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Optimum Dimensions of Concave Cable Roofs

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الأبعاد المثلى للأسقف المقعرة ذات الكابلات

الخلاصة : يتناول هذا البحث الإختيار الأمثل للأبعاد المثلى للأسقف المقعرة ذات الكابلات وذلك بعمل التحليل الإستاتيكي لها حيث تم إستخدام طريقة الطاقة المبنية على تصغير طاقة الوضع لجميع العناصر الإنشائية باستخدام طريقة نيوتن وقد تم إستخدام طريقة التكرار أخذا في الاعتبار التأثيرات الغير خطية لهندسة الشكل . وقد تم دراسة عوامل عده في هذا المجال مثل نسبة ترخيم الكابلات وارتفاعاتها إلى بحور الكمرات وكذلك الشد الابتدائي وميول الكابلات على الأفقي والمسافة بين الأربطة الرأسية وكذلك تأثير متانة الأعمدة وميولها على الرأسي ونوع الركائز آخذا في الإعتبار حالات التحميل المختلفة . وقد تناول البحث ايضا إستعراض الأمثلة التوضيحية لذلك مع ذكر أهم النتائج والتوصيات التي تم إستنتاجها .

ABSTRACT

The present work deals with the eleetion of the optimum dimensions of concave eable roofs. In the static analysis, the energy method, based on, the minimization of the total potential energy (TPE) of structural elements, via Newton Raphson technique is used. The procedure is carried out by using the iterative steps to acquire the final configurations taking into consideration the effect of geometric nonlinearitiy of the structure. Design parameters, such as both sag and rise to span ratios, initial tension in cables, distance between vertical ties, surface curvature, columns rigidity and their inclinations with horizontal, loading and support conditions are investigated. Numerical analysis of different concave cable roofs are performed. Finally the conclusions is outlined.

1- Introduction :

The development of high tensile steel cable has made it possible to transmit large axial forces in tension at a relatively low cost. Experience in this field has shown that the cable roof structure is a theortically pleasing. Their use has often result in attractive shape, with structures that are stable and efficient since a large proportion of the main loads carrying members are in tension. A concave eable roof comprises main cable, roofing cable and suspenders is one of the most common types of roofs . Which used to cover large span areas as independent structures or may be arranged in geometrical patterns to consist the roof nets in rectangular , circular , trapezoidal and elliptical in plane .

In structural analysis using the stiffness method , the formulation of structure's stiffness matrix is required , whilst the analysis using the energy method doesn't need the assemblage of the structure's stiffness matrix . So that especially in large structures , the later method is more advantageous to available personal computer . The analysis is carried out by minimization of the total potential energy of the structure using iterative procedure . Theoretically , this minimization can take place along any defined descent vector . Mathematically,a number of options exist [1,2] .

To understand the static behaviour of concave cable roofs shown in Fig. (1) , many design parameters are taken into consideration . Design parameters as spacing between vertical ties , both sag and rise to span ratios , initial tensions in cable elements , inclination of both suspenders and vertical columns and rigidity of columns and their supports are taken into consideration . At first , a concave cable roof for 30m span with and without diagonal members arrangement as shown in Figs. (2a) and (2b) , respectively have been studied in order to obtain the optimum dimensions taking into consideration the above mentioned different design parameters . Then , with optimum dimensions for design parameters obtained for concave cable roof for 30m span , a complete analysis for concave roofs with 60m and 90m spans is achieved .

Newton-Raphson or instantaneous stiffness matrix method [3 , 4] ,which , converges more quickly in terms of iterations and gives a high degree of accuracy is used . The success of this method depends upon :

a) The justification of ignoring the cubic and higher order terms in the Taylor expansion series , and

b) The condition of the stiffness matrix for any displaced form of the structure .

More details about Newton-Raphson method using iterative procedure are given by [5 , 6] . Finally , numerical analysis for all design parameters including the conclusions are reported .

2- Nonlinear response by minimization of the total potential energy.

The Newtow-Raphson iterative technique [2, 7, 8, 9, 10] is reported for nonlinear analysis. A method of nonlinear analysis using the Newtow-Raphson technique is developed in this section.

2-1 Assumptions :

In the following development, it is assumed that [7]

a) The cable cannot resist bending,

b)The forces in the cable are of sufficient magnitude to prevent non-linear load-extension curves of cable links due to their weight [11],

c) The elastic extensions of the cable are small compared with their lengths, and

d) The loads are applied at points of intersection of the cables.

2-2 The gradient vector of the total potential energy at a stationary point in displacement space :

Considering U is the elastic strain energy stored in structure's members and V is the potential energy of the applied load system, the TPE, W of the structural system is given by :

$$W = U + V \qquad (1)$$

For constant loading, if the datum is taken as the unloaded configuration of the system, equation (1) may be written as[7] :

$$W = \Sigma_{m=1}^{M} U_m - \{F\}^T\{x\} \qquad (2)$$

Where

$\{F\}$ = column vector of the external applied loads,

$\{x\}$ = column vector of the joint displacements,

U_m = strain energy stored in any link m, and

M = total number of links or members.

