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DYNAMIC RESPONSE OF GUYED MASTS TO STRONG MOTION EARTHQUAKE

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ABSTRACT

A nonlinear dynamic response calculation method is presented, based on step by step response calculation in the time domain for which equilibrium of the dynamic forces at the end of each time increment is established by minimisation of the total potential dynamic work. The dynamic loading is generated as a series of cross correlated earthquake histories defined by base frequency envelope of typical recorded earthquake histories with inclusion of main frequency components of the structure.

The analysis method is applied to the numerical model of a 327 metres tall guyed mast with five levels of stay and a cantilever at the top. In the numerical model, the mast shaft and its cables are all modelled together and analysed as a whole single unit. Three ground accelerations in two horizontal and one vertical directions are generated and applied to the mast. The response of the structure to generated strong motion earthquakes is calculated. A mass-spring system model of the mast is also constructed with the masses concentrated on the stay levels, the cables are represented by their spring stiffness. Comparative study of the results obtained from nonlinear dynamic analysis and conventional mass-spring system approach is made and presented.

KEYWORDS

Guyed masts; dynamic response; nonlinear analysis; frequency domain; time domain.

LINEAR AND NONLINEAR ANALYSIS METHODS

Guyed masts are highly dynamic sensitive structures and will undergo large deflections under dynamic loading, this introduces geometrical nonlinearity and effects the calculation of forces in cables and therefore the reactions at the base of the mast. Strong motion earthquake introduces base vibration to the cable anchor blocks and the mast base, causing the cables to vibrate and produce large changes in the tension of the cables. These fluctuations should be considered in the design of the shaft and the base foundation. The method developed and used for nonlinear dynamic response analysis of the mast to strong motion earthquake in this work is based on step-by-step integration of the total potential dynamic work (Buchholdt *et al.*, 1986).

The concept of dynamic response analysis by energy minimisation method for large structural assemblies, (Moossavi Nejad, 1985), comprises the following four parts.

1- Formulation of the total potential dynamic work for the assembly.
2- Difference equations expressing the relationship between displacement, velocity and acceleration with respect to time.
3- A minimisation scheme to minimise the total potential dynamic work.
4- Description of the dynamic loading either by a function or as a series of tabulated time histories.

Complete formulation of the method is given by (Moossavi Nejad, 1985) and is not repeated here. The implementation of the method related to the Newmark's ß=¼ difference equations, (Newamk 1959) and the conjugate gradients minimisation method, is employed to calculate the dynamic response of the following guyed mast to a generated strong motion earthquake. The earthquake histories are generated by applying cross correlation to typical recorded earthquake histories containing frequency components of the mast. The earthquake history in the Y-Direction and its frequency spectrum are given in Figs. 1a and 1b respectively.

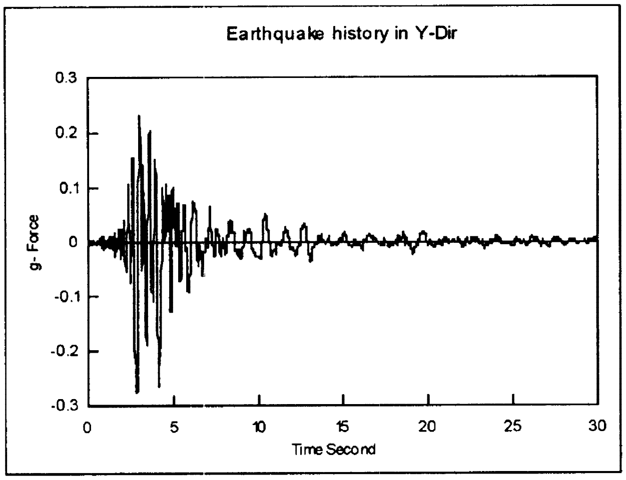

Fig. 1a Earthquake history.

