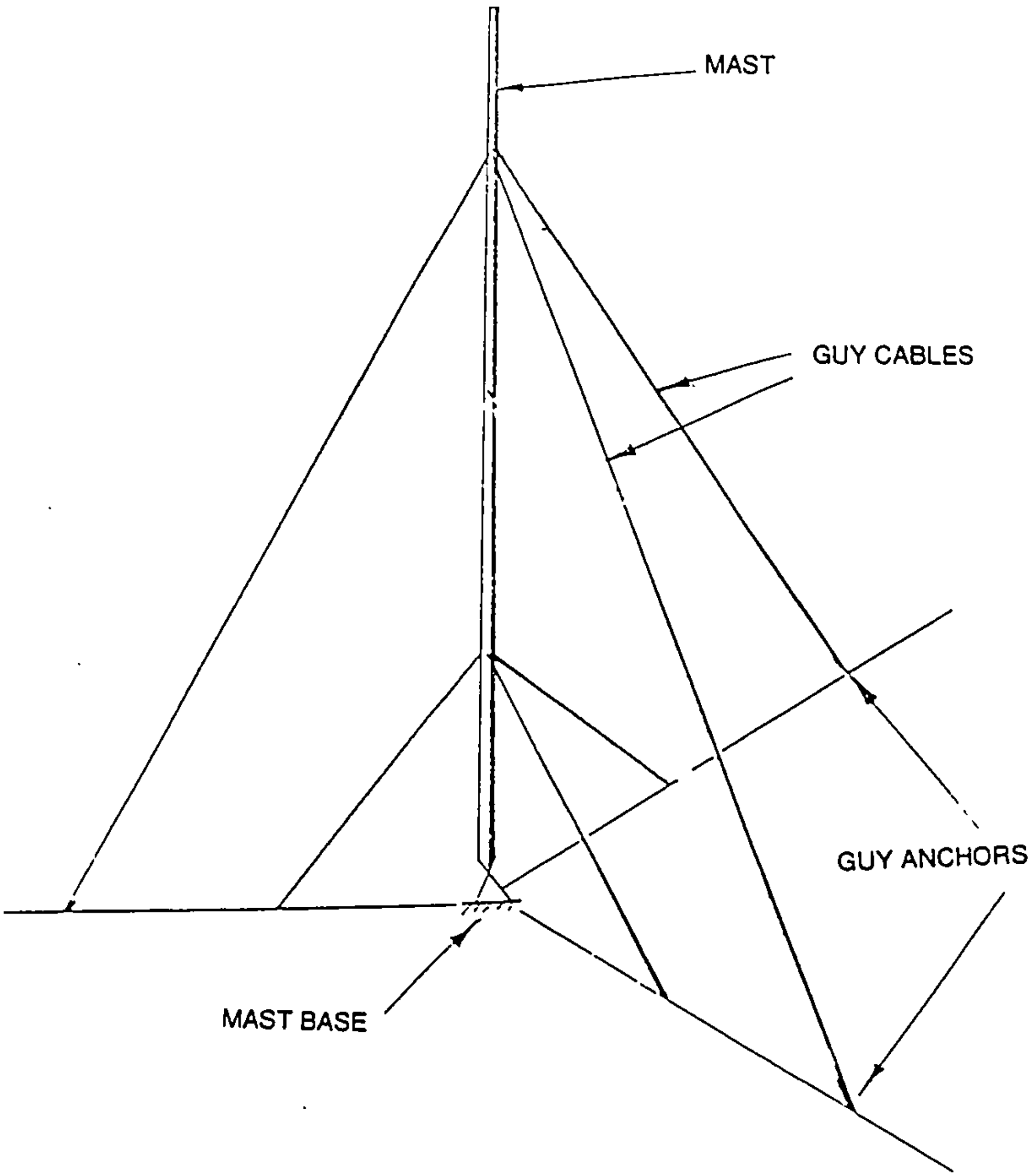


FIG 1.2 : 100 m HIGH GUYED TOWER WITH 2 GUY LEVELS

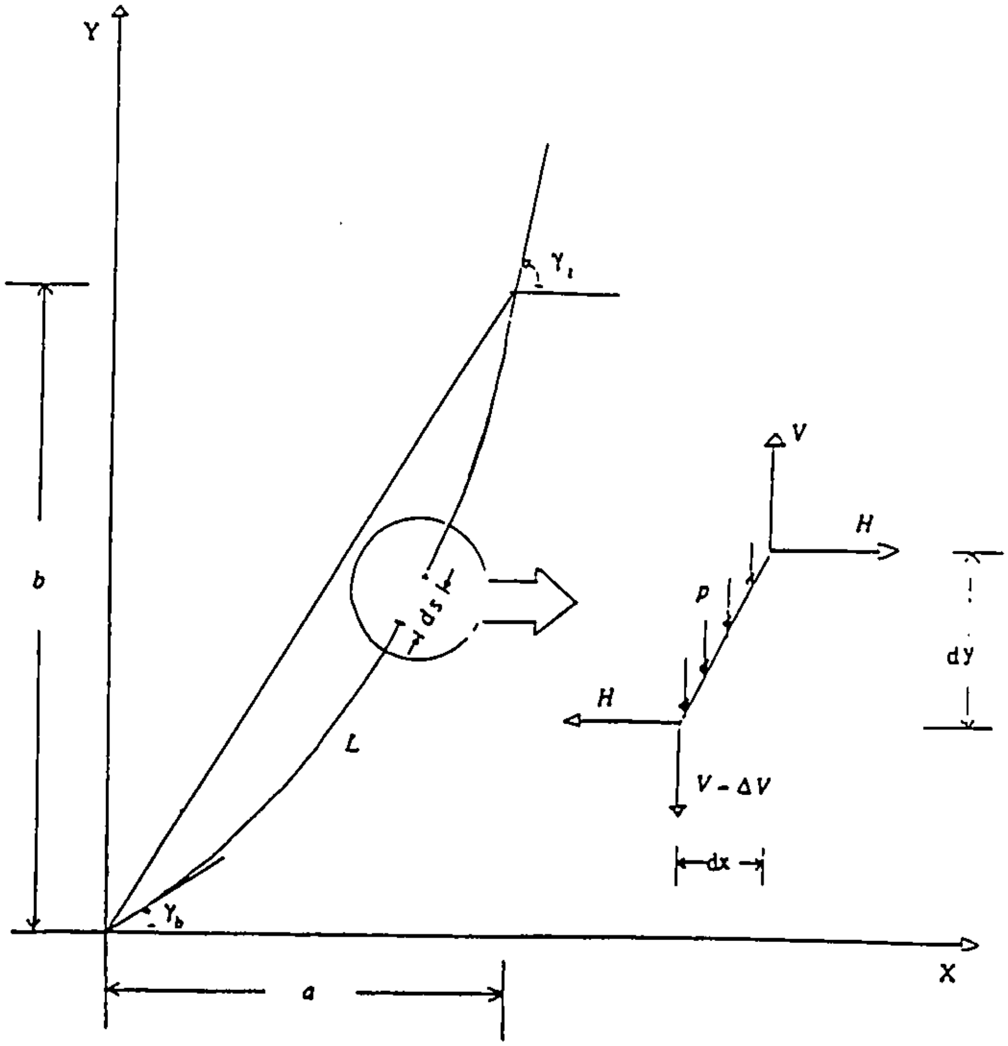


FIG 3.1 : GUY CABLE GEOMETRY AND LOADING

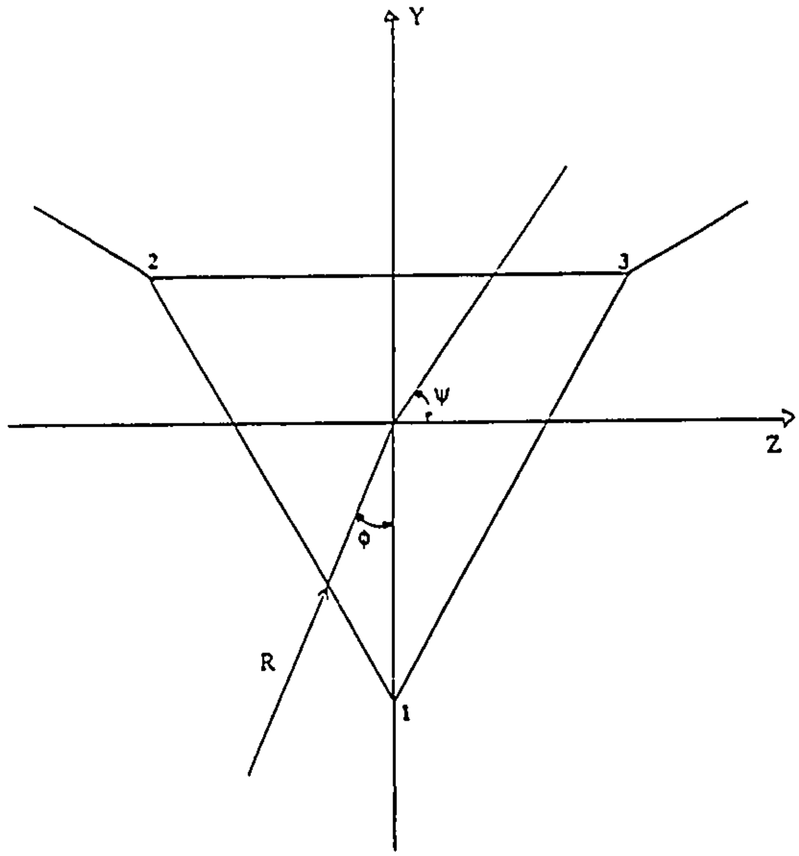


FIG.3.2 TRIANGULAR MAST SUBJECTED TO WIND LOAD AT AN ARBITRARY ANGLE

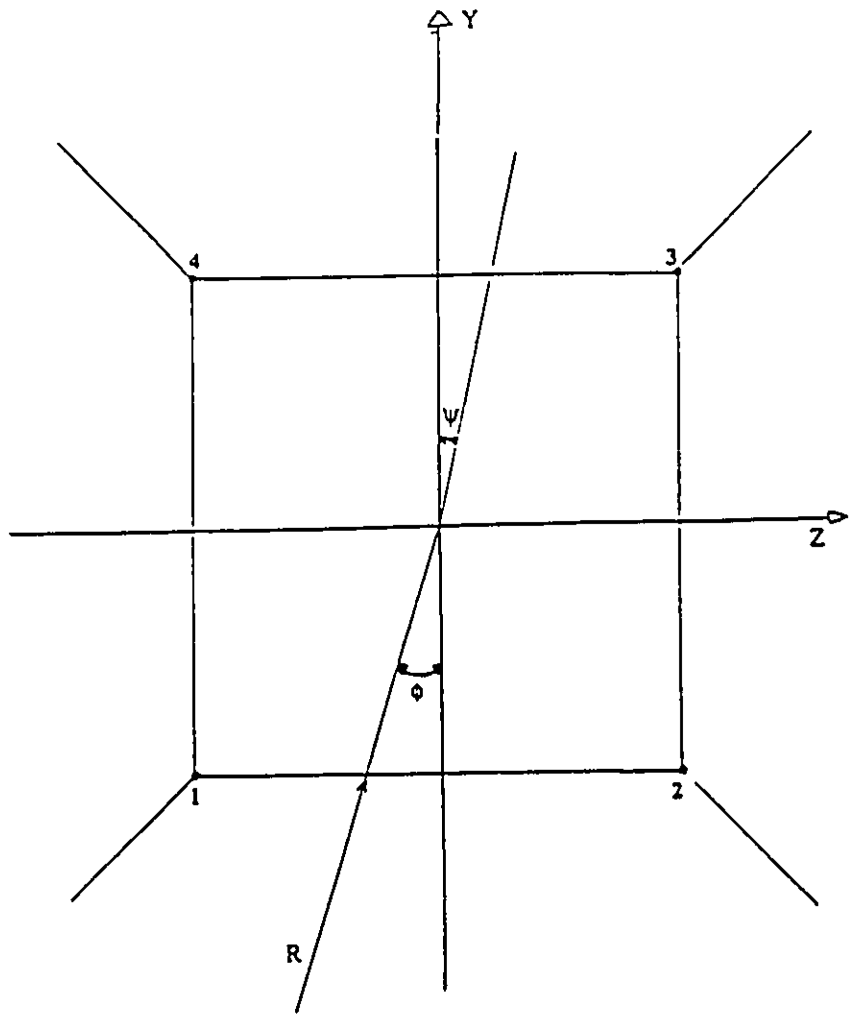


FIG.3.3 SQUARE MAST SUBJECTED TO WIND LOAD AT AN ARBITRARY ANGLE

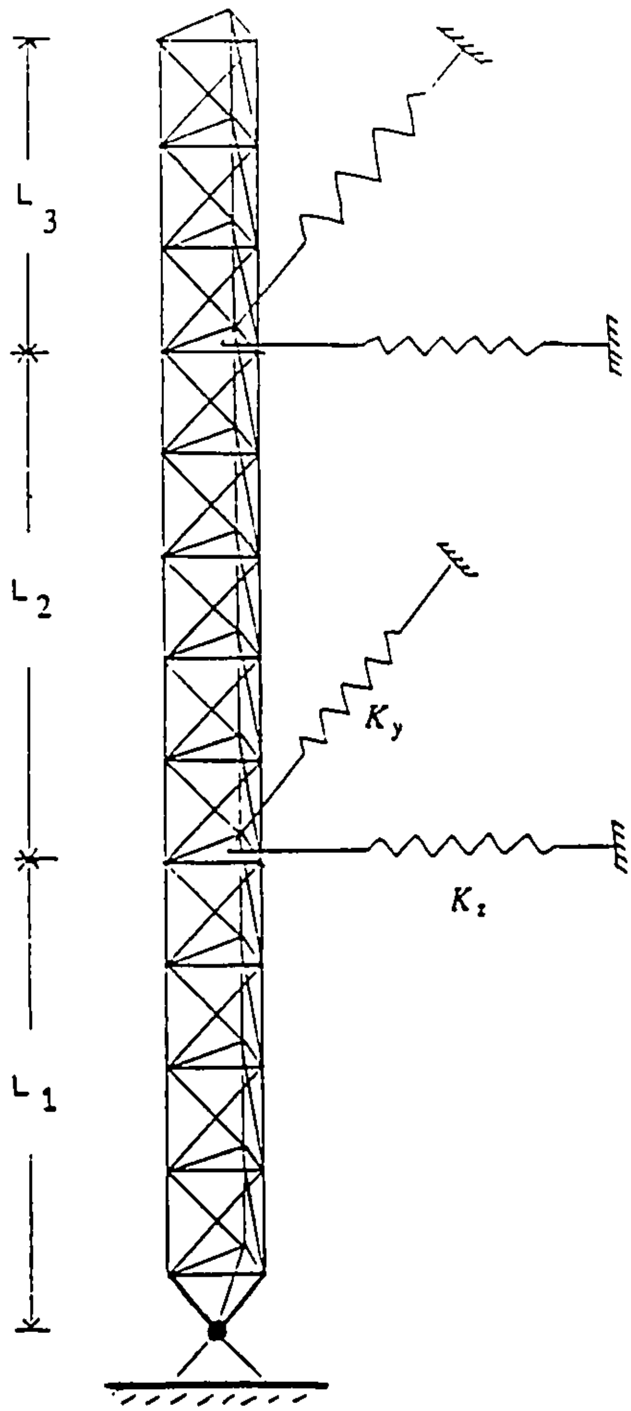


FIG 4.1 : GUYED TOWER-MODELLED AS TRANSLATIONAL SPRINGS

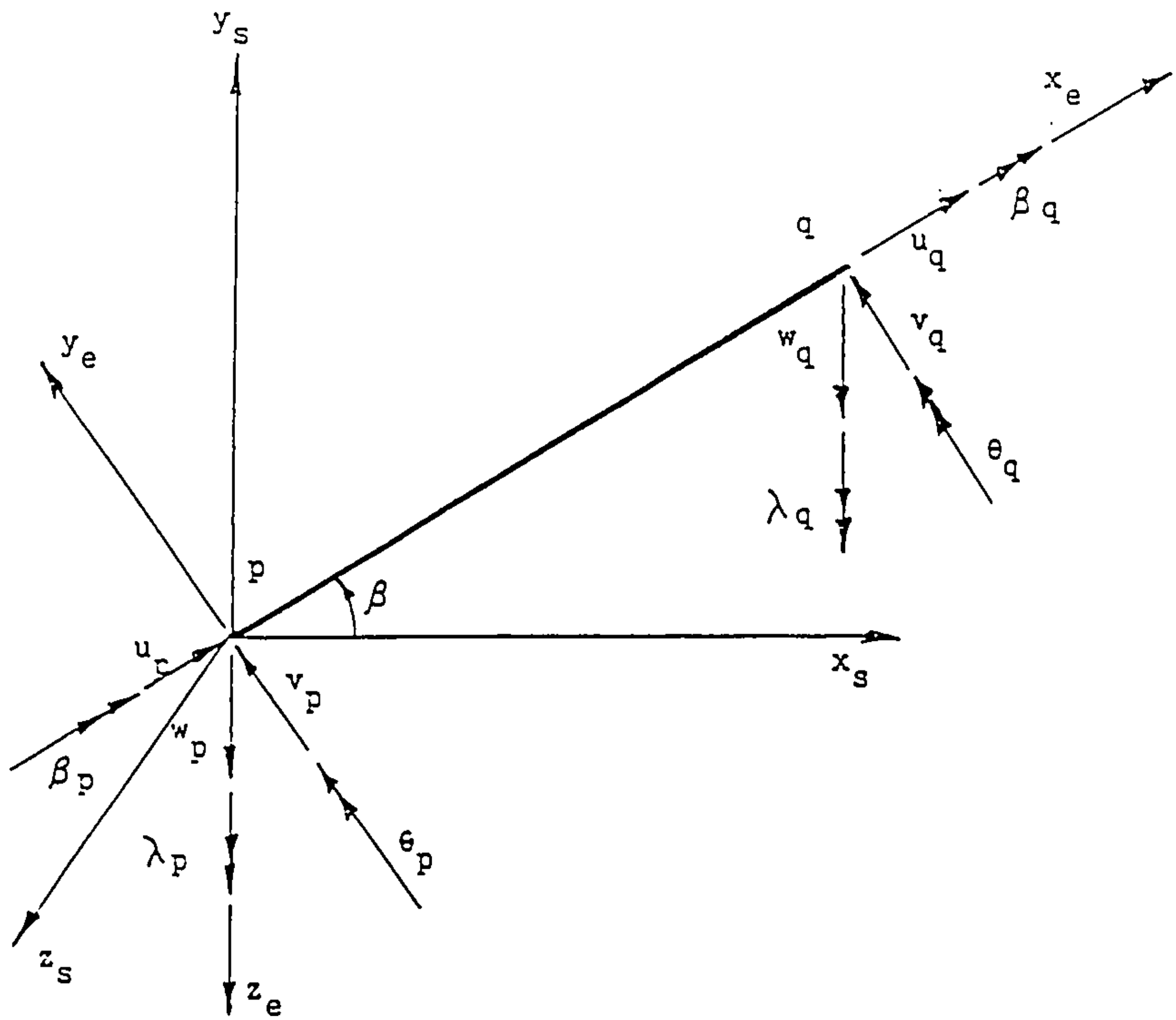
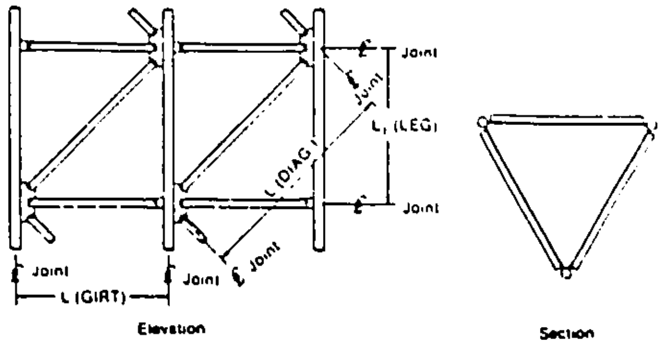
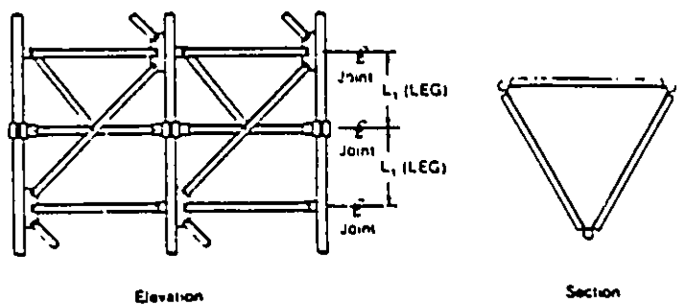


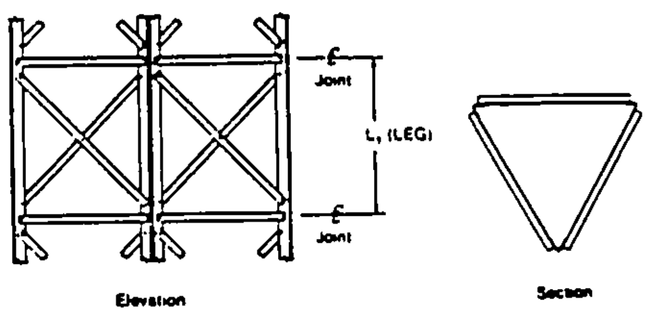
FIG 4.2 : GENERAL ELEMENT DISPLACEMENTS



Web Member System



Web Member System with Additional Girt



Double Web Member System

FIG 4.3 : TYPICAL TOWER SECTIONS

Source

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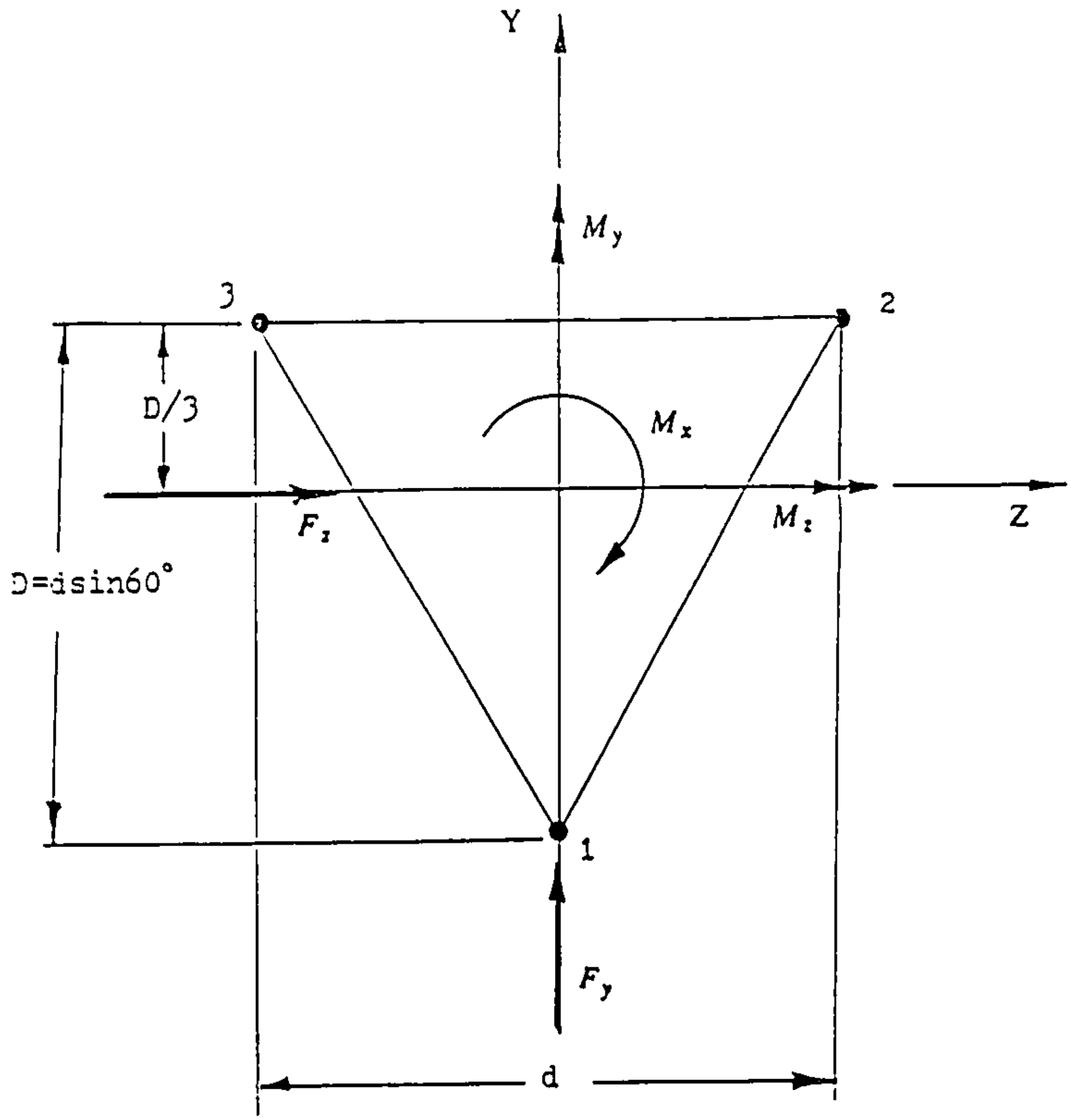


FIG.4.4 TRIANGULAR MAST CROSS SECTION WITH INDUCED FORCES AND MOMENTS

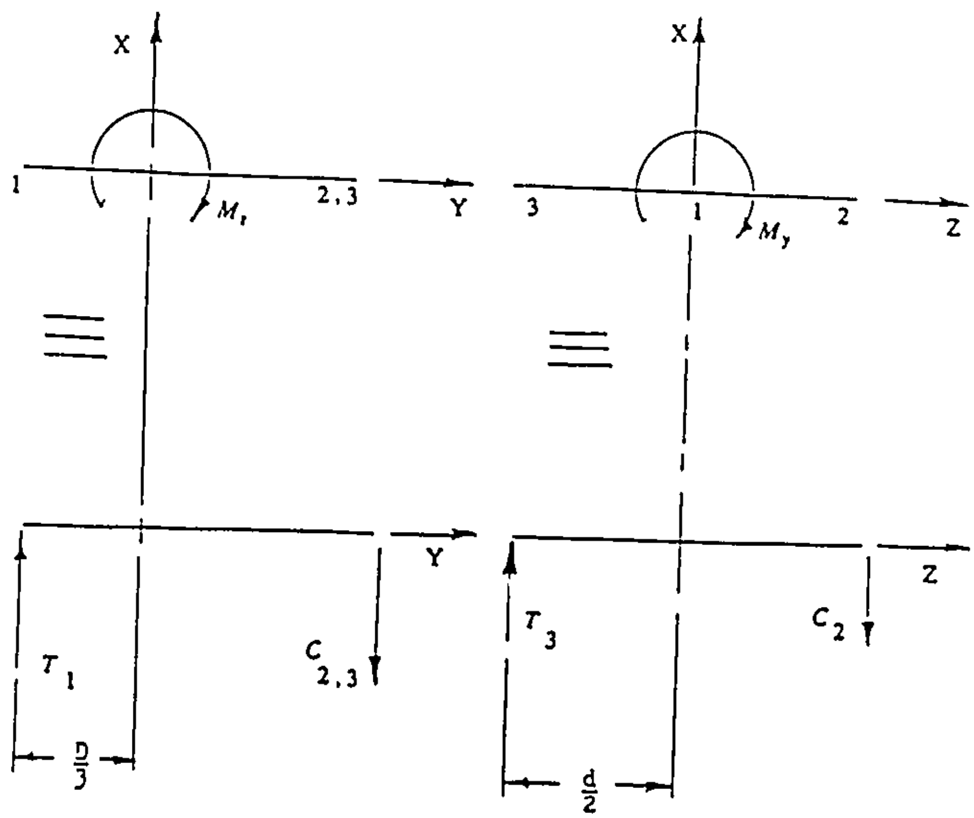


FIG 4.5 : AXIAL FORCES INDUCED IN LEG MEMBERS
BY MOMENTS My AND Mz FOR A TRIANGULAR MAST

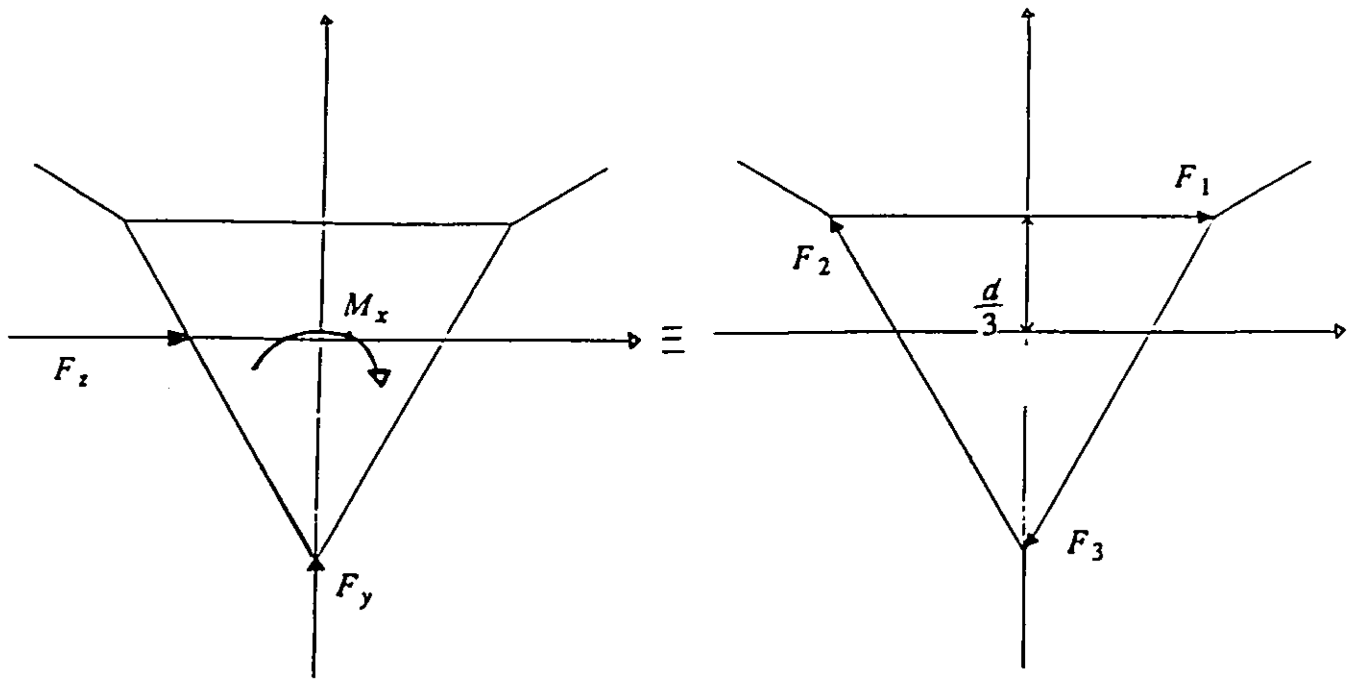


FIG 4.6 : AXIAL FORCES INDUCED IN WEB MEMBERS

BY MOMENT Mx AND SHEAR FORCES Fy & Fz FOR A TRIANGULAR MAST

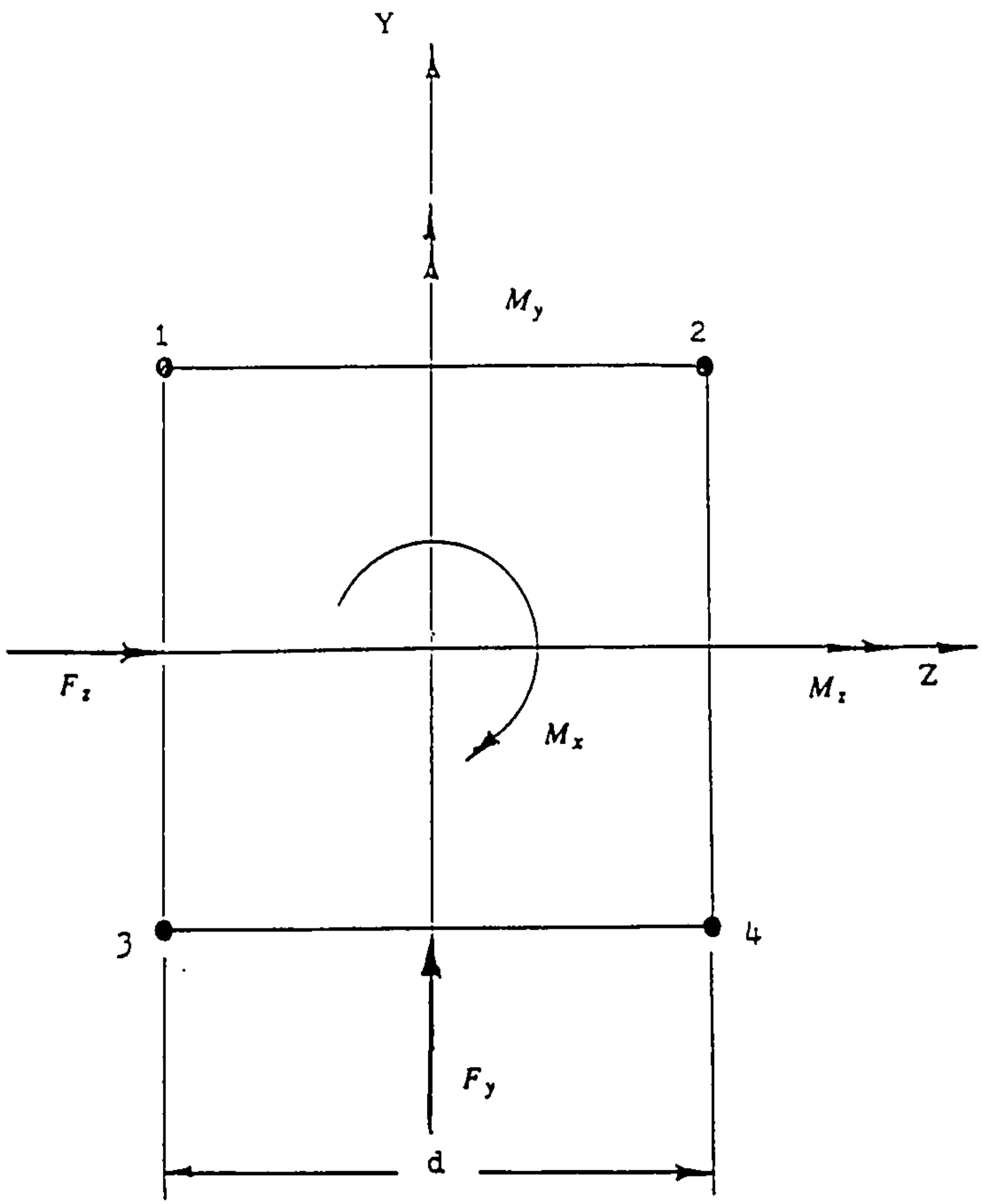


FIG 4.7 : SQUARE MAST CROSS SECTION WITH INDUCED FORCES AND MOMENTS

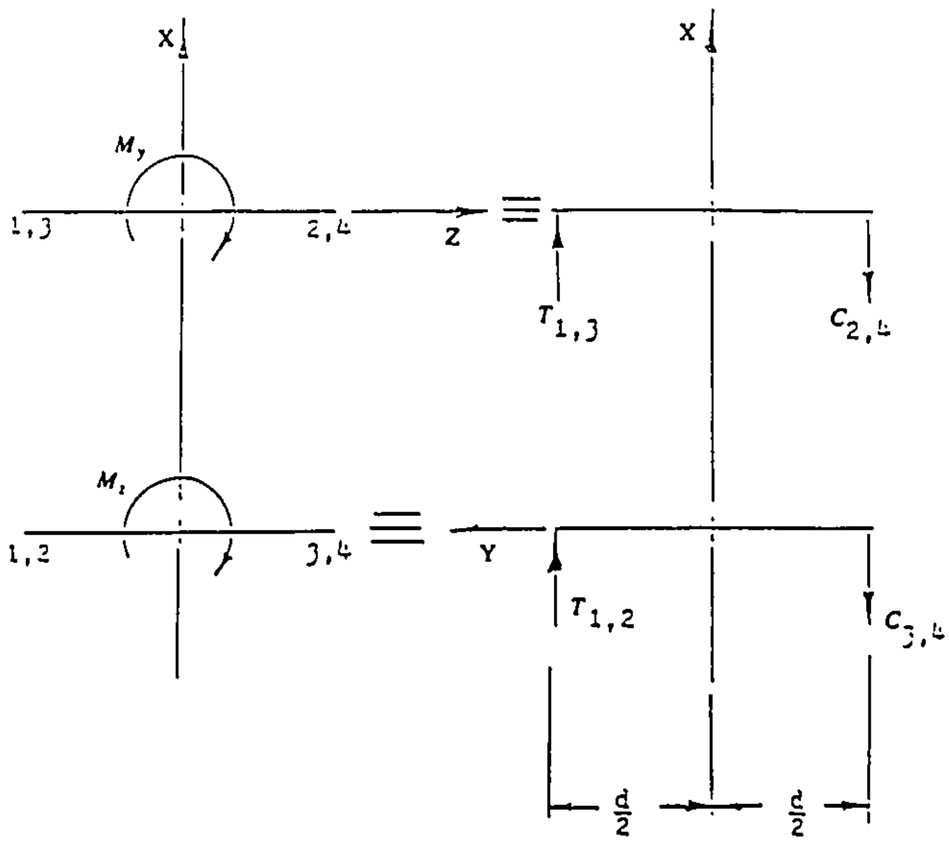


FIG 4.8 : AXIAL FORCES INDUCED IN LEG MEMBERS
BY MOMENTS My AND Mz FOR A SQUARE MAST

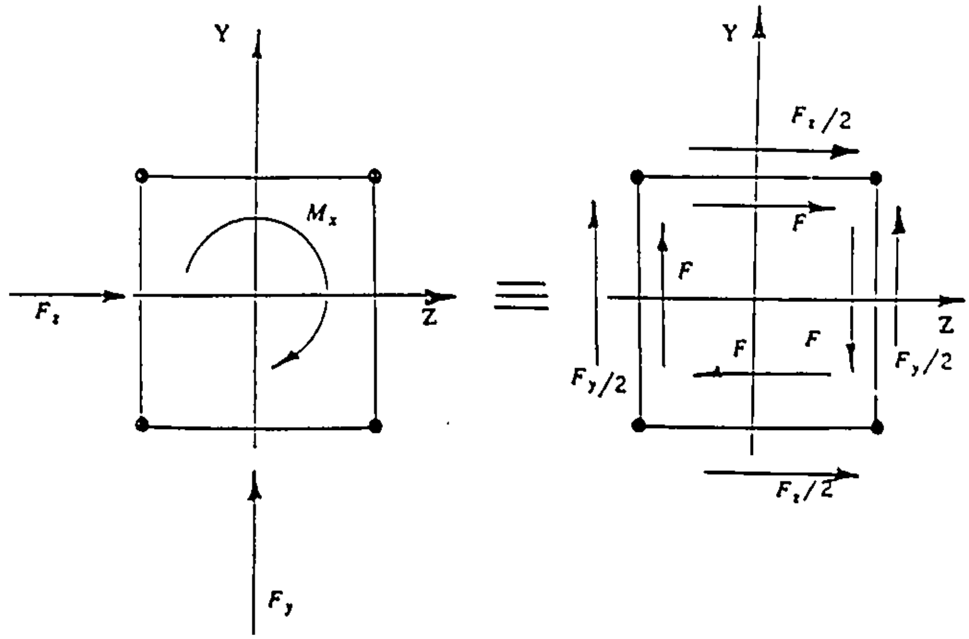
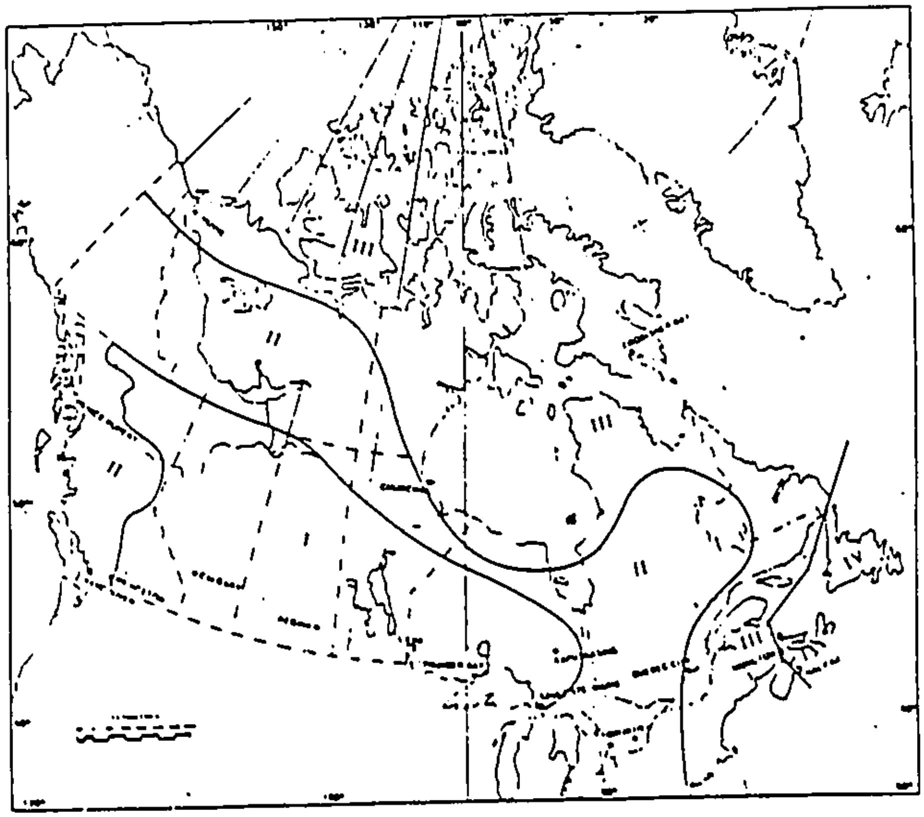


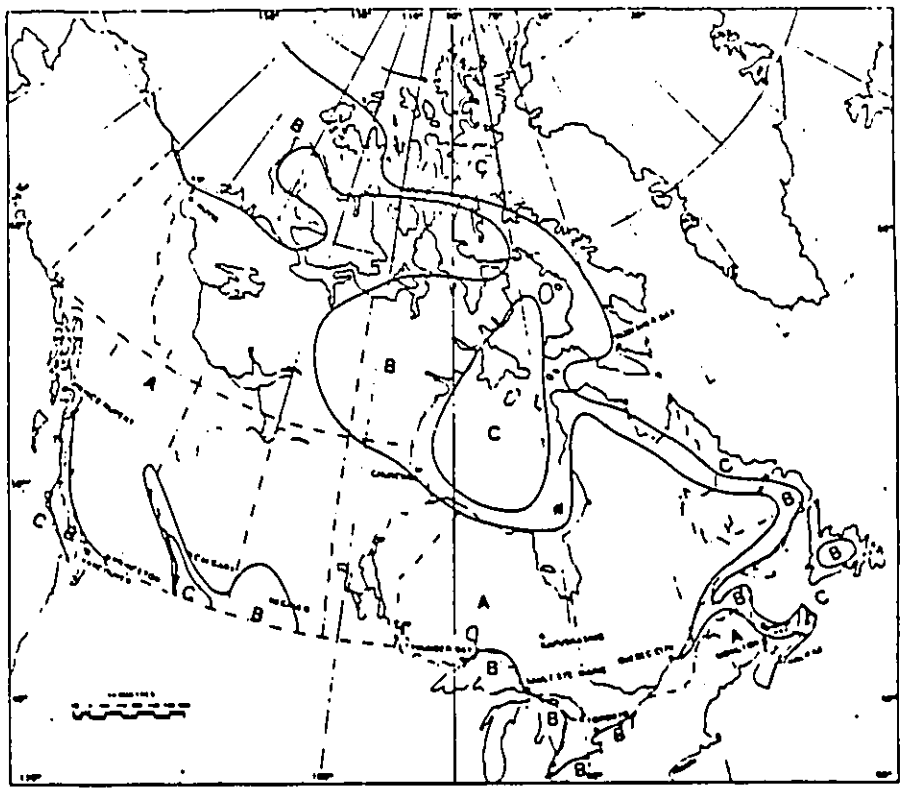
FIG 4.9 : AXIAL FORCES INDUCED IN WEB MEMBERS
BY MOMENTS Mx AND SHEAR FORCES Fy & Fz FOR A SQUARE MAST



Class	Nominal ice thickness, mm
I	10
II	25
III	40
IV	50

FIG 5.1 : ICE MAP OF CANADA

(Source: Environment Canada)



Class	Reference velocity pressure (q), Pa
A	450
B	600
C	850

FIG.5.2 WIND MAP OF CANADA

(Source: Environment Canada)

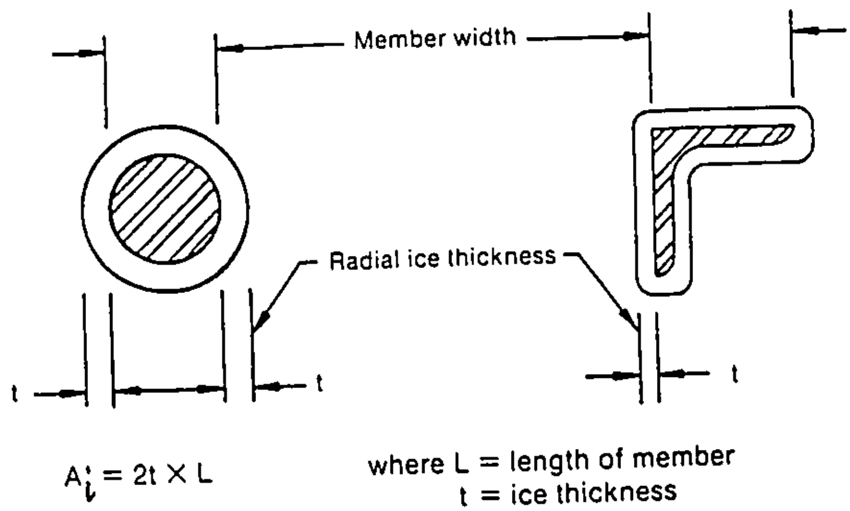


FIG 5.3 : FACE AREA OF RADIAL ICE

Source

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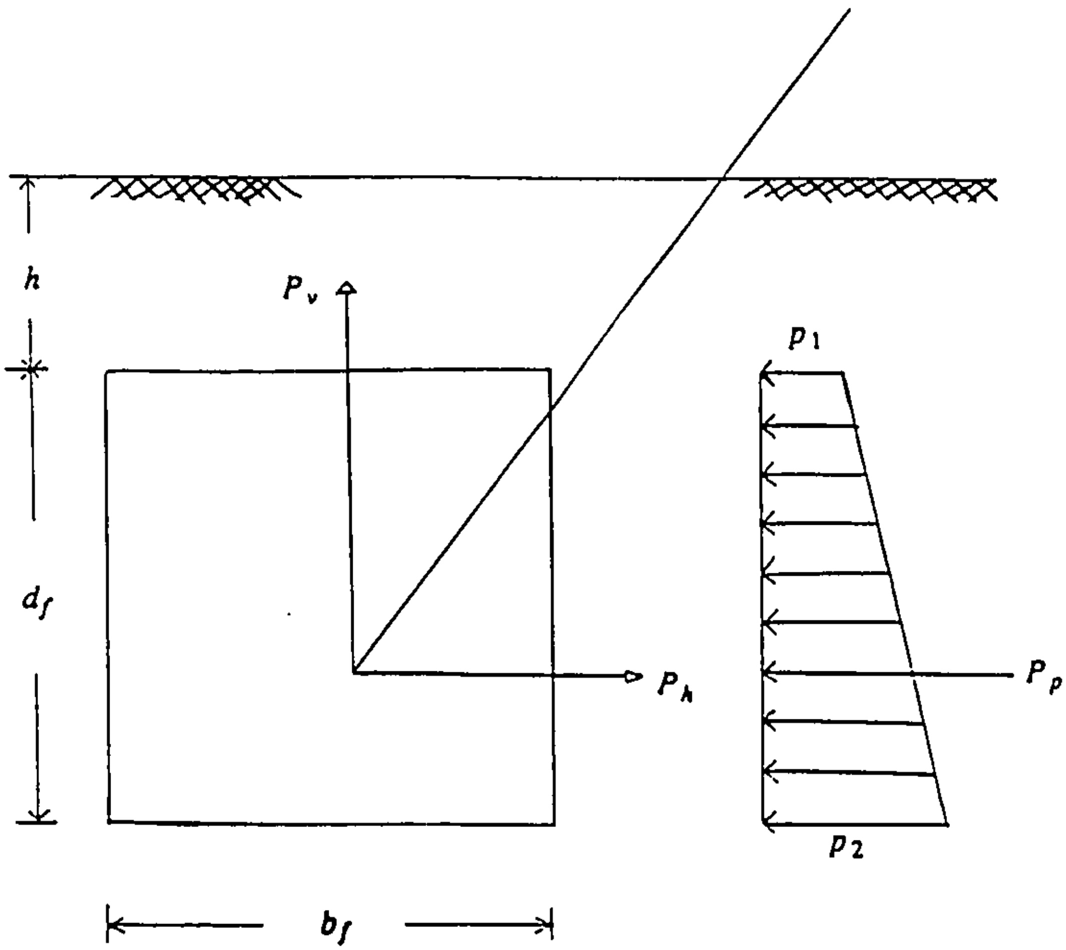