The equilibrium position of a structure corresponds to a stationary minimum point in the n-dimensional space of the TPE and occurs when x :

$$\partial W / \partial x_i = \{g_i\} = 0, \; i = 1,2,\text{---}, f \qquad (3)$$

Where f is the total degree of freedom of all joints. Differentiating equation (2) with respect to x_{ji}, gives :

$$\partial W / \partial x_{ji} = \Sigma_{n=1}^{N} \{\partial U_{jn} / \partial x_{ji}\} - F_{ji} \tag{4}$$

Where N = number of links at joint j .

Considering the tensile force in any link be t_{in} and the elongation of the link e_{in} , then

$$\{ \partial U_{in} / \partial x_{ii} \} = \{ \partial U_{in} / \partial e_{in} \} \{ \partial e_{in} / \partial x_{ii} \} = t_{in} \{ \partial e_{in} / \partial x_{ii} \} \tag{5}$$

Substituting in equation (4) gives :

$$\partial W / \partial x_{ji} = \Sigma_{n=1}^{N} t_{jn} \{ \partial e_{jn} / \partial x_{ji} \} - F_{ji} \tag{6}$$

let L_{ino} is the initial prestressed length of link jn , then :

$$(L_{jno} + e_{jn})^3 = \Sigma_{i=1}^{3} ((X_{ni} - X_{ji}) + (x_{ni} - x_{ji}))^2 \tag{7}$$

Differentiating equation (7) with respect to x , and substituting for $\{ \partial e_{in} / \partial x_{ii} \}$ in equation (6) gives :

$$\partial W / \partial x_{ji} = -\Sigma_{n=1}^{N} [t_{jn}((X_{ni} - X_{ji}) + (x_{ni} - x_{ji}))^2 / (L_{jno} + e_{jn})] - F_{ji} \tag{8}$$

2-3 The Newton-Raphson method .

Let the change in the displacement vector at the end of the k^{th} iteration be δx_k and the gradient vectors at x_k and x_{k+1} be respectively :

$$g_k = \partial W / \partial x_k \text{ and } g_{k+1} = \partial W / \partial x_{k+1} \tag{9}$$

Expanding the gradient at x_k in terms of a Taylor series and ignoring cubic and higher-order terms and remembering that the potential energy of the loading is linear in x , the gradients are given as :

$$g_{k+1} = g_k + (\partial^2 U / \partial x_k \partial x_k)\delta x_k \tag{10}$$

If it is now assumed that , g_{k+1} is equal to zero , equation(10)reduces to :

$$- g_k = (\partial^2 U / \partial x_k \partial x_k)\delta x_k \tag{11}$$

In which the Hessian matrix $(\partial^2 U / \partial x_k \ \partial x_k)$ can be recognized as the stiffness matrix at point x_k in displacement space . The change in the displacement vector at the end of the kth iteration is there fore , given by :

$$\delta x_k = -K_k^{-1} g_k \tag{12}$$

and thus,

$$x_{k+1} = x_k + \delta x_k \tag{13}$$

For non-linear structures successive iterations are required and considering δx_k as a descent vector along which is taken a step $S_k \delta x_k$ to a point where the TPE is a minimum. Then, the equation (13) can be written as:

$$x_{k+1} = x_k + S_k V_k \tag{14}$$

Where, The descent vector $V_k = \delta x_k$ (15)

The stiffness matrix K_k given in equation (12) for pinjointed pretensioned link at the k^{th} is given, ref. (8) as:

$$K_k = \frac{EA - T_k}{L_m} \begin{vmatrix} G_k G_k^T & -G_k G_k^T \\ -G_k G_k^T & G_k G_k^T \end{vmatrix} + \frac{T_k}{L_m} \begin{vmatrix} I & -I \\ -I & I \end{vmatrix} \tag{16}$$

Where, I is a unit matrix of dimension (3×3), T is the tension in link member, and $G_k = \{ l \;\; m \;\; n \}^T$ and l, m, n are the direction cosines of the member.

2-4 The stationary condition in the descent direction.

The TPE be expressed as a fourth order polynomial in the steplength S in the form:

$$W = C_4 S^4 + C_3 S^3 + C_2 S^2 + C_1 S + C_0 \tag{17}$$

The value of S can now be found from the condition that:

$$dW/dS = 0 \tag{18}$$

at the stationary point along the path of descent.
Differentiating equation (17) with respect to S and applying the condition given by equation (18), yields:

$$4C_4 S^3 + 3C_3 S^2 + 2C_2 S + C_1 = 0 \tag{19}$$

Using Newton's approximation formula to get the step-length as

$$S_{i+1} = S_i - \frac{dW/dS}{d^2W/dS^2} \tag{20}$$

Where i is an iteration suffix and $S_{i=0}$ is taken as zero.