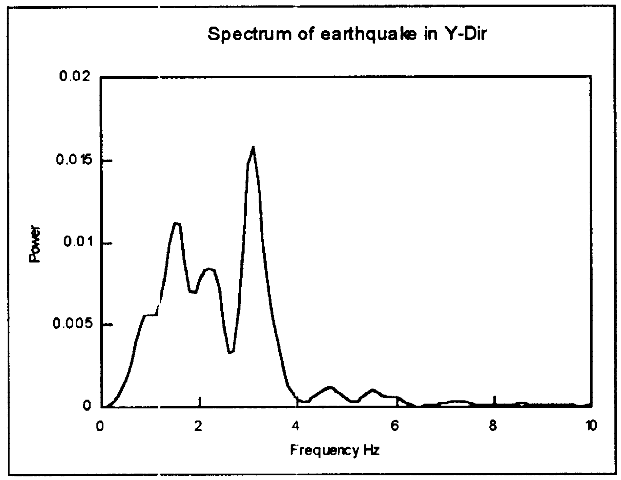

Fig. 1b. Frequency spectrum of earthquake.

The frequency domain method used for linear analysis of the mast is based on decoupling of the equation of motion for a linear system subjected to a support motion $x_g(t)$ with acceleration $\alpha\ddot{x}_g(t)$ as follows.

$$M\ddot{X} + C\dot{X} + KX = M\alpha\ddot{x}_g(t) \tag{1}$$

Where M, C and K are the mass, damping and stiffness matrices for the system and X, $\dot{X}$ and $\ddot{X}$ are the displacement, velocity and acceleration vectors respectively. α is a constant that defines the magnitude of the peak acceleration. Let Z be the matrix of normalised eigenvectors with respect to M, then for q, $\dot{q}$, and $\ddot{q}$ as decoupled displacement, velocity and acceleration, given as $X=Zq$, $\dot{X}=Z\dot{q}$ and $\ddot{X}=Z\ddot{q}$, eq. (1) can be written as:

$$Z^TMZ\ddot{q} + Z^TCZ\dot{q} + Z^TKZq = Z^TM\alpha\ddot{x}_g(t) \tag{2}$$

and for $Z^TMZ = I$, $Z^TCZ = 2\xi\omega$, and $Z^TKZ = \omega^2$ the decoupled equation of motion becomes

$$\ddot{q} + 2\xi\omega\dot{q} + \omega^2 q = Z^TM\alpha\ddot{x}_g(t) \tag{3}$$

where ξ and ω are the damping ratio and natural angular frequency of the system respectively. Using the power spectral density function and peak acceleration for a recorded or generated earthquake history and its variance σ^2 together with statistical factor κ (Clough 1993), the value of q can be calculated from $q = \kappa\sigma$ and $X = Zq$.

NUMERICAL MODELS

The guyed mast selected for the numerical analysis is a 327m tall radio mast with five levels of stay and a 27m cantilever on top. The shaft of the mast is triangular in section with solid round legs and with a constant width of 3m. The numerical model has 82 cable nodes and 28 rigid nodes with 372 translational and 129 rotational degrees of freedom. The properties of stays and the mast shaft are given in Tables 1. and 2. respectively. Insulators each weighing 145 kg are applied to the stays as given in Table 1.

Table 1. Properties of stays.

Level	Height (m)	E-Module (N/m^2)	Area (m^2)	Initial Tension (kN)	Weight (N/m)	Cable Diam. (m)	Base Radius (m)	Insulators Per stay	Ins. weight (kg)
1	300	1.6E+11	1.300E-03	50	104.00	0.046	225.000	5	2175.0
2	240	1.6E+11	1.300E-03	50	104.00	0.046	180.000	5	2175.0
3	180	1.6E+11	9.650E-04	40	77.40	0.040	180.000	5	2175.0
4	120	1.6E+11	9.650E-04	30	77.40	0.040	85.000	3	1305.0
5	60	1.6E+11	7.660E-04	30	61.30	0.035	85.000	3	1305.0

Table 2. Properties of the mast shaft.