FIG.5.4 DEADMAN ANCHORAGE : FORCES ACTING ON THE ANCHORAGE BLOCK

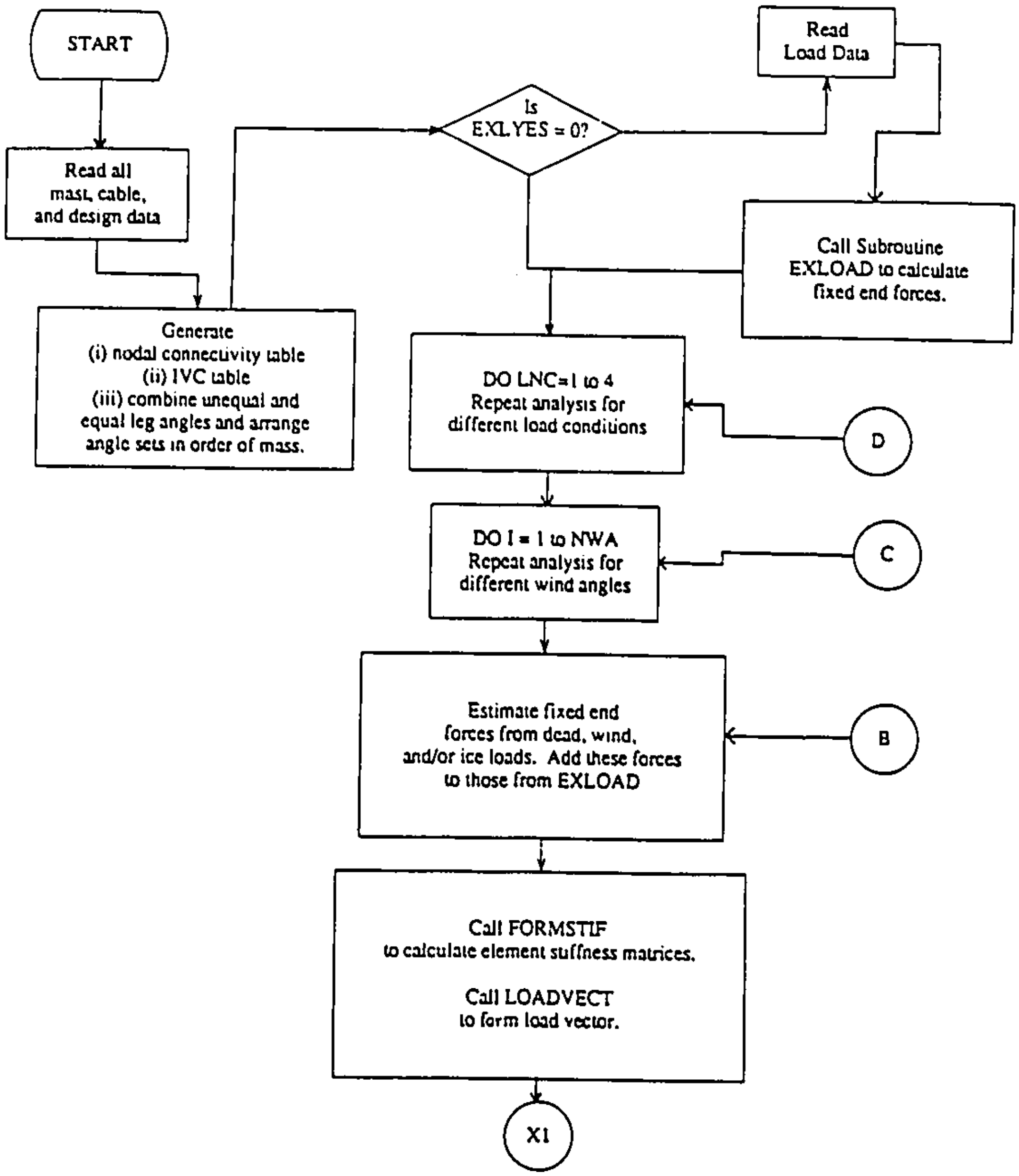

FIG.6.1a FLOW CHART FOR MAIN PROGRAM

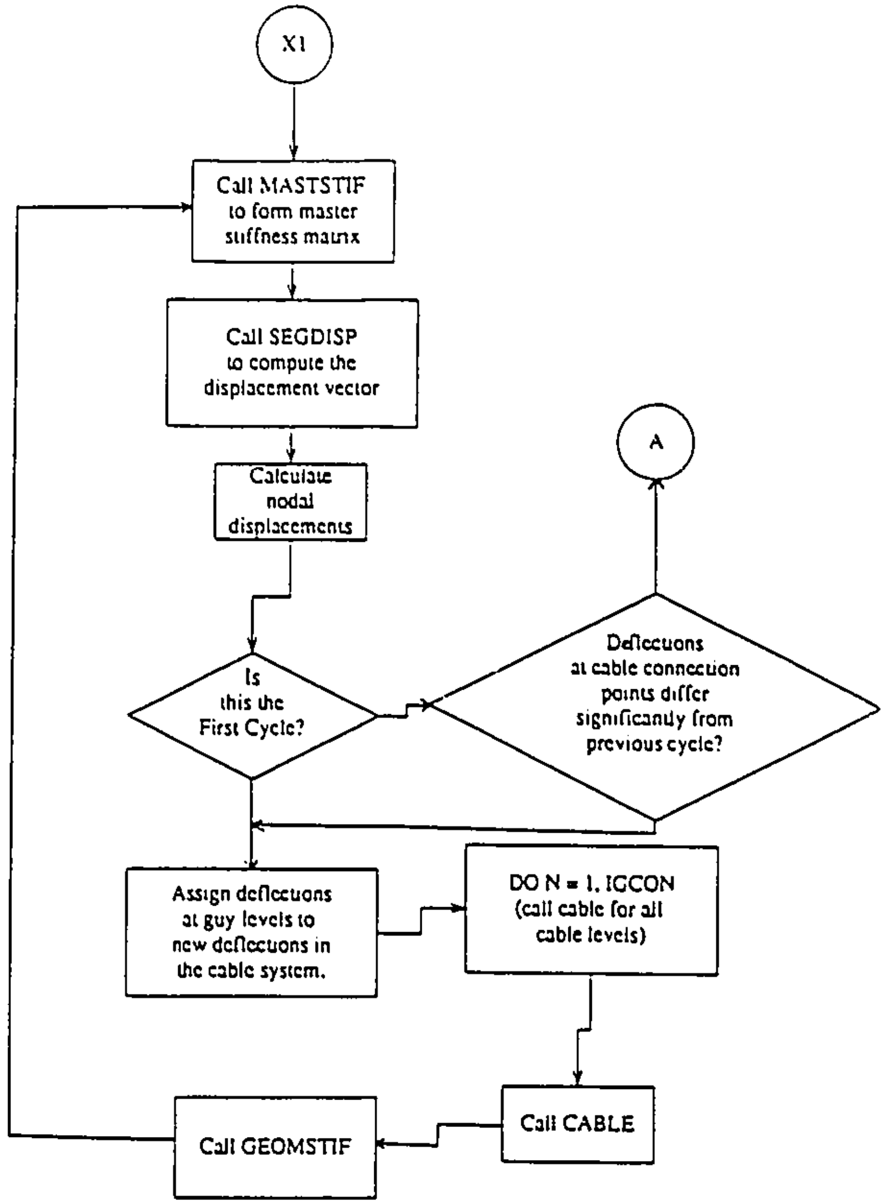

FIG.6.1b FLOW CHART FOR MAIN PROGRAM (cont'd.)

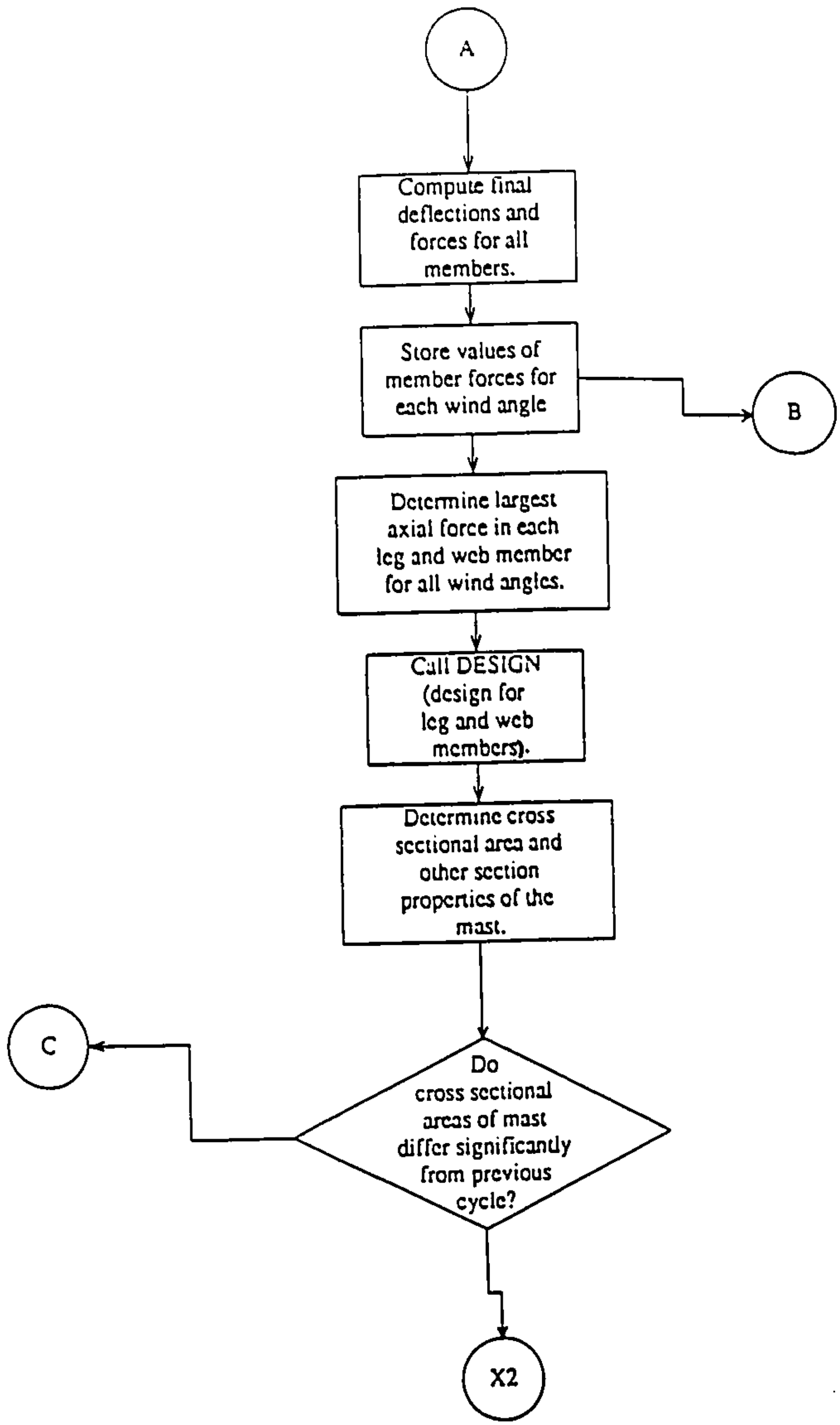

FIG.6.1c FLOW CHART FOR MAIN PROGRAM (cont'd.)

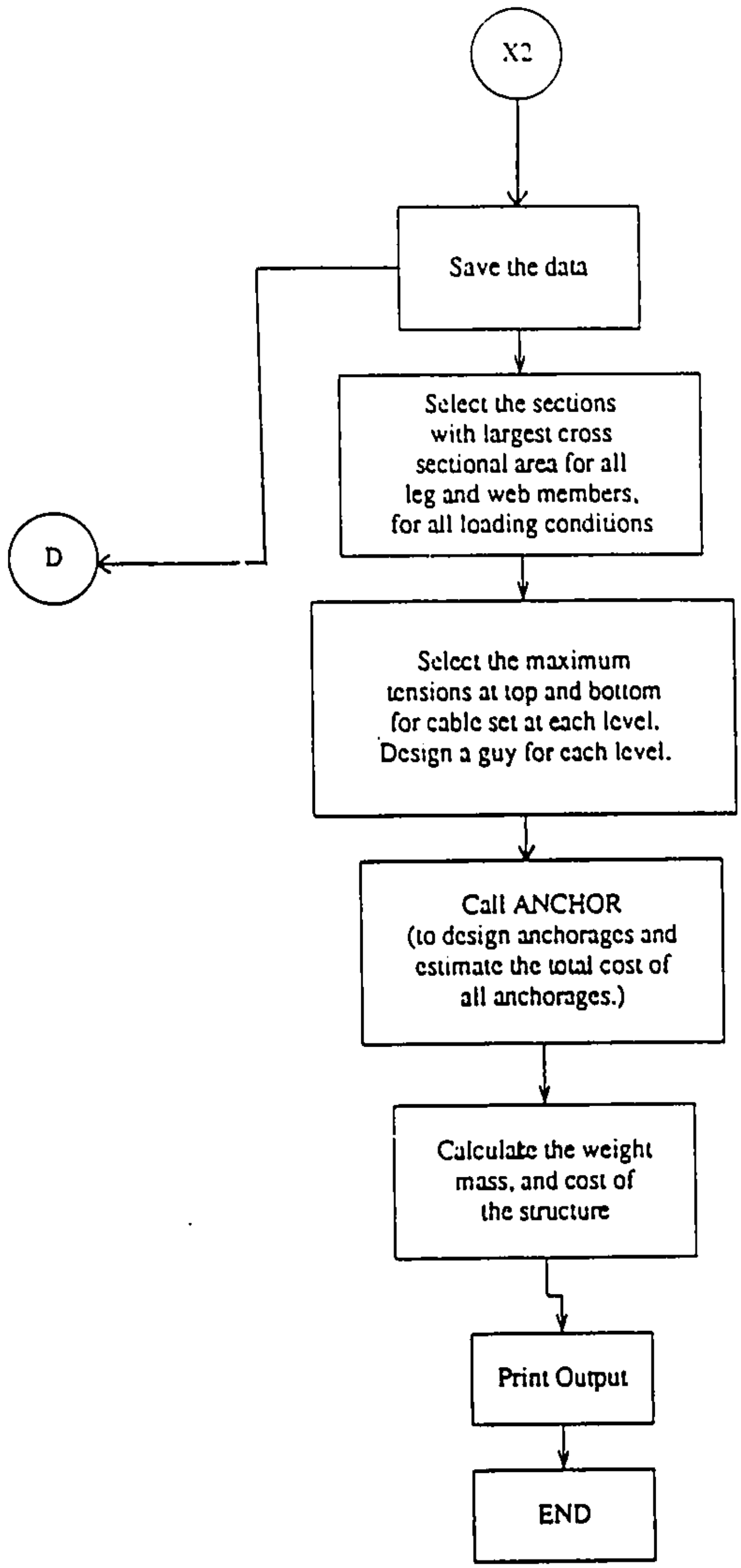

FIG.6.1d FLOW CHART FOR MAIN PROGRAM (cont'd.)

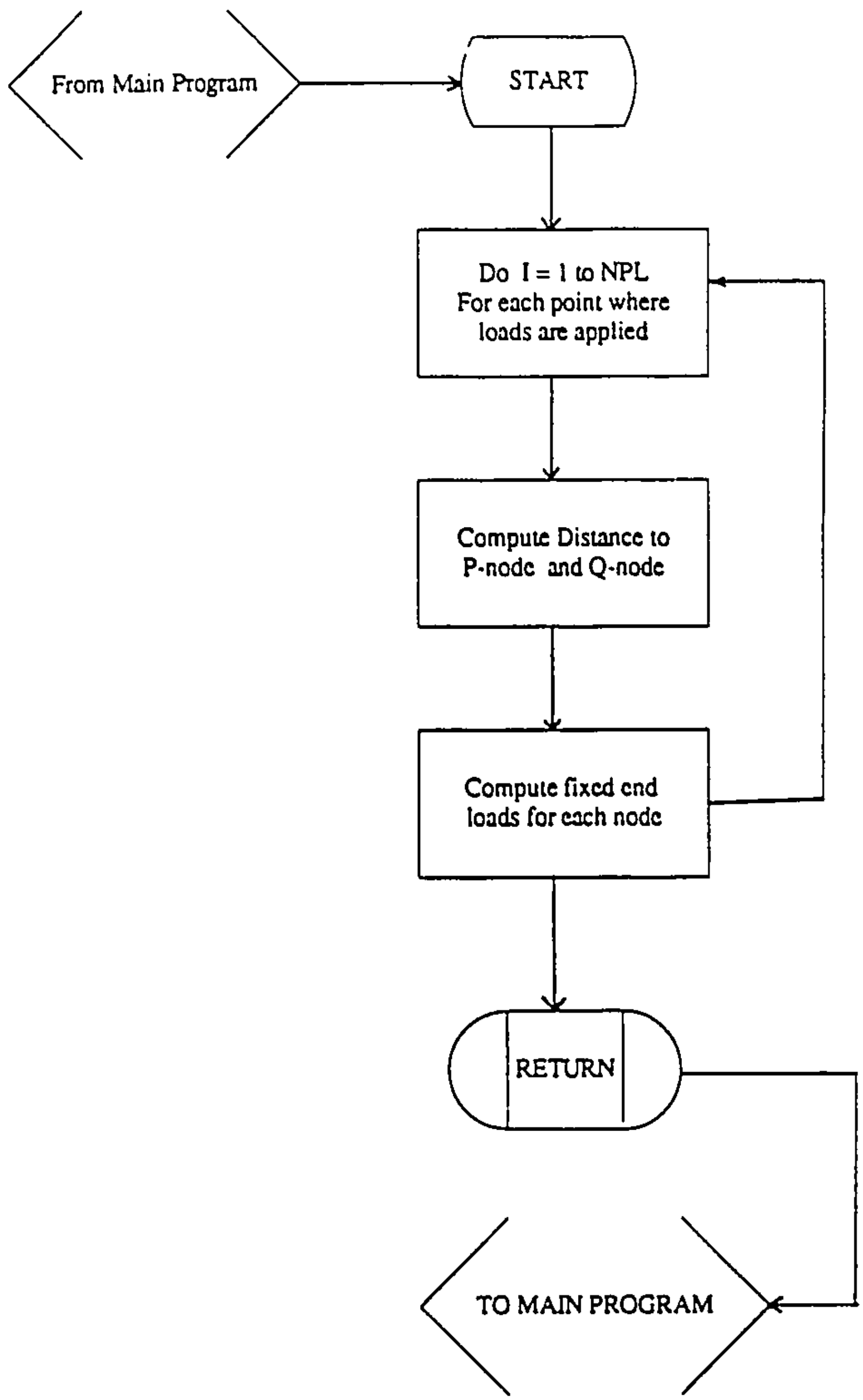

FIG.6.2 FLOW CHART FOR SUBROUTINE EXLOAD

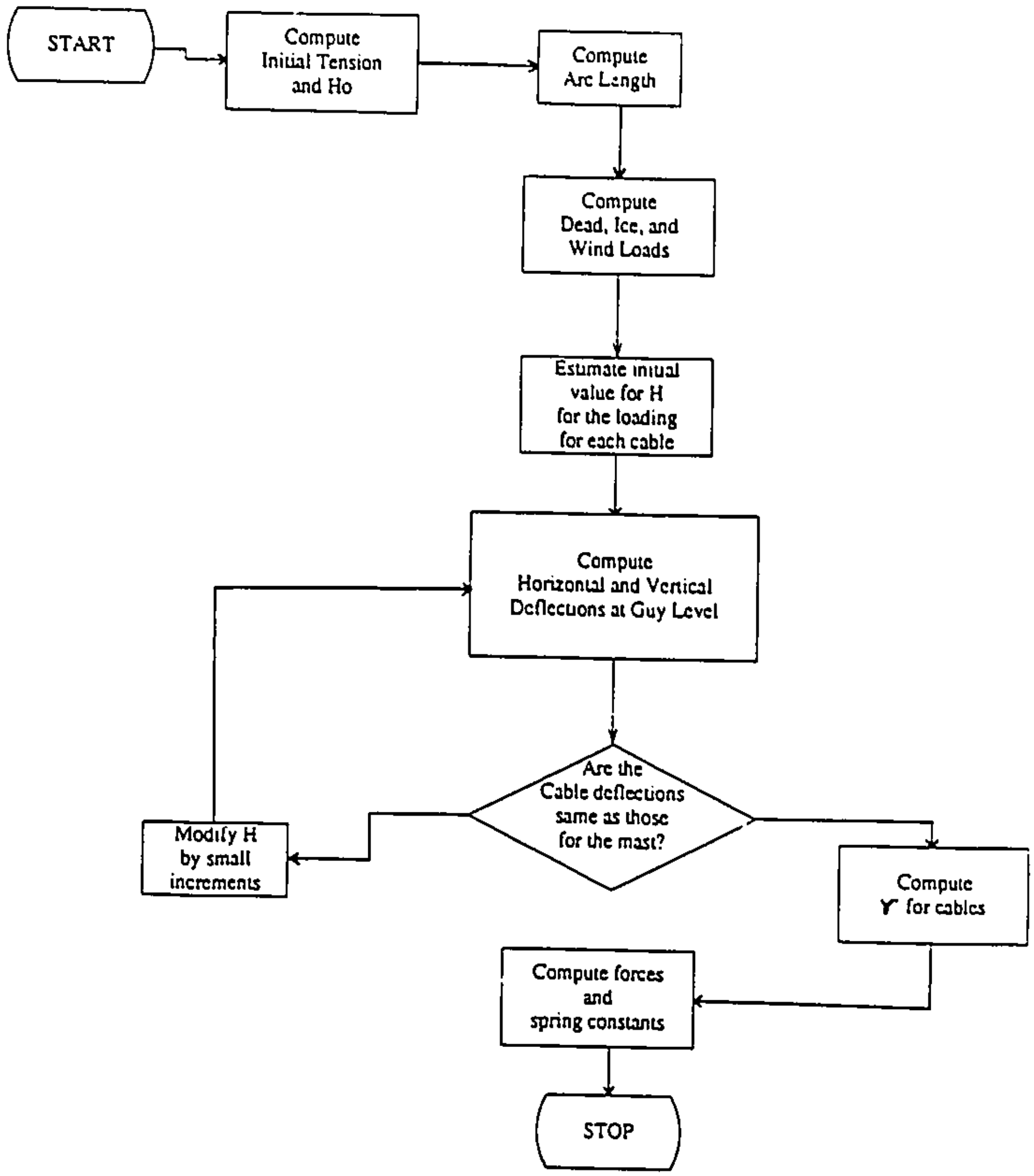

FIG.6.3 FLOW CHART FOR SUBROUTINE CABLE

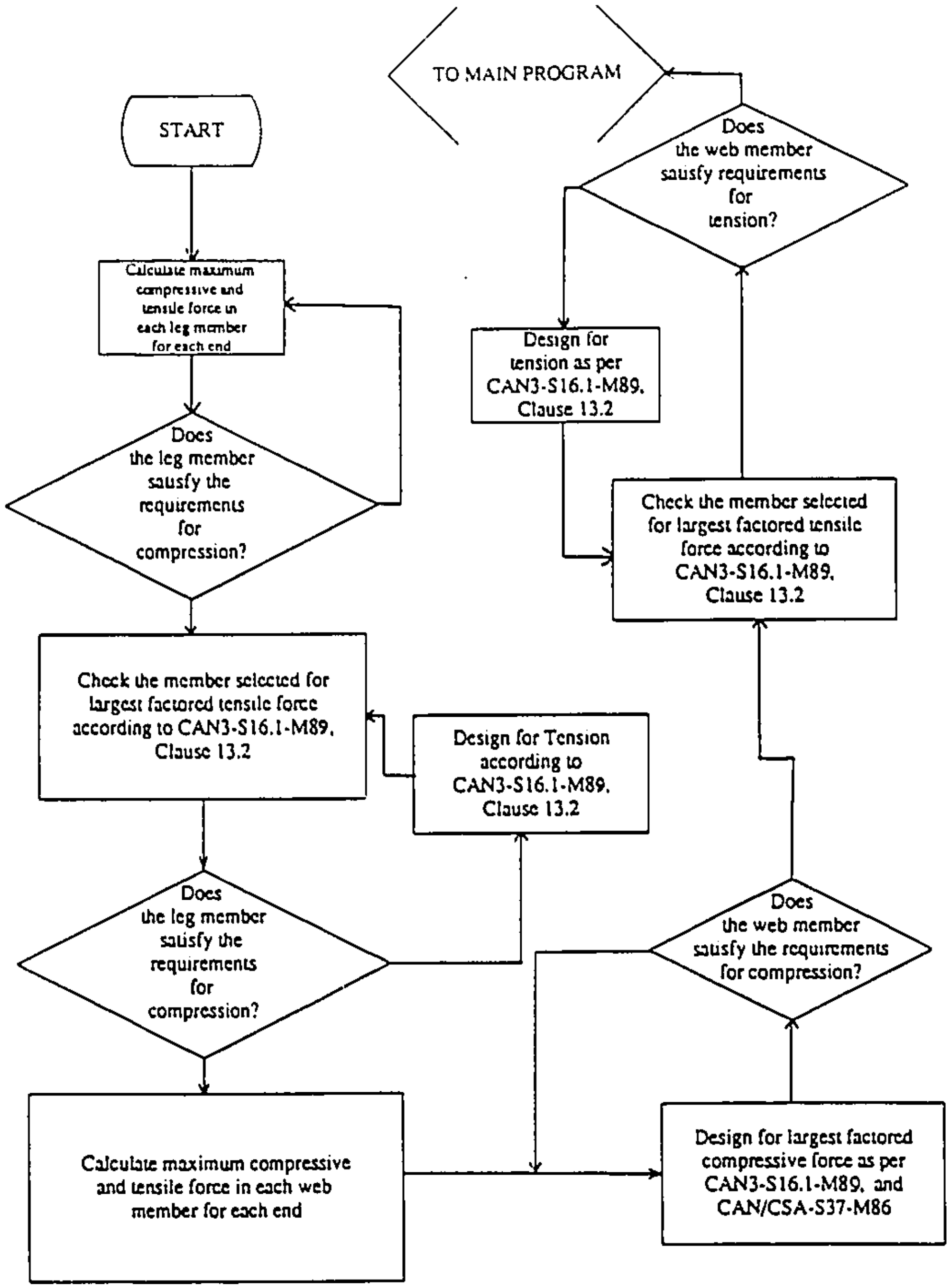

FIG.6.4 FLOW CHART FOR SUBROUTINE DESIGN

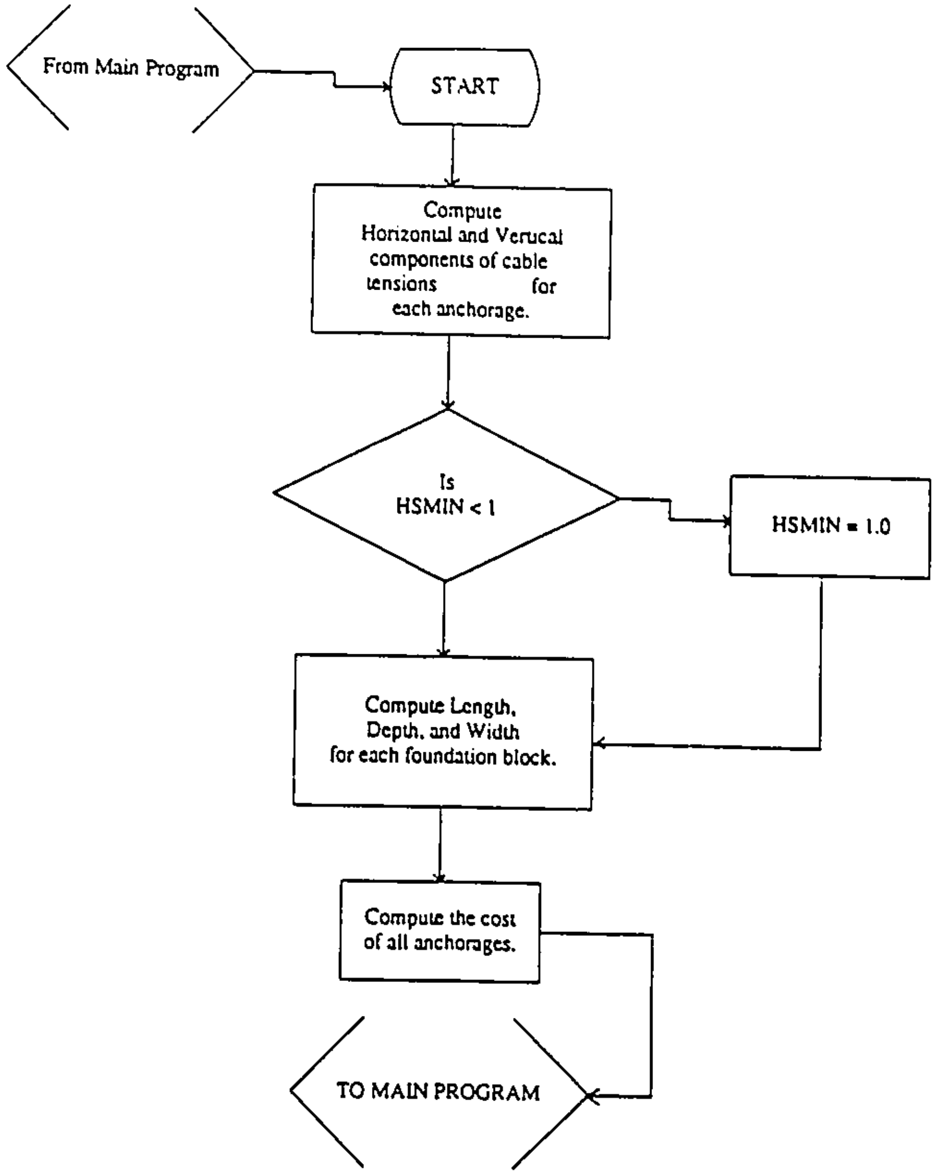

FIG.6.5 FLOW CHART FOR SUBROUTINE ANCHOR

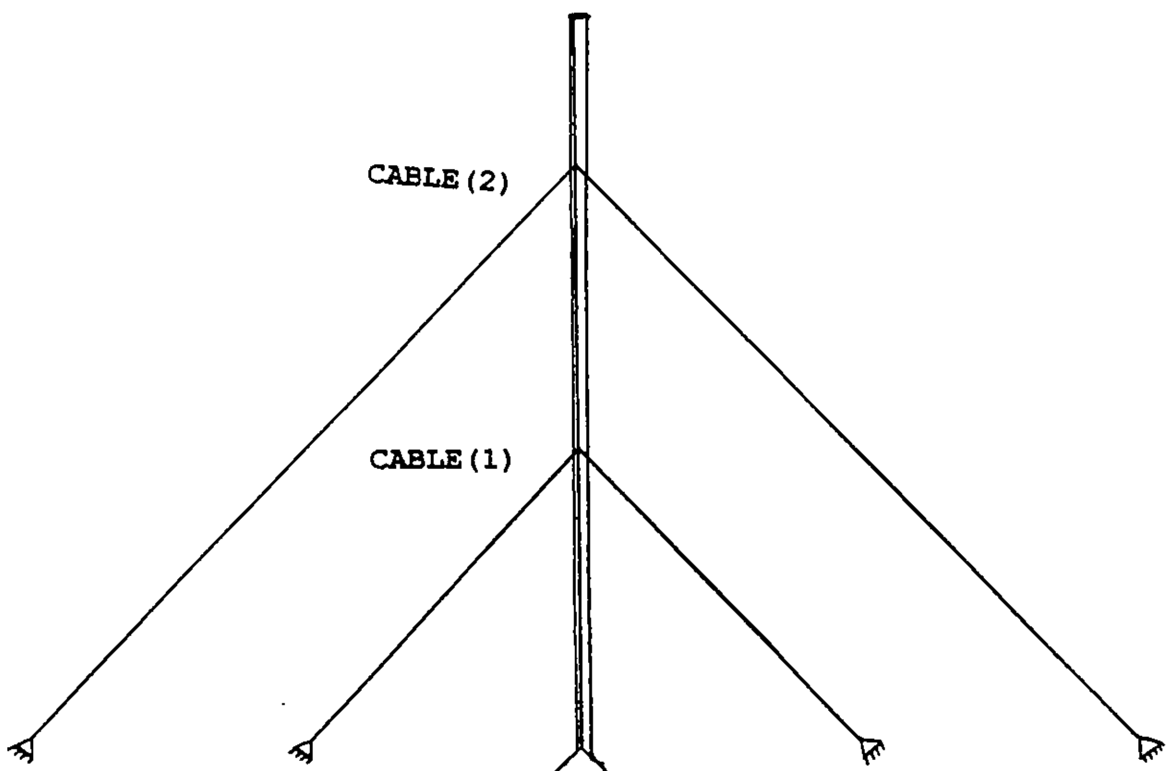

FIG 7.1 : 100 m HIGH GUYED TOWER WITH 2 GUY LEVELS

MAST SIZE : 2.0m ; MAST HEIGHT : 100m
ALL CABLE CHORDS ARE AT 45 DEGREES INCLINATION
ALL CABLE CONNECTION POINTS ARE EQUALLY SPACED
WEB INCLINATION = 45 DEGREES
LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

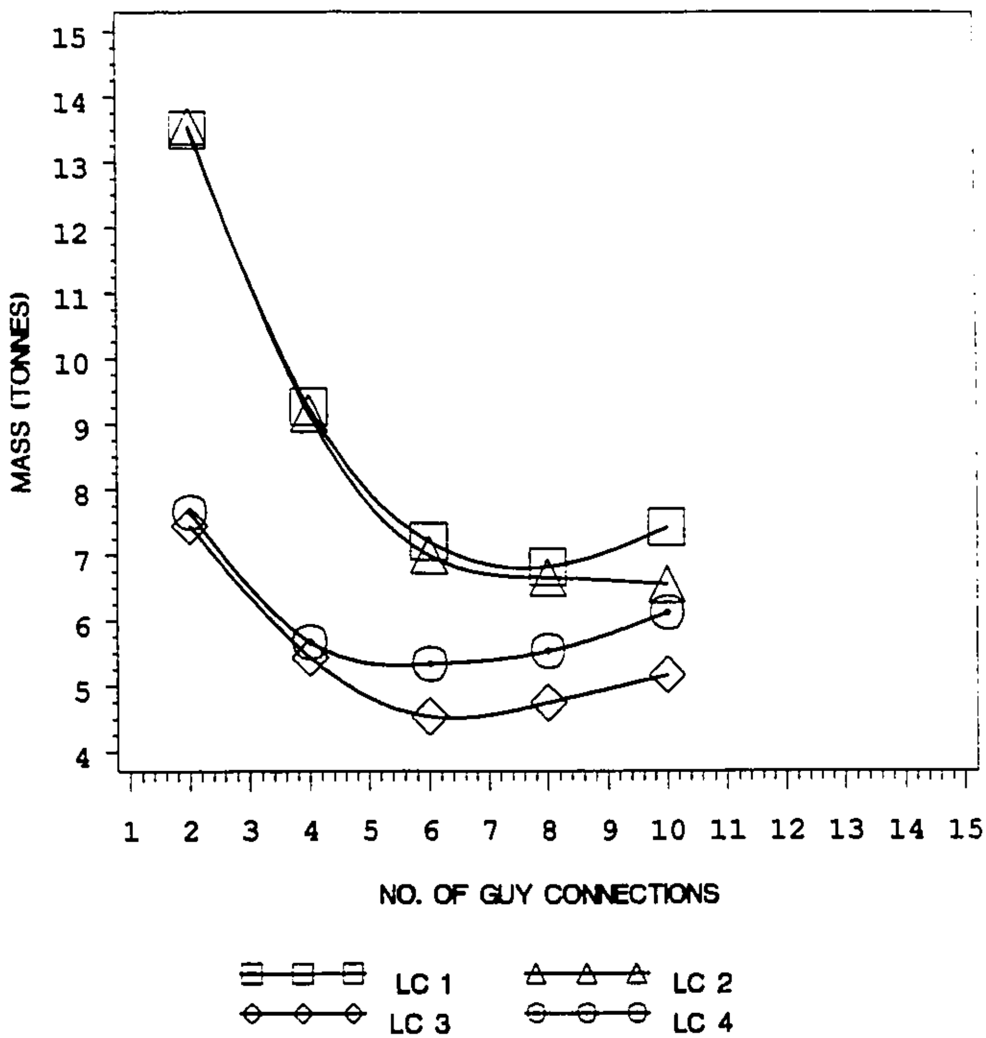

FIG 7.2 : MASS vs NO. OF GUY CONNECTIONS

MAST SIZE : 2.0m ; MAST HEIGHT : 100m
ALL CABLE CHORDS ARE AT 45 DEGREES INCLINATION
ALL CABLE CONNECTION POINTS ARE EQUALLY SPACED
WEB INCLINATION = 45 DEGREES
LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