The coefficients C_4 , C_3 , C_2 , C_1 are givein as :

$$C4 = \Sigma_{n=1}^{p} (EAa_3^2/2L_0^3)_n \tag{21a}$$

$$C3=\Sigma_{n=1}^{p} (EAa_2a_3/L_0^3)_n \tag{21b}$$

$$C2 = \Sigma_{n=1}^{p}(t_0a_3+EA(a_2^2+2a_1a_3)/2L_0^3)_n+\Sigma_{n=1}^{f}\Sigma_{s=1}^{12}\Sigma_{r=1}^{12}(\tfrac{1}{2}v_sk_{sr}v_r)_n \tag{21c}$$

$$C1 = \Sigma_{n=1}^{p}(t_0a_2+EAa_1a_2)/L_0^3)_n+\Sigma_{n=1}^{f}\Sigma_{s=1}^{12}\Sigma_{r=1}^{12}(x_sk_{sr}v_r)_n-\Sigma_{n=1}^{N}F_nv_n \tag{21d}$$

Where

$$a_1 = \Sigma_{i=1}^{3}((X_{ni}-X_{ji})+\tfrac{1}{2}(x_{ni}-x_{ji}))(x_{ni}-x_{ji}) \tag{22a}$$

$$a_2 = \Sigma_{i=1}^{3}((X_{ni}-X_{ji})+(x_{ni}-x_{ji}))(v_{ni}-v_{ji}) \tag{22b}$$

$$a_3 = \Sigma_{i=1}^{3}\tfrac{1}{2}(v_{ni}-v_{ji})^2 \tag{22c}$$

Where

f = number of flexural members ;

p = number of pinjointed members and cable links ;

k_{sr} = element of stiffness matrix in global coordinates of a flexural member ;

t_0 = initial force in a pinjointed member or cable link due to pretension ;

x = element in displacement vector due to applied load ; and

F_n = element in applied load vector .

3- Analysis considerations :

The analysis is carried out similar to the study of optimum shape of convex and concave-convex cable roofs [12 , 13] . It was achieved for 30m , 60m , and 90m spans of beams with the unbalanced force vector is less than 0.001 of the initial values . The cable and flexural members properties for all spans are given in Table 1 .The static analysis of the design parameters is carried out for the following cases of loading including the self weight of structural elements .

Case 1 : uniformly distributed dead loads = 0.15kN/m^2

Case 2 : uniformly distributed live load over full span = 0.55 kN/m^2 .

Case 3 : uniformly distributed live load on left half of the span .

Case 4 : Combination of case 3 and wind loads [14] .

Wind loads are treated according to " Egyptian Standard Specification "[15].

To election the optimum dimensions of the coneave cable roof shown in Fig.2 , the analysis is earried out for different concave roofs taking the following assumptions .

a) For initial tension in cables as a percentage of minimum breaking loads , three stages are considered as :
stage 1 : 20% and 10% for guys and other cable elements, respectively .
stage 2 : 20% and 10% for guys and rising cable and sagging cable , respectively .
stage 3 : 10% for all cable elements .

b) For sag and rise to span ratios , three options are considered as :
option 1 : rise to span ratio is 3% and the sag to span ratio is variable .
option 2 : rise to span ratio is 4% and the sag to span ratio is variable .
option 3 : both sag and rise to span ratios are variables .

4- Static analysis of concave cable beam .

4.1 Parametric studies

4.1.1 Spacing between vertical tie elements :

For a 30m span roof with option 3 , both sag and rise to span ratios of 5% , and stage 2 , the analysis is carried out . The results are given in Figs. (4 to 7) . These results showed that , case of loading 2 gave a maximum deflection at midspan while case of loading 3 gave a maximum deflection at roof span's quarter . The variation of spacing between vertical ties in the range up to 4m , has no significant effect on the responses of the structure . Finally , from obtained results and upon experience , it can be concluded that , the optimum spacing between vertical ties lies between 2 to 3m . In the following studies the spacing between vertical ties is considered as 2.5m .

4.1.2 Initial tension in cable elements .

With both rise and sag to span ratios of 5% , the results given in Figs. (8 to 11) which are carried out for three stages mentioned in item 3 showed that :

1) the deflection and tension in sagging cable decreases with increasing the initial tension in all stages and all cases of loading while the tension in rising cable increases .

2) stage 2 which considered that the initial tension for inclined cables and other cable elements were 20% and 10% of minimum breaking load gave a good results .

4.1.3 Both sag and rise to span ratios :

With the obtained results in first two items and options mentioned in item 3 , the analysis is achieved . The results are given in Figs. (12to17) . The results demonstrate that , increasing both sag and rise to span ratios the deflections and final tensions in sagging cable decreasing while the tension in rising cable increasing . So that , the best choice for sag and rise to span ratios is option 2 which considered that these ratios are 5% and 4% for sag and rise , respectively.

4.1.4 Inclination of guy cable with horizontal .

The analysis of all proposed cases of loading is carried out and some results are given in Figs. (18 to 25) . It can be summarized that the optimum inclination of the guys ranges between 30 and 50 degrees .

4.1.5 Inclination of columns on horizontal :

The analysis is carried out taking into consideration all the optimum design parameters obtained from the previous investigations with guy inclination of 30 degree . The results given in Figs. (24 to 28) showed that the best angle of inclination for columns with vertical doesn't exceed than 15° .