Level	Height (m)	E-Module (N/m^2)	Area (m^2)	G-Module (N/m^2)	Weight (N/m)	Ixx (m^4)	Iyy (m^4)	Izz (m^4)	Shaft width (m)
Cantilever	327	2.05E+11	0.03393	7.89E+10	4807.0	0.04	0.0508938	0.0508938	3.000
1	300	2.05E+11	0.03393	7.89E+10	4807.0	0.04	0.0508938	0.0508938	3.000
2	240	2.05E+11	0.04618	7.89E+10	5788.0	0.04	0.0692721	0.0692721	3.000
3	180	2.05E+11	0.04618	7.89E+10	5788.0	0.04	0.0692721	0.0692721	3.000
4	120	2.05E+11	0.06032	7.89E+10	6867.0	0.04	0.0904779	0.0904779	3.000
5	60	2.05E+11	0.06032	7.89E+10	6867.0	0.04	0.0904779	0.0904779	3.000

A simplified mass-spring system model of the mast, containing 6 lumped masses concentrated on stay levels of the shaft is also constructed for frequency domain analyses. The line diagram of the mast and the numbering arrangements for the nodes and members together with a three dimensional view of the mast showing the stay arrangements are given in Fig. 2. The figure also contains the numbering arrangements for the mass-spring system model.

NUMERICAL ANALYSIS

The guyed mast is subjected to static analyses, eigenvalue analysis and linear and nonlinear dynamic response analysis as described below. The design of the mast is based on British Standards BS8100 Parts 1 and 4, for a wind speed of 45 m/s.

Static Analyses

To establish design forces and deflections, static analyses of the mast were carried out for the self weight and static wind load in the x and the +ve and -ve y directions, see Fig. 2. Concentrated static load of 50 kN was applied to each of joints 97, 100, 105, 110, 115 and 120 in turn and for each case the deflections of all these joint, referred to from now on as the lumped mass joints, were noted in a table. The values in the table, ie the deflections, were then divided by the load to provide the flexibility matrix for the mass-spring system model. The stiffness matrix for the model was obtained from the inverse of this matrix.

This method of calculation of stiffness matrix for the model is fundamentally different from that obtained by the usual assembly of the stiffness matrix from the properties of the shaft and the assumed stiffness of springs representing the cables. In this method, since the static analyses are carried out on the full model of the mast, the effects of the stays and their change of tension are implemented in the stiffness matrix directly. To verify the stiffness matrix, a set of direct point loads were applied to the mast column and to the relative nodes of the mass-spring model, the resultant displacement vectors are given in Table 3. and show less than 7% difference between the two. The calculated stiffness matrix is given in Table 4. The diagonal lumped mass matrix, representing the effective mass of lumped mass joints was also constructed and is given in Table 5

Table 3 Static deflection, (m), of lumped mass joints for the guyed mast and the mass-spring model.

Node	97	100	105	110	115	120
Load (kN)	20.0	60.0	30.0	30.0	30.0	30.0
Guyed Mast	0.9885	0.7560	0.3749	0.2078	0.1413	0.0762
Mass-spring Model	0.9400	0.7150	0.3500	0.1960	0.1380	0.0760
% Difference	4.9064	5.4233	6.6418	5.6785	2.3355	0.2625

Table 4 Stiffness matrix of the mass-spring system model.

K =

527600.	-837900.	408000.	-119400.	33470.	-9332.
-844500.	1533000.	-846000.	327400.	-92760.	28670.
406800.	-849800.	1001000.	-657400.	289900.	-76030.
-122300.	338200.	-662500.	1090000.	-742600.	309400.
31160.	-90870.	293000.	-740500.	1189000.	-730500.
-11820.	30740.	-89010.	316900.	-744200.	1194000.

N/m

Table 5. Mass matrix of the mass-spring system model.

M =

6735.	0.	0.	0.	0.	0.
0.	21875.	0.	0.	0.	0.
0.	0.	32835.	0.	0.	0.
0.	0.	0.	35835.	0.	0.
0.	0.	0.	0.	39135.	0.
0.	0.	0.	0.	0.	42435.

kg

Eigenvalue Analysis

An eigenvalue analysis of the full mast was carried out and provided the full range of 372 frequencies of the mast. These frequencies started from 0.159 Hz and continued in bands of close frequencies to 10.0 Hz for the first 200 modes. The study of mode shapes related to these frequencies showed the first six modes of the mast shaft being in the range of 0.3 to 3.0 Hz. Unfortunately, the available space does not permit the presentation of the mode shapes.