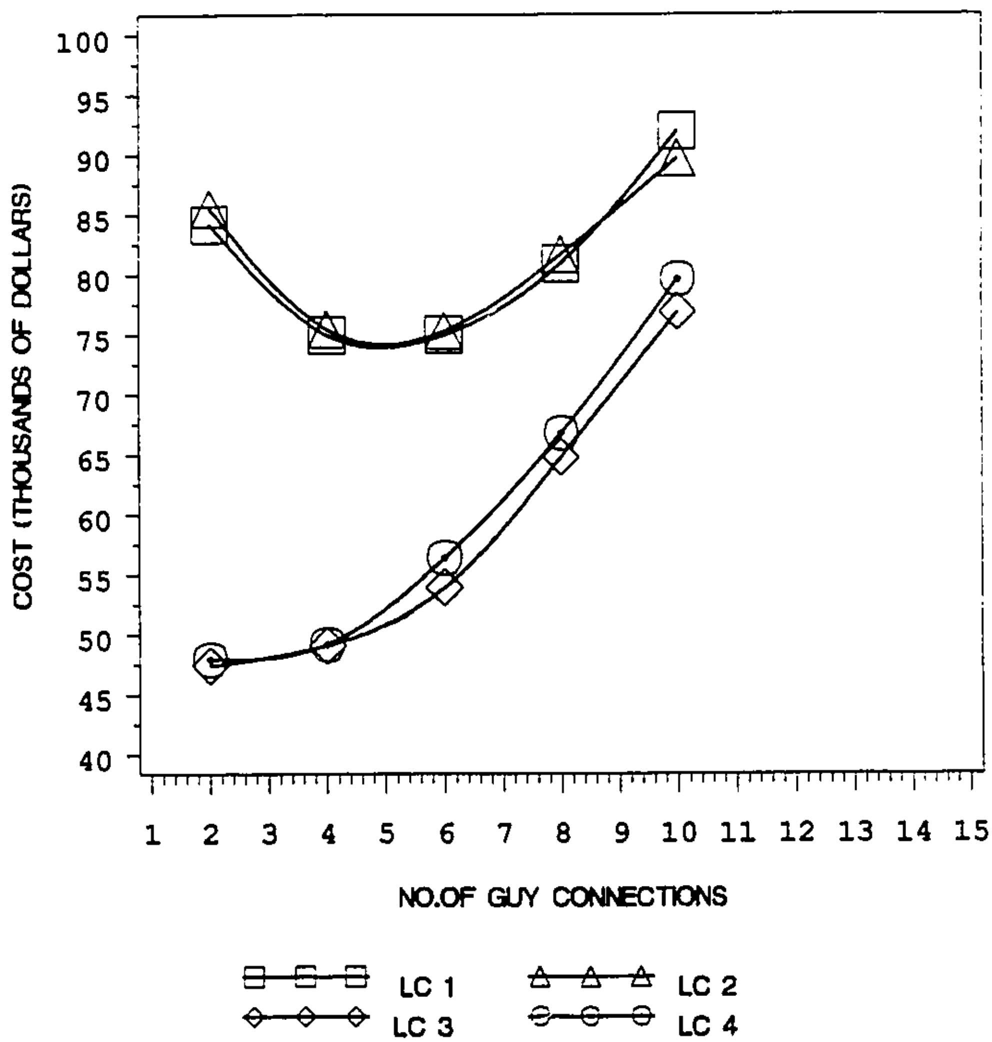

FIG 7.3 : COST vs NO. OF GUY CONNECTIONS

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

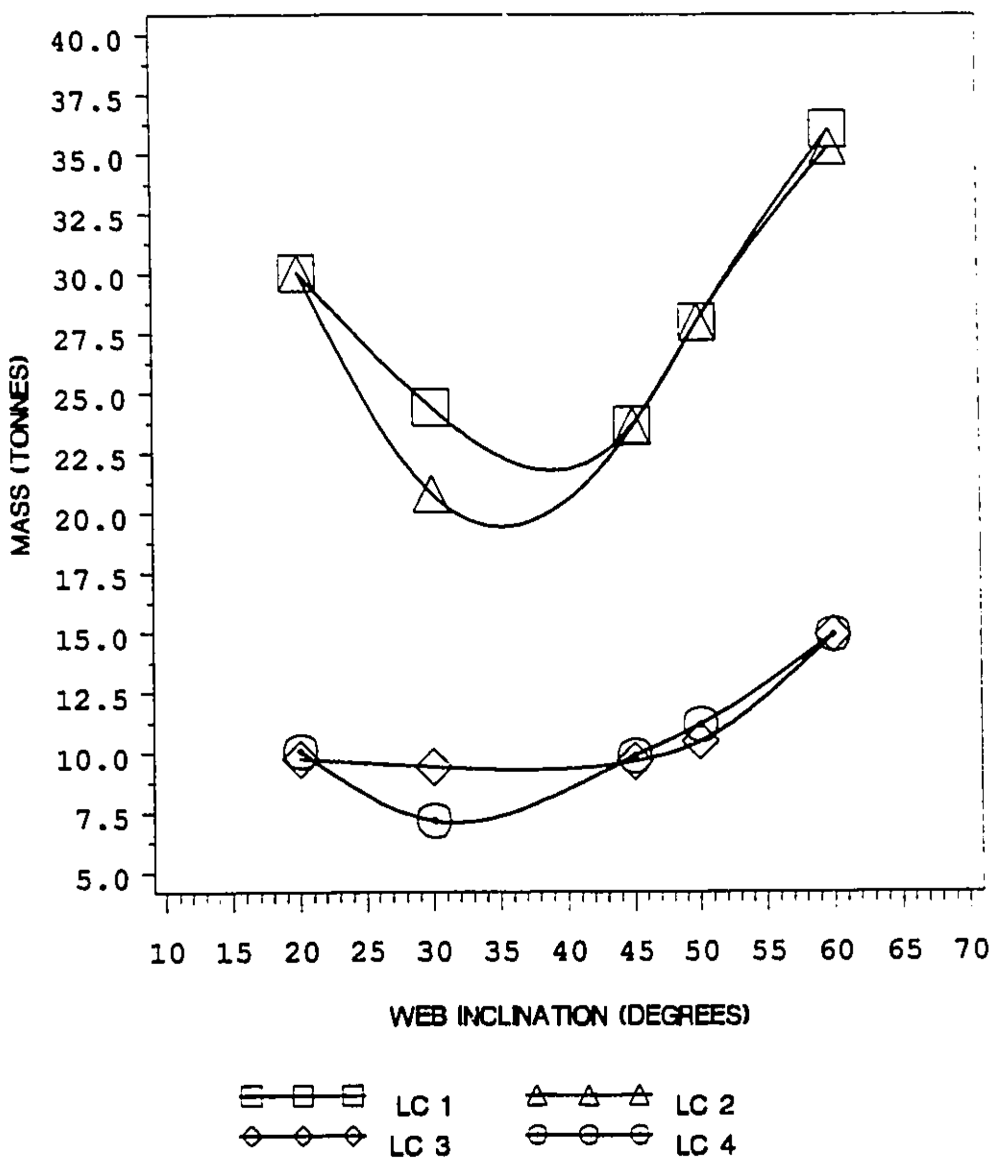

FIG 7.4 : MASS vs WEB INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

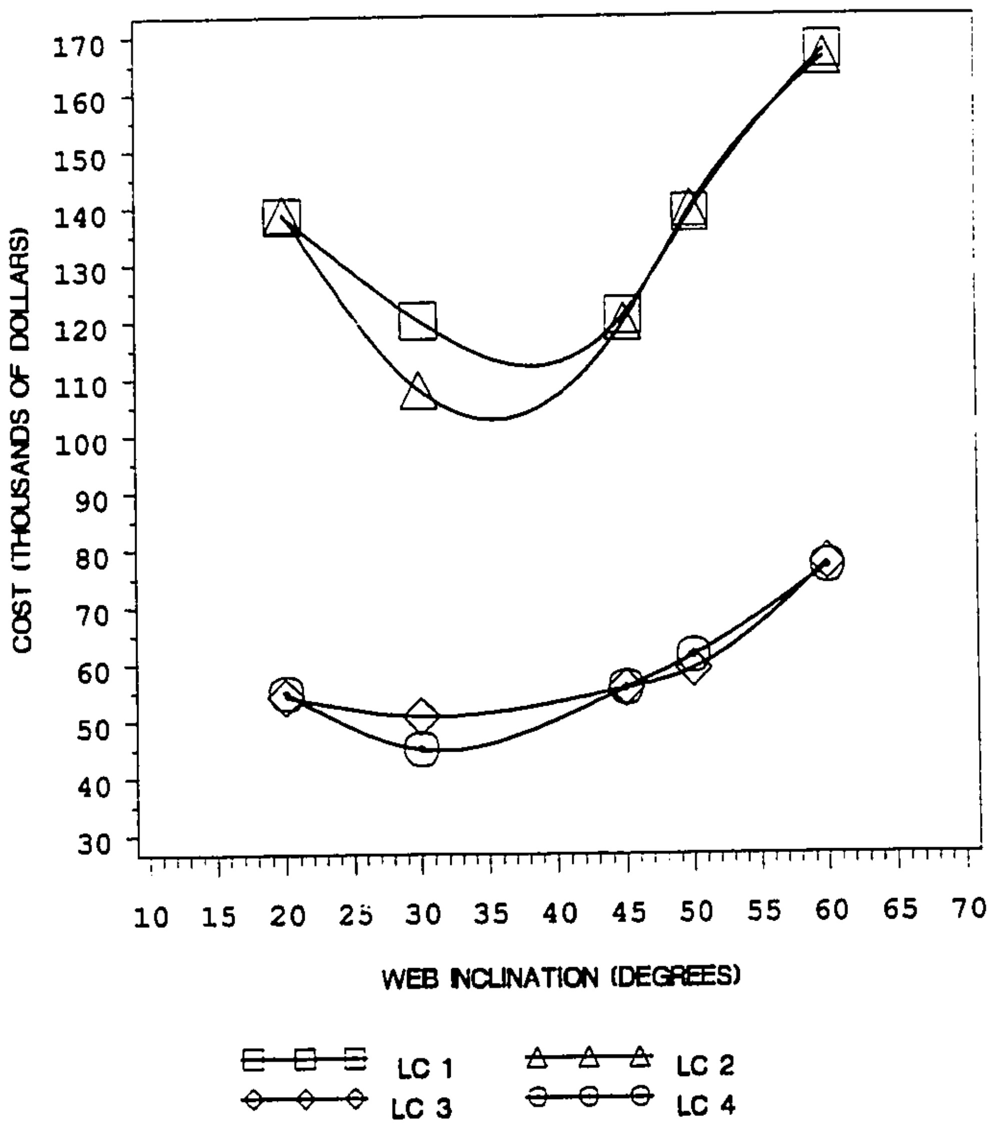

FIG 7.5 : COST vs WEB INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

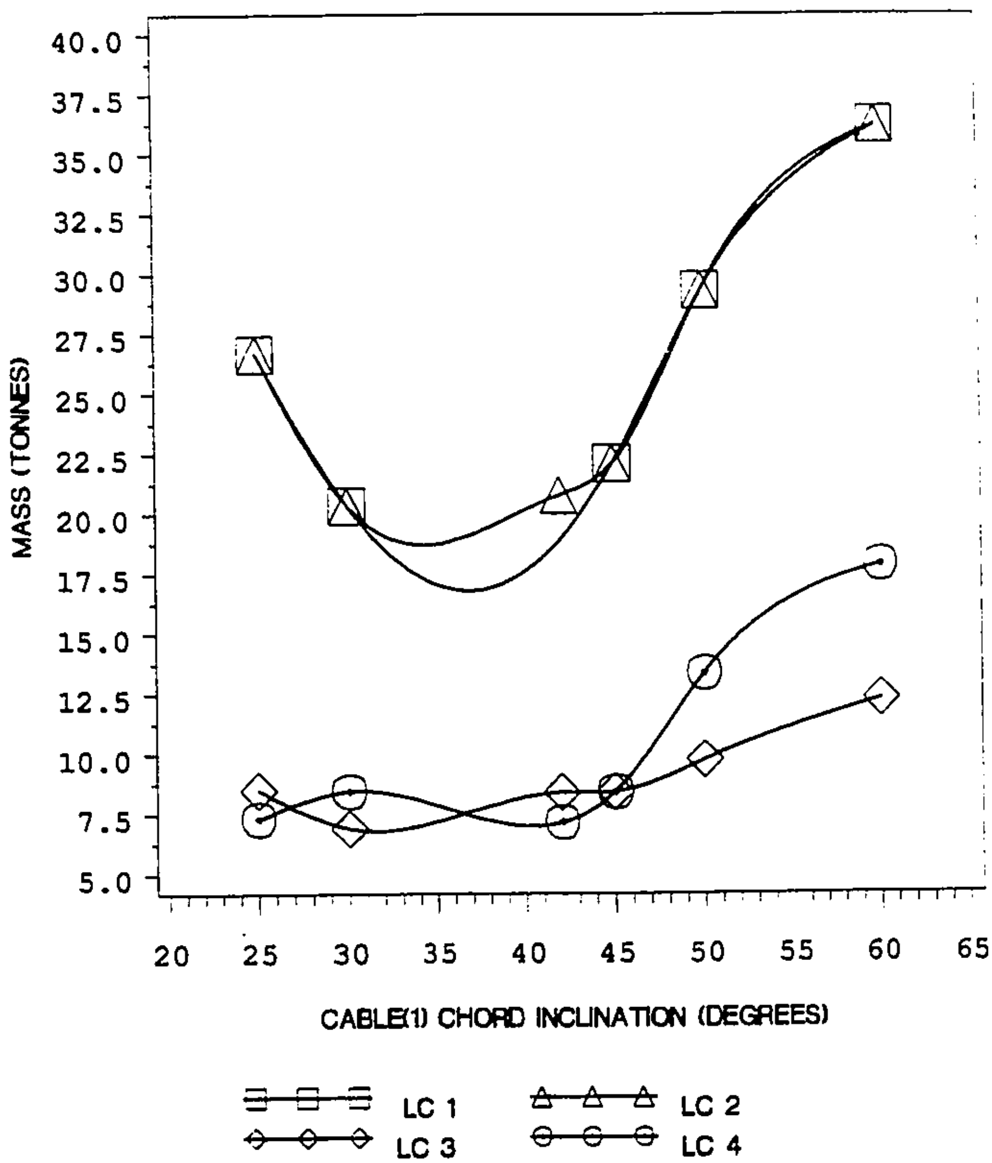

FIG 7.6 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

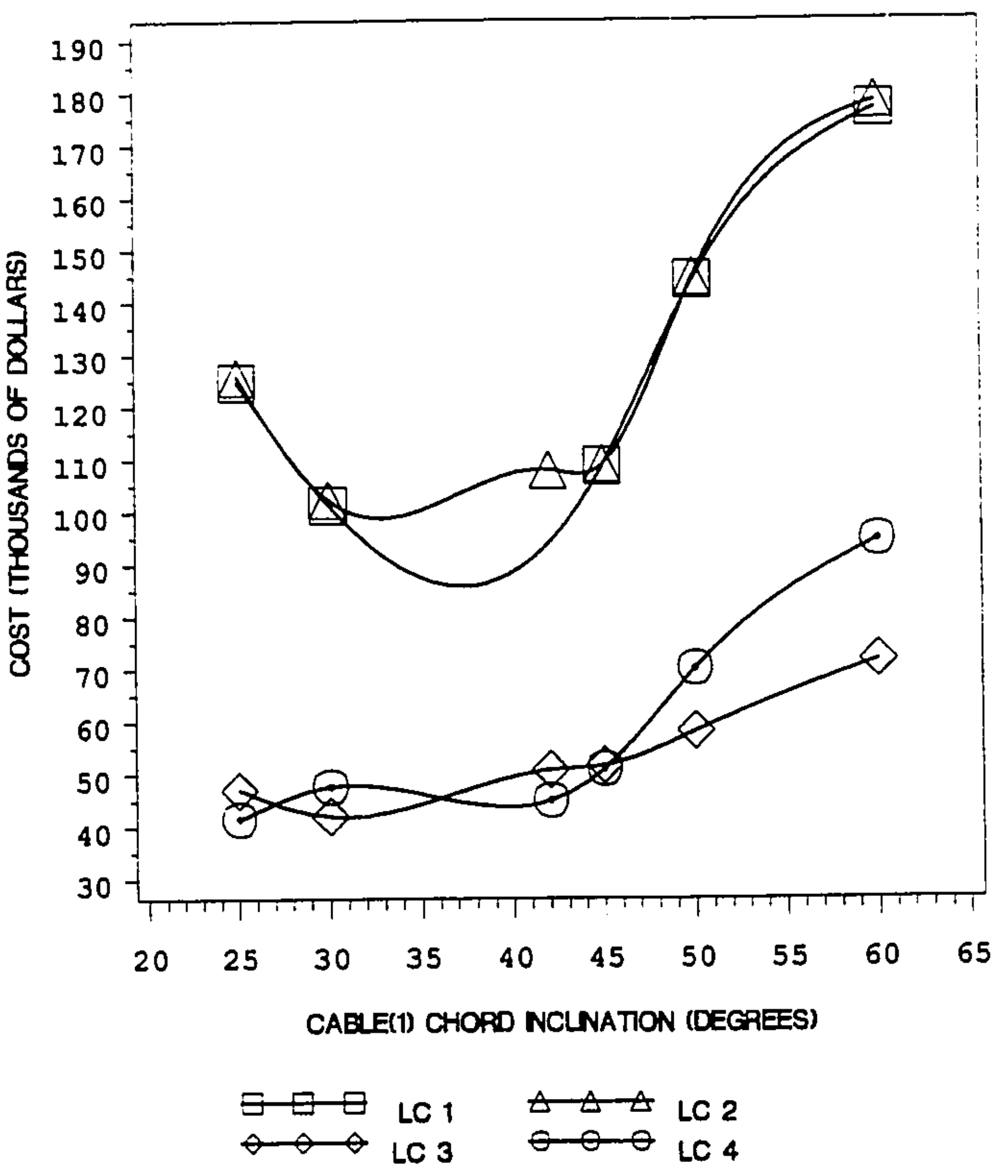

FIG 7.7 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 60.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m FOR LC 1,2&3, 50.0m FOR LC 4 ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

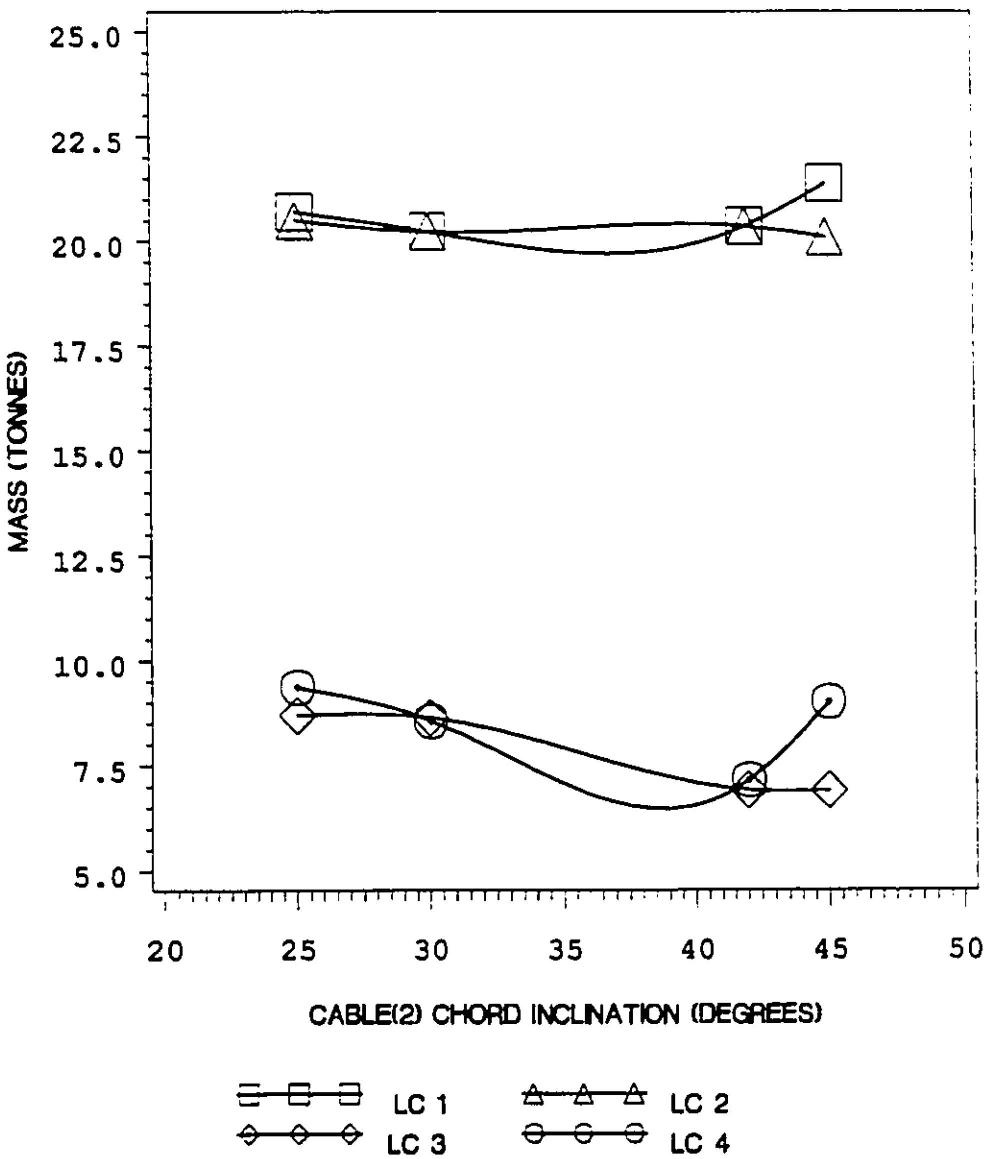

FIG 7.8 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m FOR LC 1,2&3, 50.0m FOR LC 4 ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

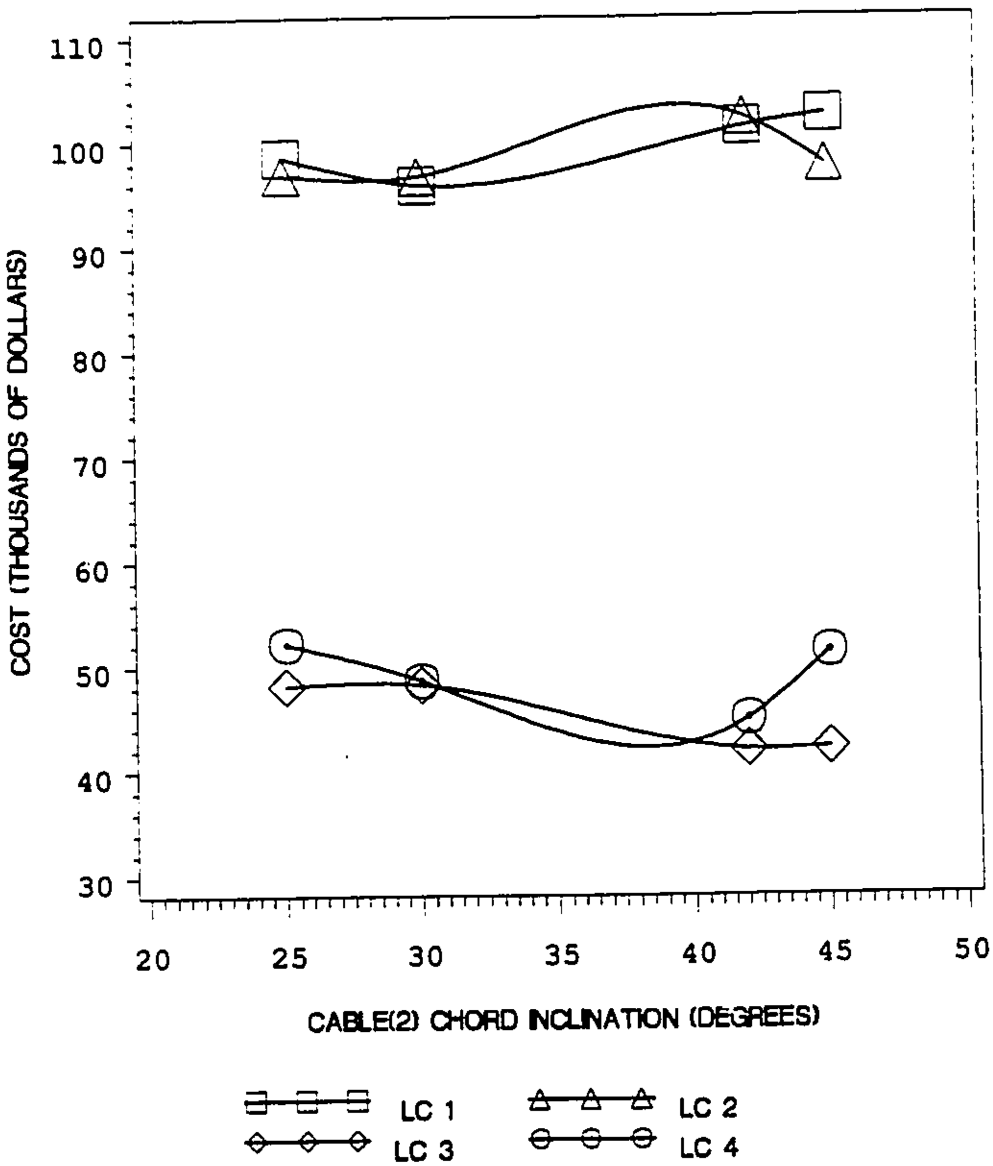

FIG 7.9 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m FOR LC 1,2&3, 50.0m FOR LC 4 ;
CABLE(2) = 155.88m FOR LC 1&2 , 100.0m FOR LC 3&4

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

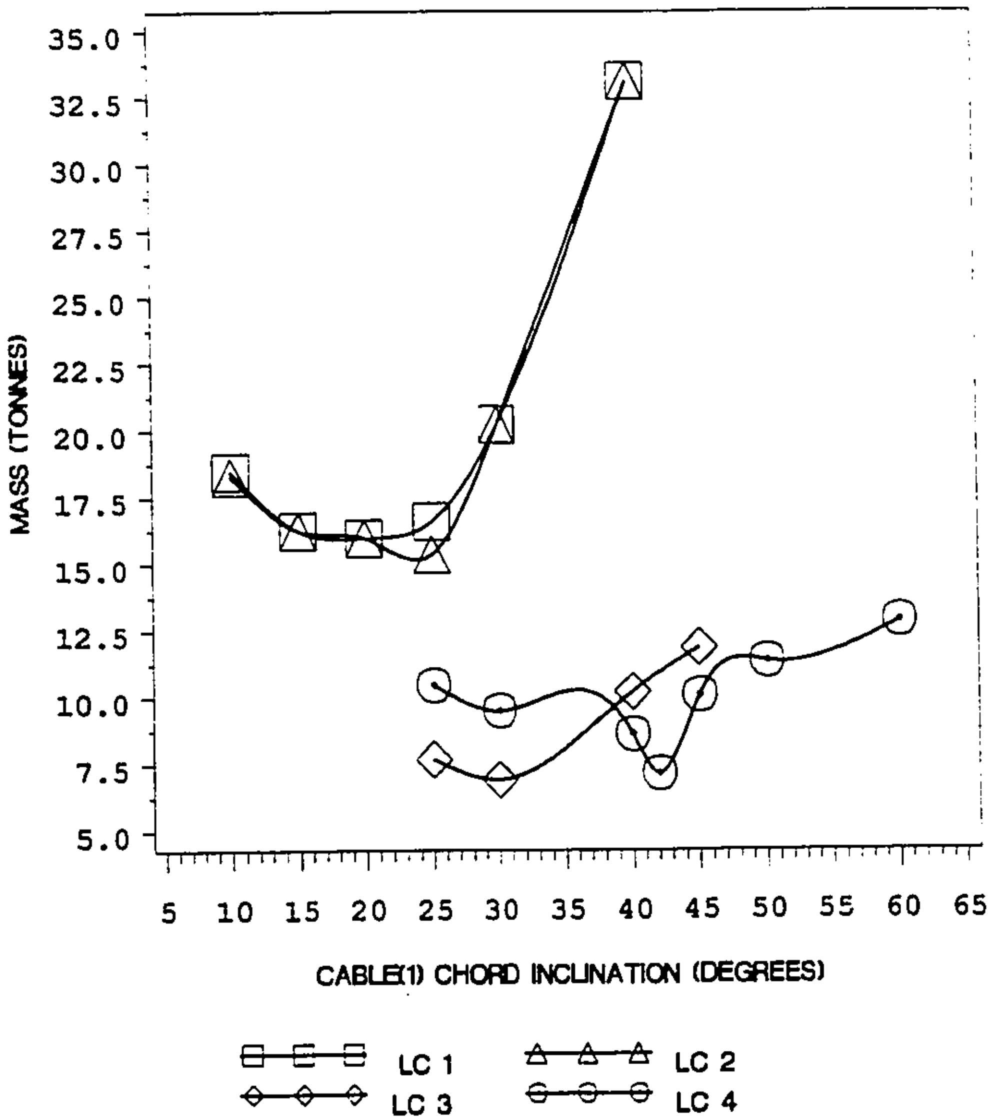

FIG 7.10 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m FOR LC 1,2&3, 50.0m FOR LC 4 ;
CABLE(2) = 155.89m FOR LC 1&2 , 100.0m FOR LC 3&4

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

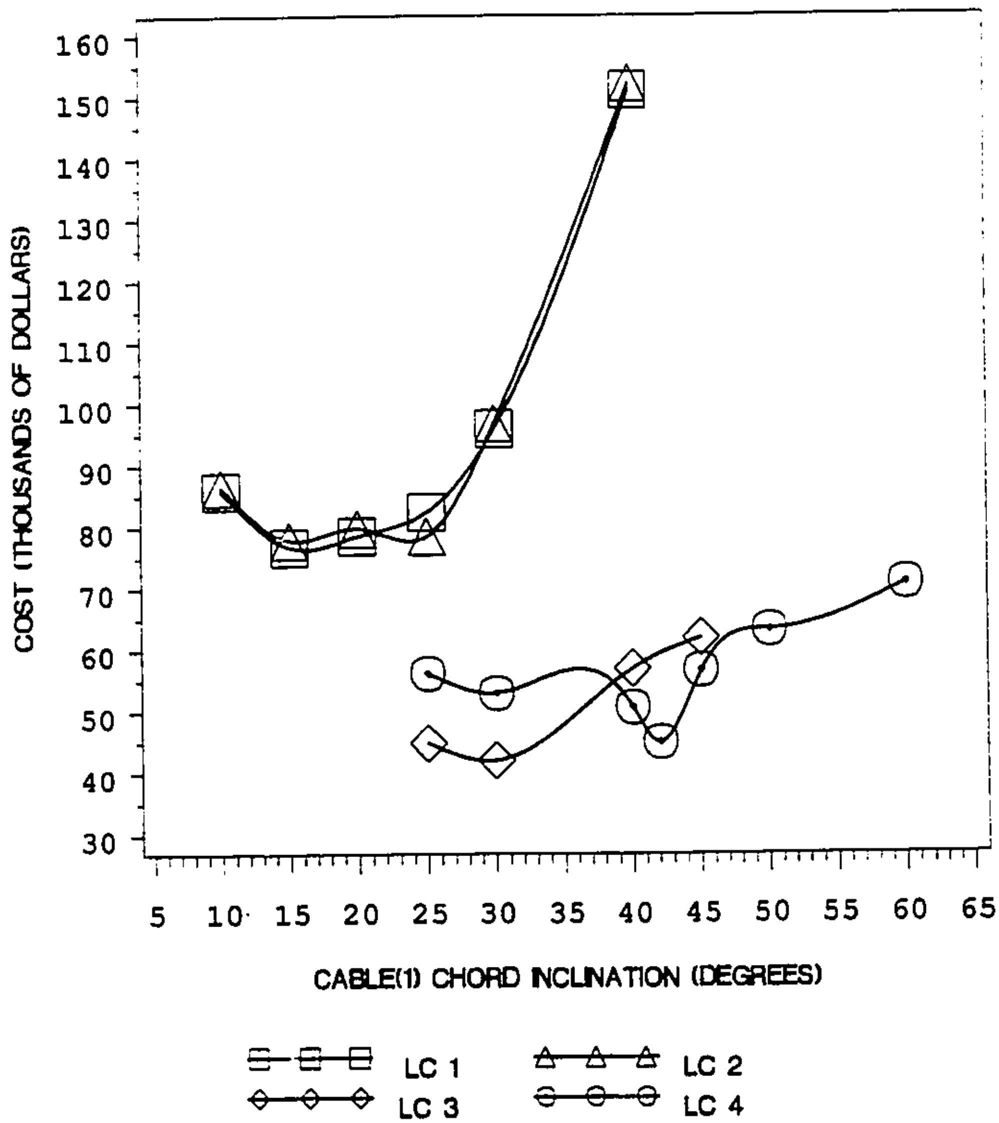

FIG 7.11 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 36.34m FOR 1&2 , 45.0m FOR 3&4 ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m FOR LC 1,2&3, 50.0m FOR LC 4 ;
CABLE(2) = 155.89m FOR LC 1&2 , 100.0m FOR LC 3&4

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

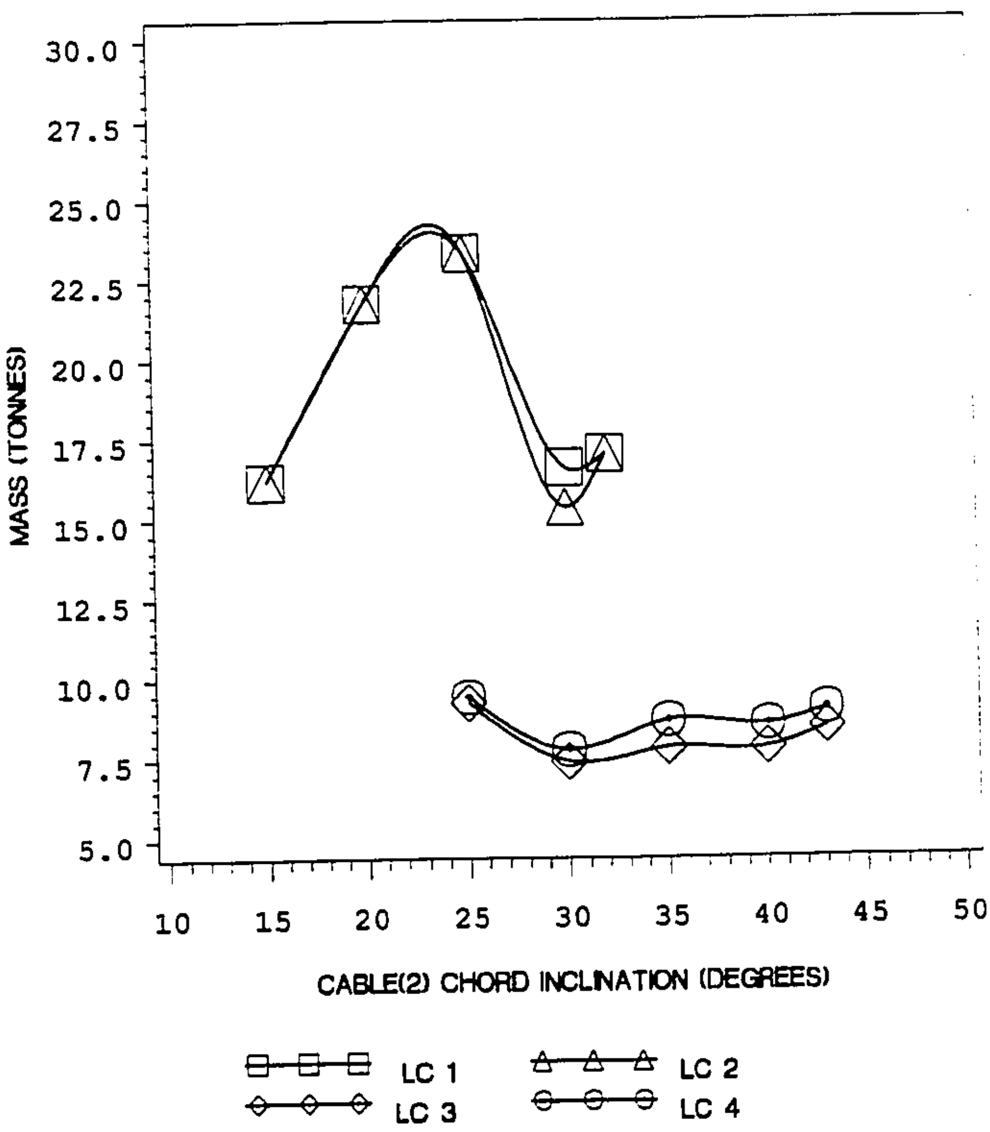

FIG 7.12 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 38.34m FOR 1&2 , 45.0m FOR 3&4 ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m FOR LC 1,2&3, 50.0m FOR LC 4 ;
CABLE(2) = 155.89m FOR LC 1&2 , 100.0m FOR LC 3&4

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

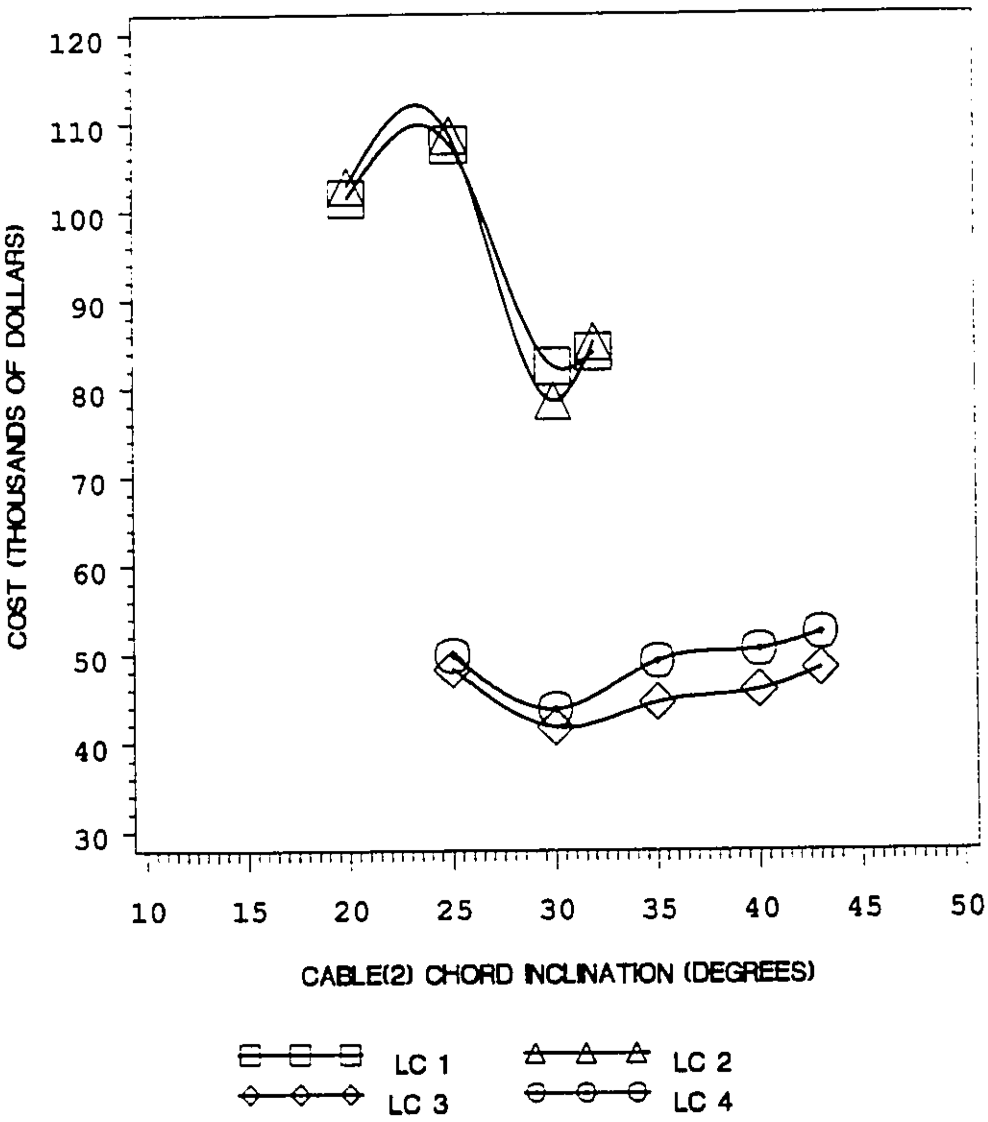

FIG 7.13 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : VARIES ; MAST HEIGHT : 100m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

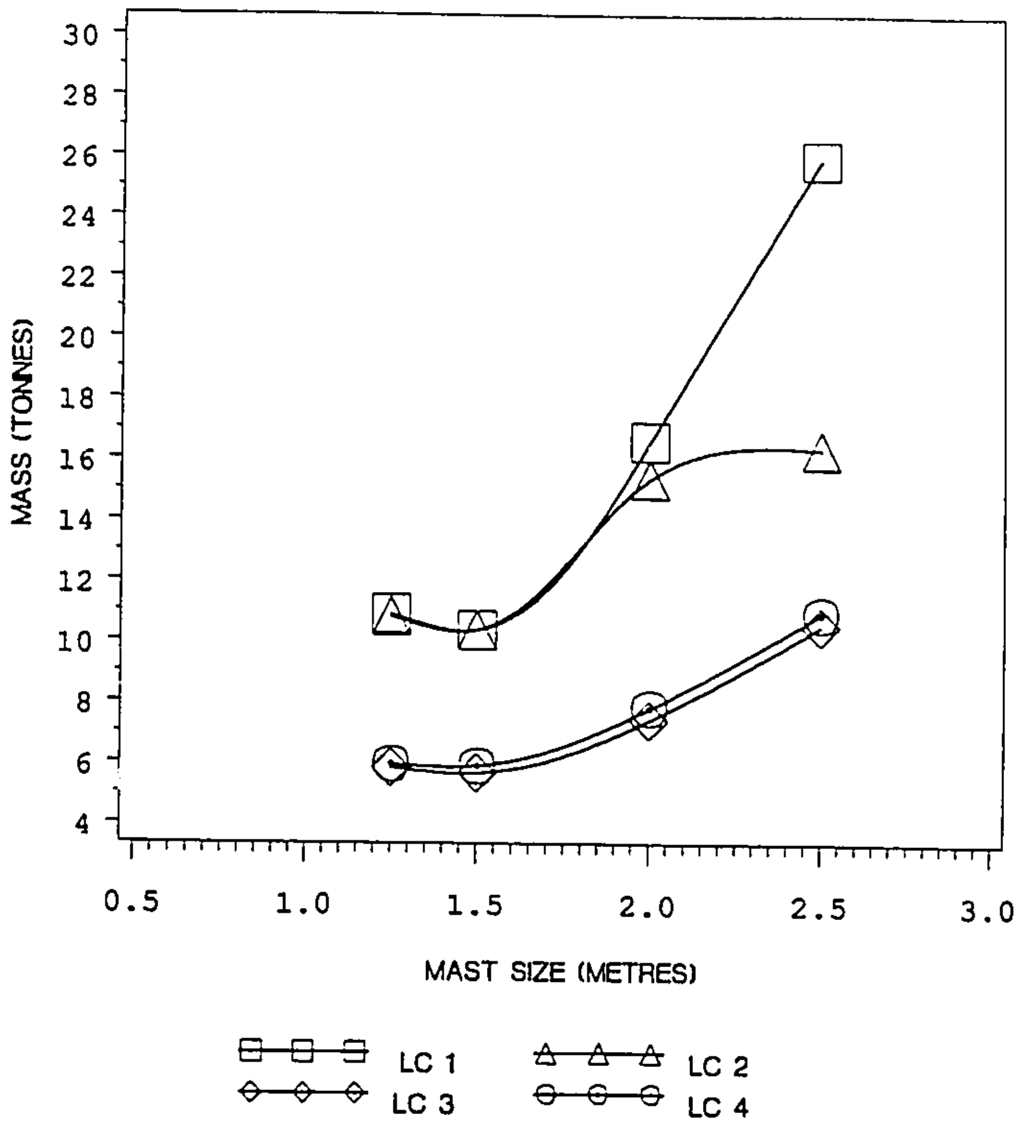

FIG 7.14 : MASS vs MAST SIZE

MAST SIZE : VARIES ; MAST HEIGHT : 100m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

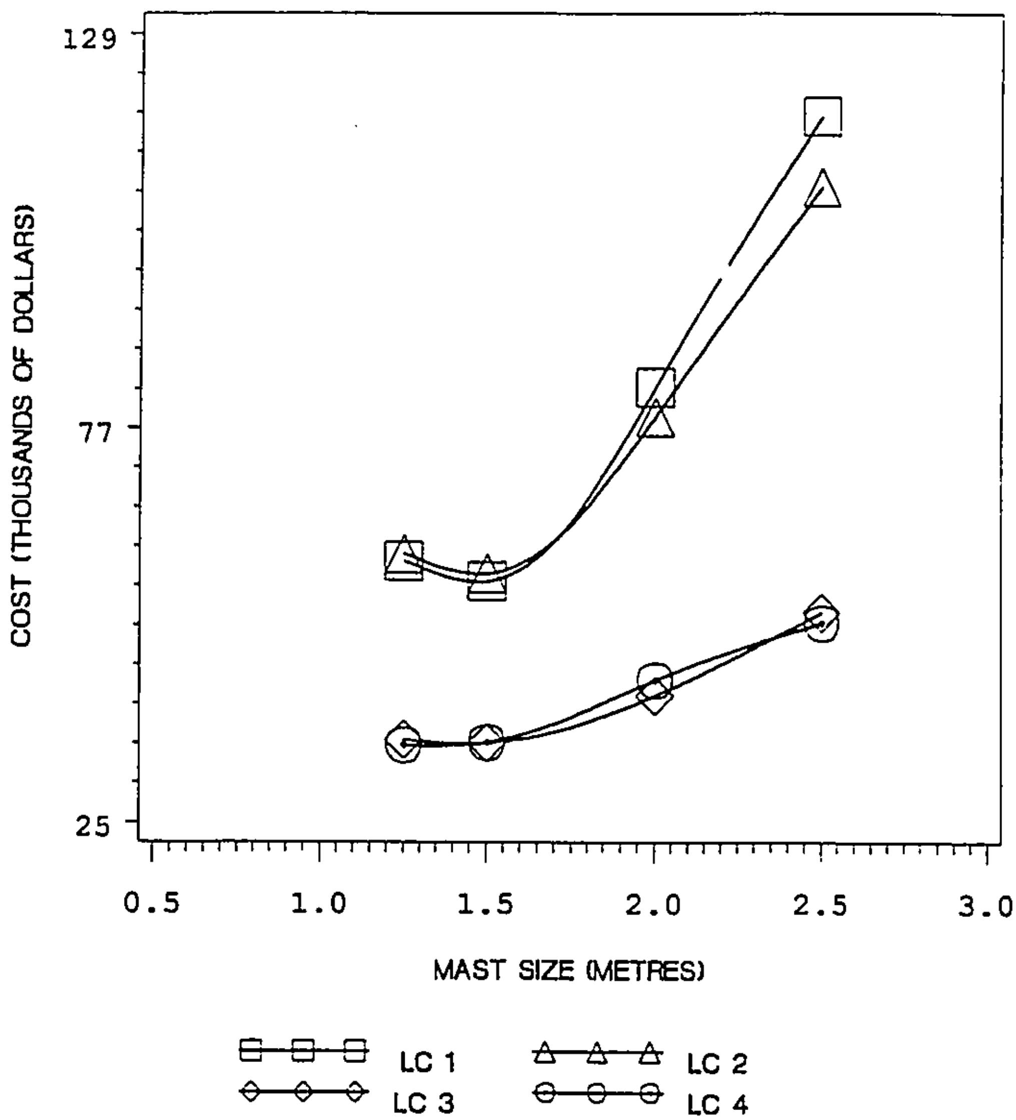

FIG 7.15 : COST vs MAST SIZE

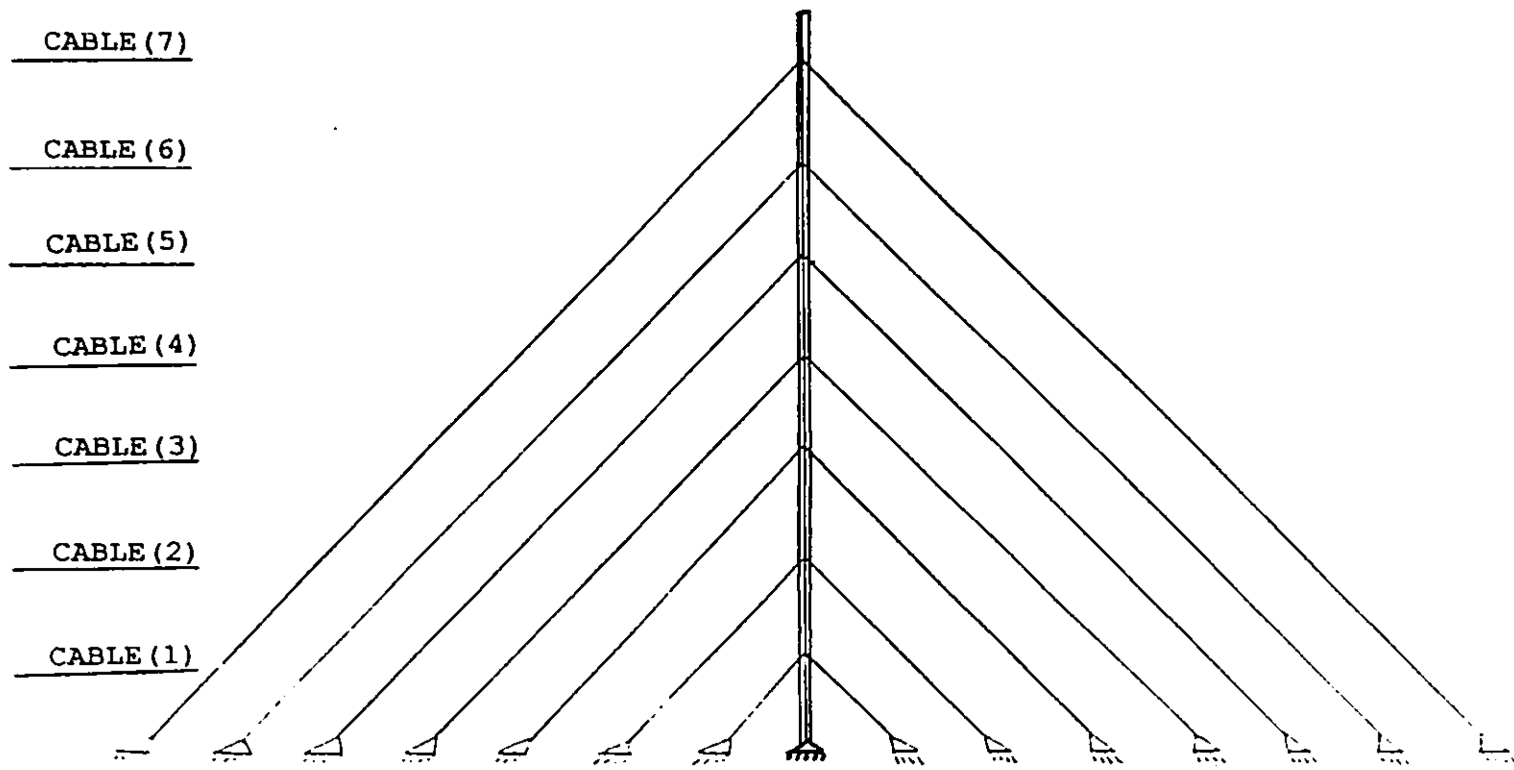

FIG 7.16 : 350 m HIGH GUYED TOWER WITH 7 GUY LEVELS

MAST SIZE : 2.0m ; MAST HEIGHT : 350m
ALL CABLE CHORDS ARE AT 45 DEGREES INCLINATION
ALL CABLE CONNECTION POINTS ARE EQUALLY SPACED
WEB INCLINATION = 45 DEGREES
LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

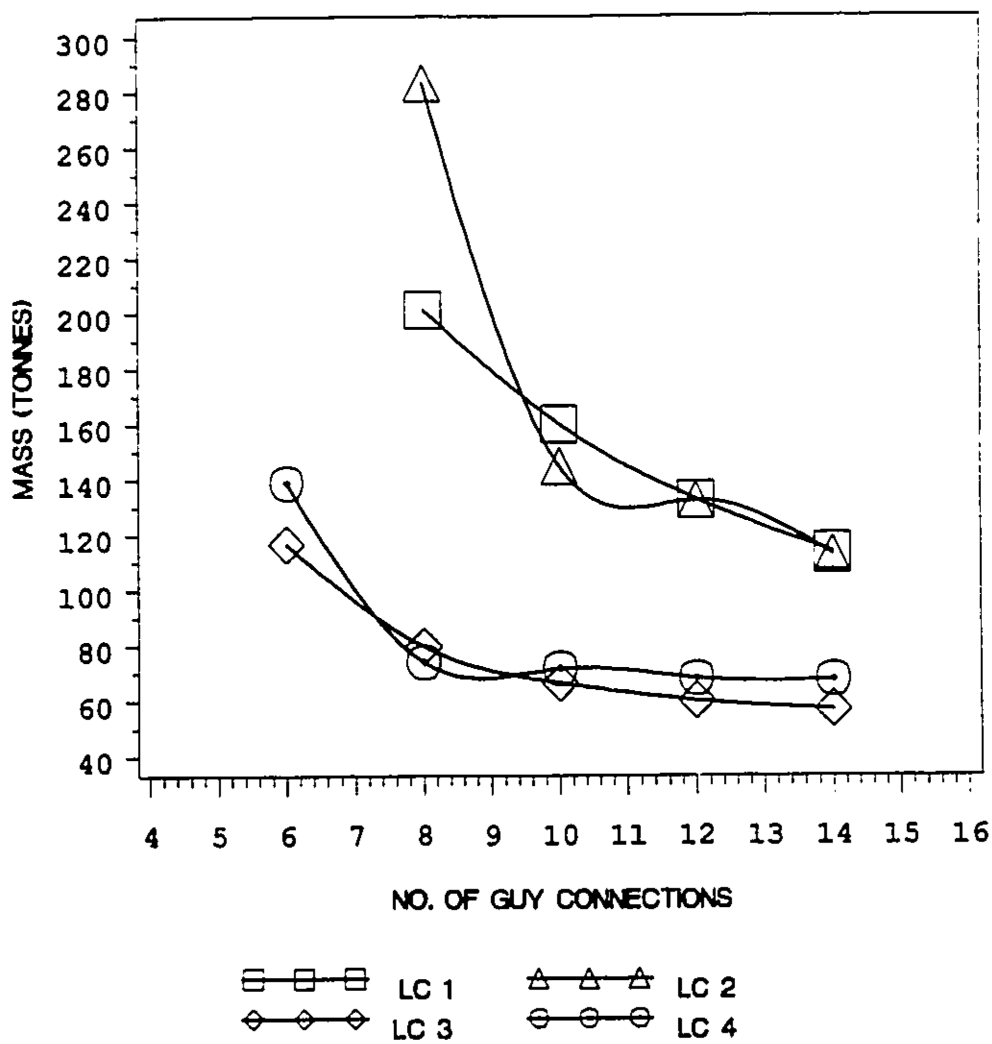

FIG 7.17 : MASS vs NO. OF GUY CONNECTIONS

MAST SIZE : 2.0m ; MAST HEIGHT : 350m
ALL CABLE CHORDS ARE AT 45 DEGREES INCLINATION
ALL CABLE CONNECTION POINTS ARE EQUALLY SPACED
WEB INCLINATION = 45 DEGREES
LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