4.1.6 Effect of diagonal cables on the analysis :

The analysis is carried out for roof shown in Fig. (2b) considering vertical columns , 45° as inclination of lower guy , stage 2 of initial tension and option 2 for sag to span ratio . Some results are given in Table 2 . It can be noted that for beam shown in Fig. (2b) , the tensions forces in sagging and rising cables decrease while in inclined guys increase . Also , the columns sway with beam

shown in Fig. (2b) is greater than that for beam shown in Fig. (2a) . The significant factor is the number of iteration required to get the specified accuracy reduced by a considerably numbers in case of beam Fig. (2b) . So that , it isn't important to use diagonal elements with light weight .

4.1.7 Rigidity of columns and their supports type :

With all assumptions given in item 4.1.6 the analysis is achieved and the results are given in Tables 2 and 3 . It can be concluded that , a hinged supports reduced the bending moment in columns by about 10% and increased the final tension in sagging , rising and upper guys by small values . Also , with increasing the rigidity of columns all responses decrease .

4.3 A complete analysis of 60m and 90m span beams :

The analysis is achieved taking into consideration all the optimum values of studied parameters obtained for span of 30m . A sample of results are given in Table 3 .

5- Conclusions :

The general conclusions can be summarized as :

(1) A Newton-Raphson method is appropriate for carrying out the analysis of this type of structures .

(2) The number of iterations required to give the specified accuracy of analysis decreases with the increase of the initial tensions in cables and increase with increasing of loads , especially for case of unsymmetrical loading . High rigidities columns with the arrangement of the diagonal ties in the cable beam reduce considerably the number of iteration required for a specific accuracy .

(3) The optimum values of the design parameters concluded from the present work can be summarized as :

(a) The spacing between vertical ties ranges between 2 to 3m .
The inclination of columns about 15° with vertical decreases both sag and rise to span ratios .

(b) In order to ensure that the all cable elements remain in tension for all case of loading , the value of initial tension for inclined guys must be twice the values of initial tension for all other cables .

(c) The sag and rise to span ratios are 5% and 4% , respectively .

(d) The inclination of guys with horizontal ranges between 30° to 50° .

(e) The inclination of columns with vertical does not exceed of 15° .

(f) In case of light loads , the arrangement of diagonal cable elements isn't necessary .

(g) The hinged support for columns is better than the fixed support with high initial tension in guys .

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Table (1) : Cables and stays properties for concave cable roof.

Item	span m	Type	Diameter, cm	Area, cm2	Breaking load ,(KN)	Weight N/m'	Modules,E KN/mm2
All cables of beam	30m	BS5896	1.8	2.23	380	17.17	150
	60m	spiral	3.31	6.78	910	54.8	169.7
	90m	locked	4.8	15.3	2001.3	125.7	158.4
Guy Stays	30m	spiral	2.5	3.74	557.2	29.9	169.7
	60m	spiral	4.2	10.33	1500.9	84.8	169.7
	90m	locked	5.51	18.4	2452.5	150.8	158.4
Flexural members	A=124.54cm^2, J =3419cm^4, Ix= 11850cm^4, Iy=11850cm^4, E=21000KN/cm^2, Weight = 962N/m'						

Table (2) : Some results for cable roofs shown in figs. (2a and 2b) :

Items	Fixed supports				Hinged supports			
	Case (2)		Case (3)		Case (2)		Case (3)	
	[A]	[B]	[A]	[B]	[A]	[B]	[A]	[B]
Defl. at n, cm	-18.3	-9.21	-17.2	-7.4	-19.3	-10.1	-18	-8.04
Defl. at m, cm	-23.8	-24.1	-13.9	-13.9	-25.8	-25.2	-14.8	-14.7
Defl. at o, cm	-18.3	-9.2	-3.77	-3.51	-19.3	-10.1	-4.32	-3.97
Sway at a, cm	0.61	1.16	0.36	0.88	0.8	1.32	0.49	0.99
Sway at b, cm	-0.61	-1.16	0.4	-0.22	-0.8	-1.32	-0.3	-0.32
Max. sway, cm	1.7	1.41	1.53	1.2	2.4	2.02	2.13	1.72
Tension in 1, KN	103	87.6	95.3	78.9	103	87.6	95.2	78.5
Tension in 2, KN	124	137	118	130	129	141	121	133
Tension in 5, KN	14.8	6.6	21.2	14.9	17.2	8.6	23	16.5
Tension in 6, KN	69.7	82.9	71.8	85.6	62.9	76.9	65.4	80.3
N. F., KN	180	199	154	175	178	199	154	176
B.M., KN.m	64.5	63	55.1	52	58	57.1	49.8	46.9
Iteration number	560	258	702	494	568	299	720	529

Table (3) : Some responses for infinitely rigid columns beam :