Utilising the stiffness and mass matrices of the mass-spring model, its frequencies and mode shapes were calculated by an eigenvalue analysis and are given in Table 6.

Table 6. Frequencies of the mass-spring model, Hz.

Mode	1	2	3	4	5	6
Frequency Hz	0.295	0.352	0.506	0.813	1.257	2.019

Dynamic Analysis

The guyed mast was subjected to earthquake loading in 3 directions. Strong motion histories were generated for 30 seconds in the x, y and z directions, with intensities of 0.3g in the horizontal and 0.2g in the vertical direction, a sample history is shown in Fig. 1a. The nonlinear dynamic analysis was carried out for a period of 120 seconds to allow the structure to respond to all frequency components and to contain the peak response of the stays and the mast shaft. A recording of the maximum amplitude of each node in all 3 directions and the maximum force encountered by each member were kept together with their time of occurrence.

The mass-spring model was analysed for earthquake loading, using the linear frequency domain method given above. The peak acceleration factor, α, was taken as 0.3 and the frequency spectrum of the generated earthquake history for y-direction was used to calculate the variance and other statistical factors for the frequency domain calculations. In order to investigate the effect of damping, both linear and nonlinear dynamic analysis were carried out for two values of logarithmic damping, $\delta = 4\%$ and $\delta = 8\%$ equivalent to damping ratios $\xi = 0.006$ and $\xi = 0.012$. The peak amplitude of lumped mass joints were recorded for the four cases analysed and are given in Table 7. Response of joint 4, middle of the top cable, in y-direction and its frequency spectrum are shown in Figs 3a and 3b, the response of joint 100, top stay level on the shaft, and its frequency spectrum are given in Figs. 4a and 4b. The fluctuation of tension of top level stay and its frequency spectrum are shown in Figs. 5a and 5b. Figure 6. shows the movement of joint 100 in the x-y plane.

Table 7. Maximum amplitude of response (m) to earthquake loading for linear and nonlinear dynamic response analyses.

Joint No	Height (m)	4% Damping Time Domain Deflection (m)	4% Damping Freq. Domain Deflection (m)	8% Damping Time Domain Deflection (m)	8% Damping Freq. Domain Deflection (m)
97	327	0.3271	0.2830	0.3451	0.2000
100	300	0.1457	0.2730	0.1262	0.1930
105	240	0.1750	0.2560	0.1725	0.1810
110	180	0.1549	0.2300	0.1590	0.1630
115	120	0.1917	0.2250	0.1746	0.1590
120	60	0.1513	0.1330	0.1573	0.0940