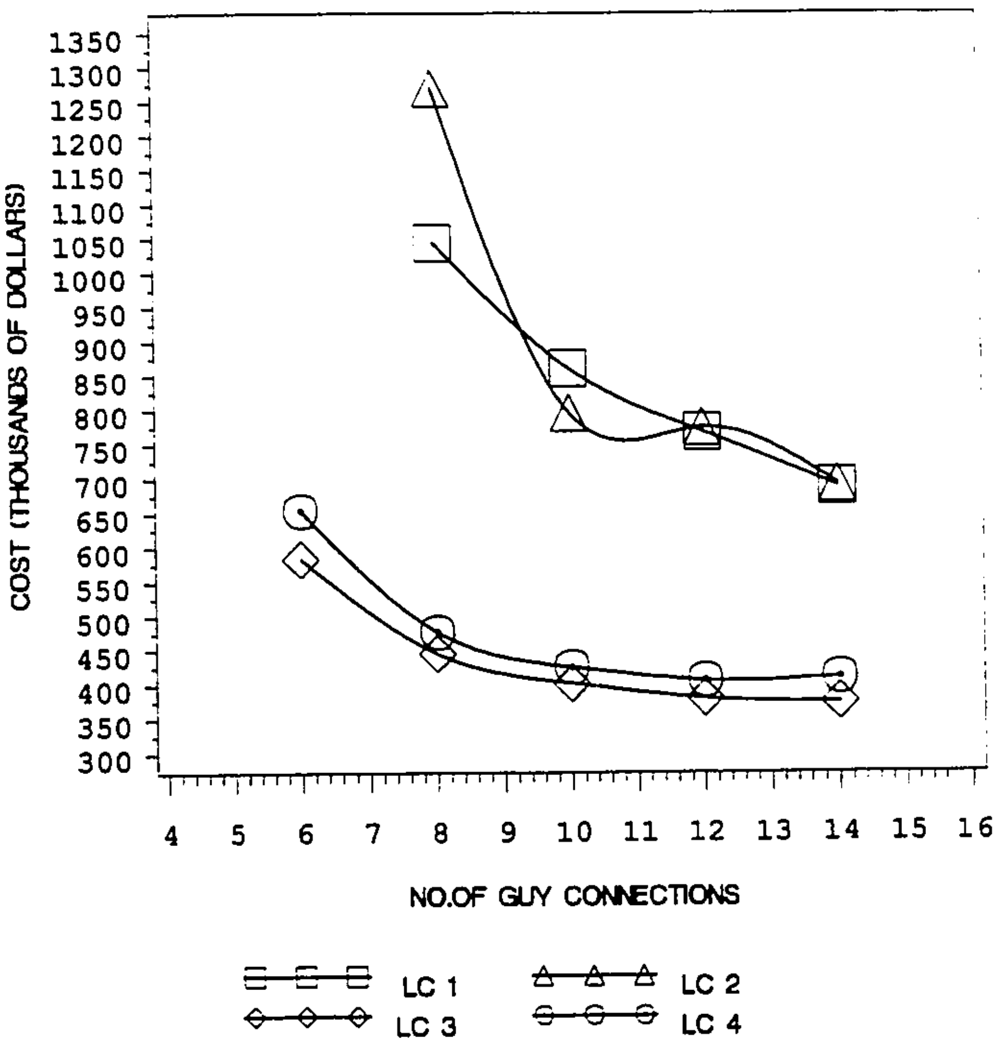

FIG 7.18 : COST vs NO. OF GUY CONNECTIONS

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

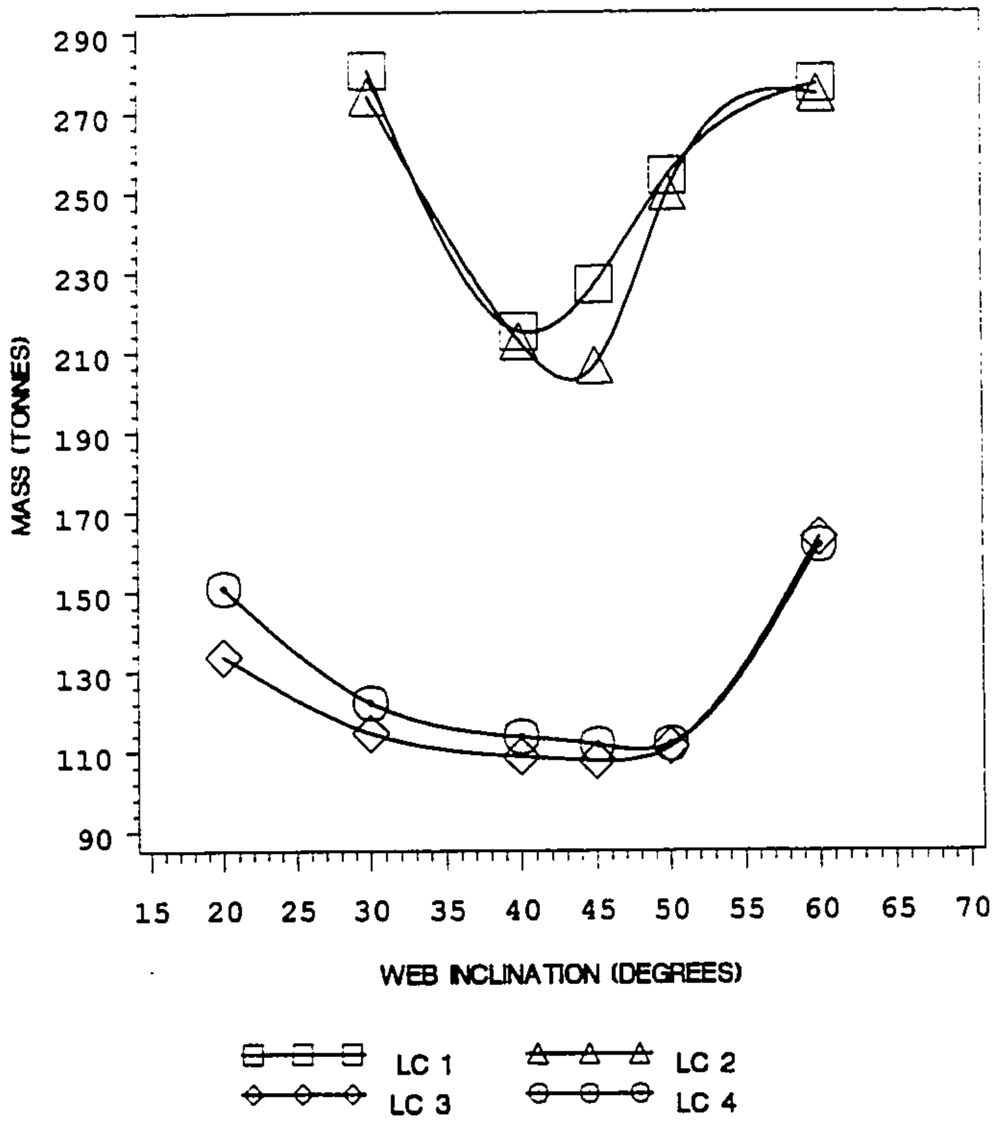

FIG 7.19 : MASS vs WEB INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

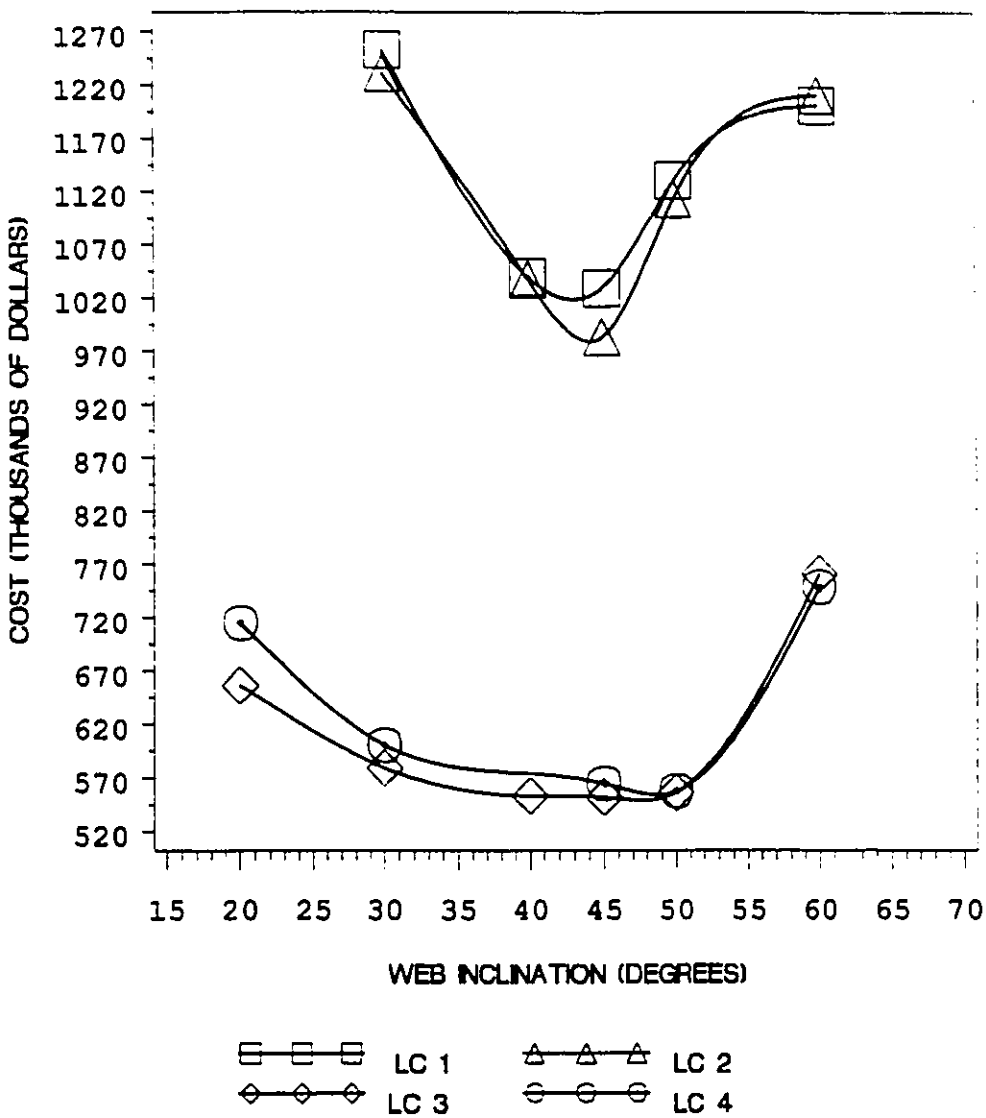

FIG 7.20 : COST vs WEB INCLINATION

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

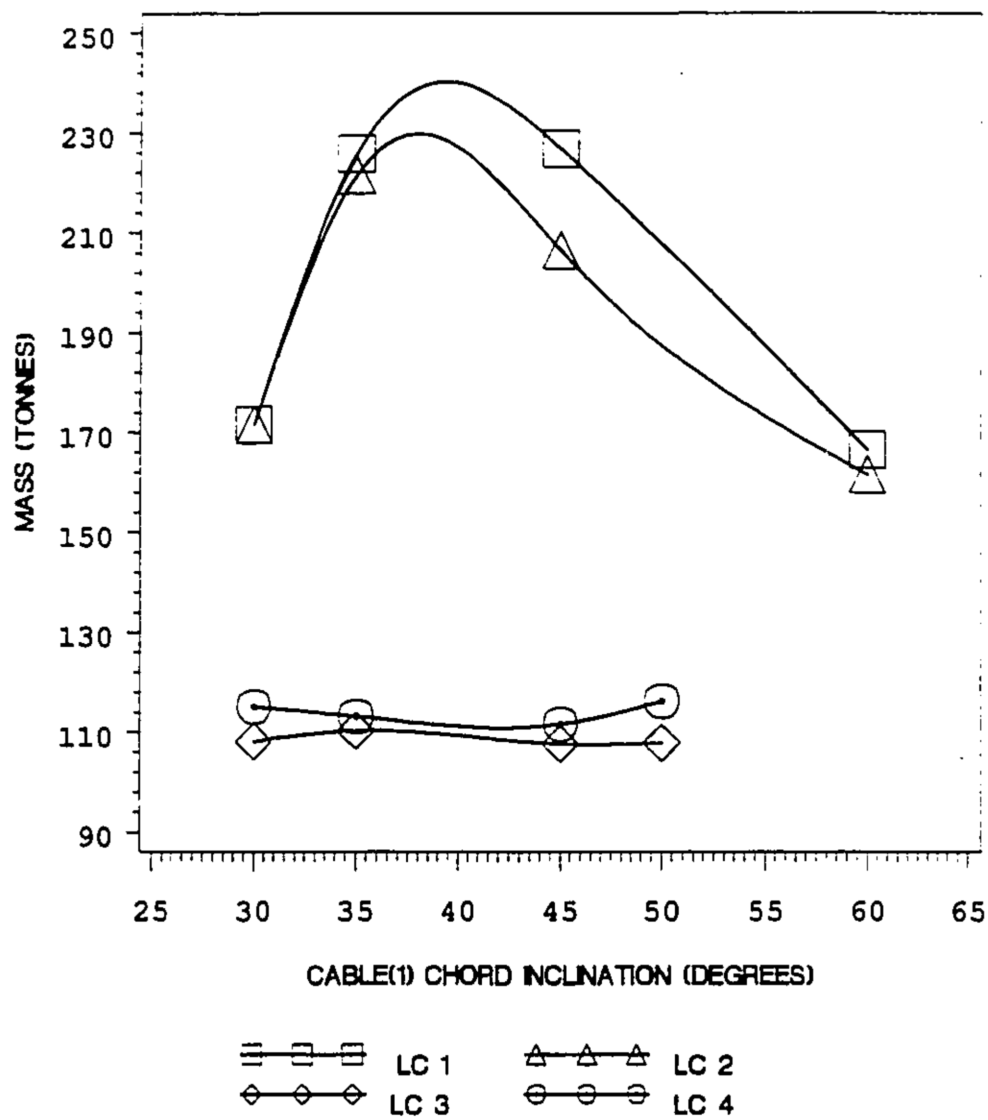

FIG 7.21 : MASS vs CABLE(1) CHORD INCLINATION (Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

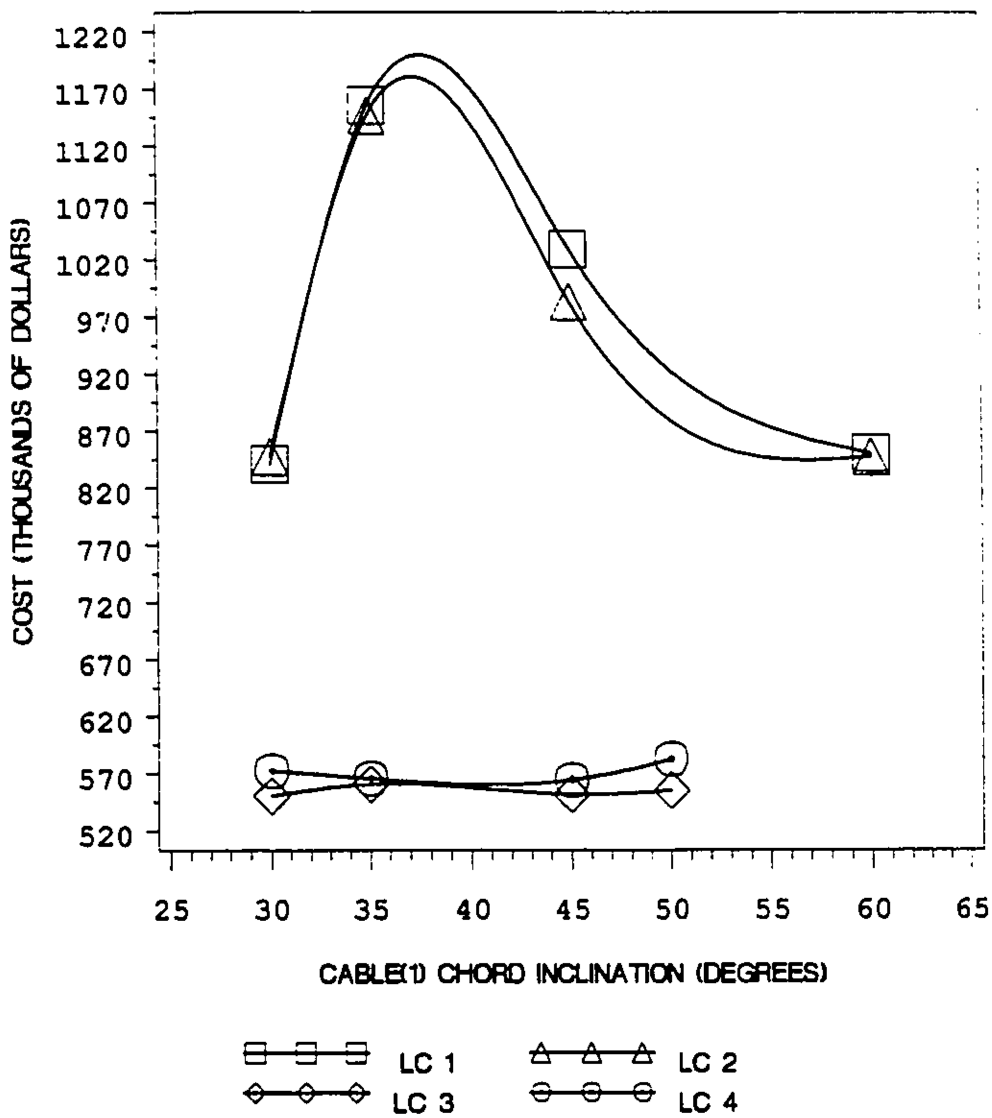

FIG 7.22 : COST vs CABLE(1) CHORD INCLINATION

(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

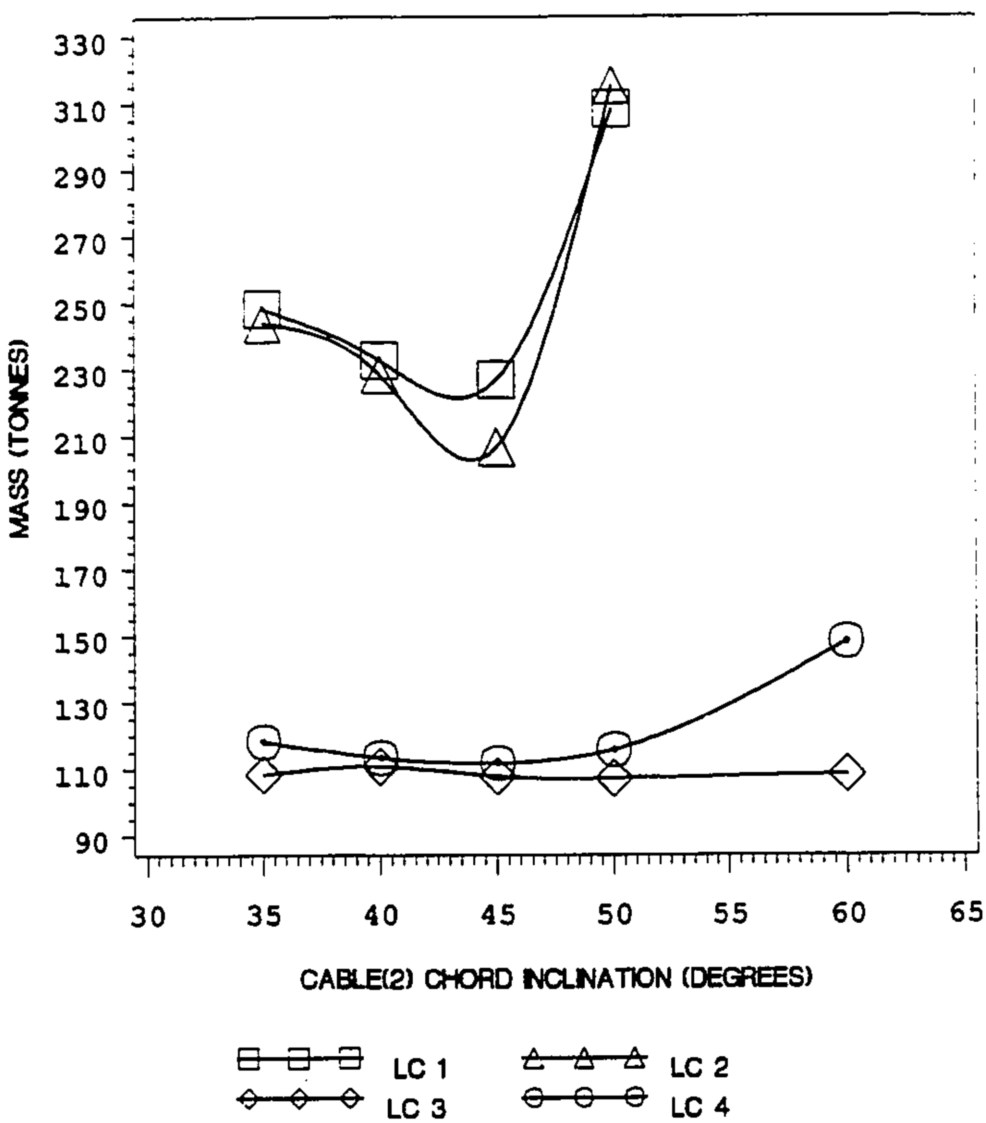

FIG 7.23 : MASS vs CABLE(2) CHORD INCLINATION
(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

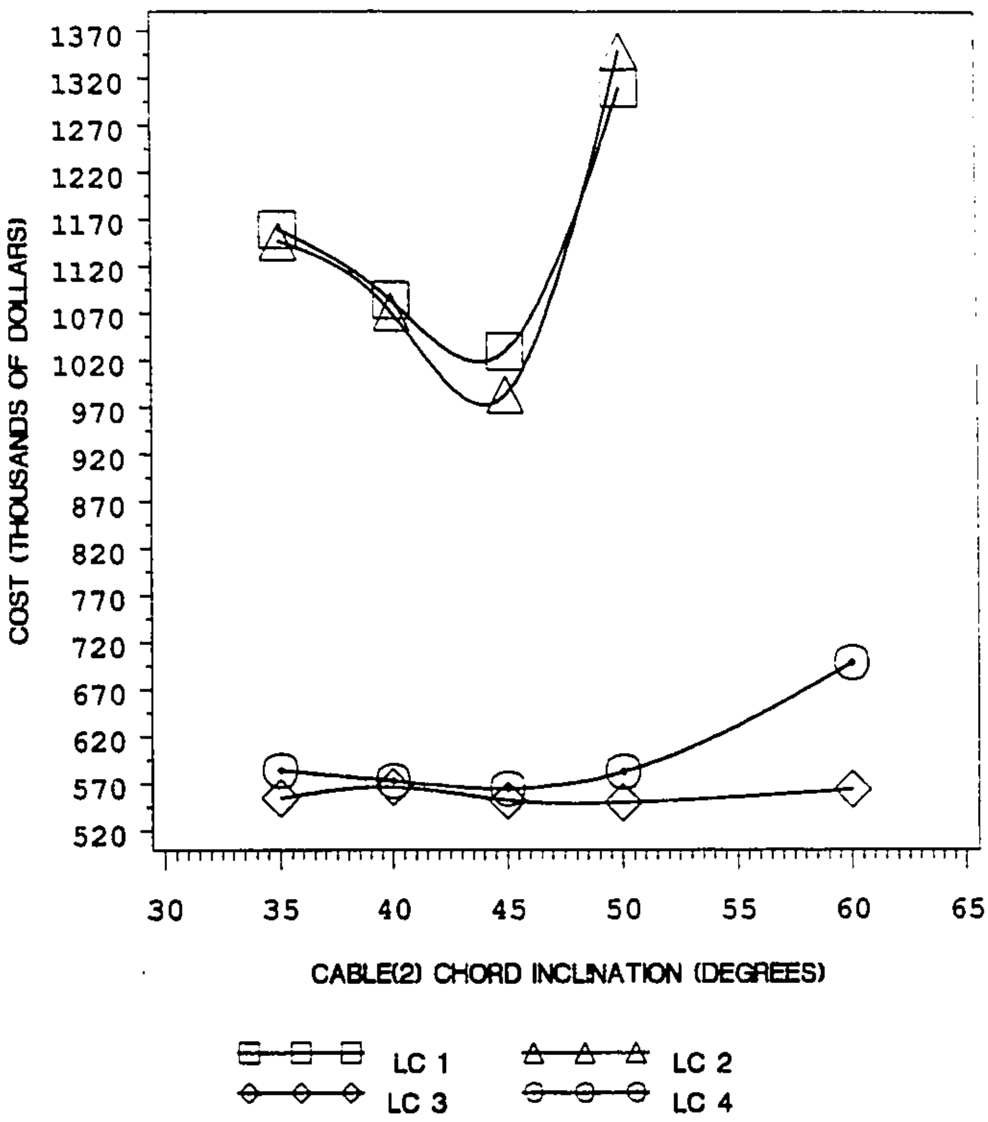

FIG 7.24 : MASS vs CABLE(2) CHORD INCLINATION
(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

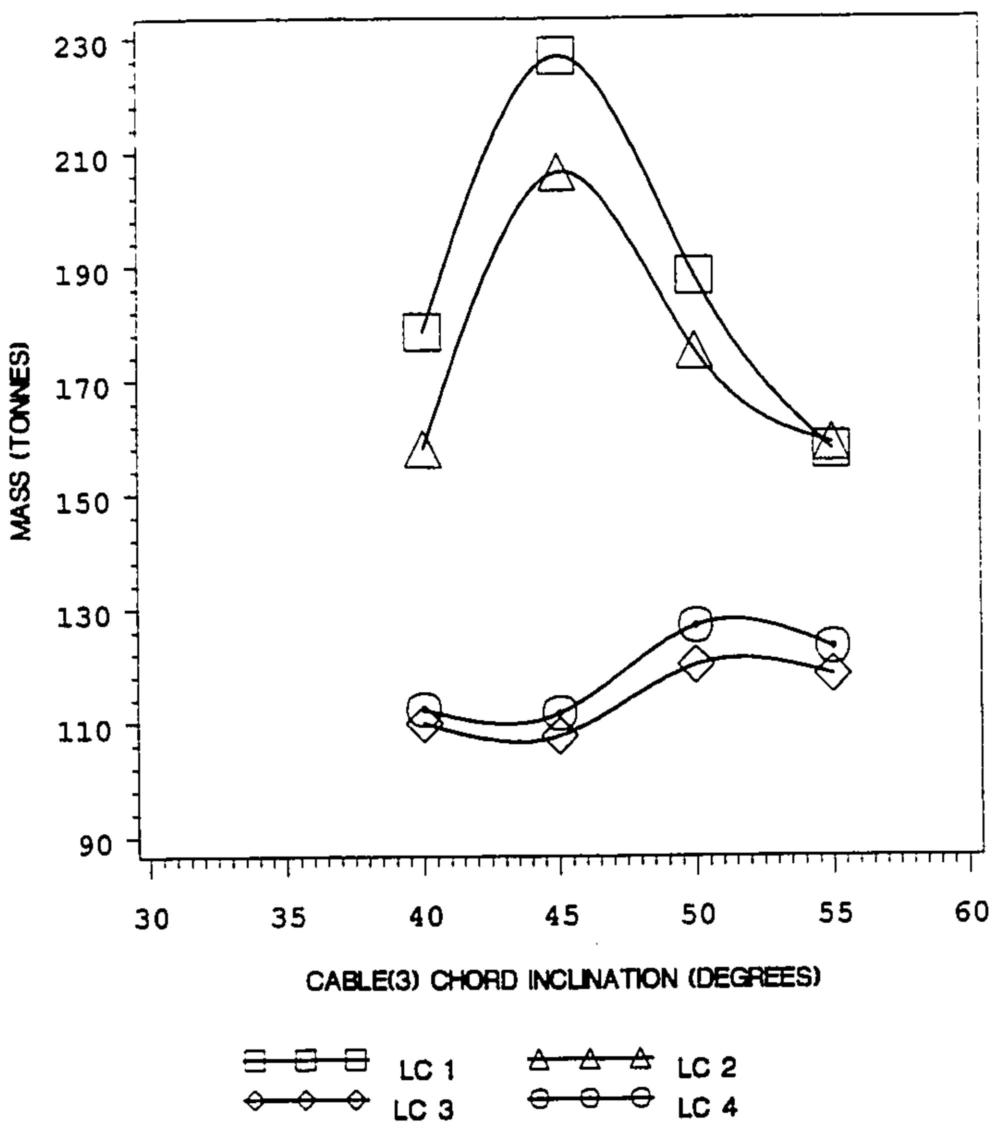

FIG 7.25 : MASS vs CABLE(3) CHORD INCLINATION
(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

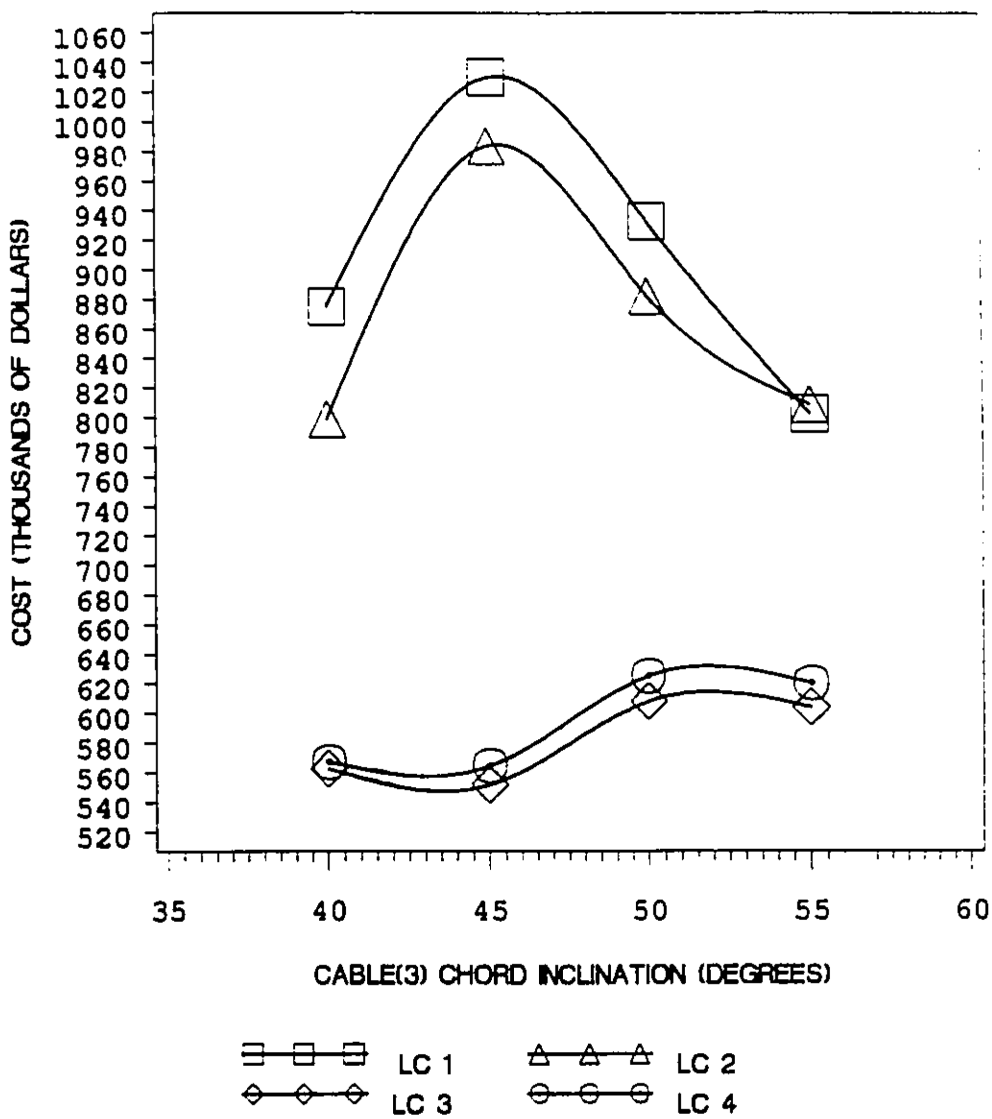

FIG 7.26 : COST vs CABLE(3) CHORD INCLINATION
(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

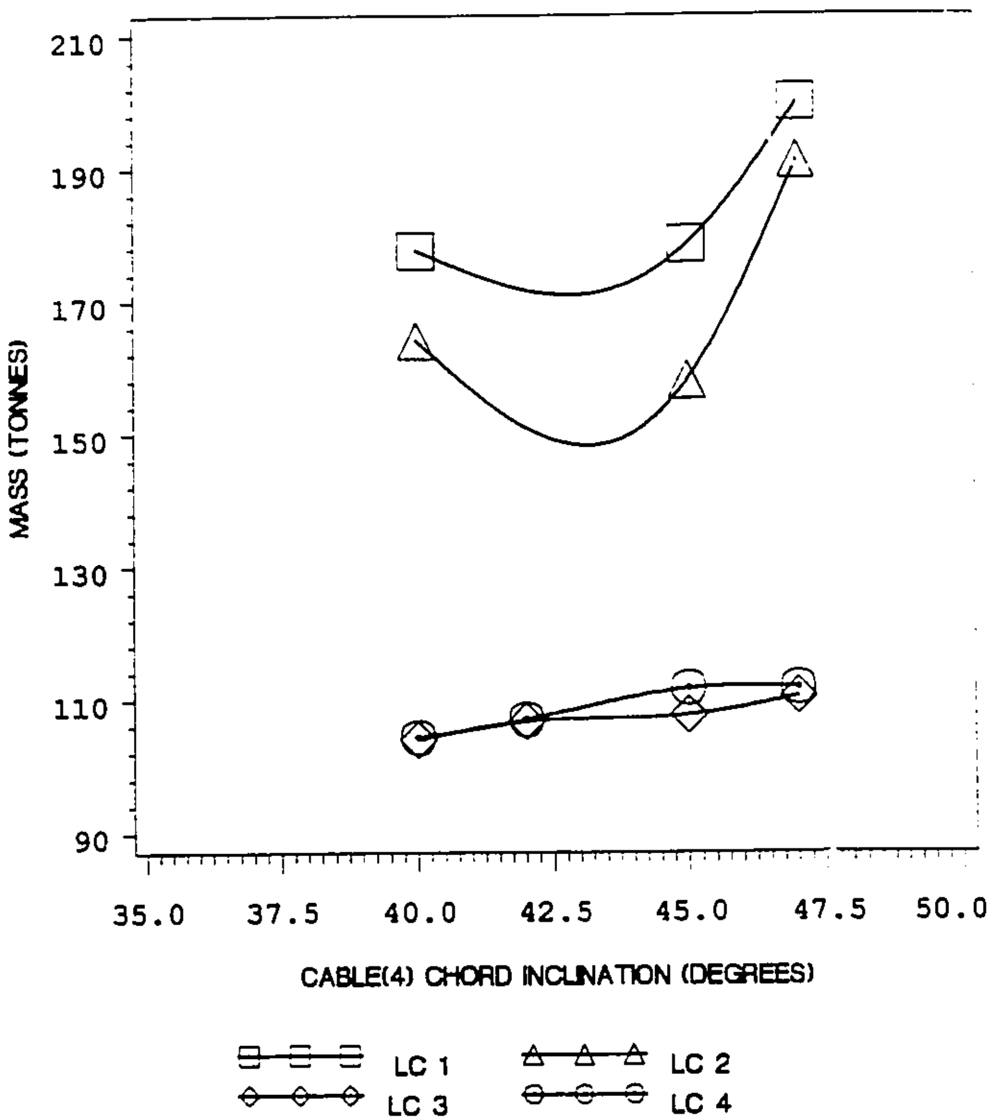

FIG 7.27 : MASS vs CABLE(4) CHORD INCLINATION
(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