		Y_n, cm	Y_m, cm	Y_o, cm	T_1,KN	T_5,KN	Iteration
Case 2	Beam B	-4	-16.85	-4	91.2	2.15	46
Case 3	Beam B	-3.3	-9.91	-1.41	81.2	6.03	84
Case 2	Beam A	-13.8	-18.37	-13.8	96	-1.48	141
Case 3	Beam A	-14.2	-10.39	-2.32	89	-0.11	253

Table (4) : Some responses for beams having span of 60m and 90m :

			Y_m	Sway	T_1	T_2	T_5	T_6	N.F.	B.M.
60m span	Beam A	Hinged	39.7	3	229	392	36.3	73	441	53
	Beam B	Hinged	45.6	2.8	228	426	37.8	98.3	493	57
	Beam A	Fixed	38.8	2.41	228	387	34.4	82.9	443	66.8
	Beam B	Fixed	44.9	2.24	227	421	35.8	108	494	68.4
90m span	Beam A	Hinged	52.8	2.98	377	748	43.8	77.3	807	48
	Beam B	Hinged	73.5	3.07	399	831	28.3	79	898	54
	Beam A	Fixed	52.3	2.50	378	744	42.6	86.6	808	60
	Beam B	Fixed	73.1	2.50	399	826	26.4	87.8	900	67.5

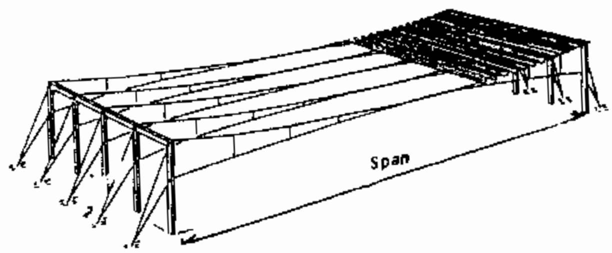

Fig. (1) :- Concave Cable Roof with Corrugated Metal Roof Deck .

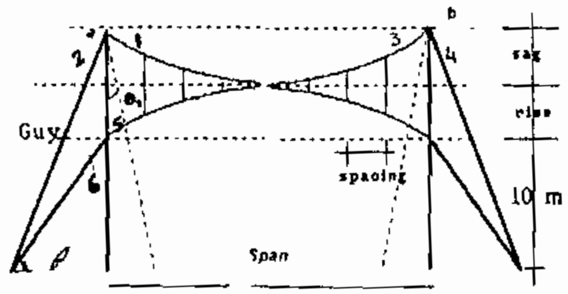

Fig. (2a) :- Concave Cable Roof without Diagonals.

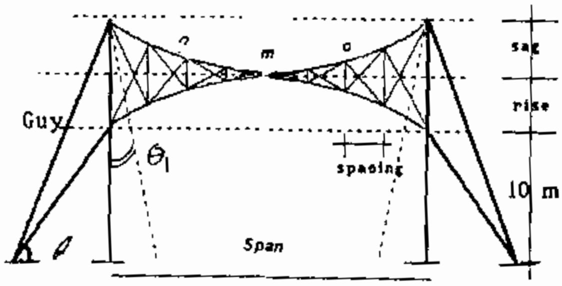

Fig. (2b) :- Concave Cable Roof with Diagonals .

Fig. (3) :- Concave Cable Roof

with Infinitly Rigid Supports .

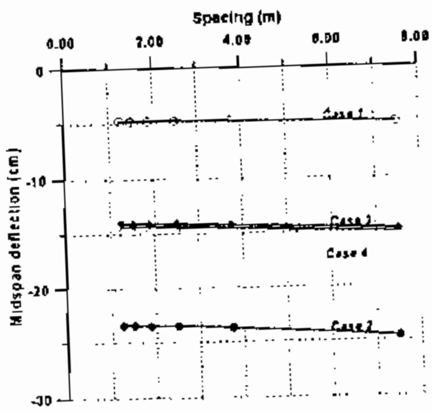

Fig. (4) Relation between spacing and deflection.

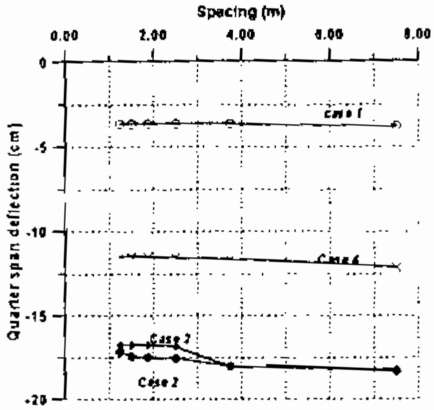

Fig. (5) Relation between spacing and deflection.

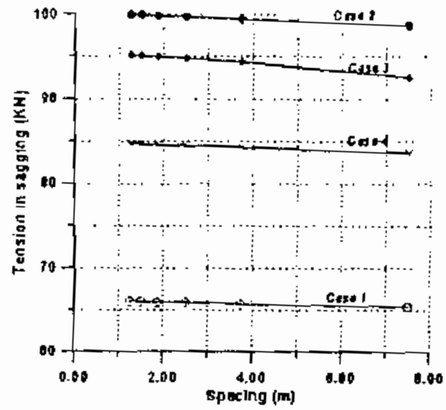

Fig. (6) Variation of spacing with tension.