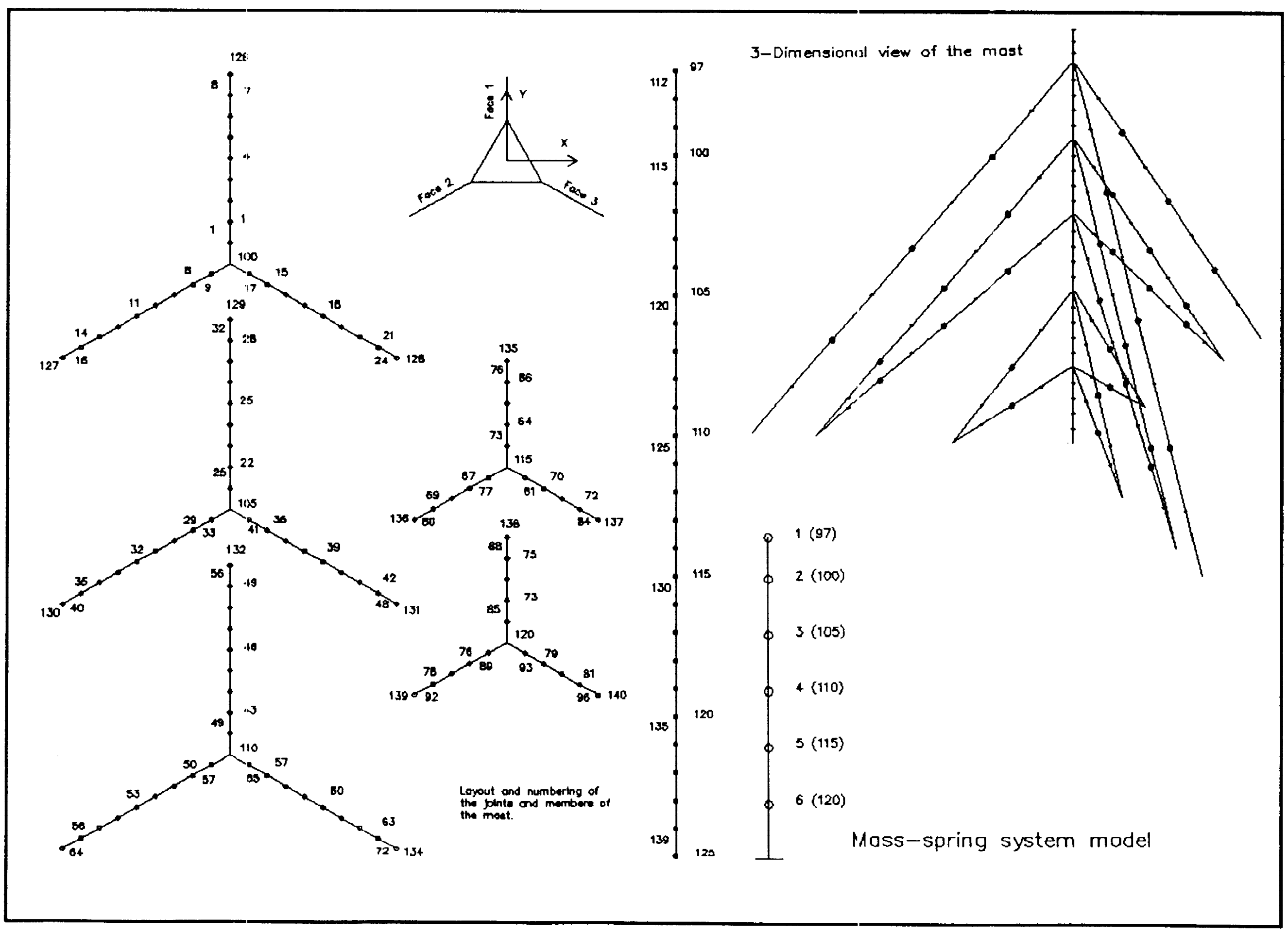

Fig. 2. Line diagram of the guyed mast model and the mass-spring system model.

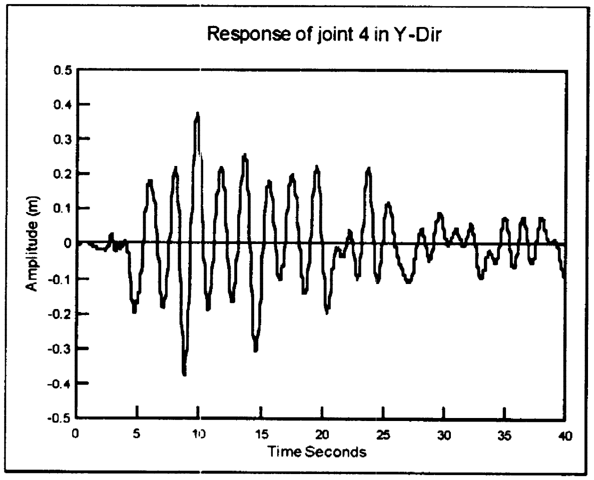

Fig. 3a. Response of joint 4.