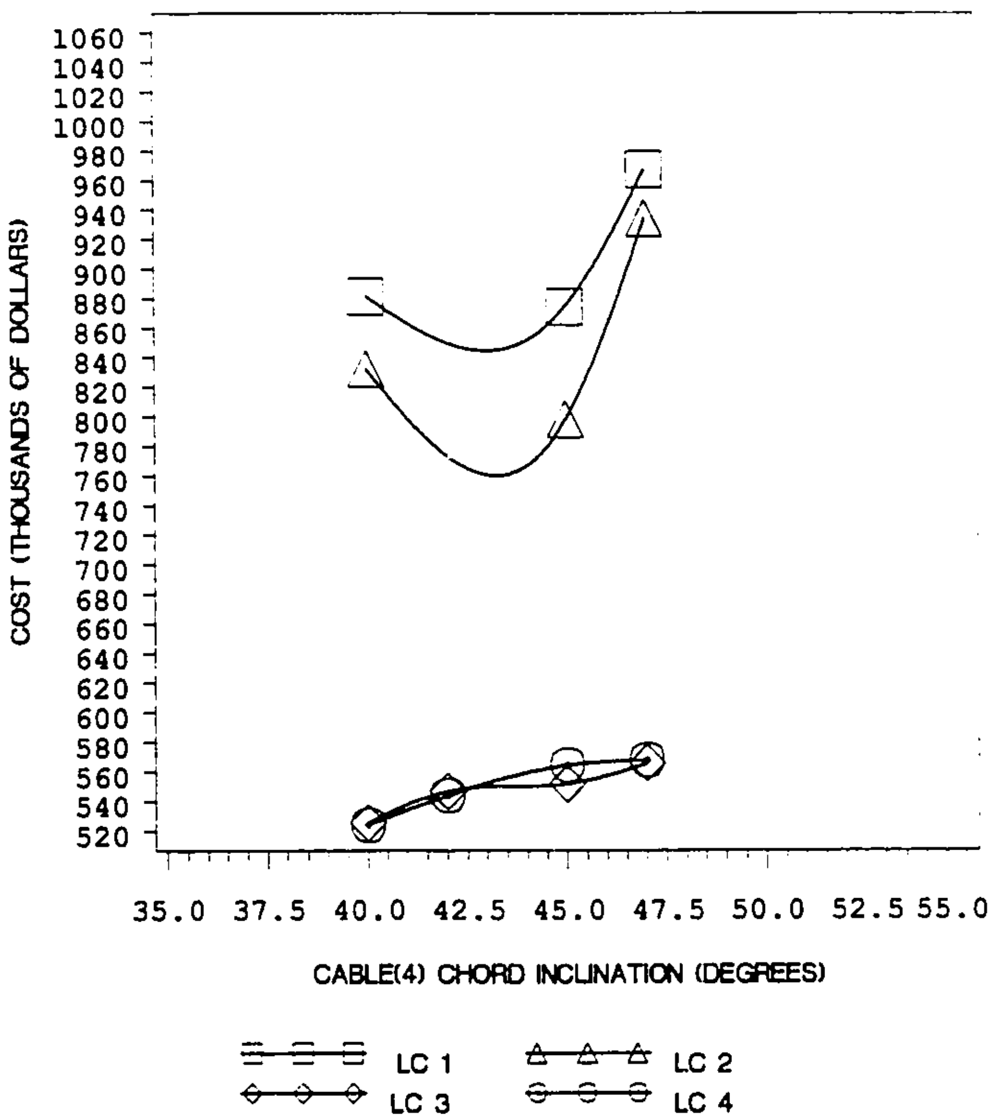

FIG 7.28 : COST vs CABLE(4) CHORD INCLINATION

(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

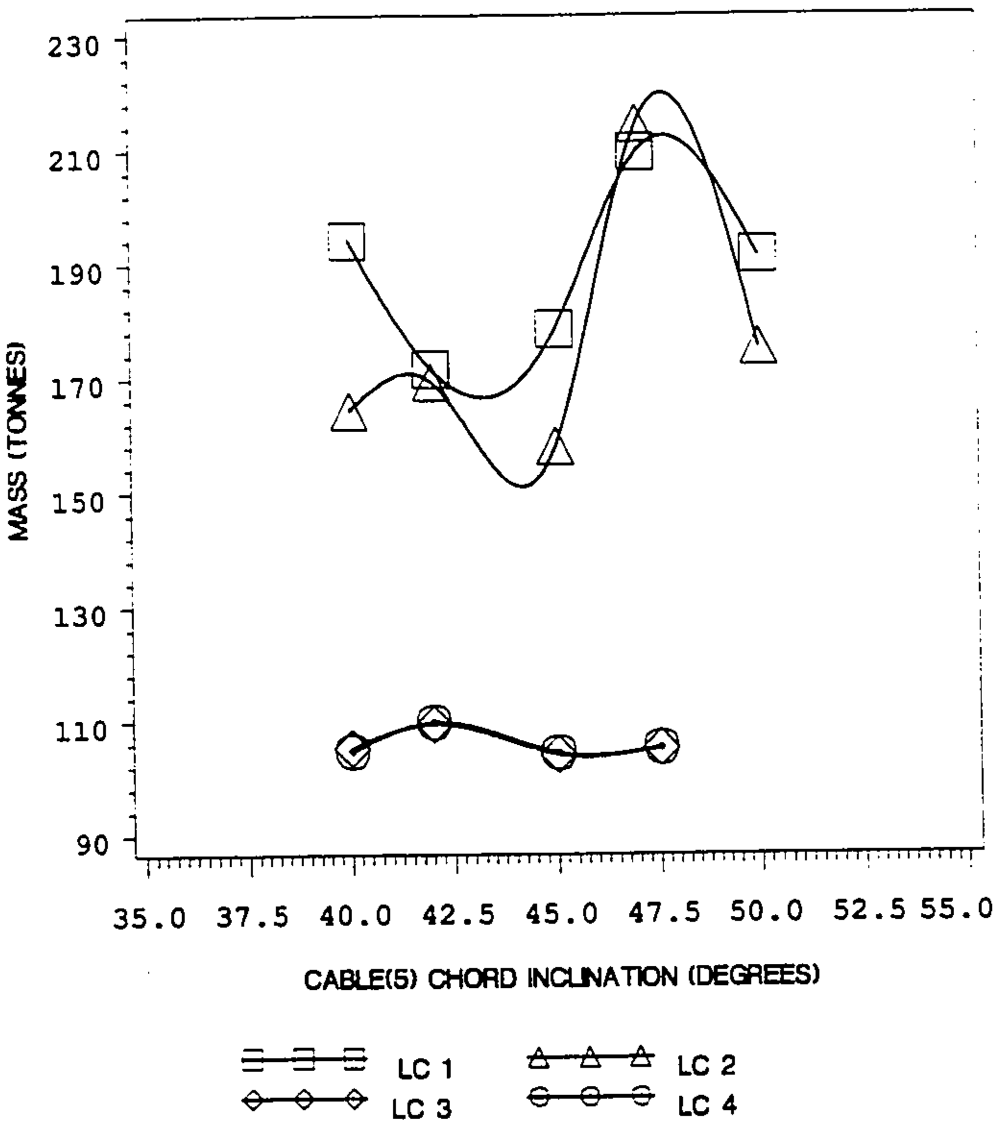

FIG 7.29 : MASS vs CABLE(5) CHORD INCLINATION

(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

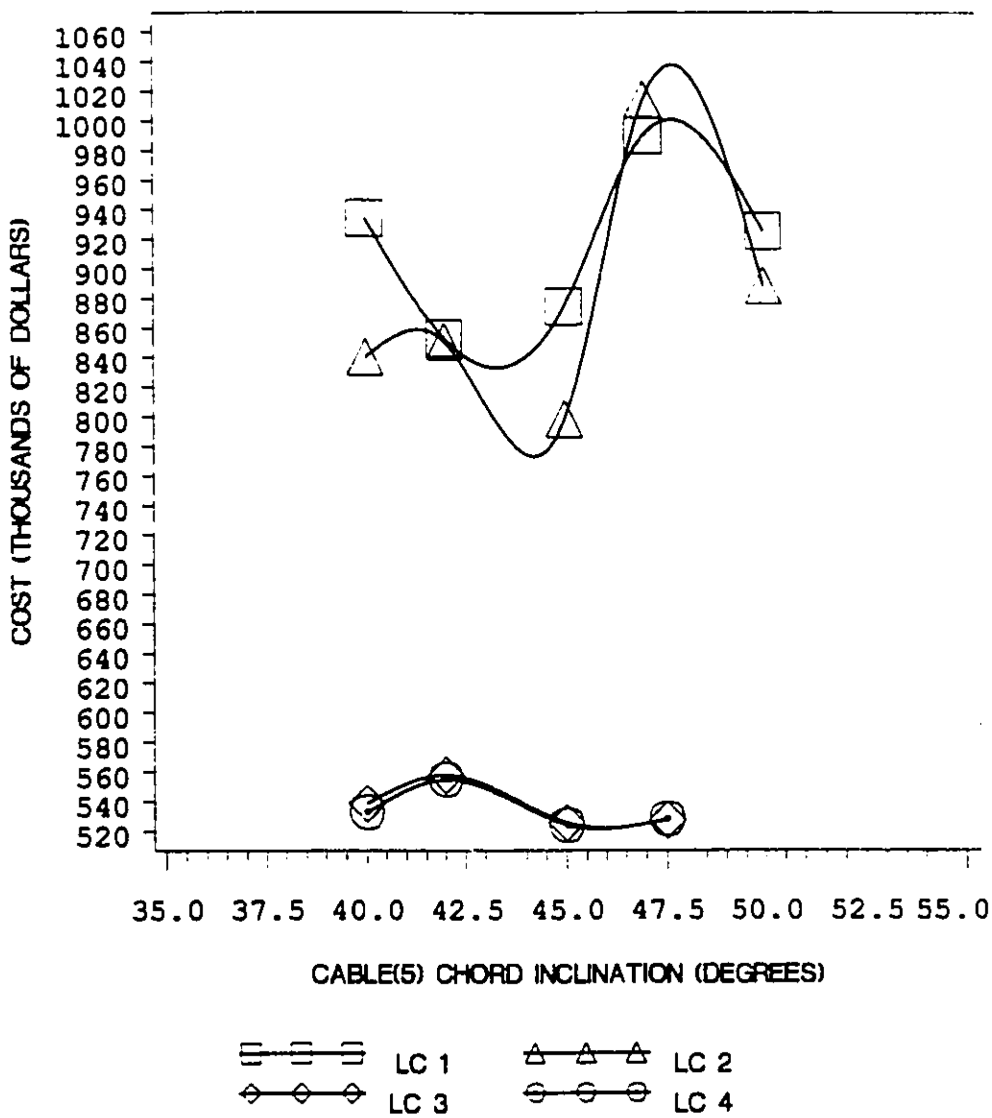

FIG 7.30 : COST vs CABLE(5) CHORD INCLINATION

(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

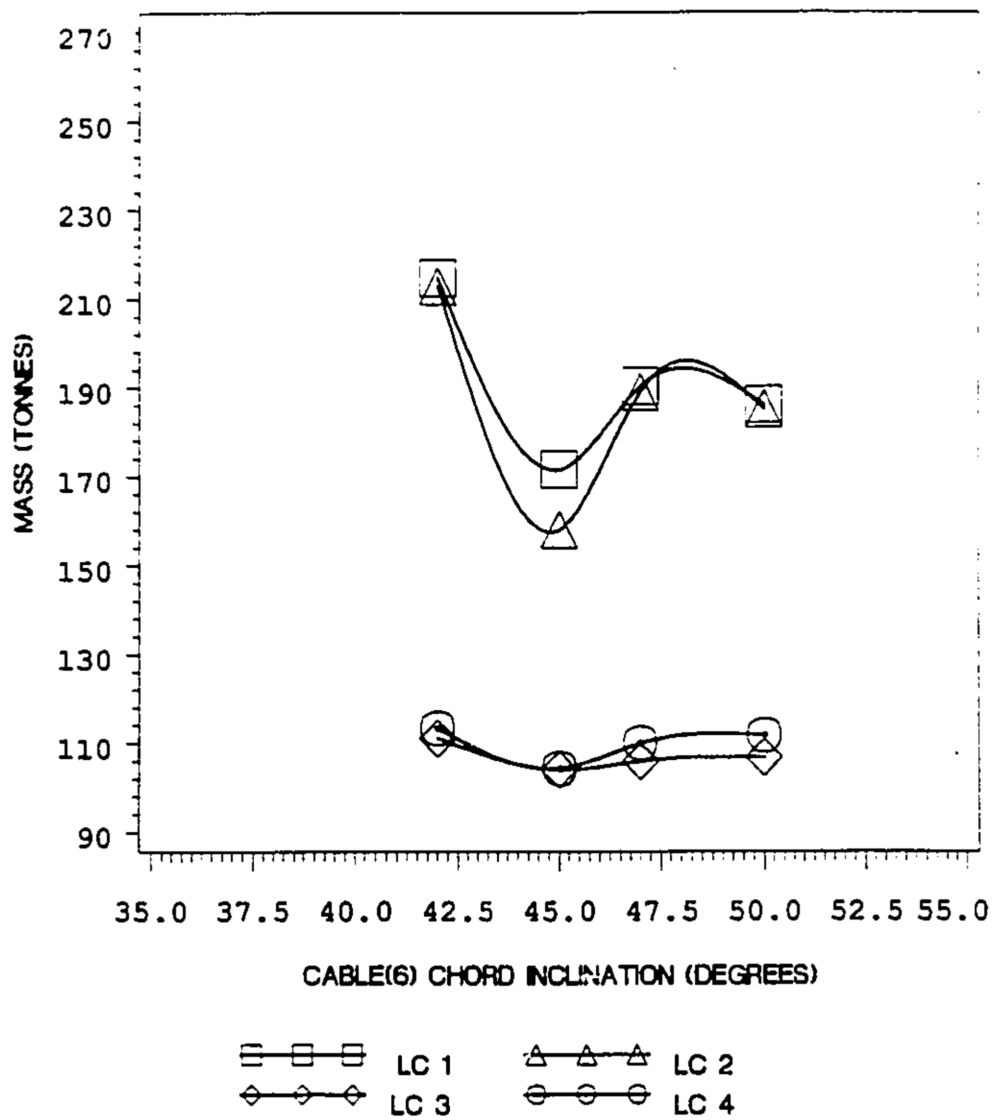

FIG 7.31 : MASS vs CABLE(6) CHORD INCLINATION

(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

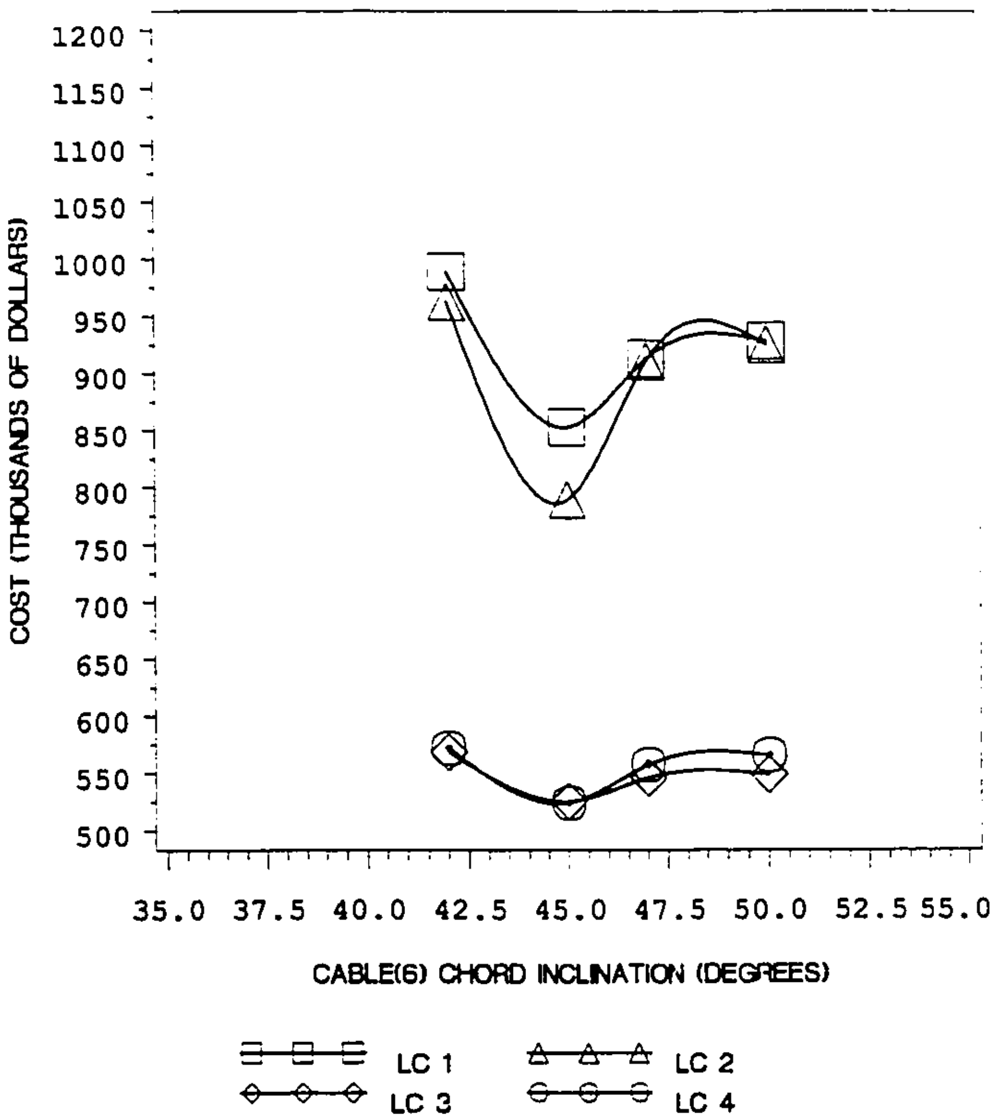

FIG 7.32 : COST vs CABLE(6) CHORD INCLINATION

(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

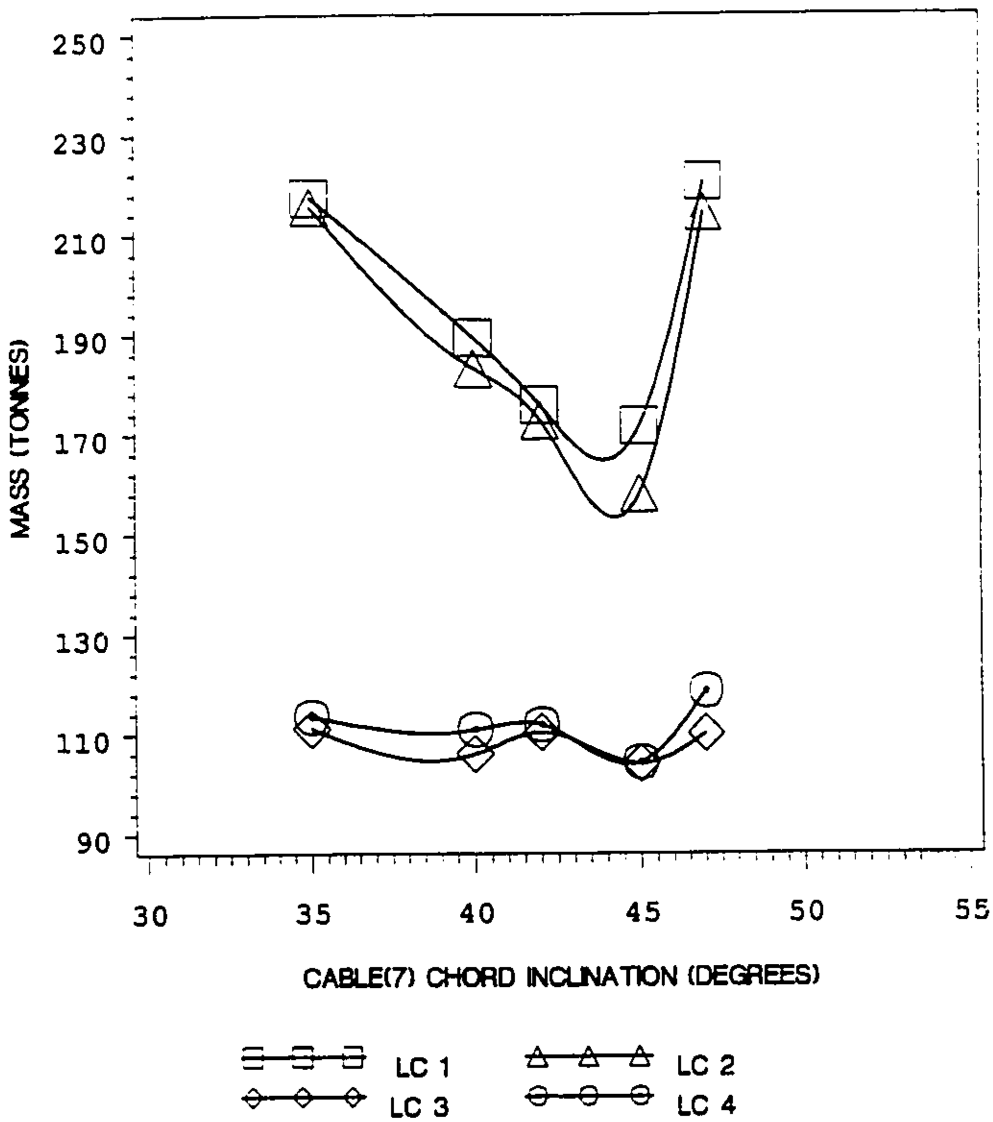

FIG 7.33 : MASS vs CABLE(7) CHORD INCLINATION (Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

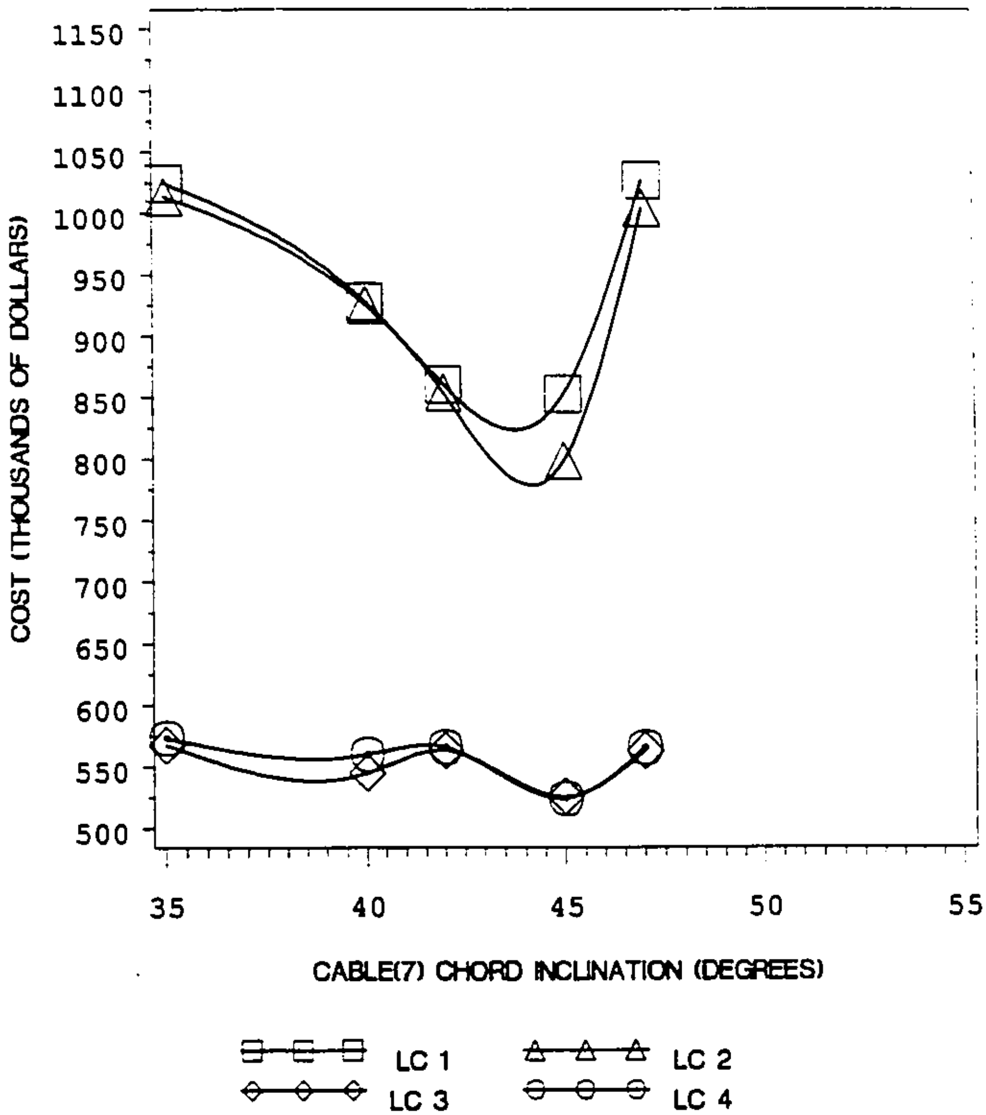

FIG 7.34 : COST vs CABLE(7) CHORD INCLINATION
(Guy Anchor Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

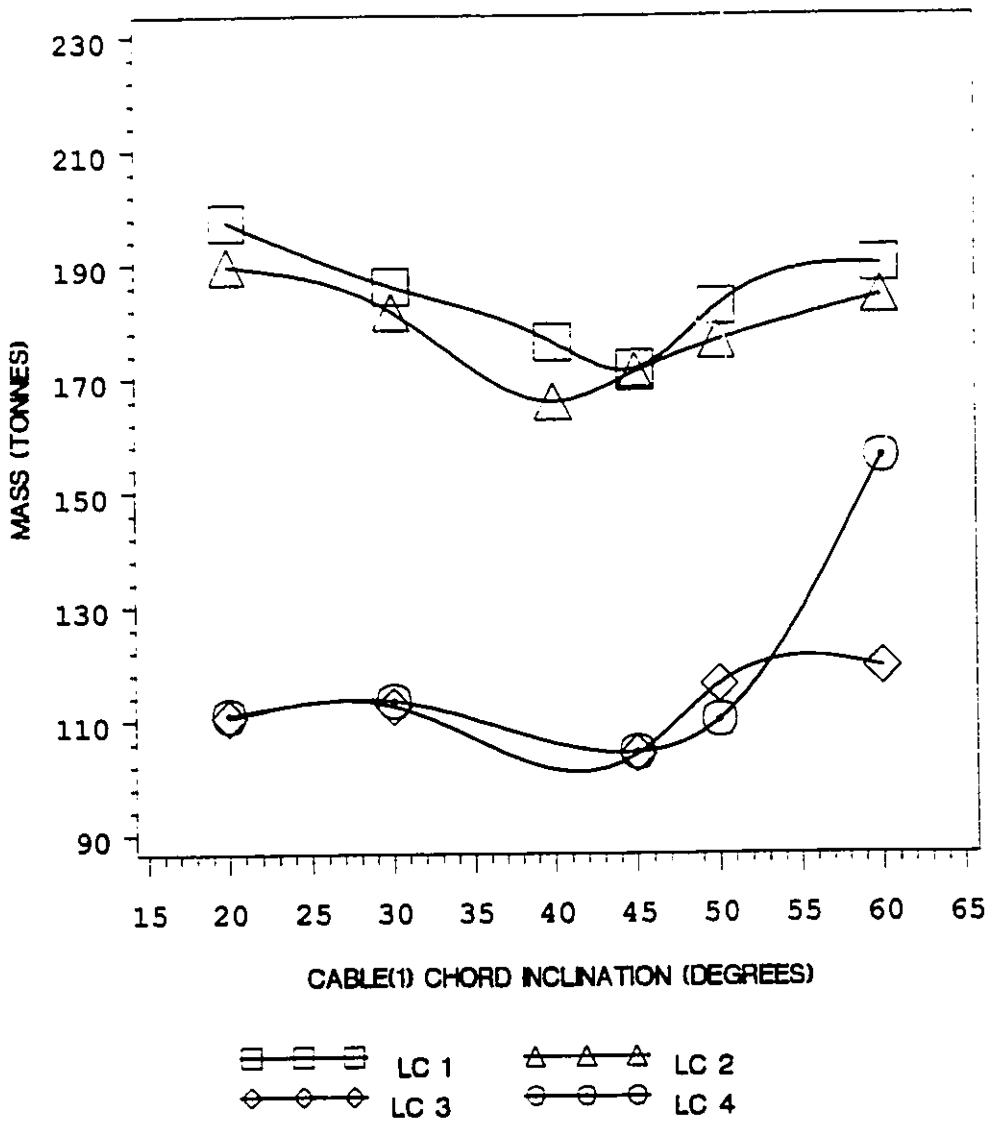

FIG 7.35 : MASS vs CABLE(1) CHORD INCLINATION
(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

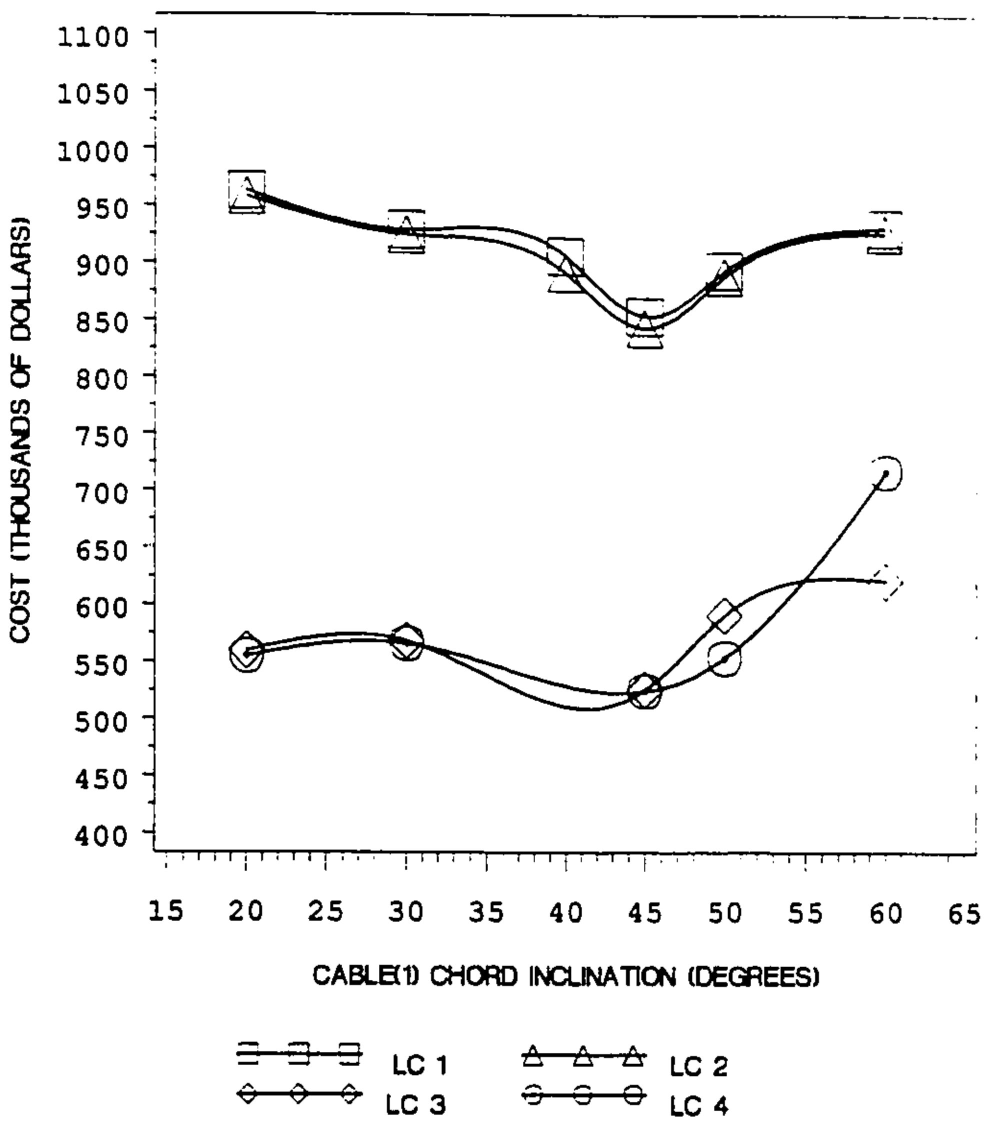

FIG 7.36 : COST vs CABLE(1) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

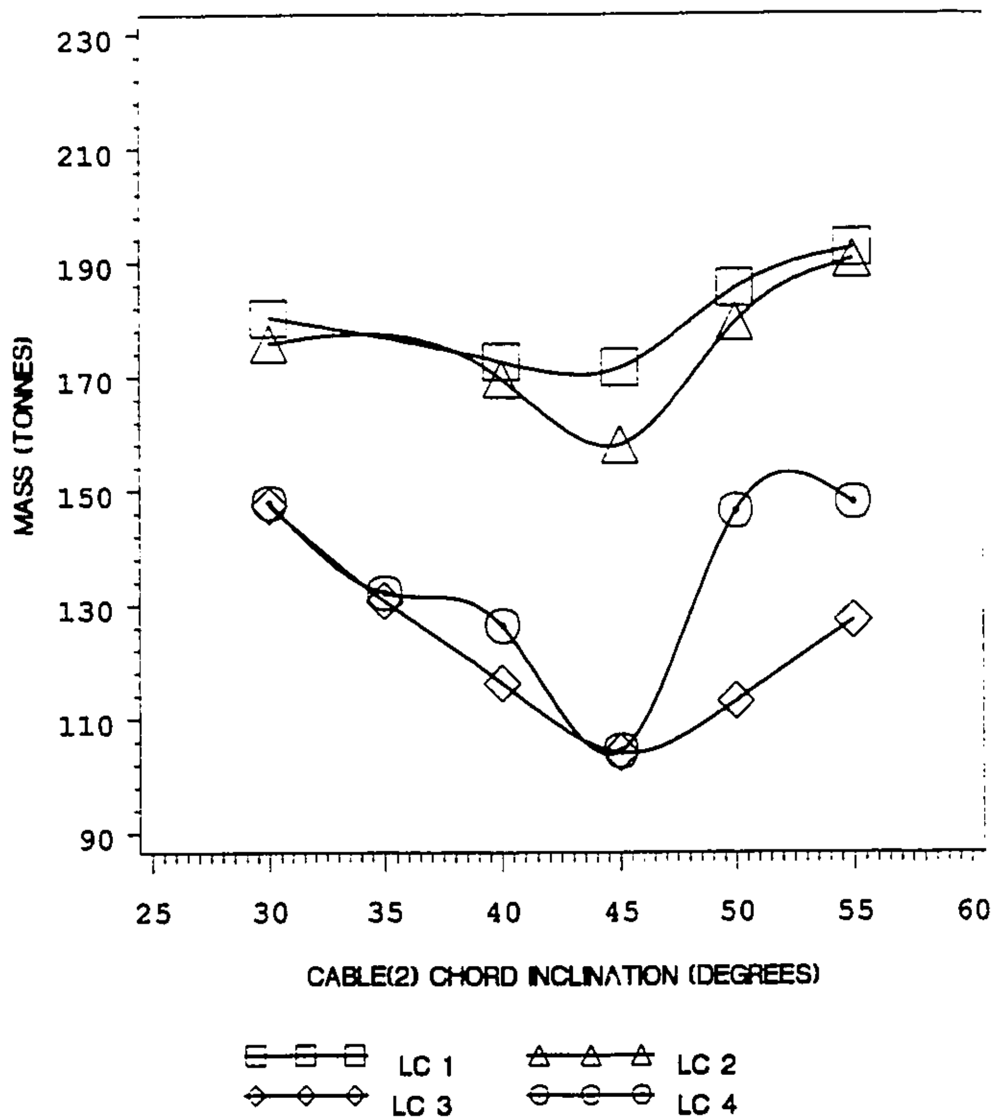

FIG 7.37 : MASS vs CABLE(2) CHORD INCLINATION (Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