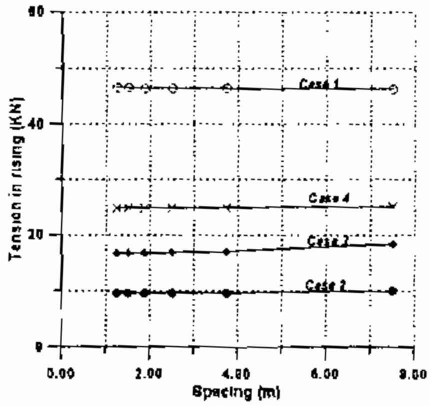

Fig. (7) Variation of spacing with tension.

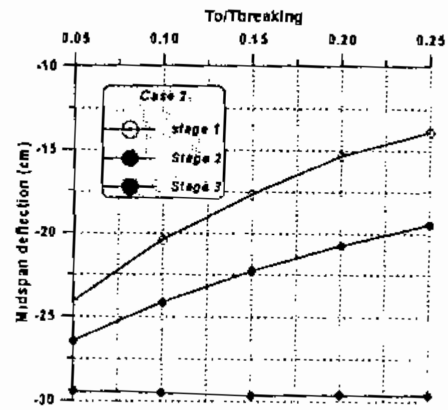

Fig. (8) variation of To/Tbreaking with deflection.

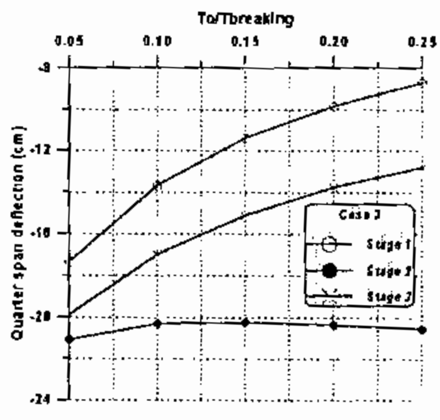

Fig. (9) Variation of To/Tbreaking with deflection.

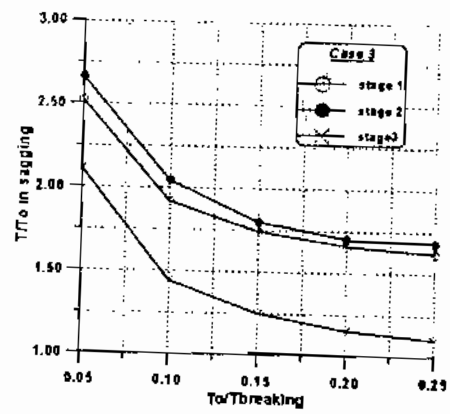

Fig. (10) Relation between To/Tbreaking and T/To.

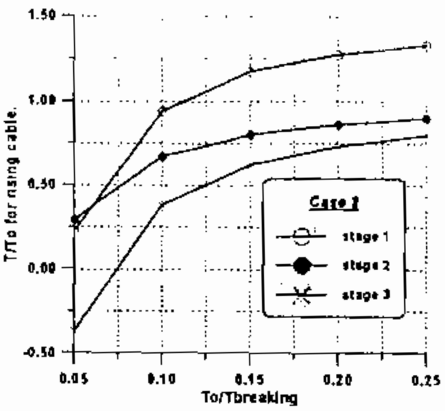

Fig. (11) Relation between To/Tbreaking and T/To

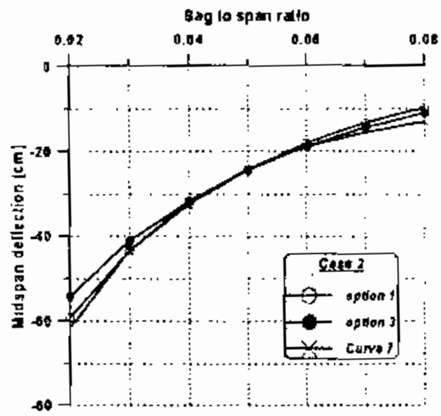

Fig. (12) Variation of Sag to span ratio with deflection.

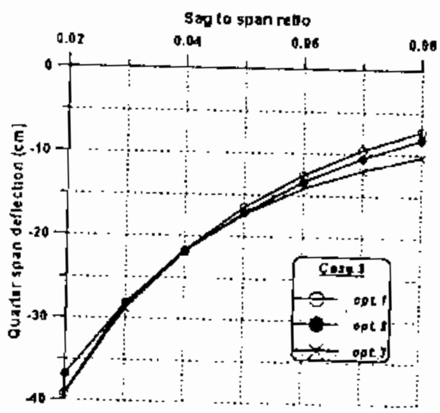

Fig. (13) Variation of sag to span ratio with deflection.

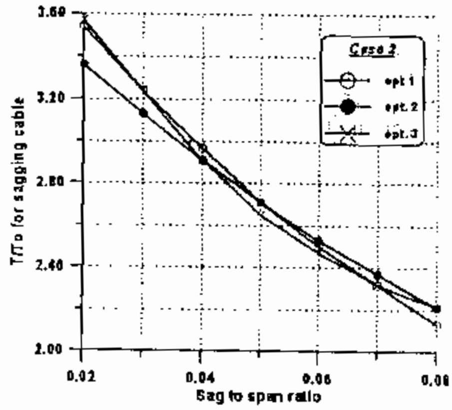

Fig. (14) Variation of T/To with sag to span ratio.

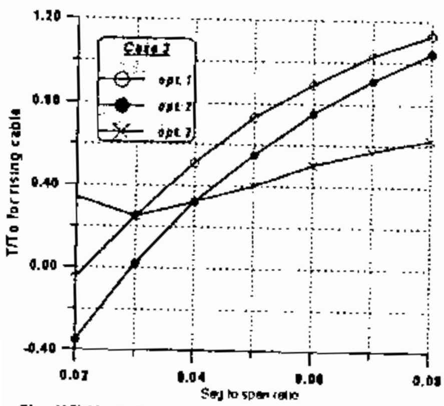

Fig. (15) Variation of T/To with sag to span ratio.

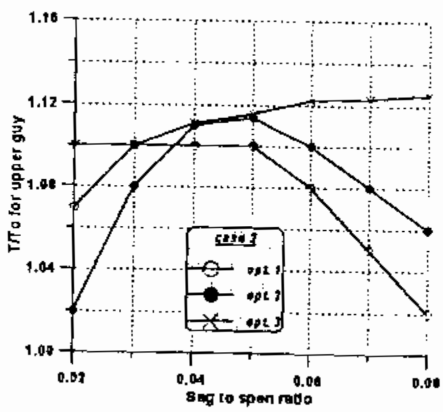

Fig. (16) Variation of T/To with sag to span ratio.

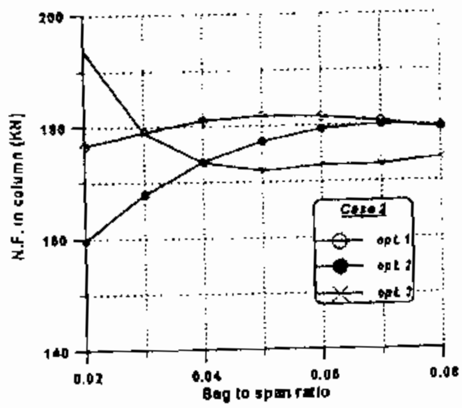

Fig. (17) Variation of N.F. with sag to span ratio.

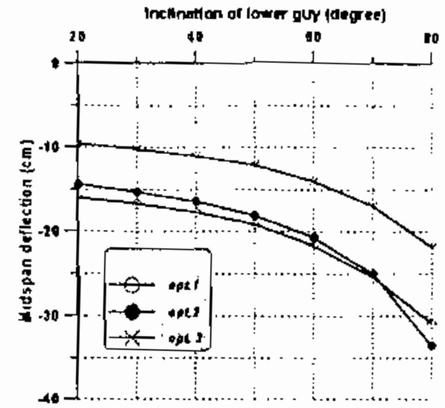

Fig. (18) Relation between inclination and the deflection.

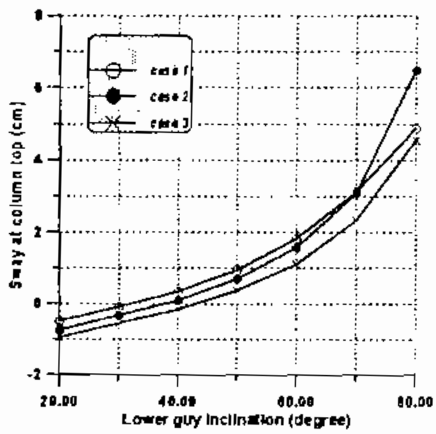

Fig. (19) Relation betweewn inclination and sway.

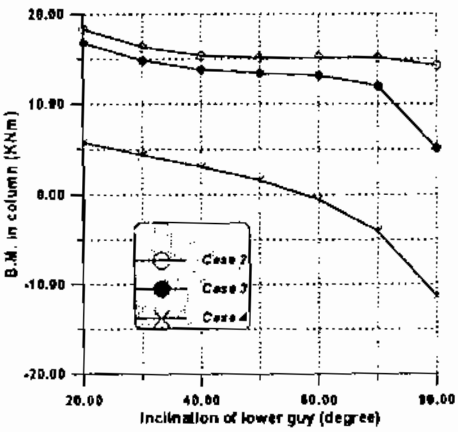

Fig. (20) Variation of B.M. with inclination.

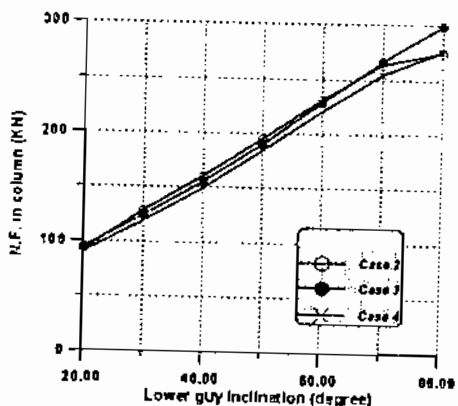

Fig. (21) Variation of N.F. with inclination.