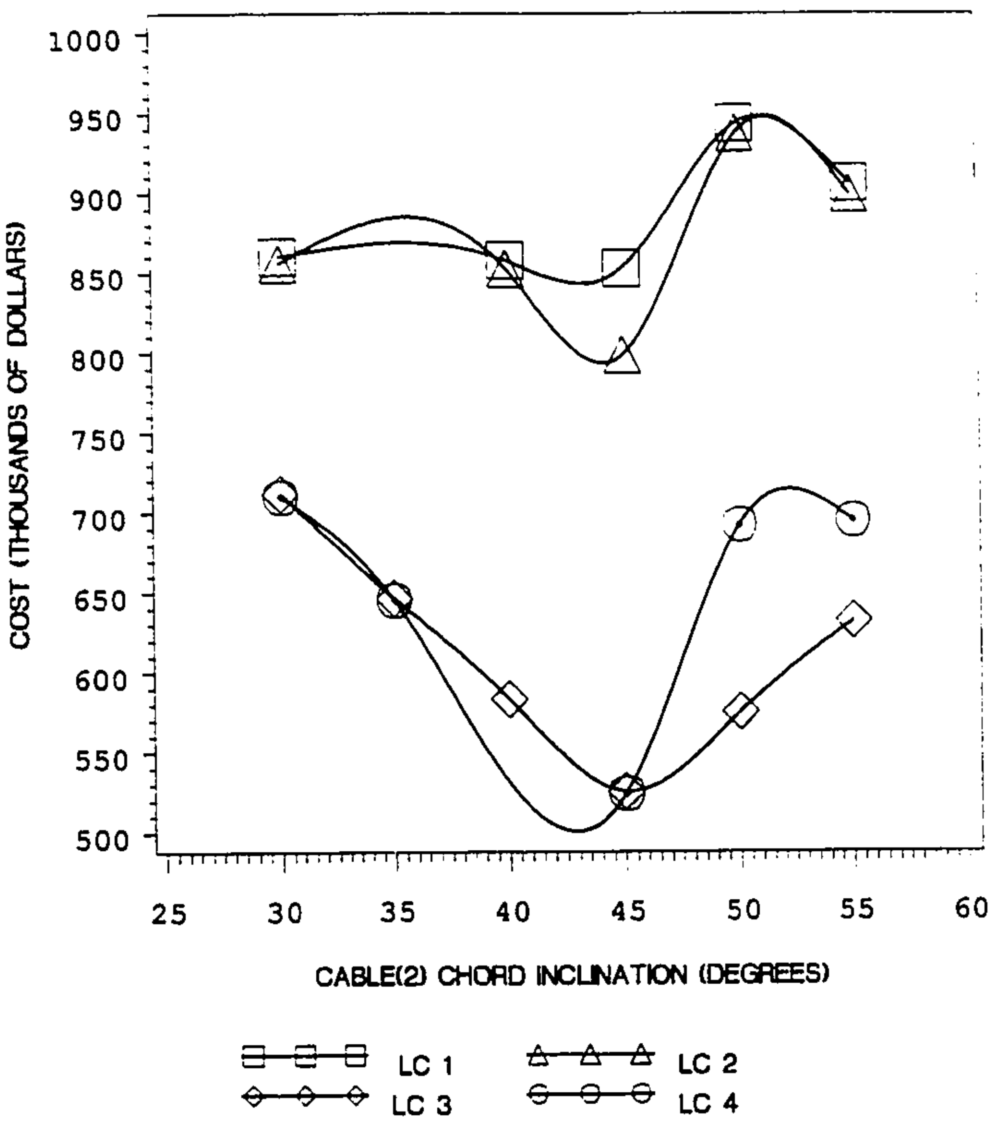

FIG 7.38 : COST vs CABLE(2) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

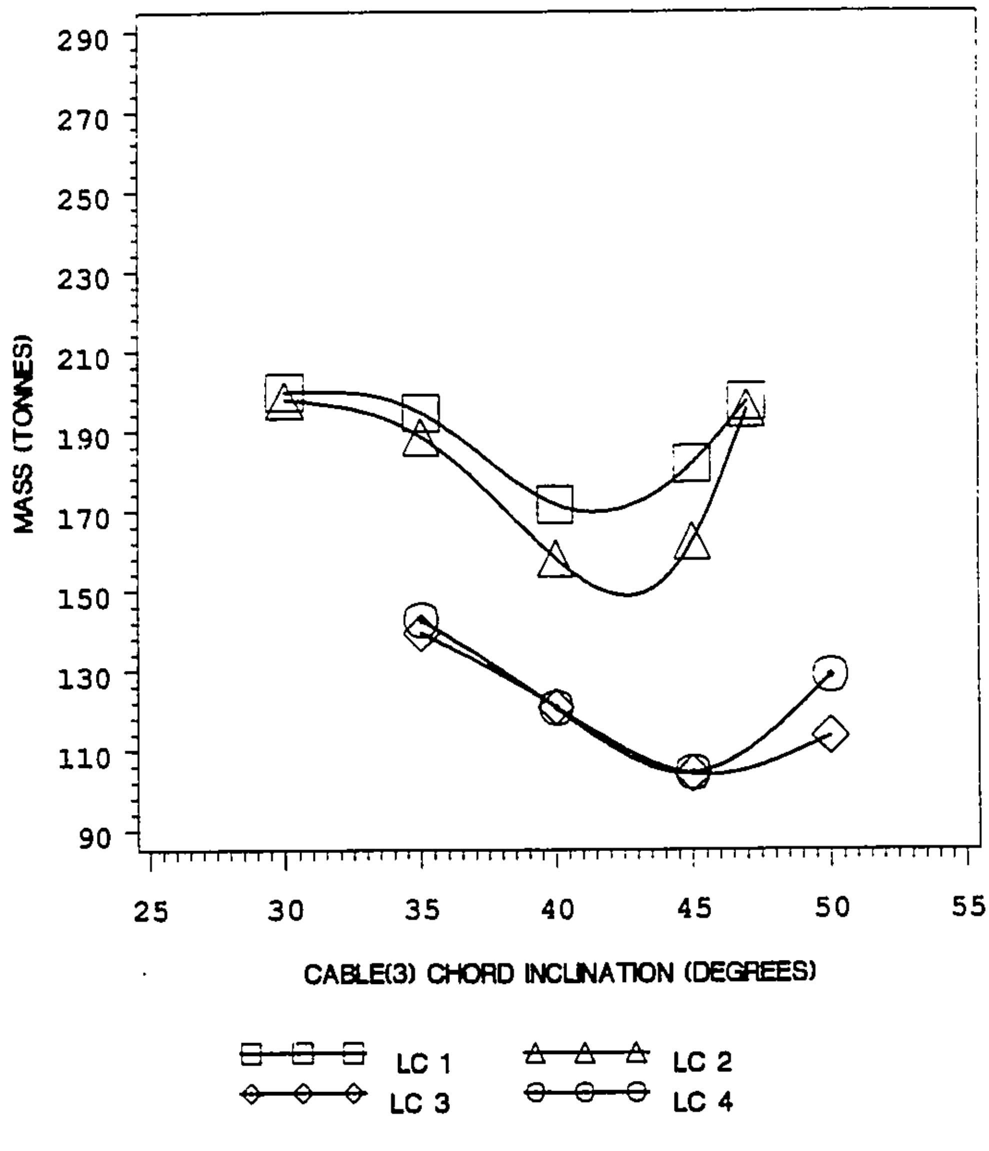

FIG 7.39 : MASS vs CABLE(3) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

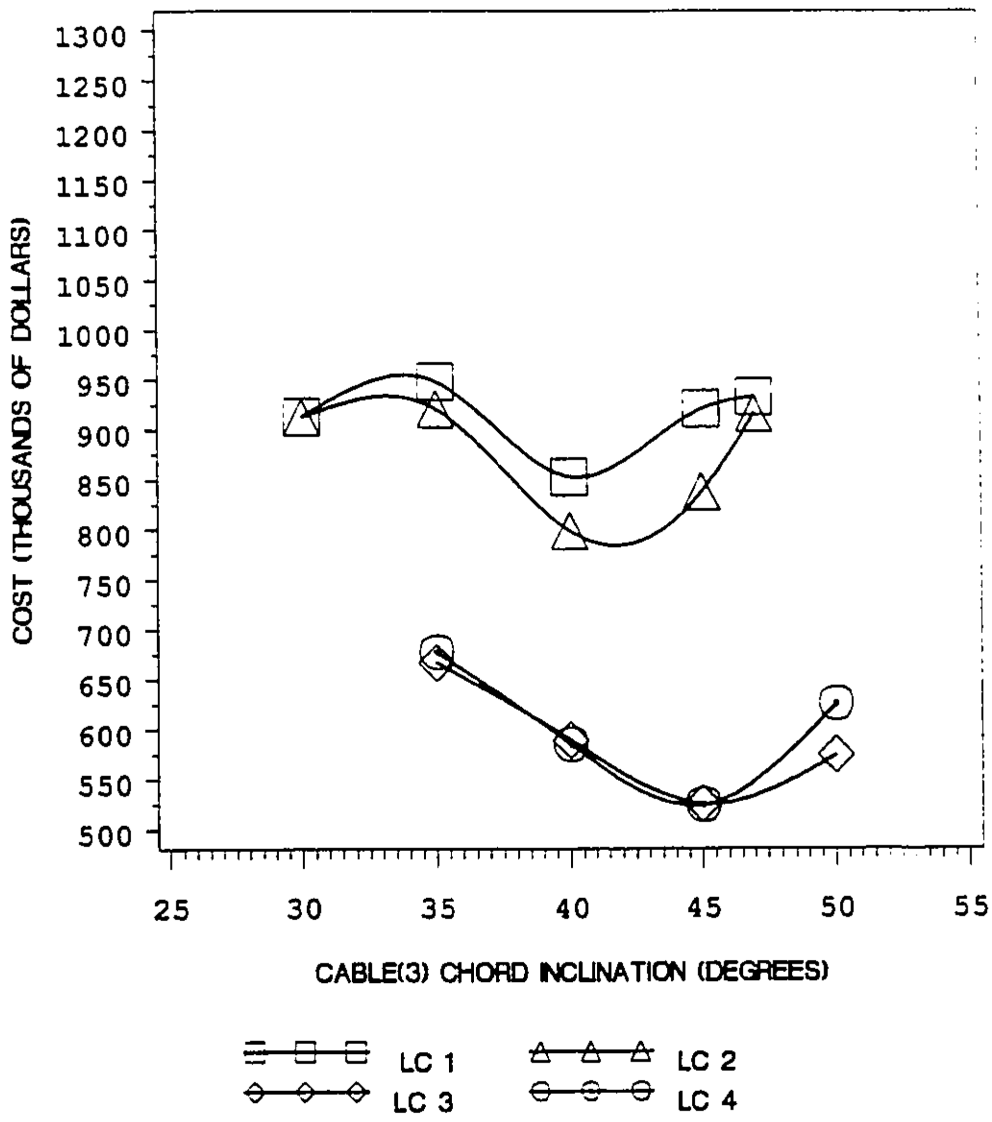

FIG 7.40 : COST vs CABLE(3) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

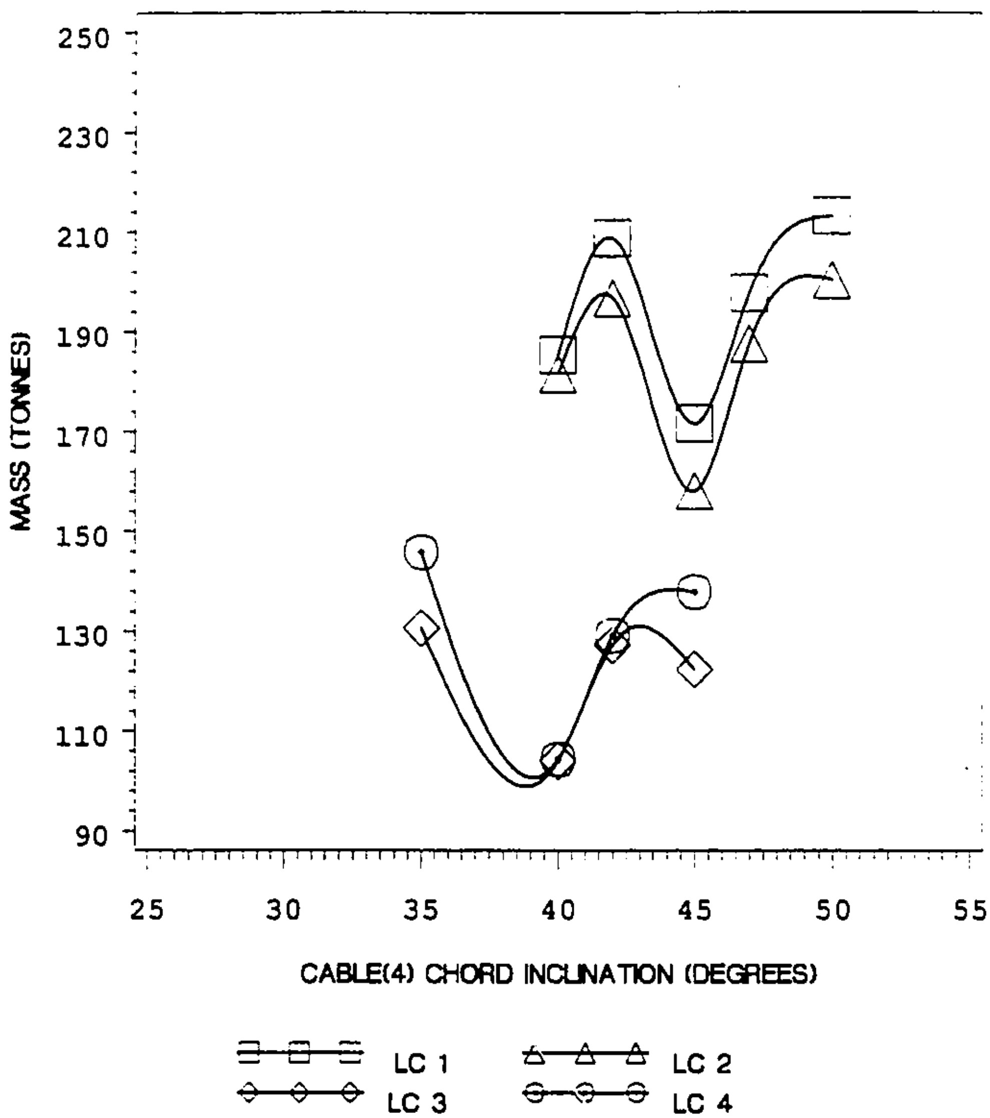

FIG 7.41 : MASS vs CABLE(4) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

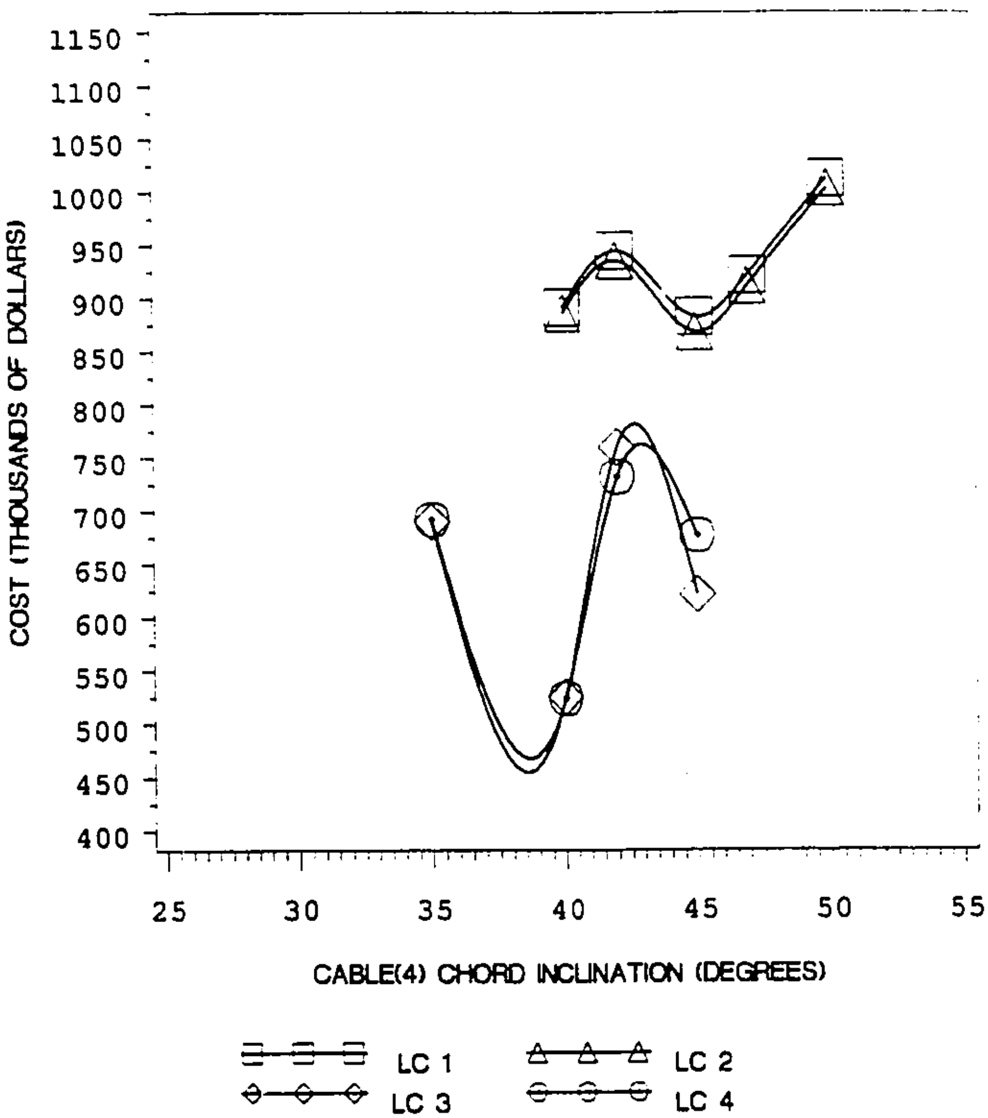

FIG 7.42 : COST vs CABLE(4) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

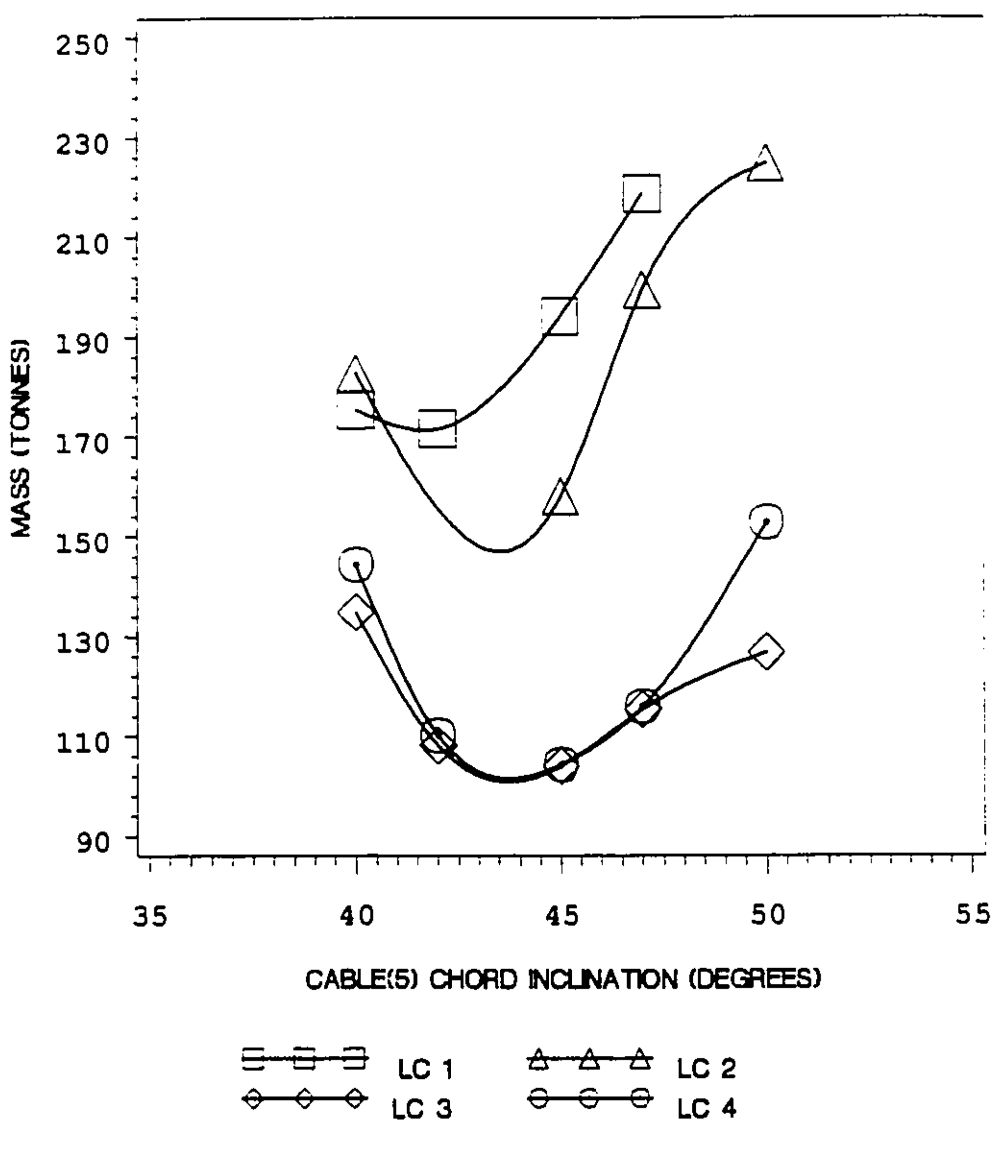

FIG 7.43 : MASS vs CABLE(5) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

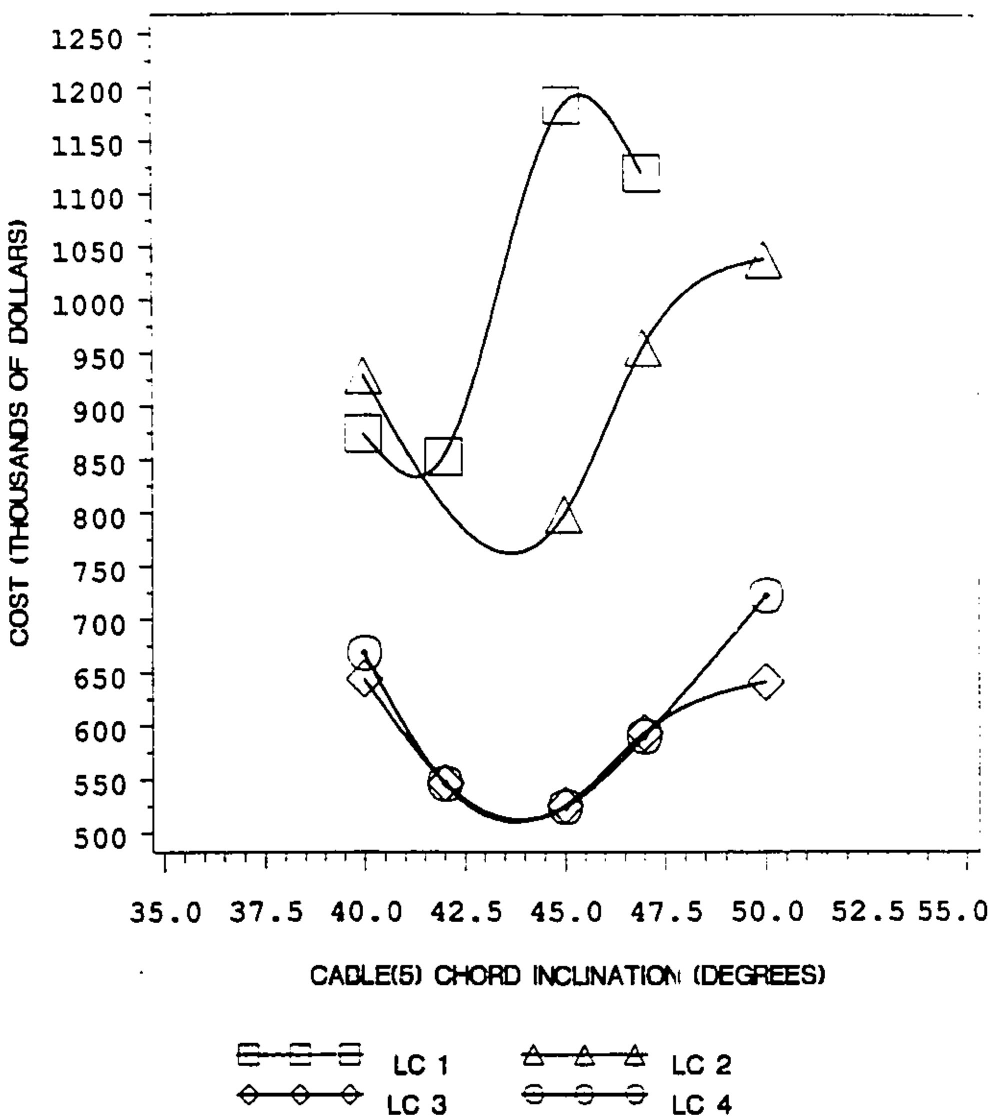

FIG 7.44 : COST vs CABLE(5) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

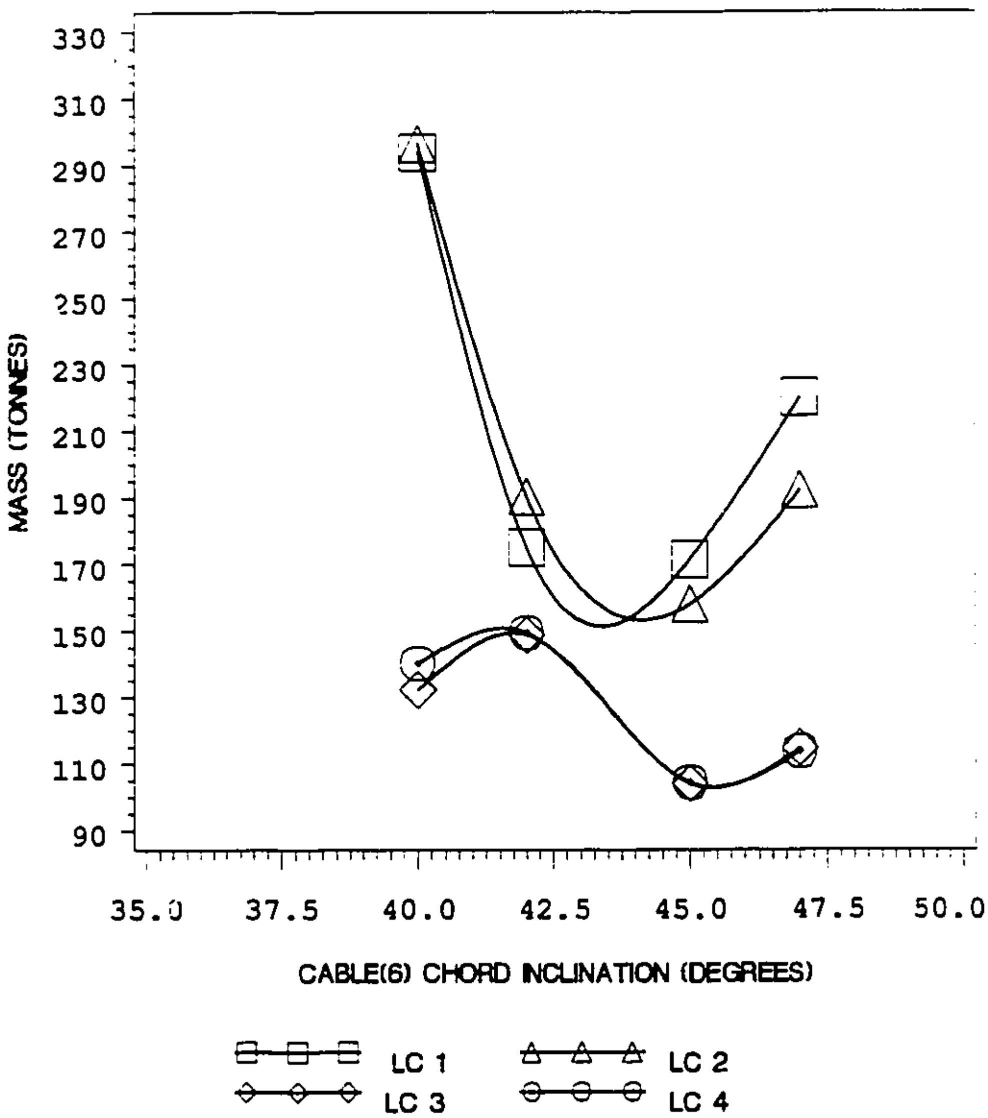

FIG 7.45 : MASS vs CABLE(6) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

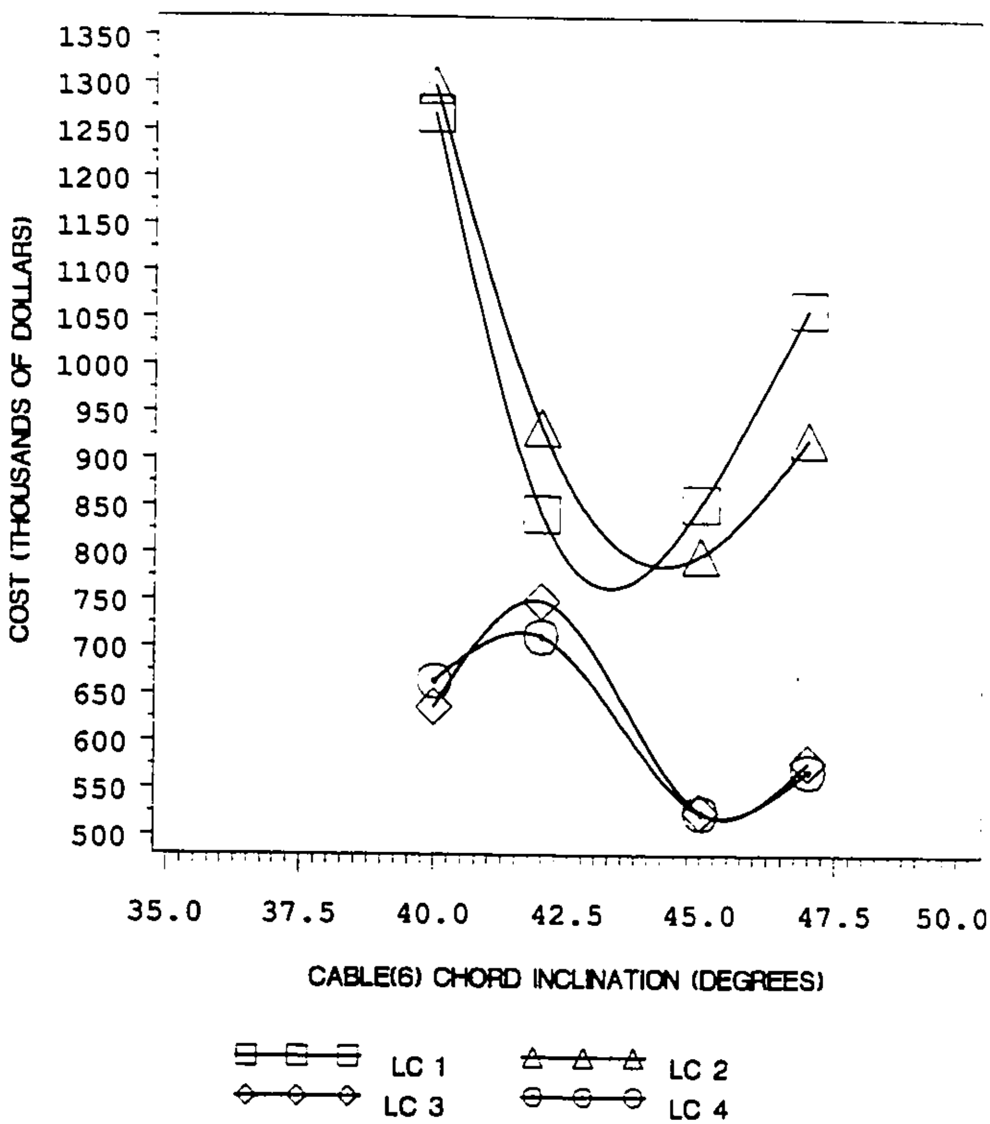

FIG 7.46 : COST vs CABLE(6) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

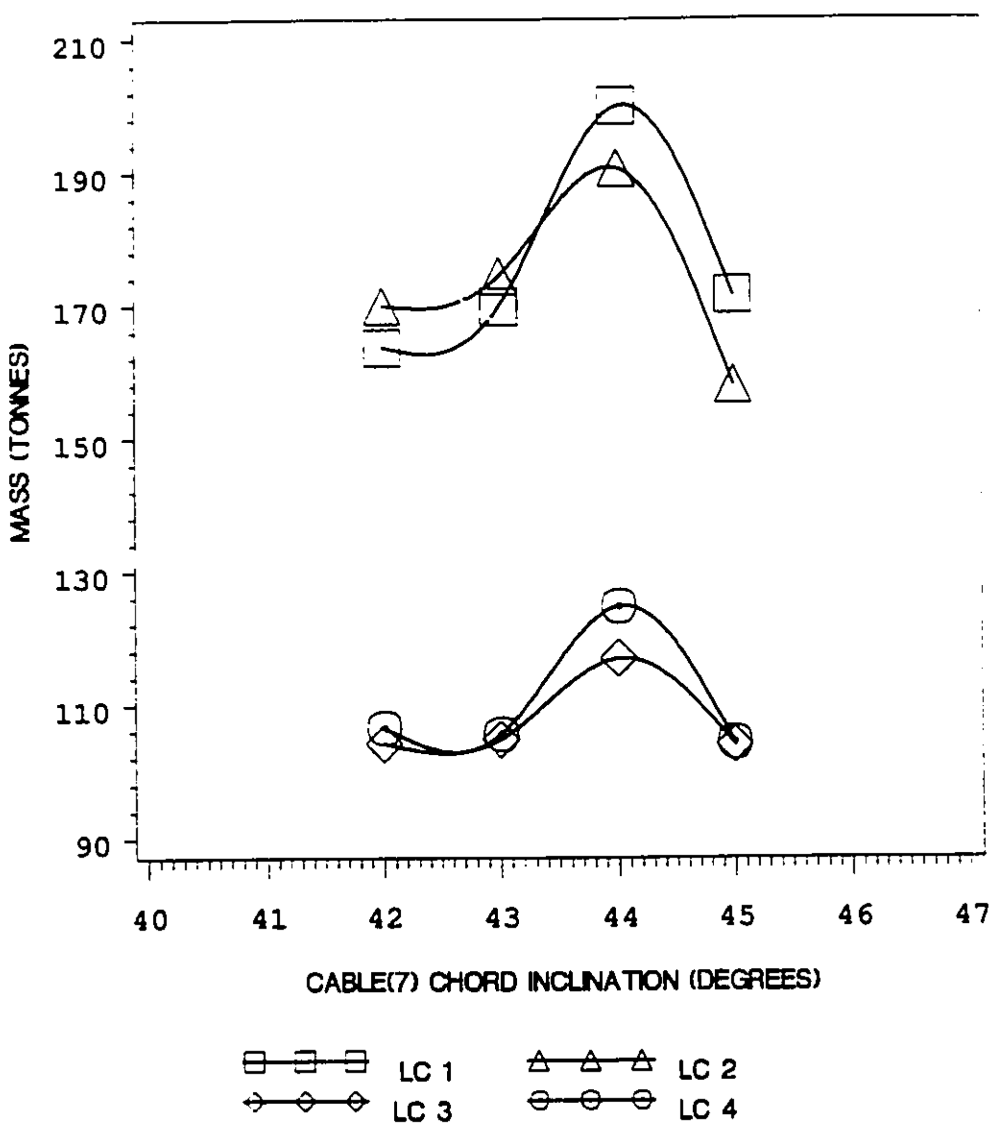

FIG 7.47 : MASS vs CABLE(7) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : 2.0m ; MAST HEIGHT : 350m ; NO. OF GUY LEVELS : 7

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

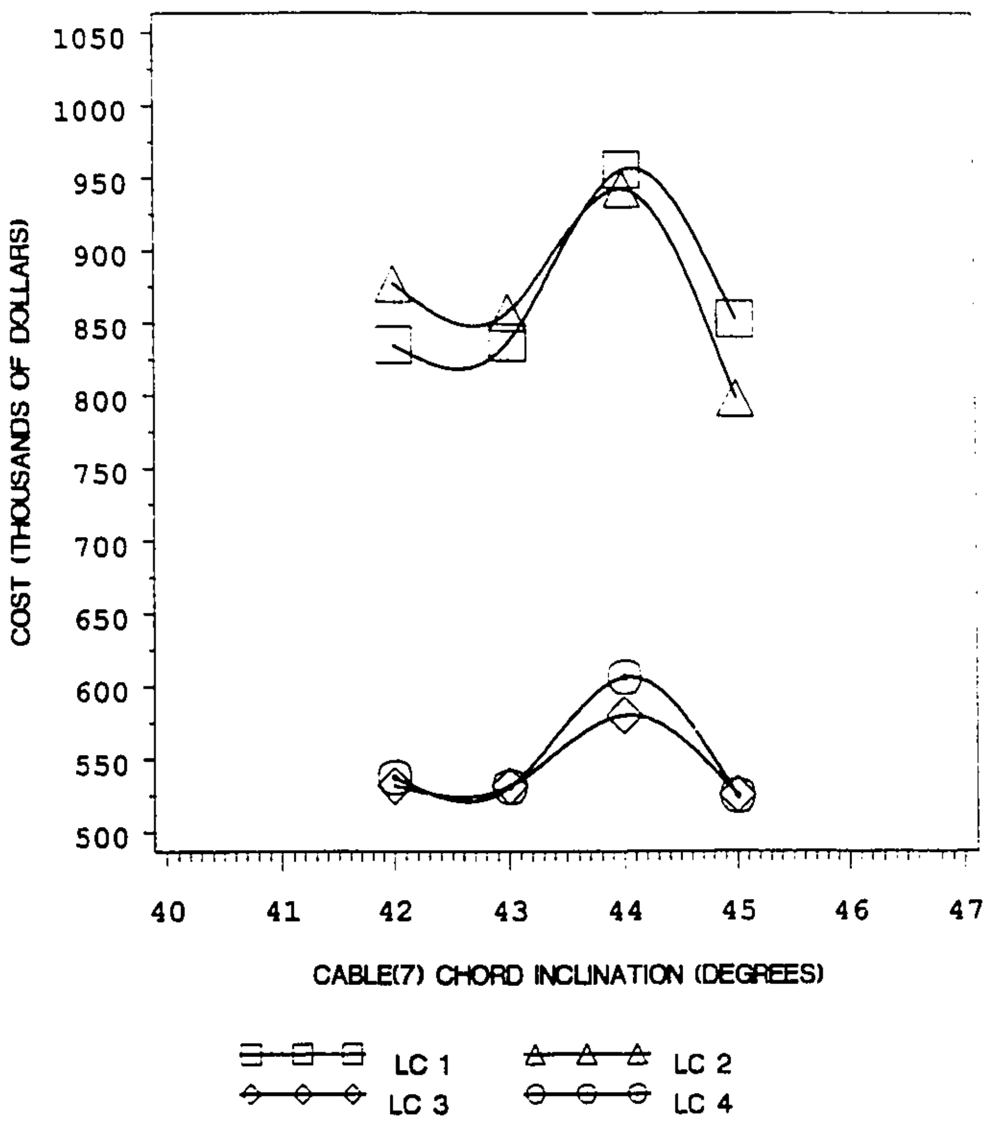

FIG 7.48 : COST vs CABLE(7) CHORD INCLINATION

(Guy Connection Point Varies)

MAST SIZE : VARIES ; MAST HEIGHT : 350m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

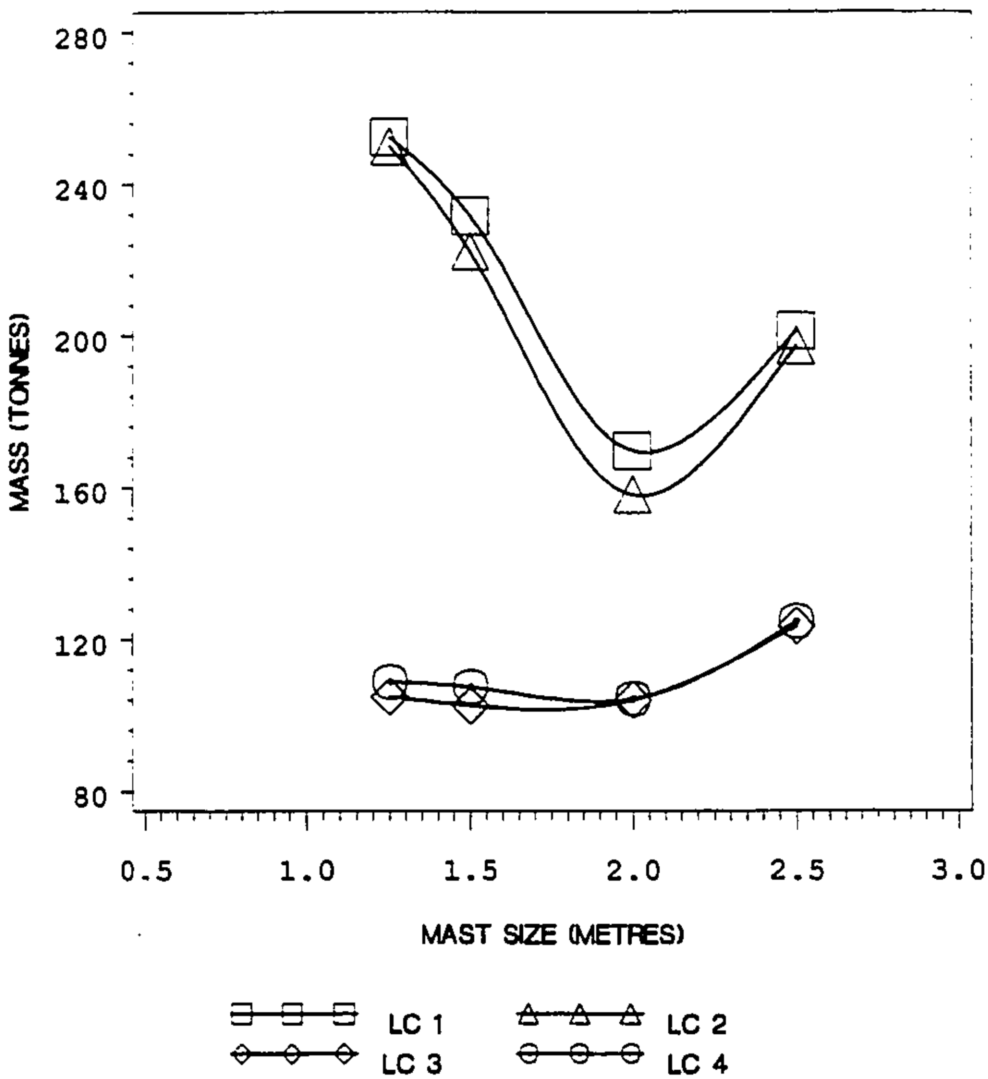

FIG 7.49 : MASS vs MAST SIZE

MAST SIZE : VARIES ; MAST HEIGHT : 350m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

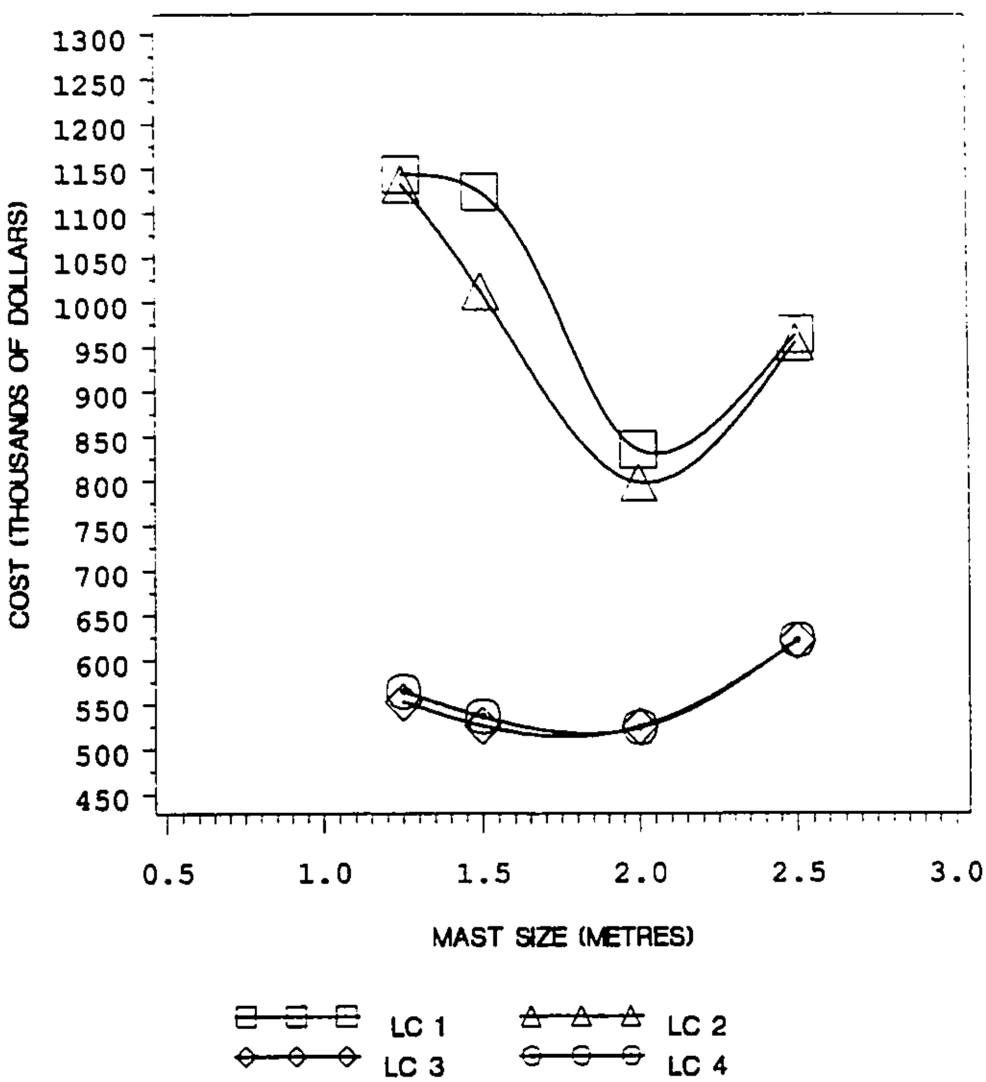

FIG 7.50 : COST vs MAST SIZE

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Appendix A

Parametric Study For A Tower of 100 m Height

With Two Guy Connections With

Mast Sizes 1.25 m, 1.5 m, and 2.5 m.

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

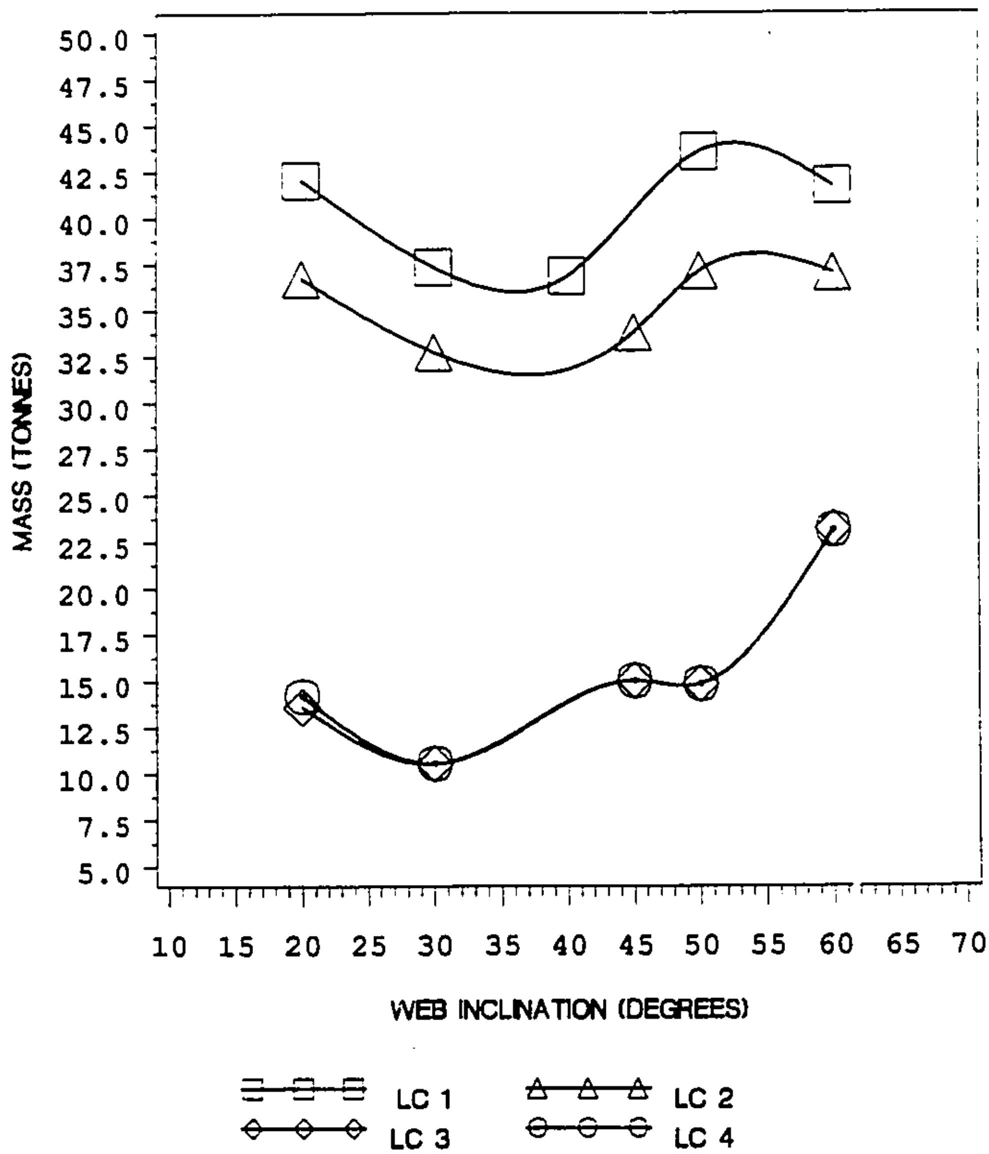


FIG A.1 : MASS vs WEB INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

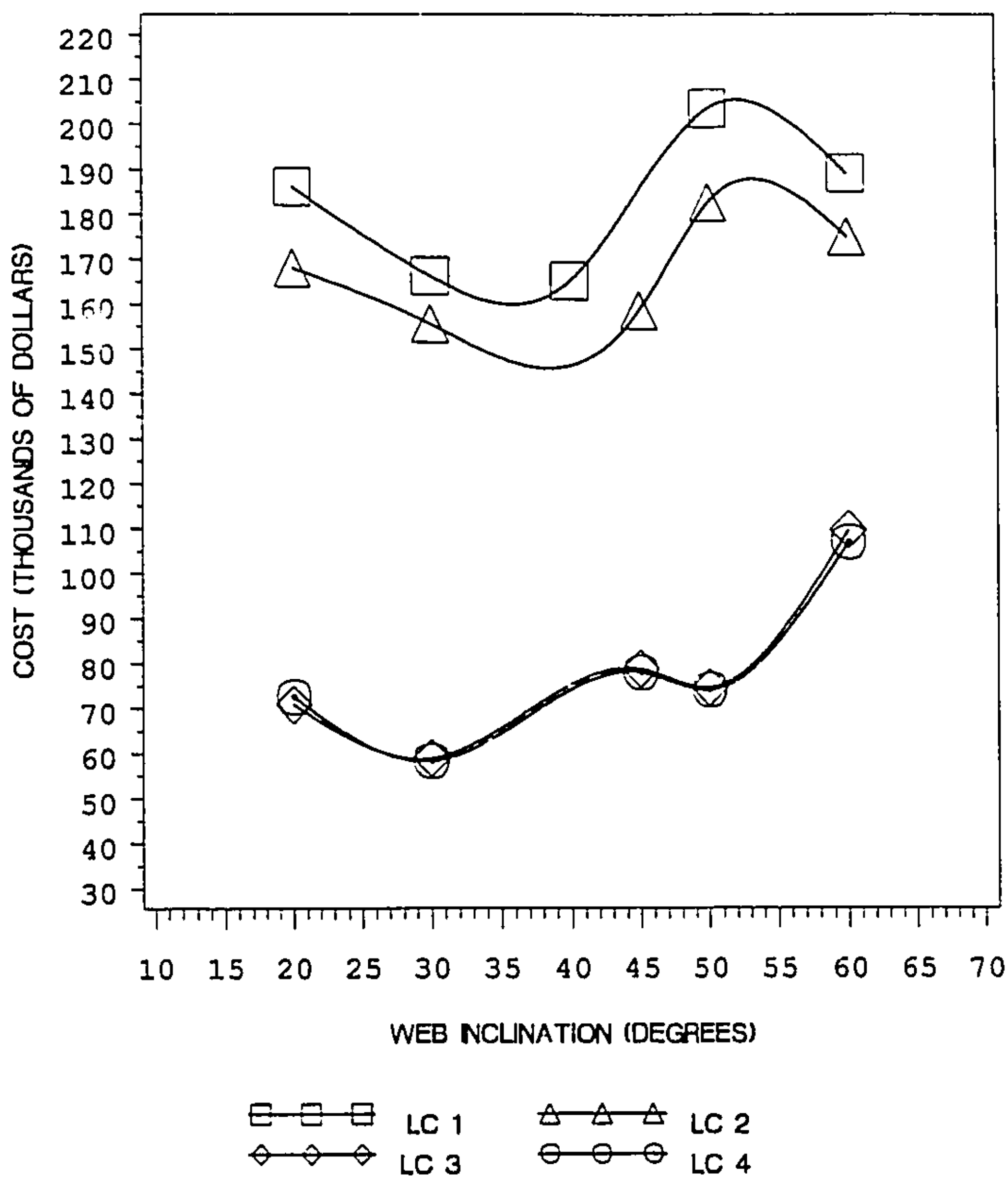


FIG A.2 : COST vs WEB INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

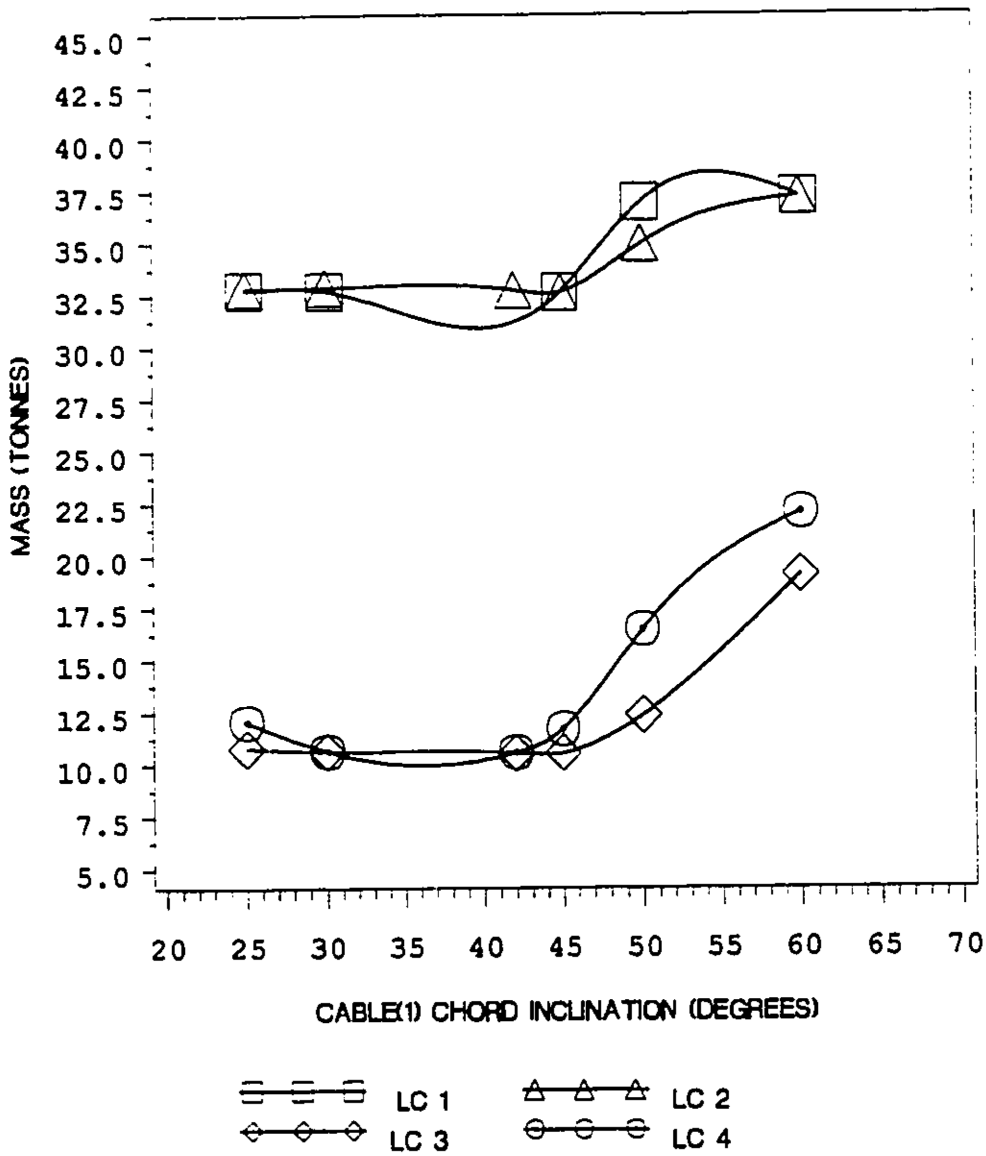


FIG A.3 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

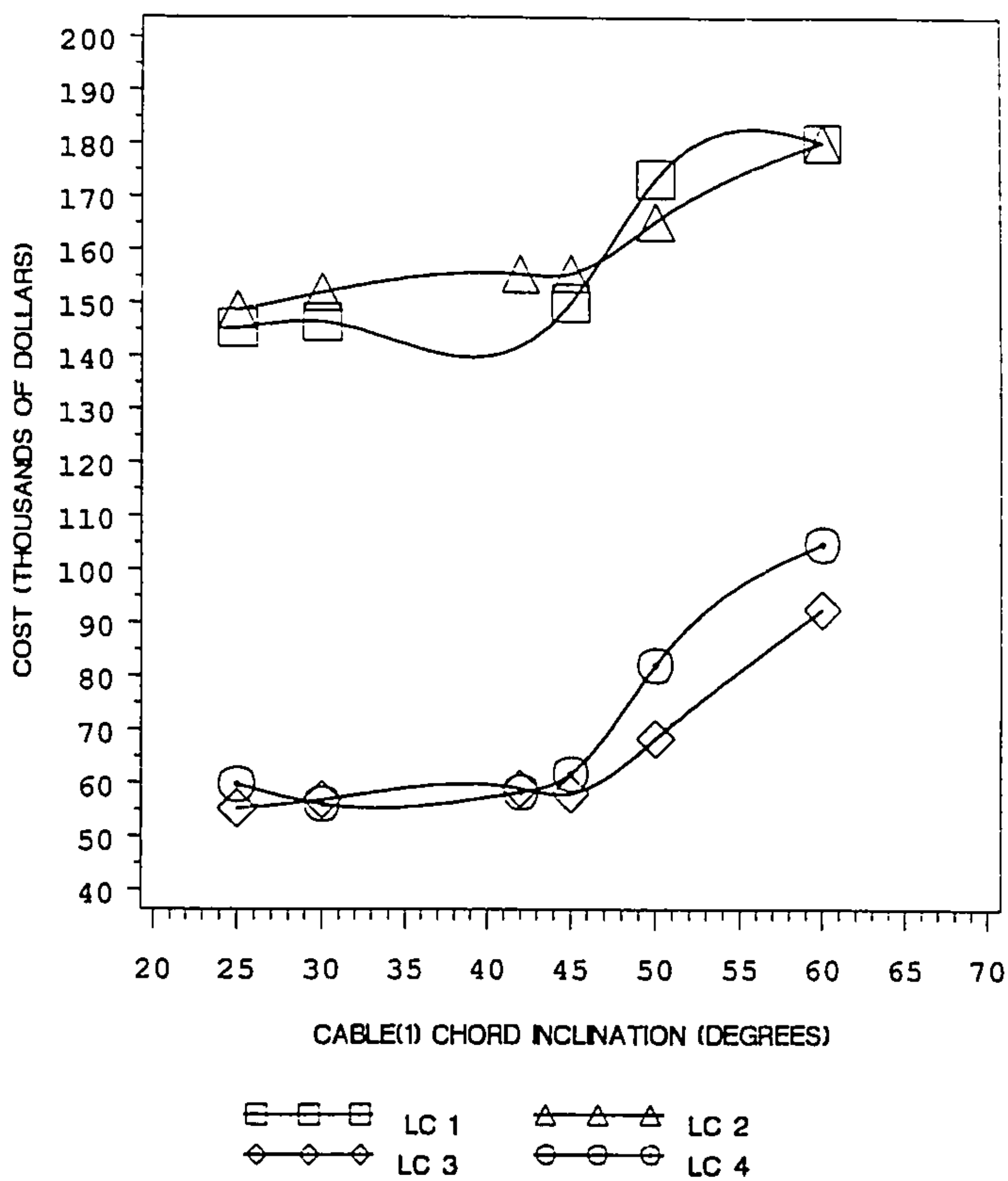


FIG A.4 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR LC 1,2&3, 77.94m FOR LC 4 ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

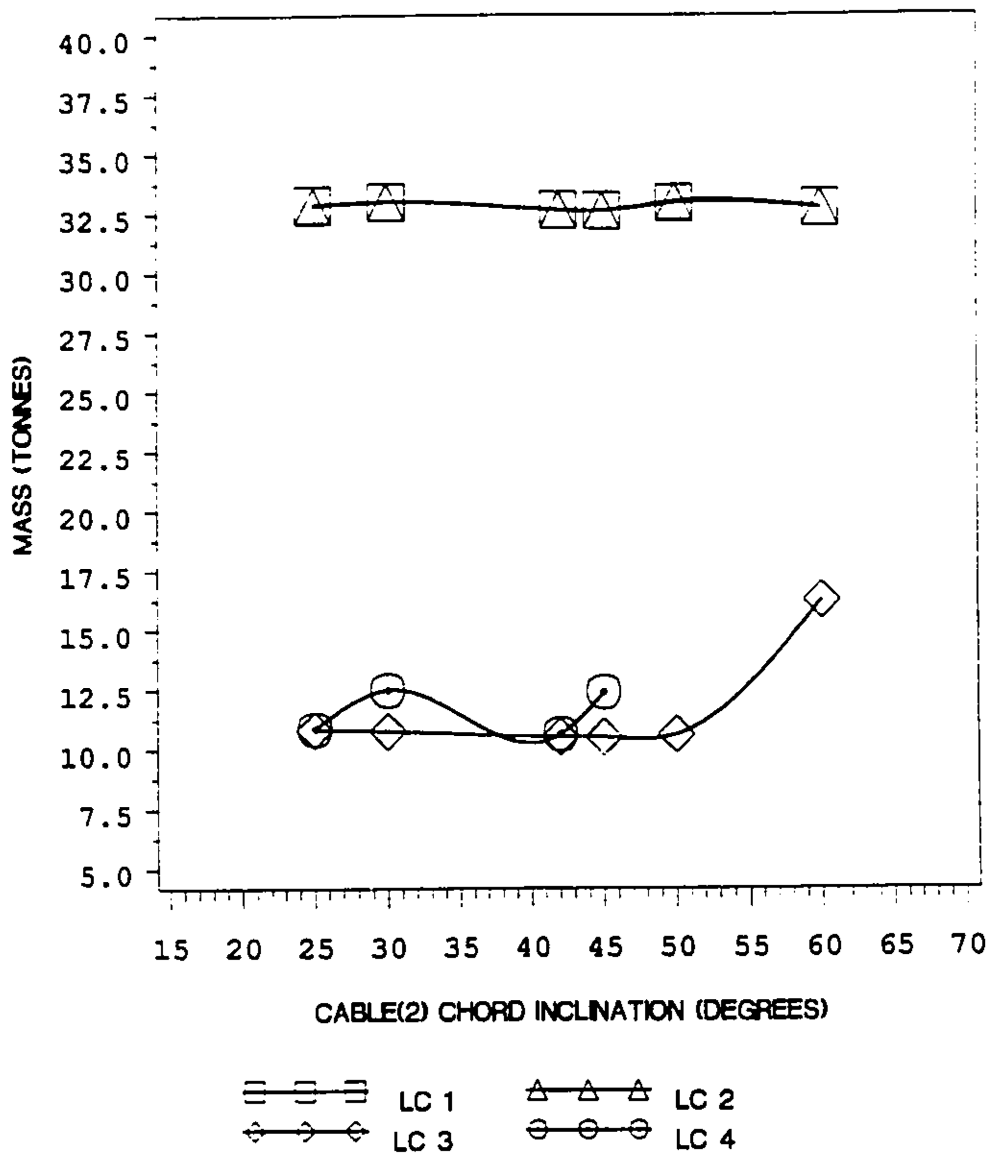


FIG A.5 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR LC 1,2&3, 77.94m FOR LC 4 ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

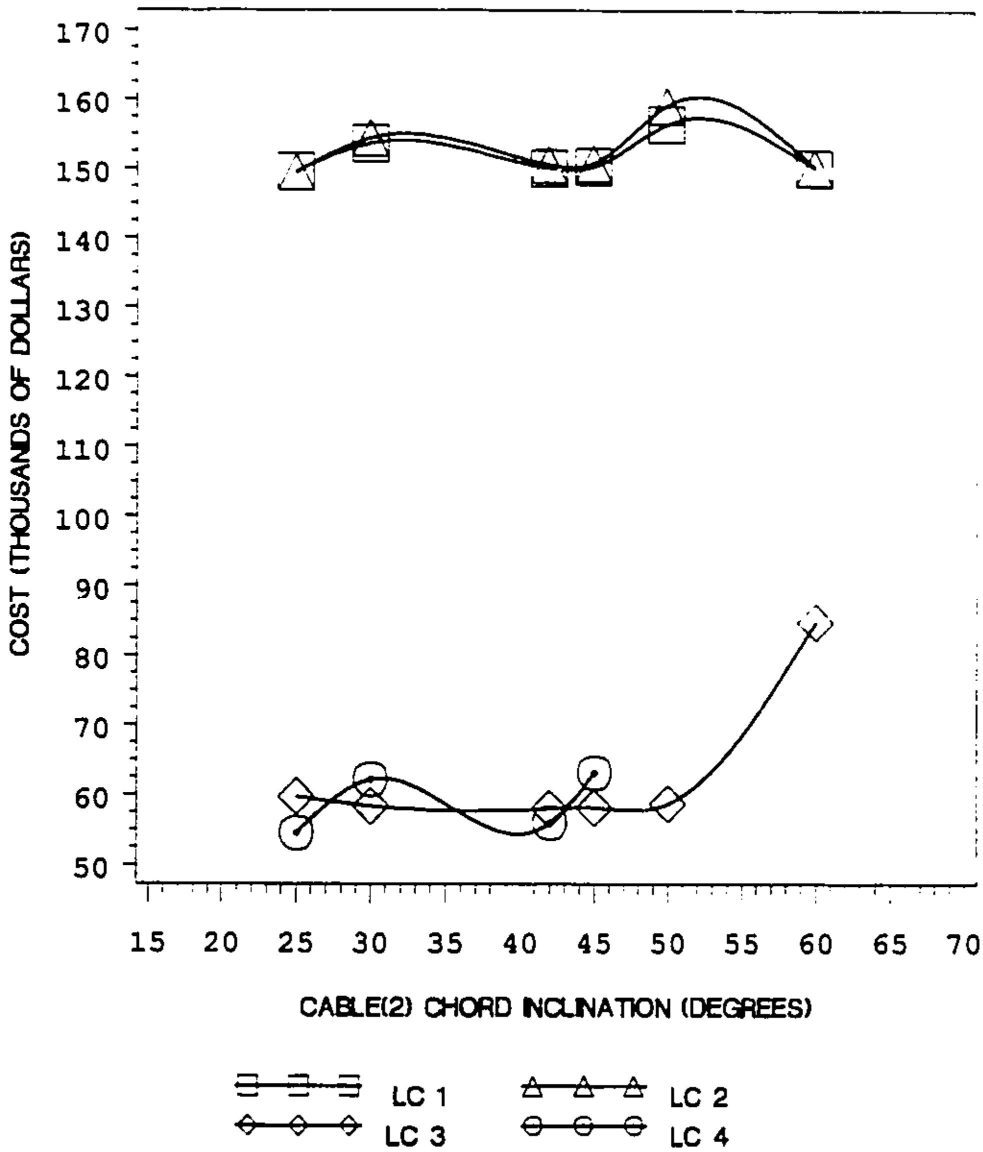


FIG A.6 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR LC 1,2&3, 77.94m FOR LC 4 ;
CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

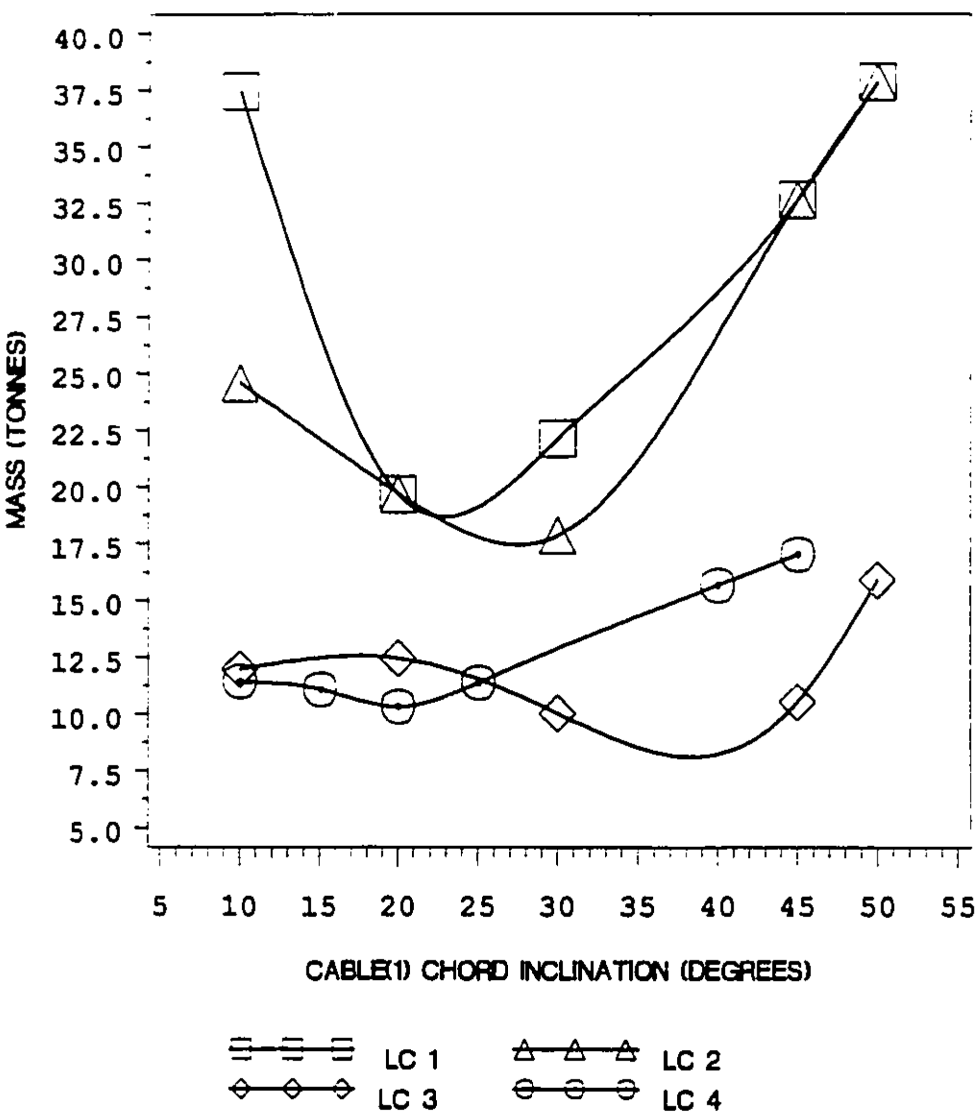


FIG A.7 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR LC 1,2&3, 77.94m FOR LC 4 ,
CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