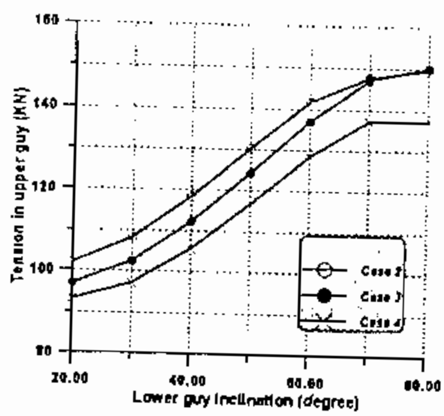

Fig. (22) Variation of inclination with tension.

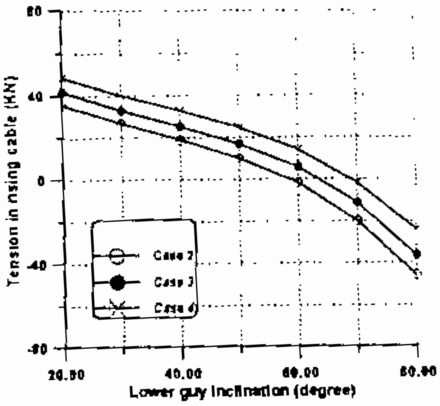

Fig. (23) Variation of inclination with tension.

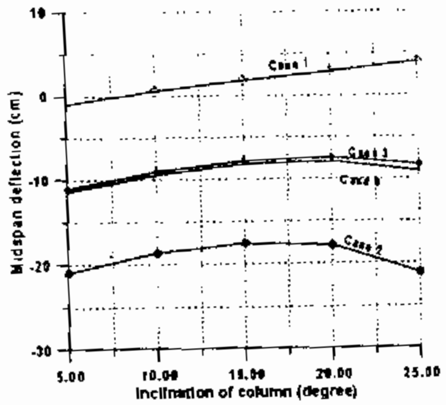

Fig. (24) Variation of column inclination with defl.

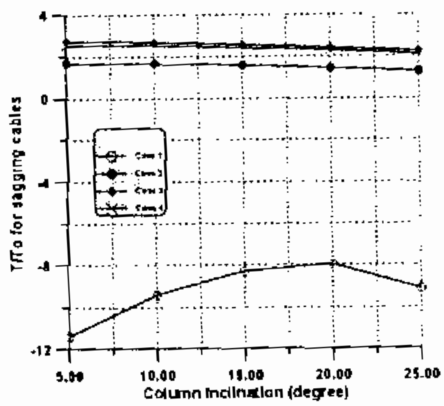

Fig. (25) Variation of column inclination with T/To.

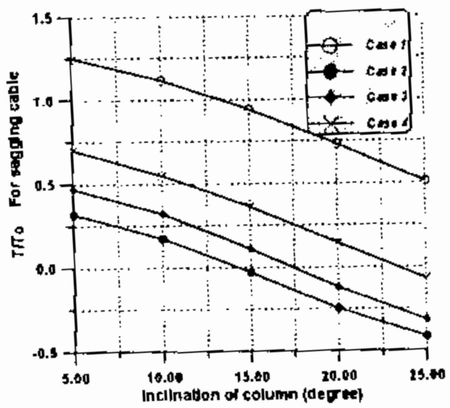

Fig. (26) Variation of T/To with column inclinatio.

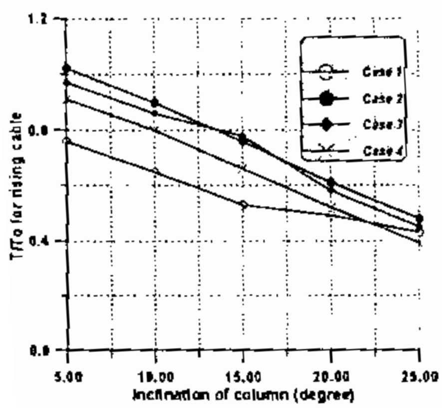

FIG.(27) Variation of T/To with column inclination

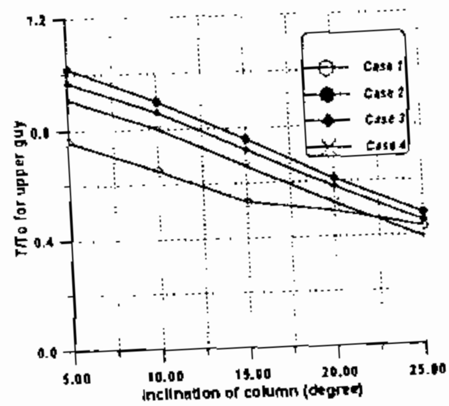

Fig. (28) Variation of T/To with column inclination.