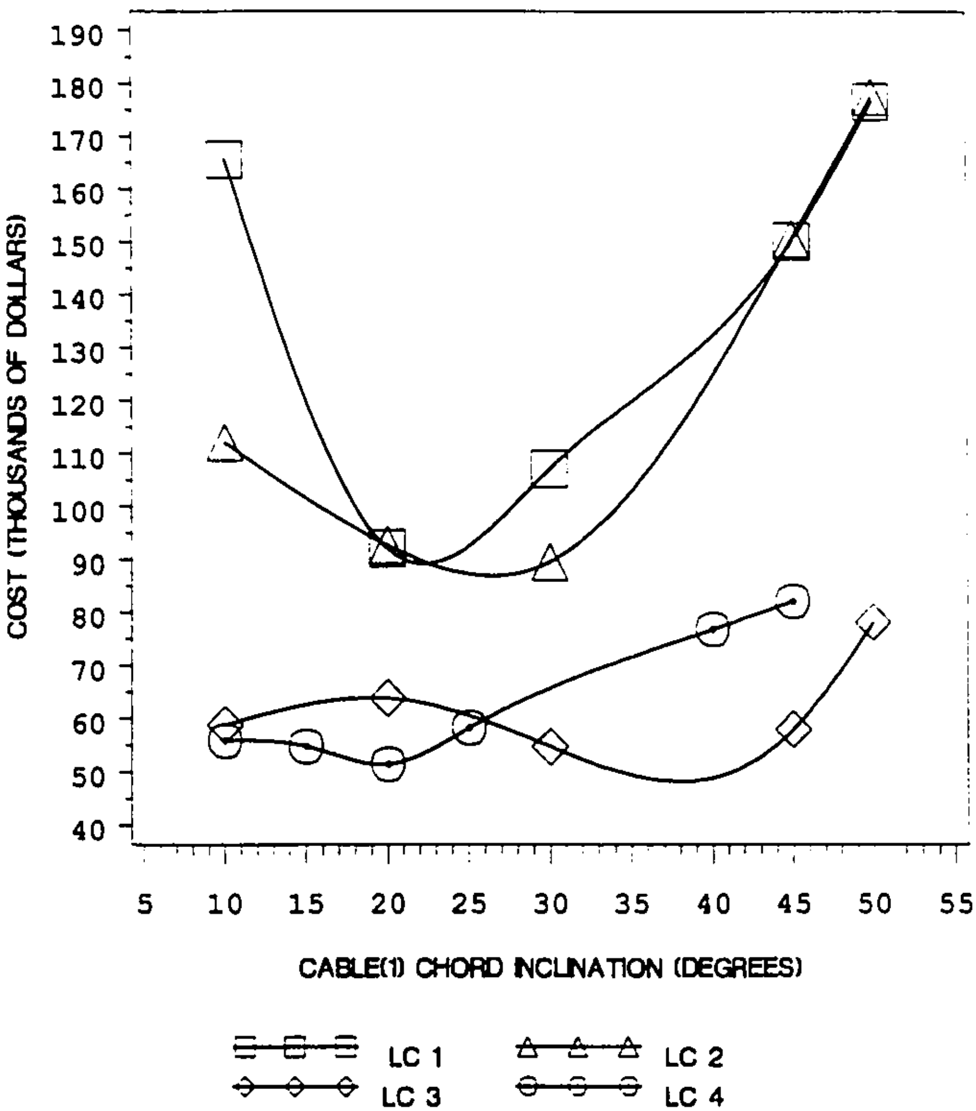


FIG A.8 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 16.38m FOR LC 1, 25.98m FOR LC 2&3, 28.37m FOR LC 4 ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR LC 1,2&3, 77.94m FOR LC 4 ;
CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

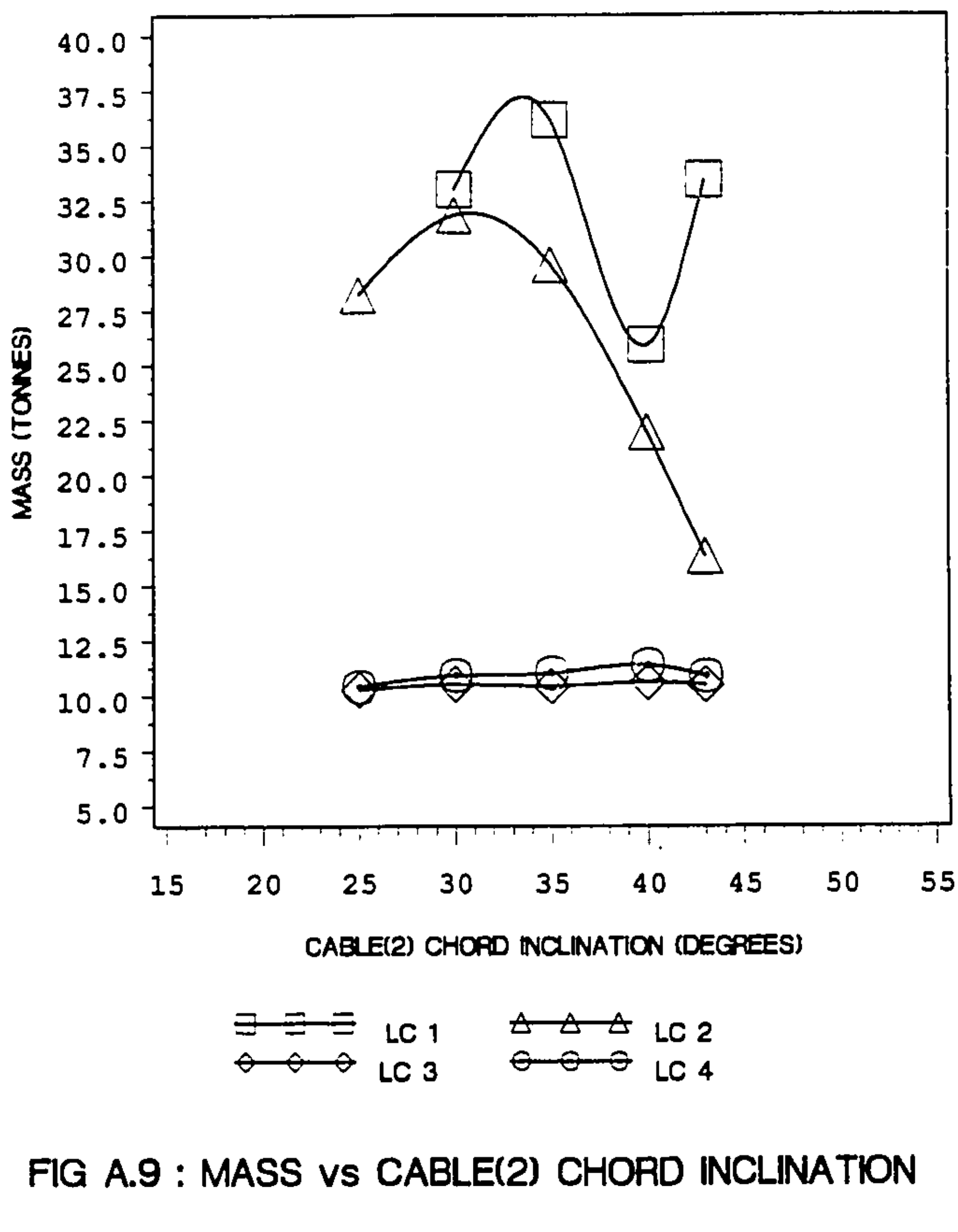


FIG A.9 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 2.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 16.38m FOR LC 1,25.98m FOR LC 2&3,28.37m FOR LC 4 ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR LC 1,2&3, 77.94m FOR LC 4 ;
CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

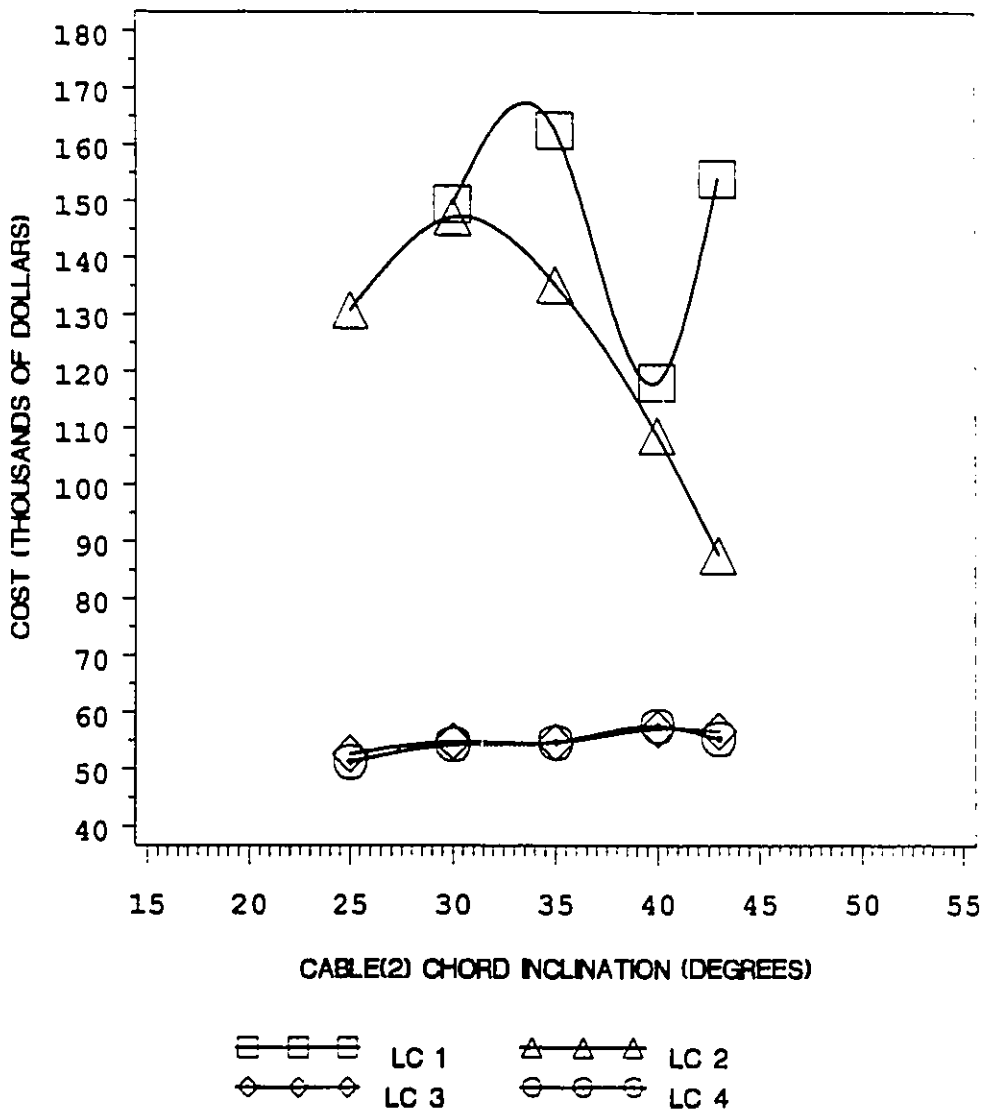


FIG A.10 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT . 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-II ICE
3. CLASS-A WIND + CLASS-II ICE
4. CLASS-A WIND + CLASS-IV ICE

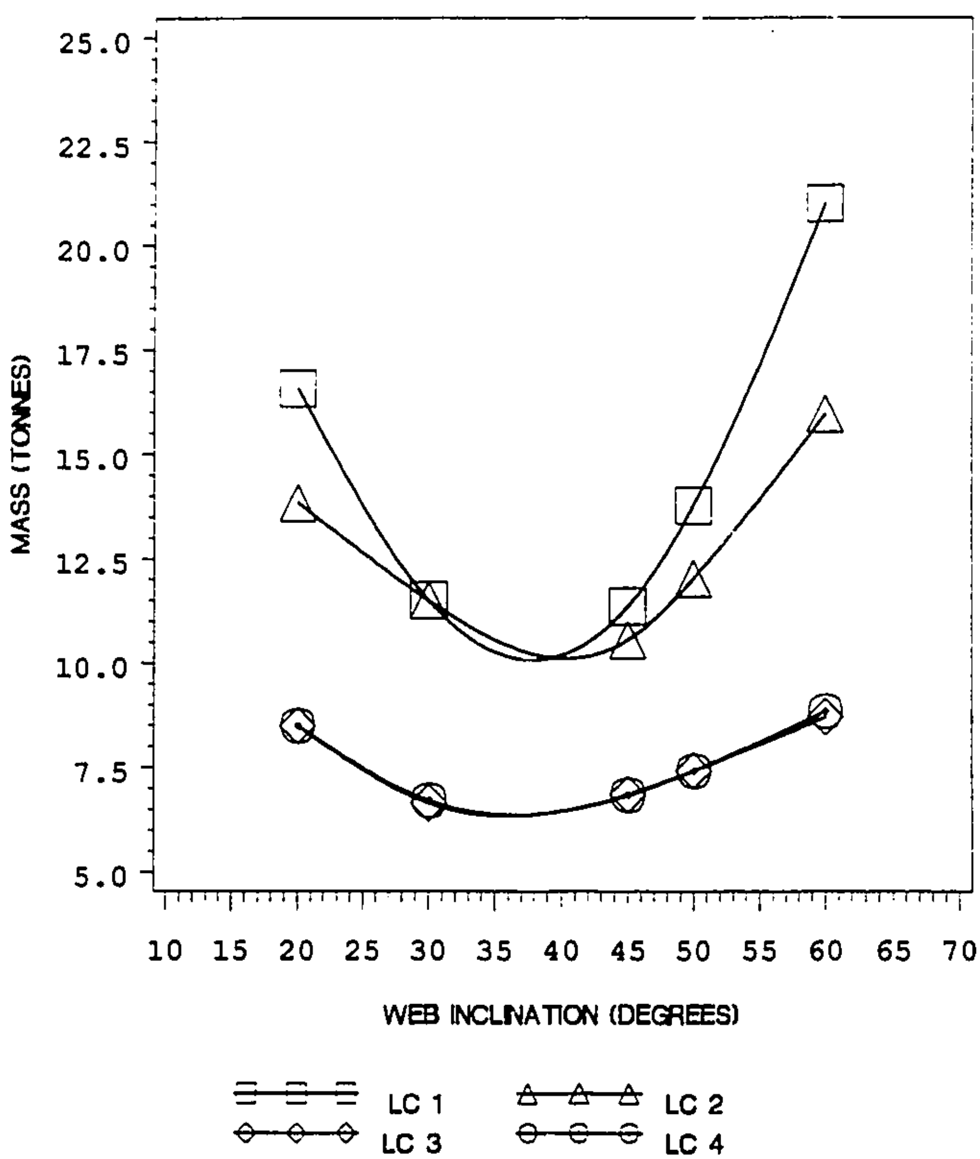


FIG A.11 : MASS vs WEB INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

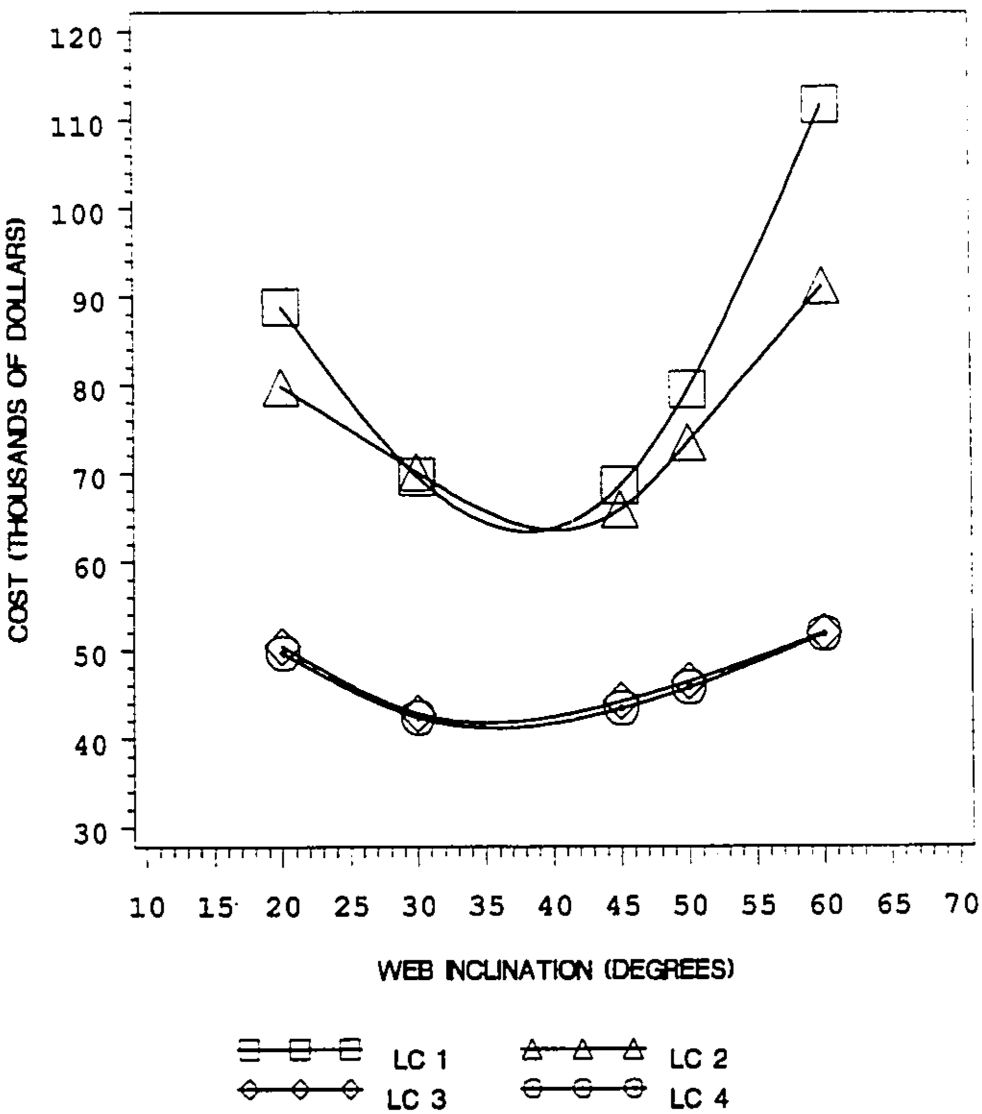


FIG A.12 : COST vs WEB INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

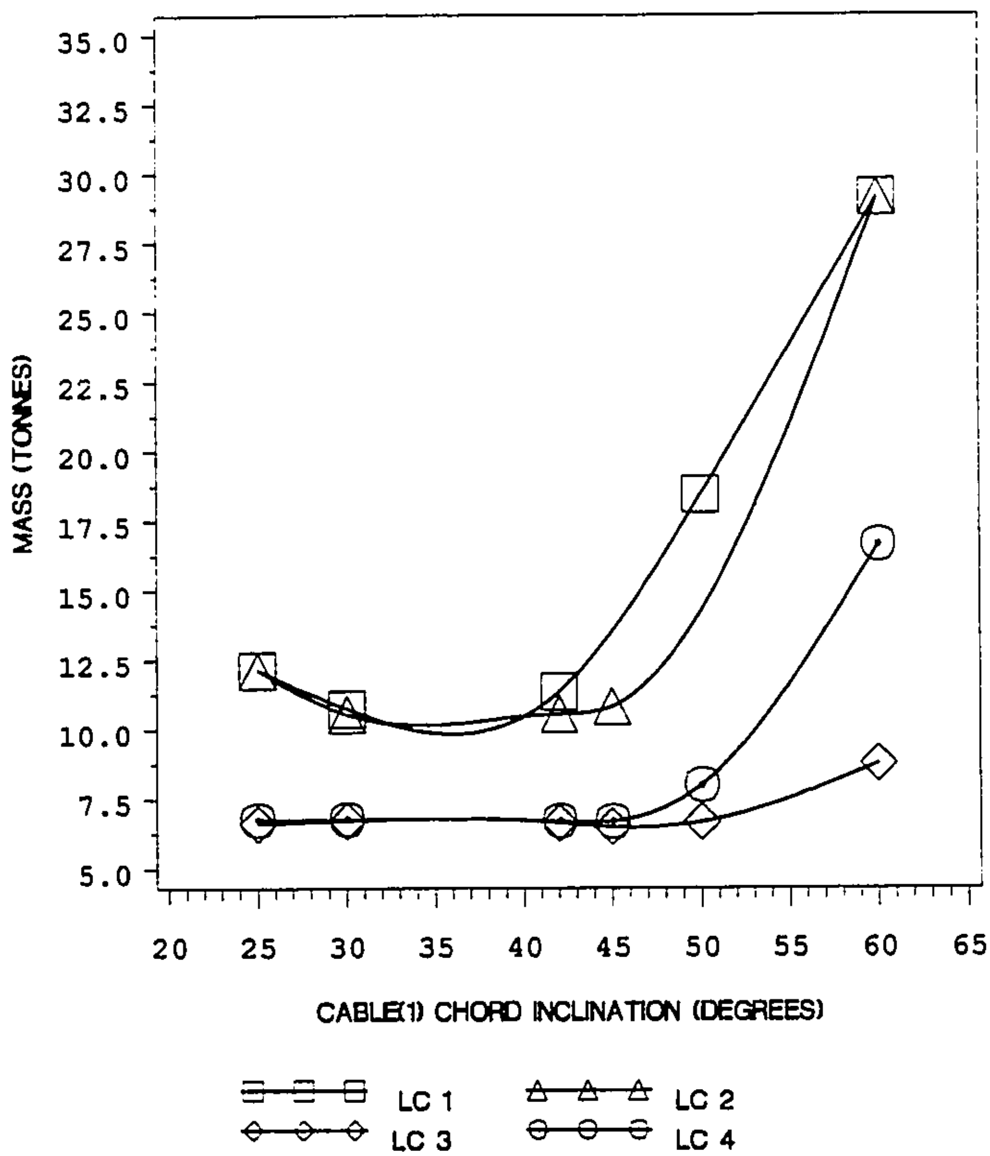


FIG A.13 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m;
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

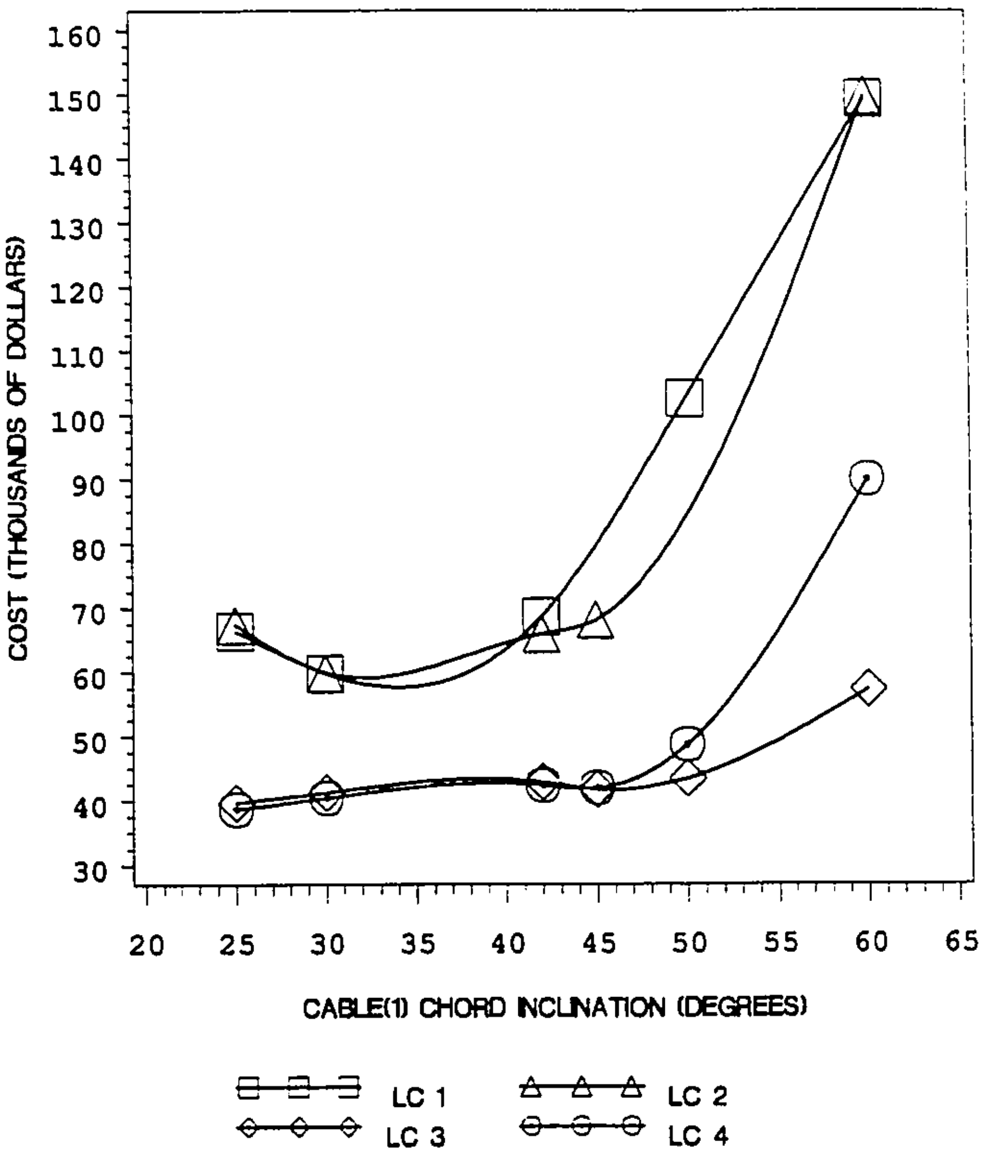


FIG A.14 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

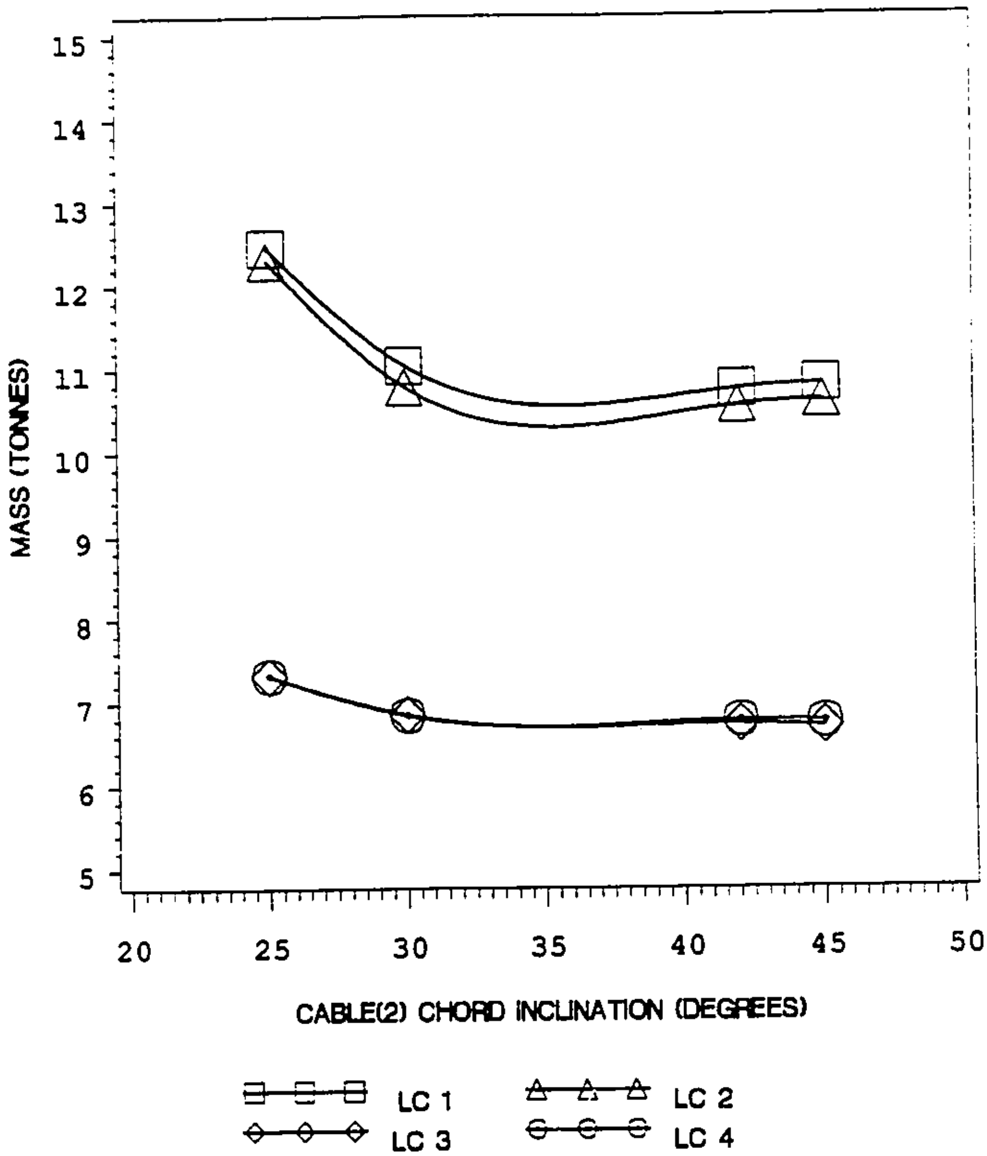


FIG A.15 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

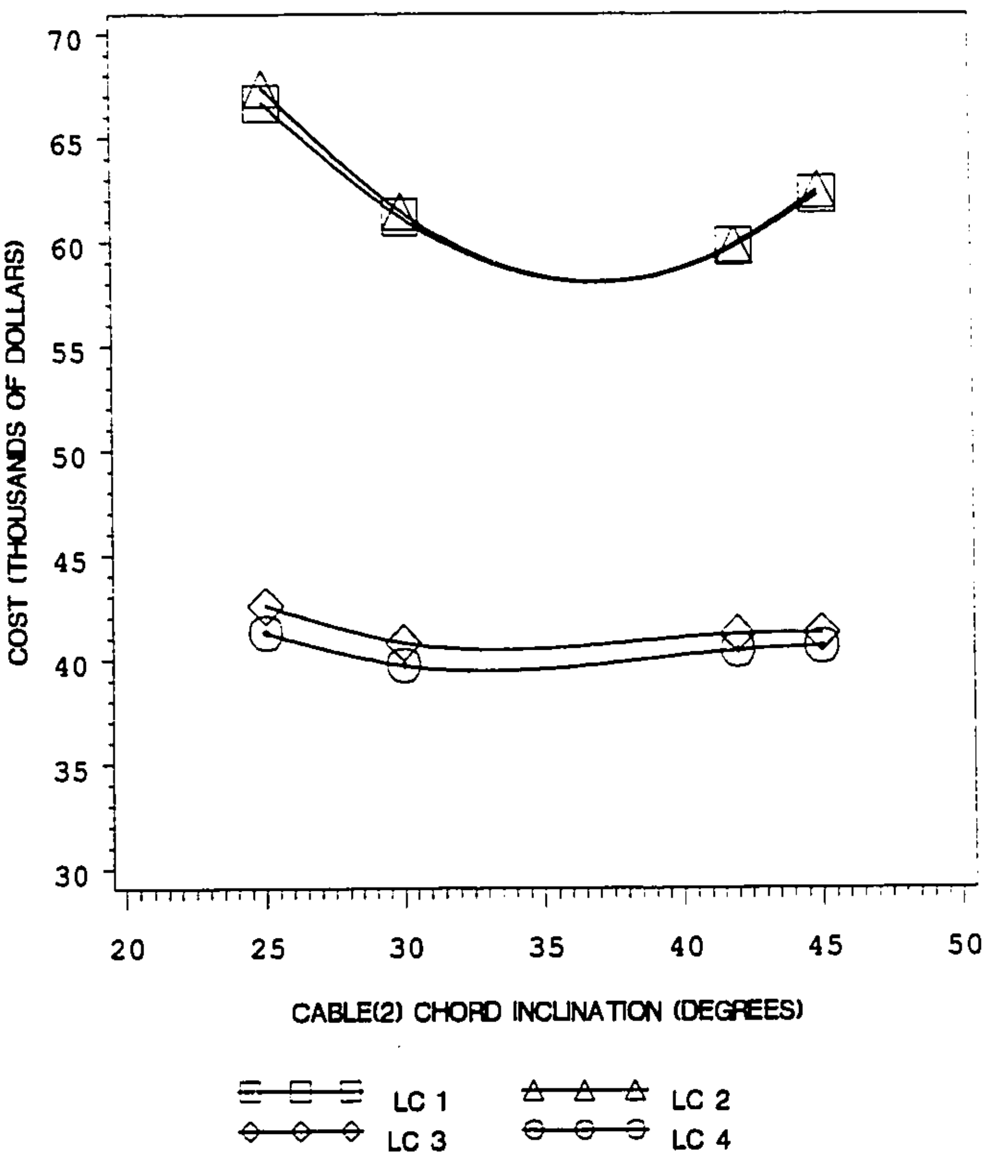


FIG A.16 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

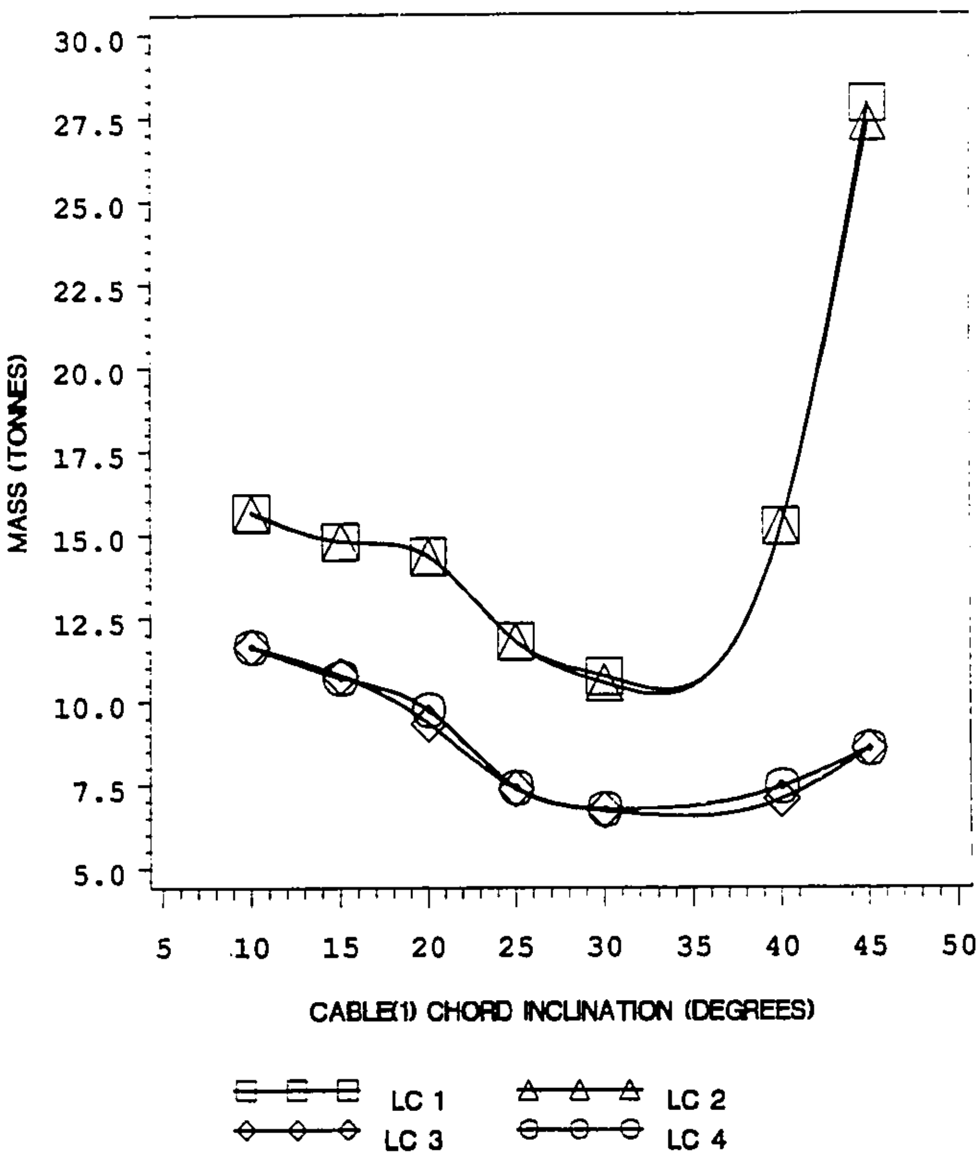


FIG A.17 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

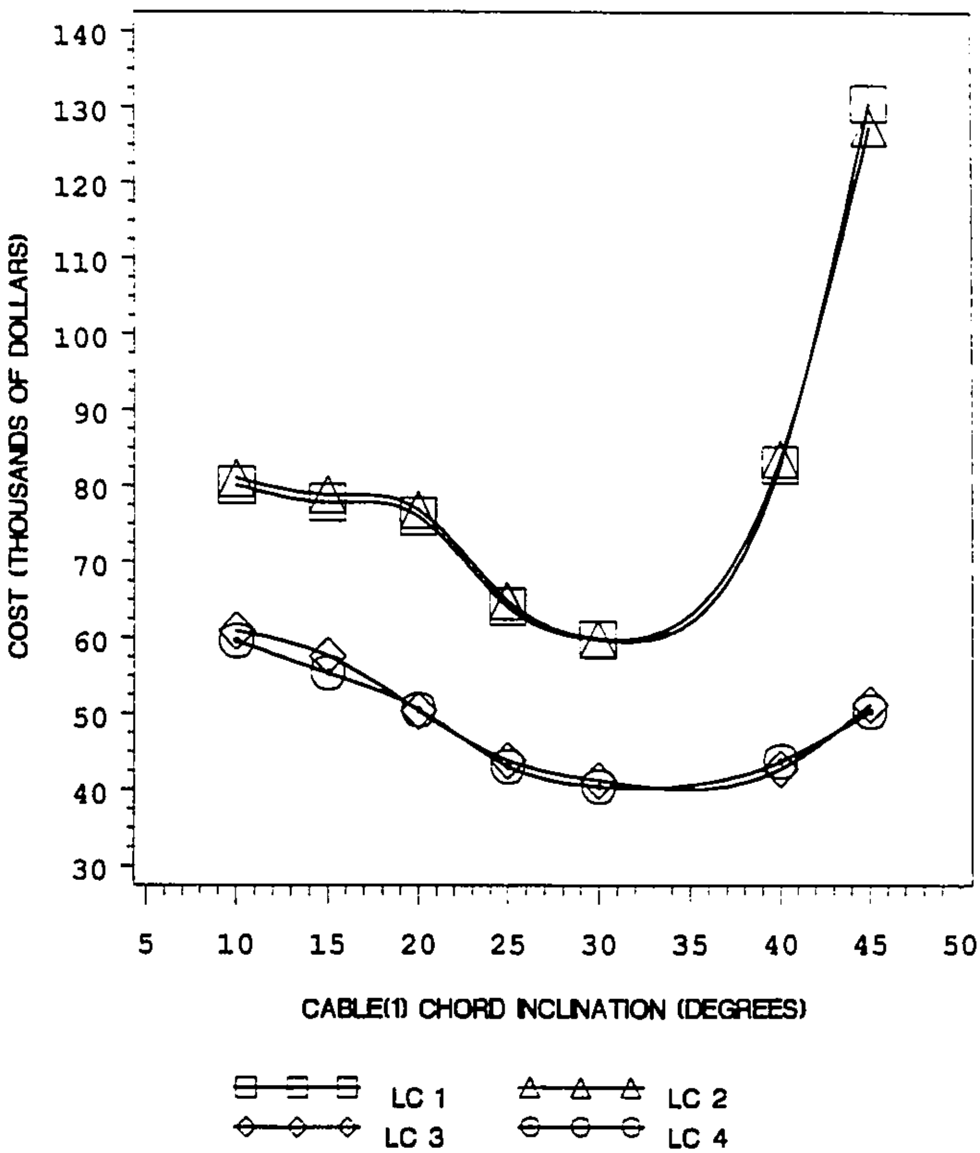


FIG A.18 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

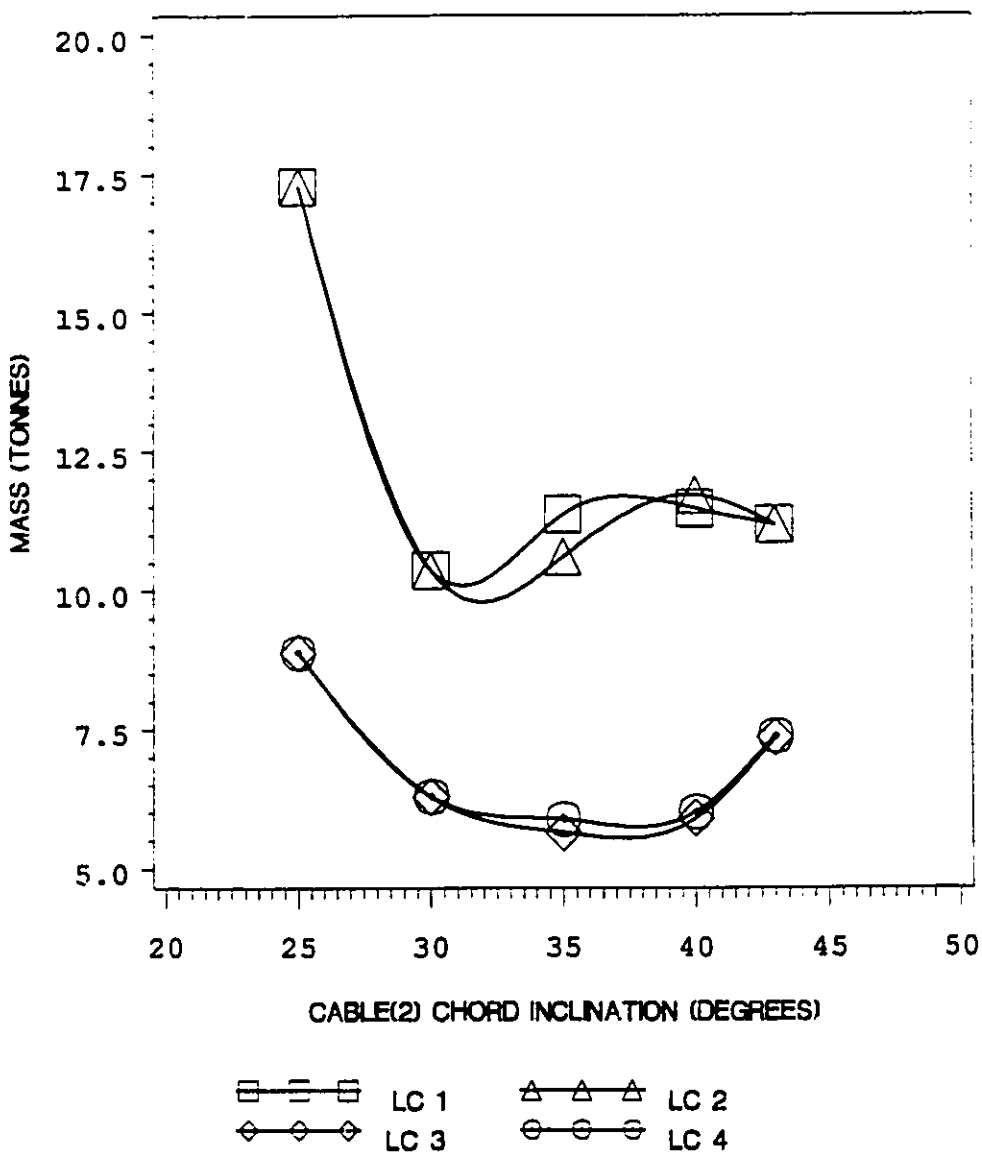


FIG A.19 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.5m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

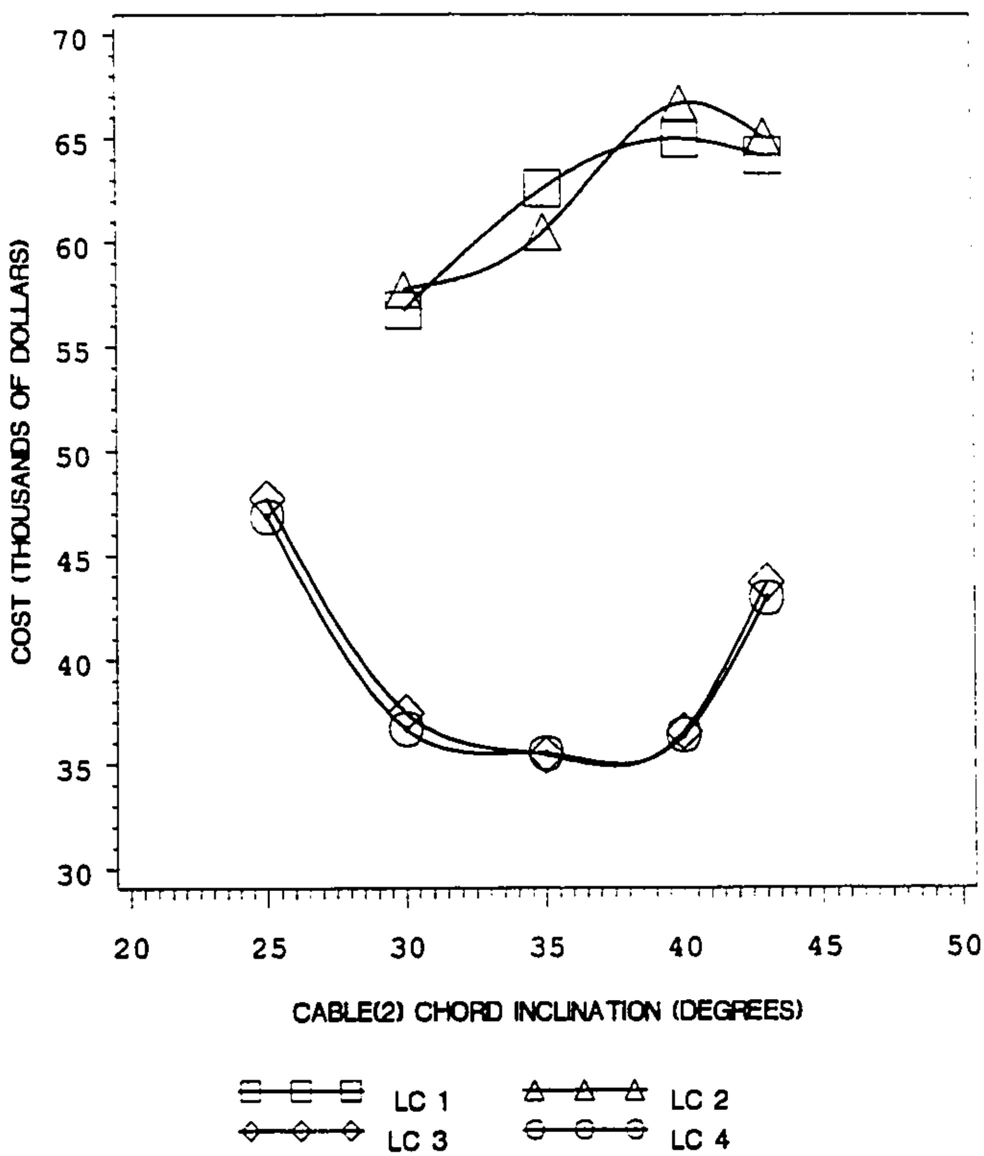


FIG A.20 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

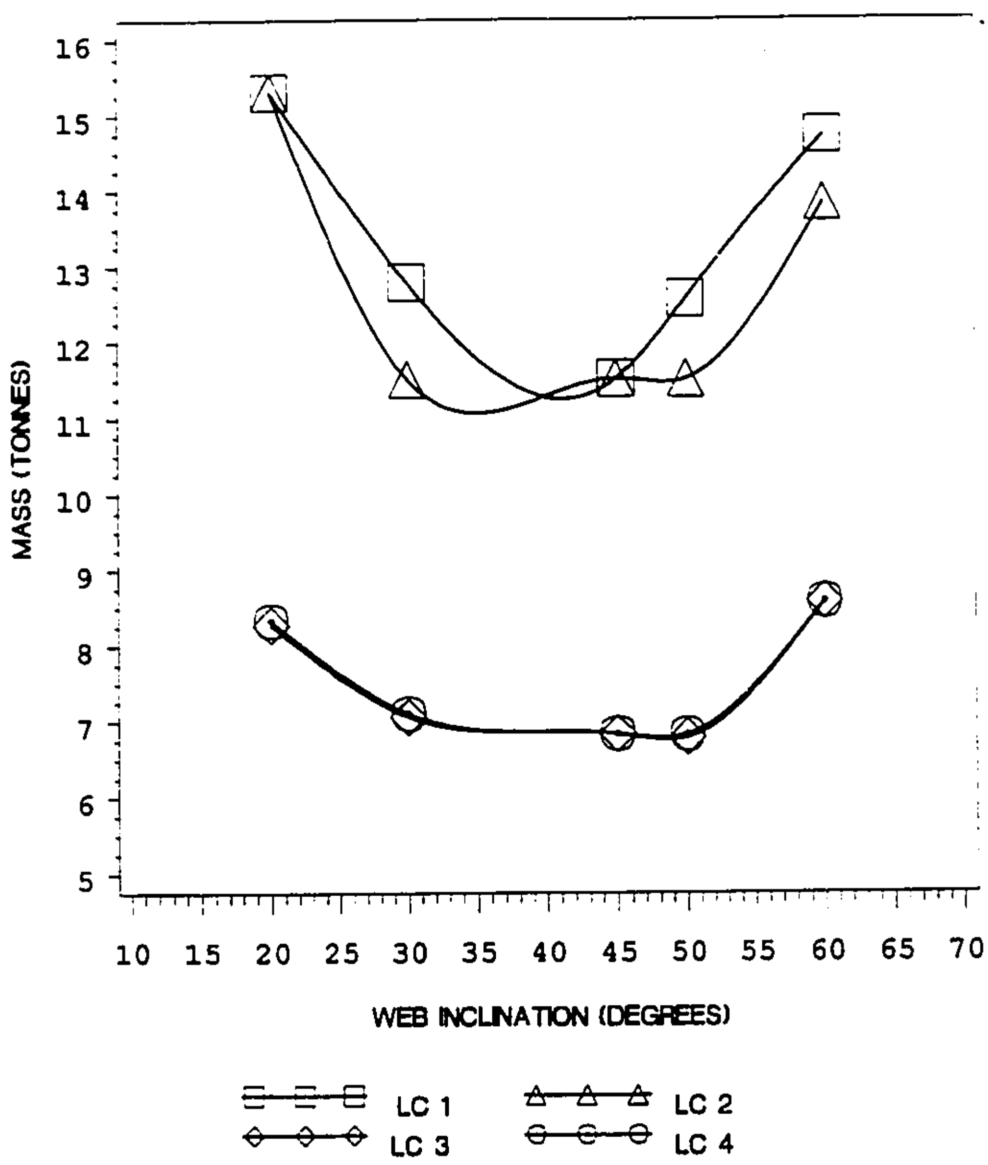


FIG A.21 : MASS vs WEB INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 50.0m ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

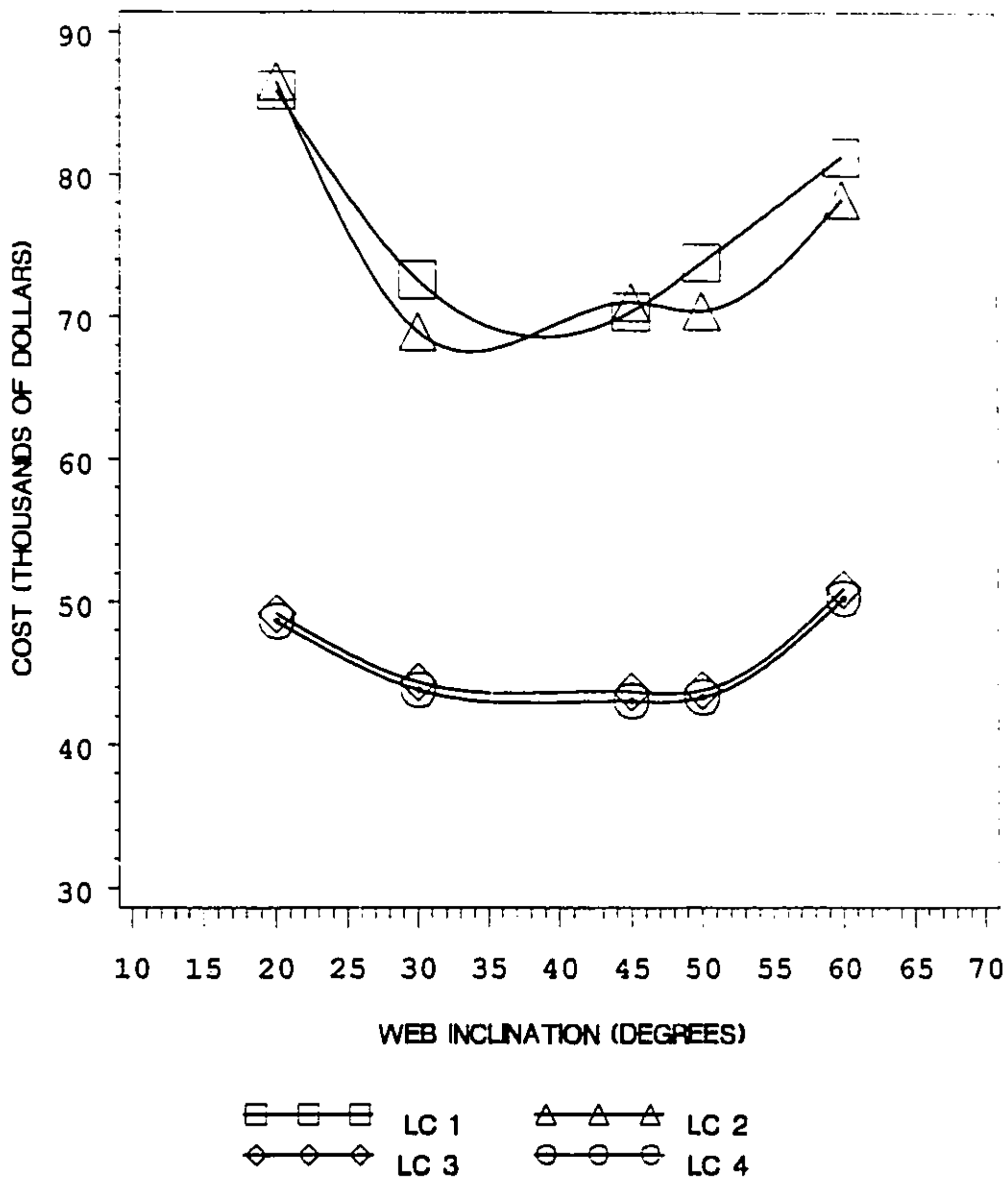


FIG A.22 : COST vs WEB INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

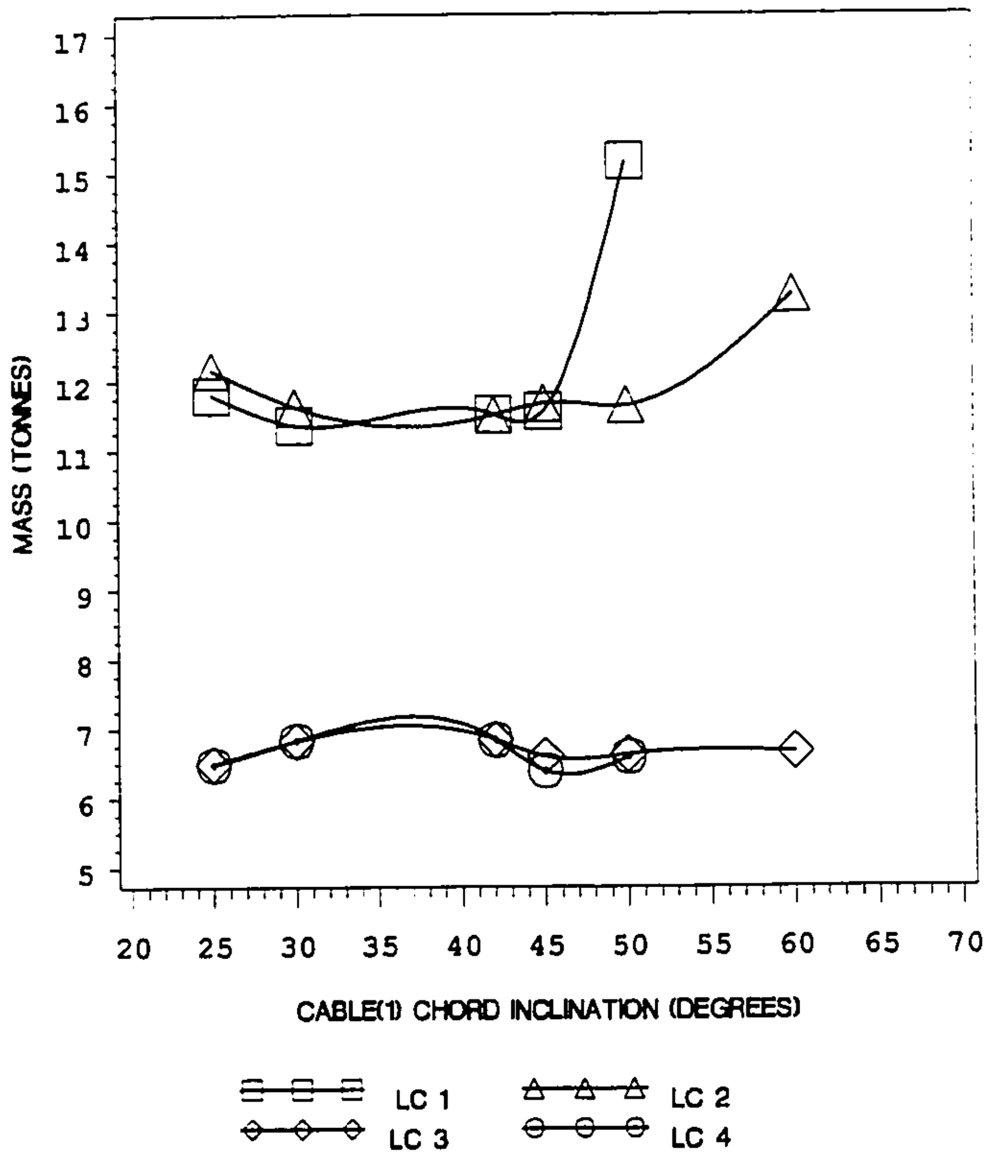


FIG A.23 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = VARIES ; CABLE(2) = 100.0m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

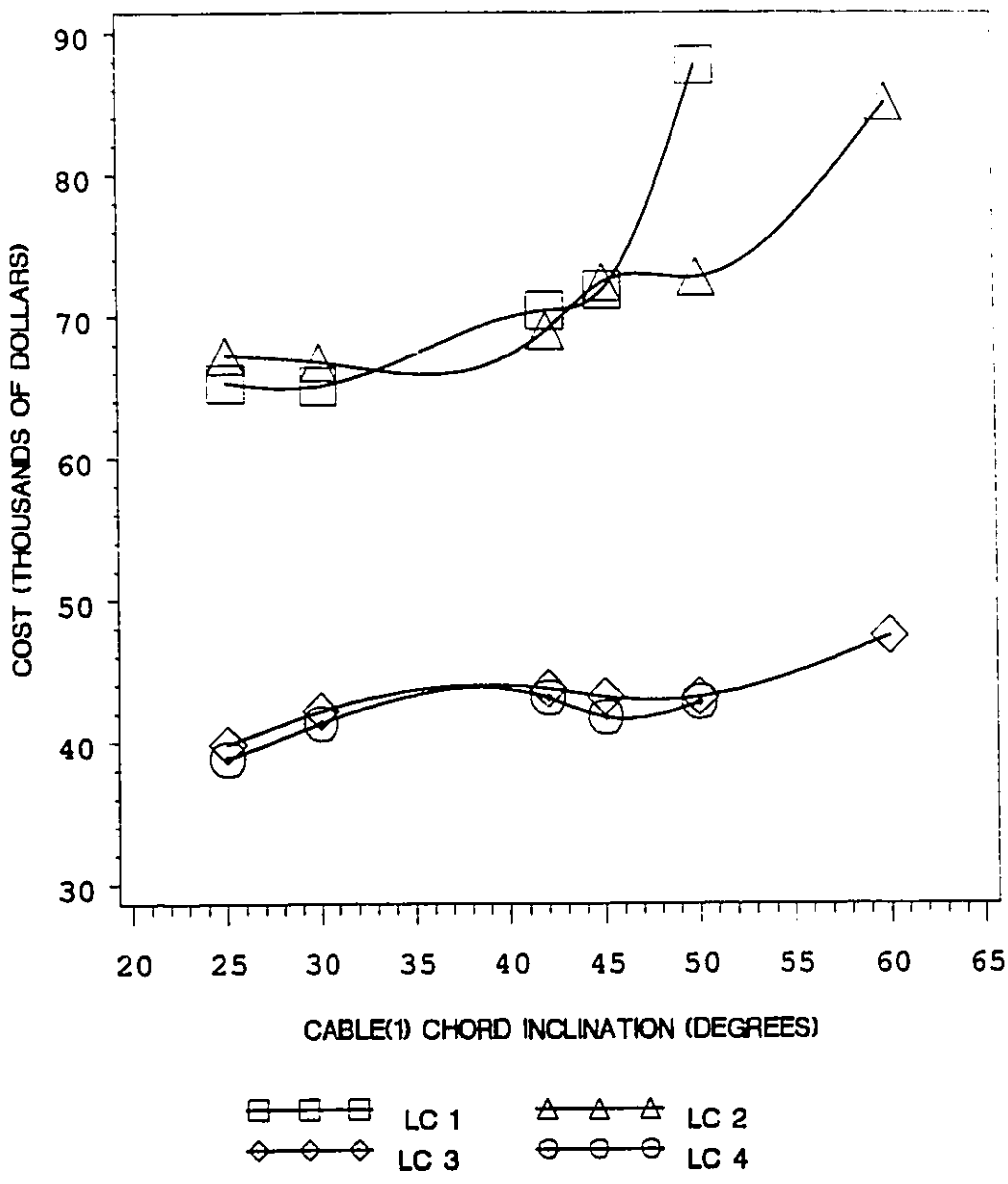


FIG A.24 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

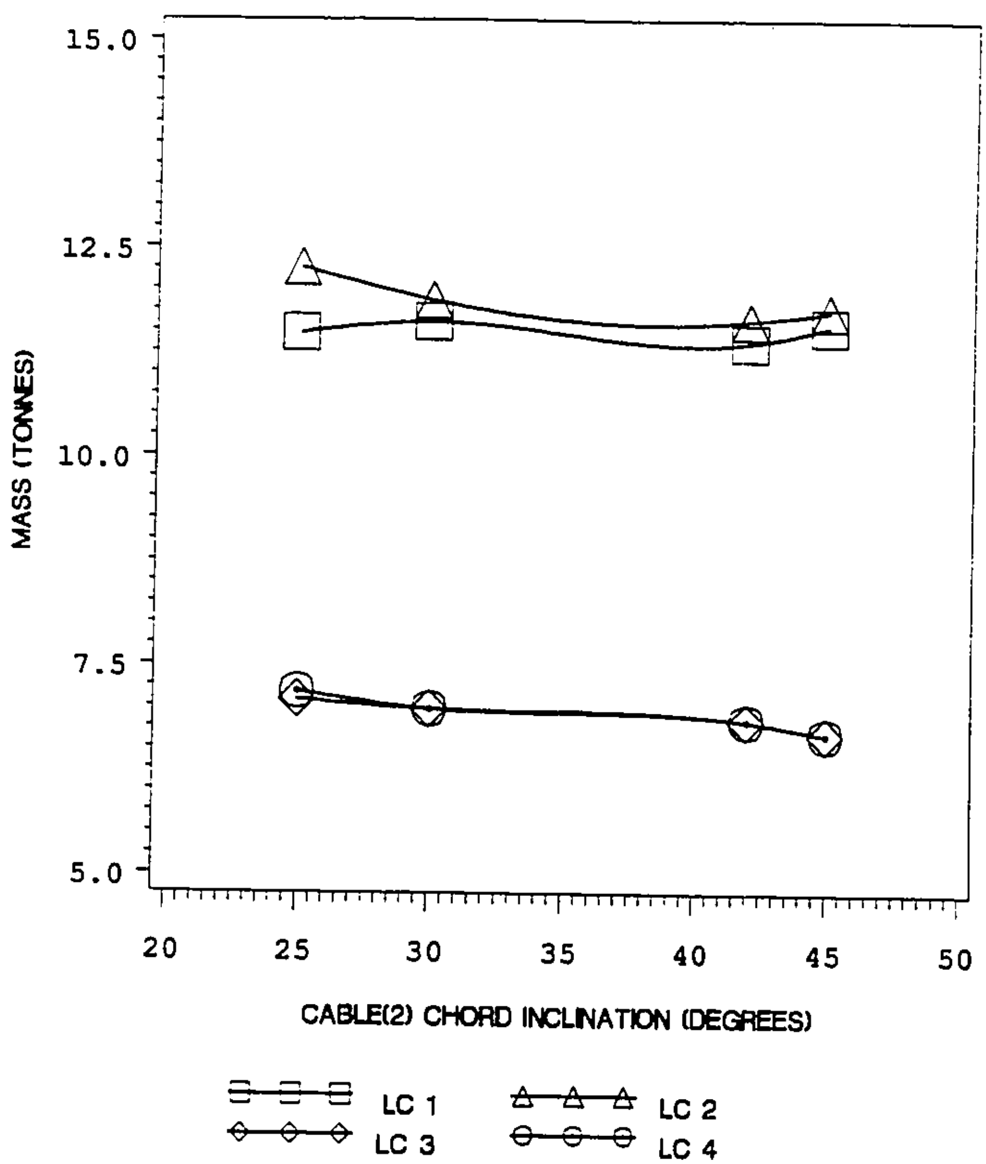


FIG A.25 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = VARIES

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

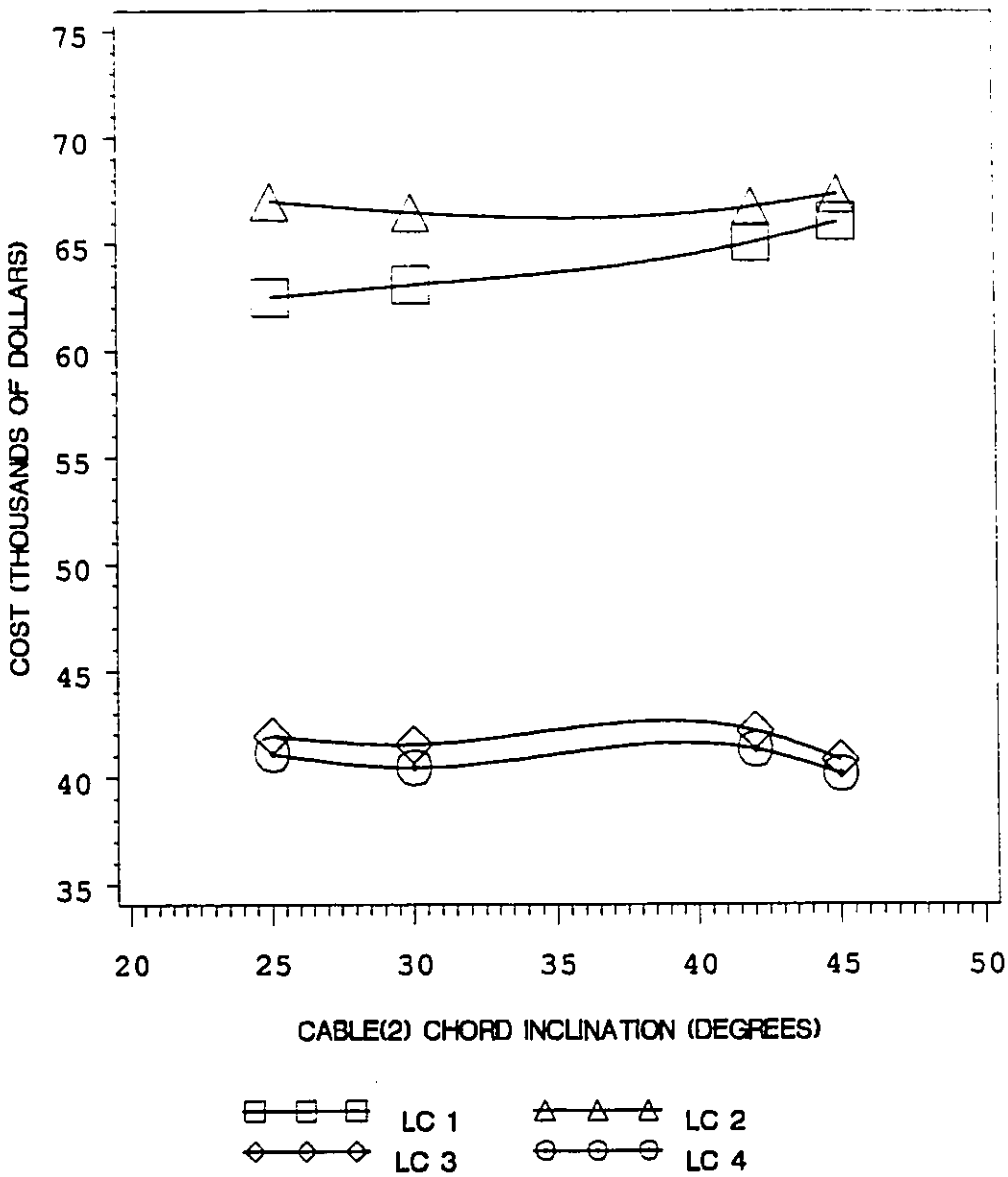


FIG A.26 : COST vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = 155.90m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

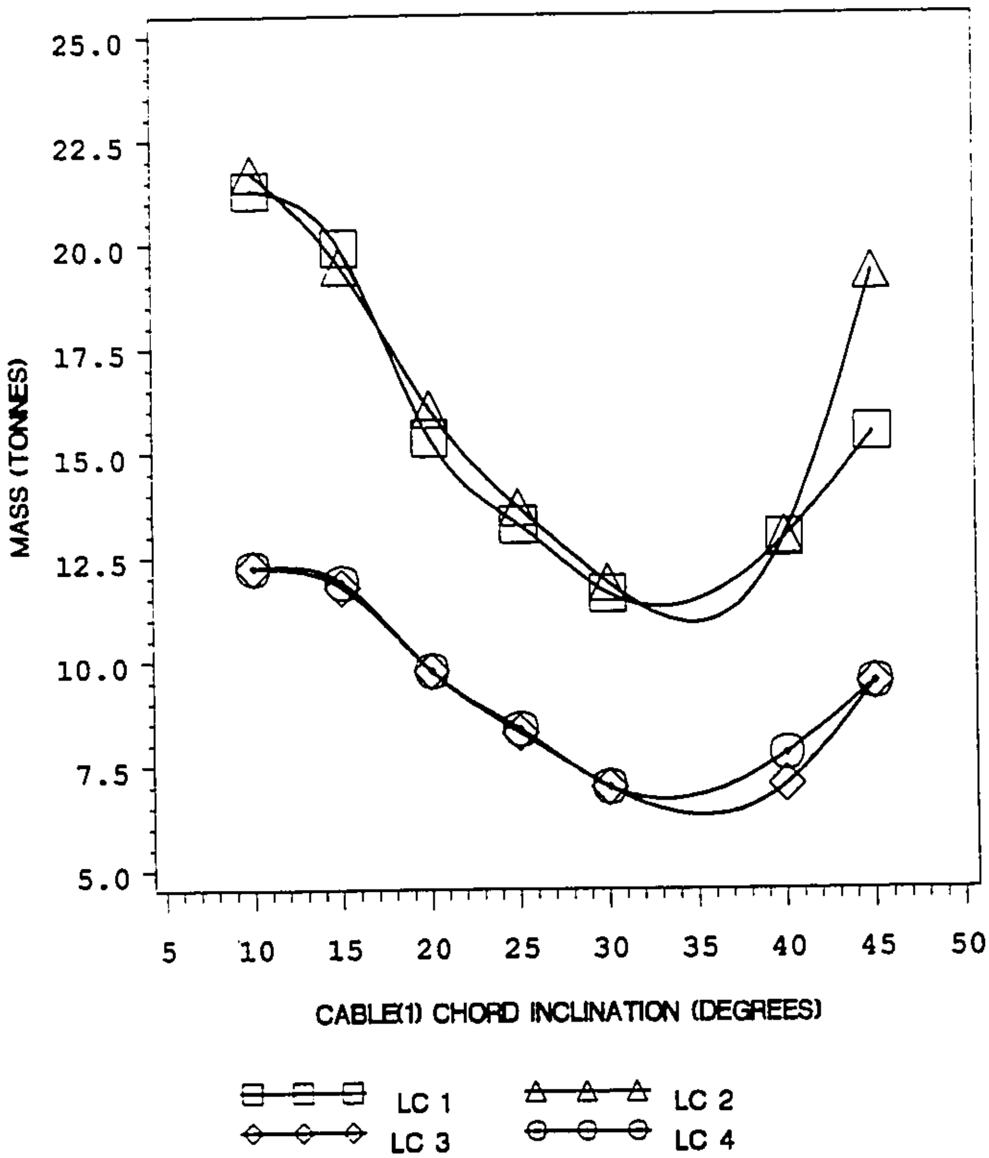


FIG A.27 : MASS vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = VARIES ; CABLE(2) = 90.0m
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 45.0m FOR 77.94m ; CABLE(2) = 155.90m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

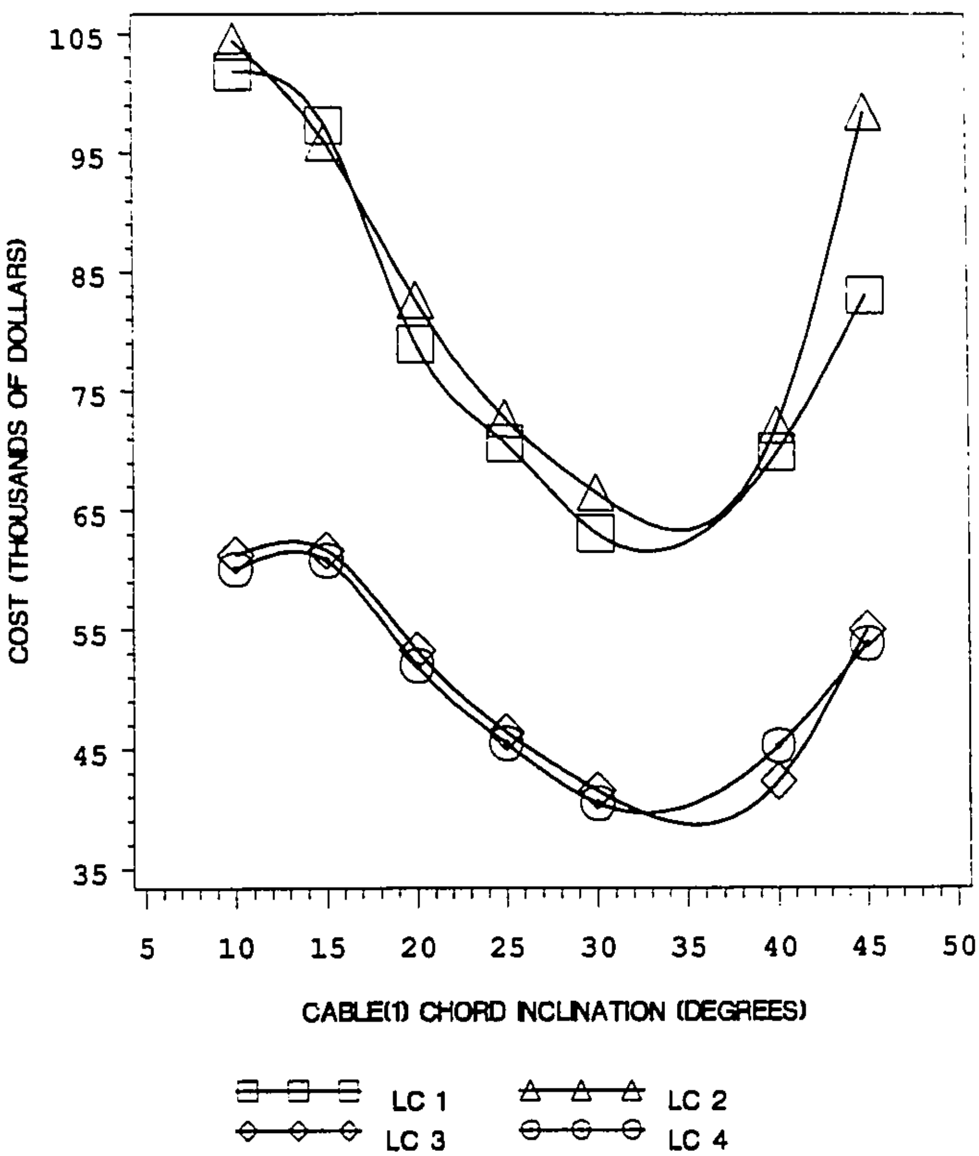


FIG A.28 : COST vs CABLE(1) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) = 155.90m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

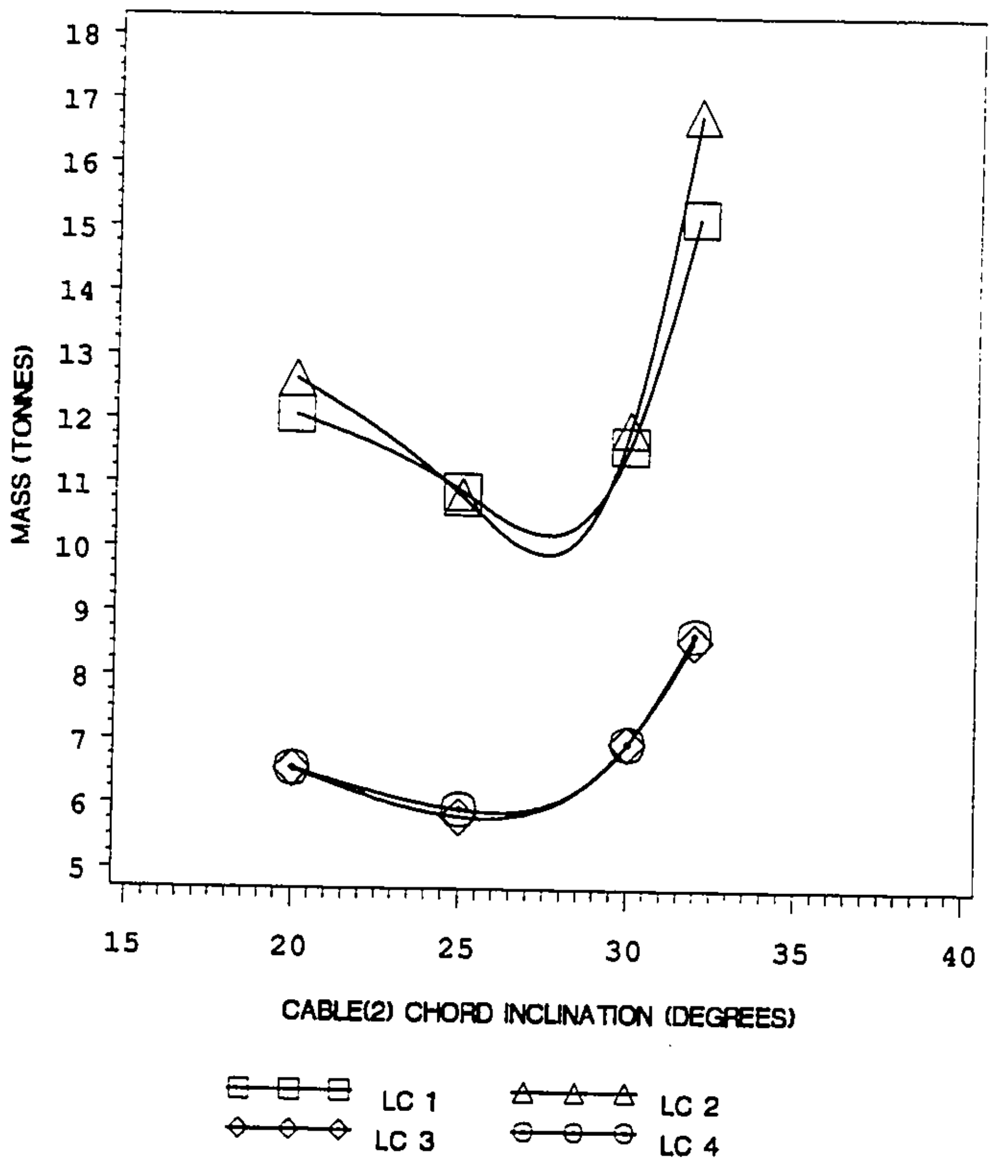


FIG A.29 : MASS vs CABLE(2) CHORD INCLINATION

MAST SIZE : 1.25m ; MAST HEIGHT : 100m ; NO. OF GUY LEVELS : 2
GUY CONNECTION POINTS :- CABLE(1) = 45.0m ; CABLE(2) = VARIES
RADIAL DISTANCE OF GUY ANCHOR POINTS :- CABLE(1) = 77.94m ; CABLE(2) =155.90m

LOADING CONDITIONS :-

1. CLASS-C WIND + CLASS-IV ICE
2. CLASS-C WIND + CLASS-I ICE
3. CLASS-A WIND + CLASS-I ICE
4. CLASS-A WIND + CLASS-IV ICE

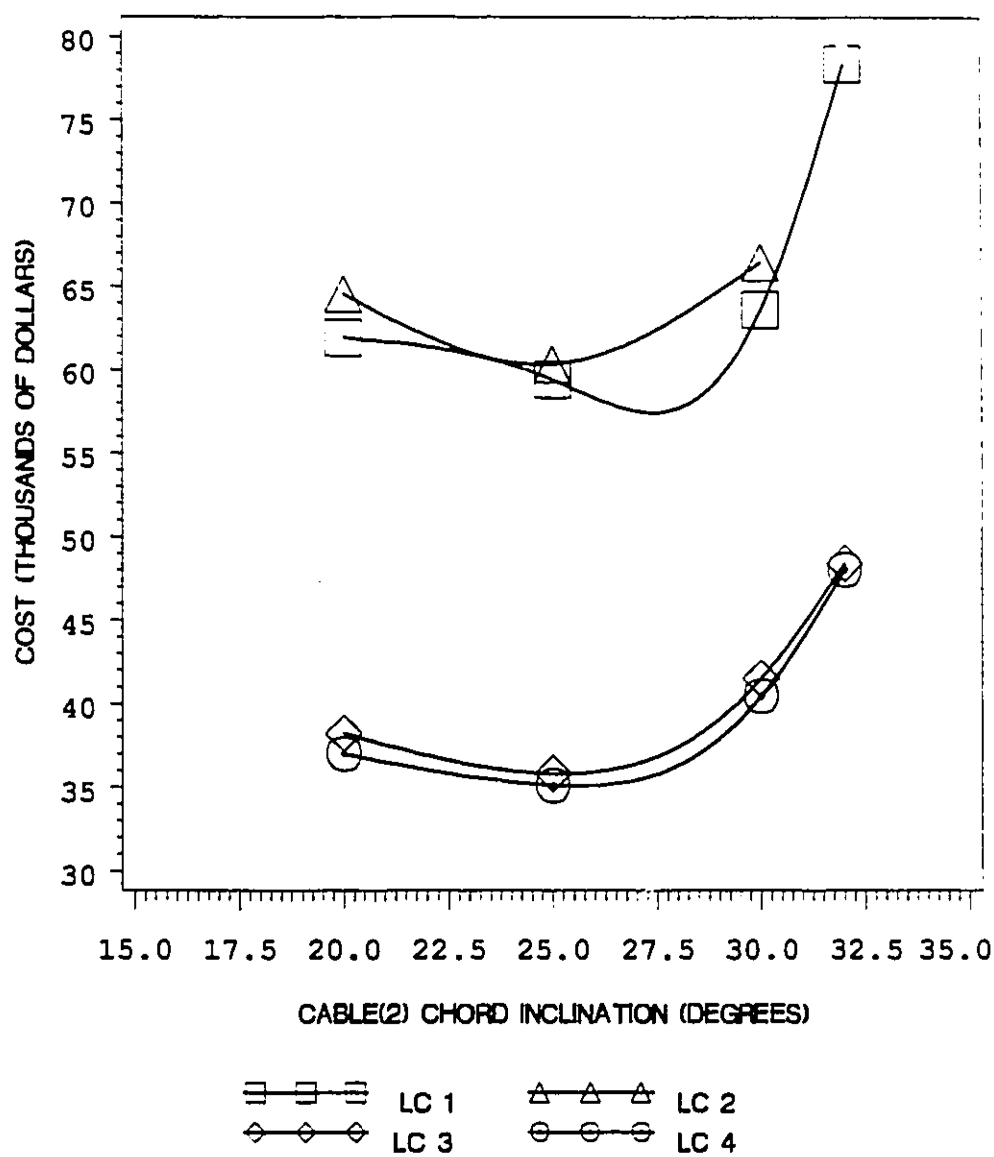


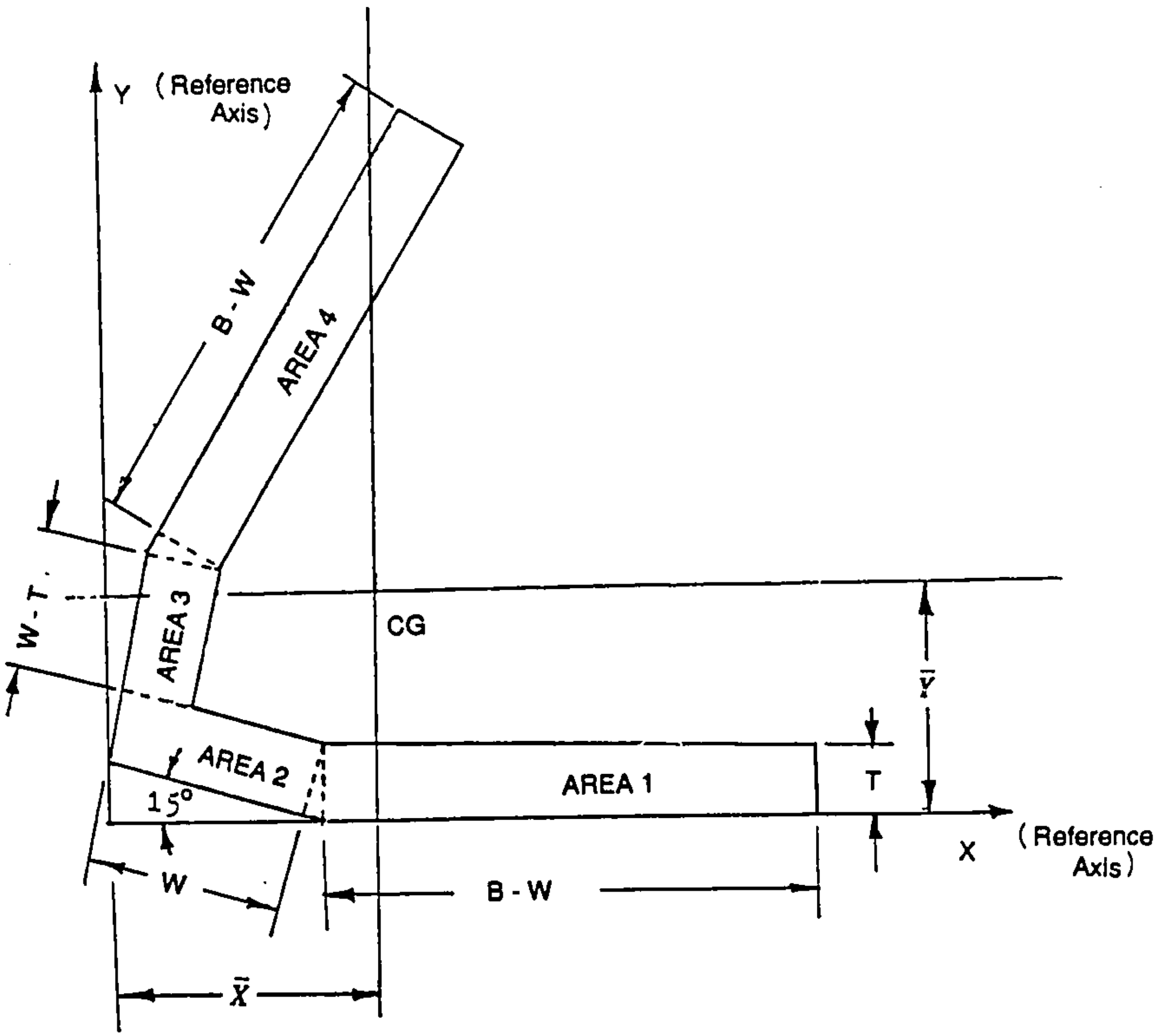
FIG A.30 : COST vs CABLE(2) CHORD INCLINATION

Appendix B

CALCULATION OF PROPERTIES OF 60^{o} SCHIFFLERIZED EQUAL LEG ANGLE

CALCULATION OF PROPERTIES OF 60 DEGREE

SCHIFFLERIZED EQUAL LEG ANGLE



$(\bar{X}_i, \bar{Y}_i)$ is the CG point for AREA i [Where i = 1 to 4].

CALCULATION OF PROPERTIES OF 60° SCHIFFLERIZED EQUAL LEG ANGLE

Calculation of Location of Centroid:

$$\overline{X_1} = \frac{\left(B - W\right)}{2} + W\cos 15^o + T\sin 15^o$$

$$\overline{Y_1} = \frac{T}{2}$$

$$\overline{X_2} = \frac{1}{2}\left(W\cos 15^o + T\sin 15^o\right)$$

$$\overline{Y_2} = \frac{1}{2}\left(W\sin 15^o + T\cos 15^o\right)$$

$$\overline{X_3} = \left(T - \left(\frac{W - T}{2}\right)\right)\sin 15^o + \frac{T\cos 15^o}{2}$$

$$\overline{Y_3} = \left(\frac{W - T}{2}\right)\cos 15^o + \left(\frac{W - T}{2}\right)\sin 15^o + T\cos 15^o$$

$$\overline{X_4} = \frac{\left(B - W\right)\sin 30^o}{2} + W\sin 15^o + \frac{T}{2\cos 15^o}$$

$$\overline{Y_4} = \frac{\left(B - W\right)\cos 30^o}{2} + W\cos 15^o + W\sin 15^o$$

$$A_1 = \left(B - W\right)T$$

$$A_2 = W\,T$$

$$A_3 = \left(W - T\right)\,T$$

$$A_4 = \left(B - W\right)\,T$$

$$A = A_1 + A_2 + A_3 + A_4$$

$$\bar{X} = \frac{\sum A_i \overline{X_i}}{A} \qquad i = 1,2,3,4$$

$$\bar{Y} = \frac{\sum A_i \overline{Y_i}}{A} \qquad i = 1,2,3,4$$

Calculation of Moment of Inertia:

For Area 1:

$$\overline{I_{x1}} = \frac{1}{12}\left(B - W\right) T^3$$

$$\overline{I_{y1}} = \frac{1}{12}\left(B - W\right)^3 T$$

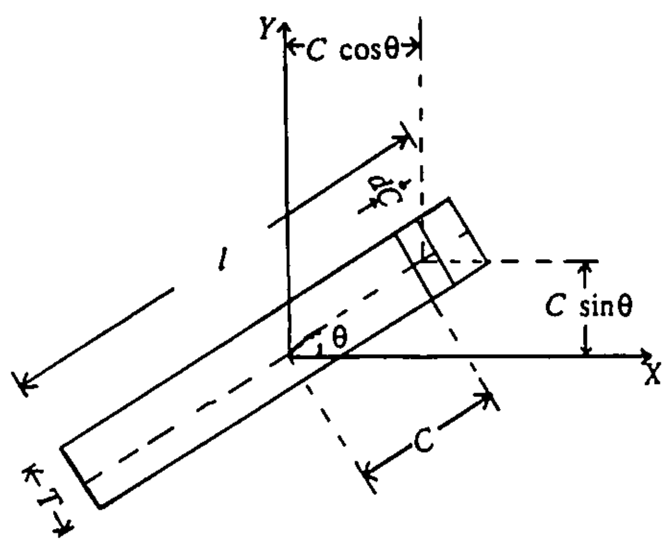
$$d_{1y} = \overline{Y_1} - \bar{Y}$$

$$I_{x1} = \overline{I_{x1}} + A_1\, d_{1y}^2$$

$$d_{1x} = \overline{X_1} - \bar{X}$$

$$I_{y1} = \overline{I_{y1}} + A_1\, d_{1x}^2$$

For Area 2:



$$I_{xx} = \int_A Y^2\, dA$$

$$= 2\int_0^{\frac{l}{2}} \left(C \sin\theta\right)^2 \times T\, dC$$

$$= \frac{T\,l^3}{12}\ \sin^2\theta \qquad \text{(a)}$$

$$I_{yy} = \int_A x^2\ dA$$

$$= 2\int_0^{\frac{l}{2}} \left(C\cos\theta\right)^2 \times T\ dC$$

$$= \frac{T\,l^3}{12}\ \cos^2\theta \qquad \text{(b)}$$

Equations (a) and (b) are used to compute the Moment of Inertia for Area 2, Area 3, and Area 4.

$$\overline{I_{x2}} = \frac{T\,W^3}{12}\ \sin^2(105^o) = \frac{T\,W^3}{12}\ \cos^2 75^o$$

$$\overline{I_{y2}} = \frac{T\,W^3}{12}\ \cos^2(105^o) = \frac{T\,W^3}{12}\ \sin^2 75^o$$

$$d_{2y} = \bar{Y} - \overline{Y_2}$$

$$I_{x2} = \overline{I_{x2}} + A_2\ d_{2y}^2$$

$$d_{2x} = \bar{X} - \overline{X_2}$$

$$I_{y2} = \overline{I_{y2}} + A_2\ d_{2x}^2$$

For Area 3:

$$\overline{I_{x3}} = \frac{T}{12}\left(W - T\right)^3\ \sin^2 75^o$$

$$\overline{I_{y3}} = \frac{T}{12}\left(W - T\right)^3\ \cos^2 75^o$$

For Area 4:

$$\overline{I_{x4}} = \frac{T}{12}\left(B - W\right)^3 \sin^2 60^o$$

$$\overline{I_{y4}} = \frac{T}{12}\left(B - W\right)^3 \cos^2 60^o$$

$$d_{4y} = \overline{Y_4} - \bar{Y}$$

$$I_{x4} = \overline{I_{x4}} + A_4\, d_{4y}^2$$

$$d_{4x} = \bar{X} - \overline{X_4}$$

$$I_{y4} = \overline{I_{y4}} + A_4\, d_{4x}^2$$

Moments of Inertia About Centroidal Axes:

$$I_{xcg} = I_{x1} + I_{x2} + I_{x3} + I_{x4}$$

$$I_{ycg} = I_{y1} + I_{y2} + I_{y3} + I_{y4}$$

Radii of gyration are given by

$$R_x = \sqrt{\frac{I_{xcg}}{A}}$$

$$R_y = \sqrt{\frac{I_{ycg}}{A}}$$

VITA AUCTORIS

Subba Rao Venkata Majety was born in Vijayawada, Andhra Pradesh, India, on July 1, 1965. In August, 1985 he graduated with a Bachelor of Technology in Civil Engineering from Jawaharlal Nehru Technological University, College of Engineering, Kakinada, India. He worked as a Design Engineer with Consulting Engineering Services (I) Pvt. Ltd., New Delhi, India during the period 1986 - 1990.

In September, 1990, he enrolled at the University of Windsor, Windsor, Ontario, Canada, to continue with his studies towards the degree of Master of Applied Science in Civil Engineering.

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