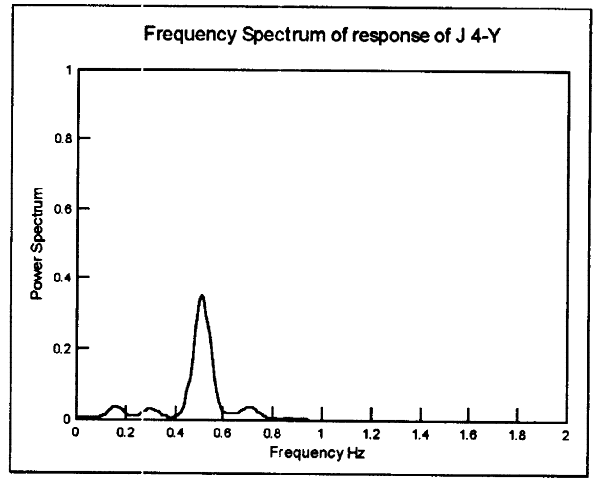

Fig. 3b. Frequency spectrum of response of joint 4.

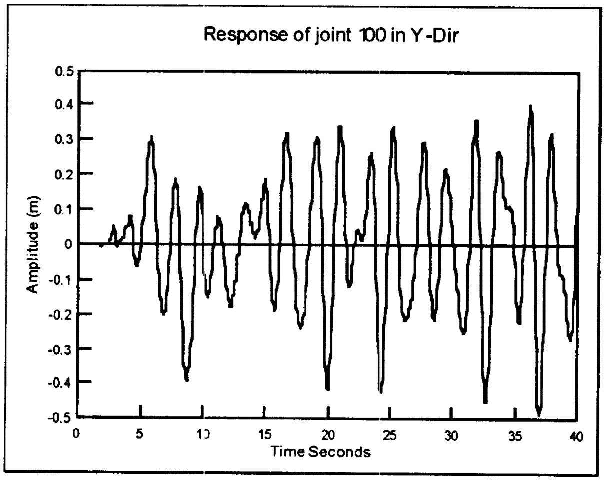

Fig. 4a. Response of joint 100.

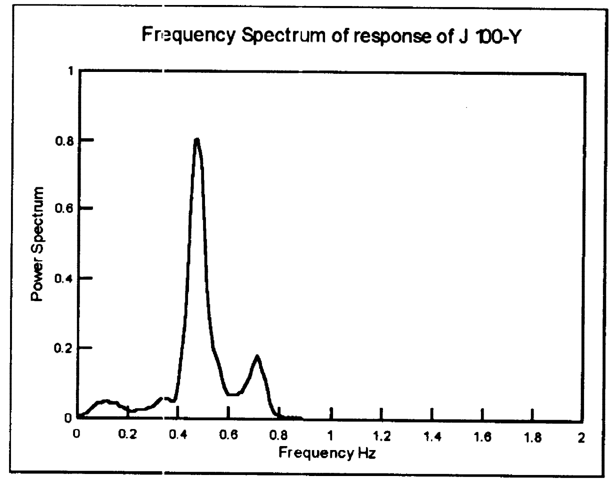

Fig. 4b. Frequency spectrum of response of joint 100.

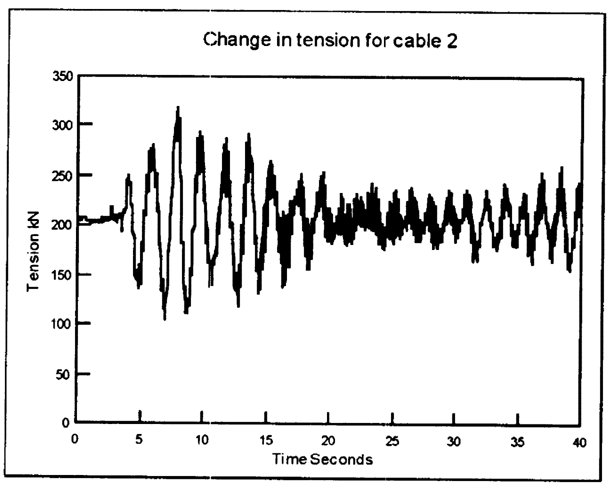

Fig. 5a. Fluctuation of tension of cable at stay level 2.

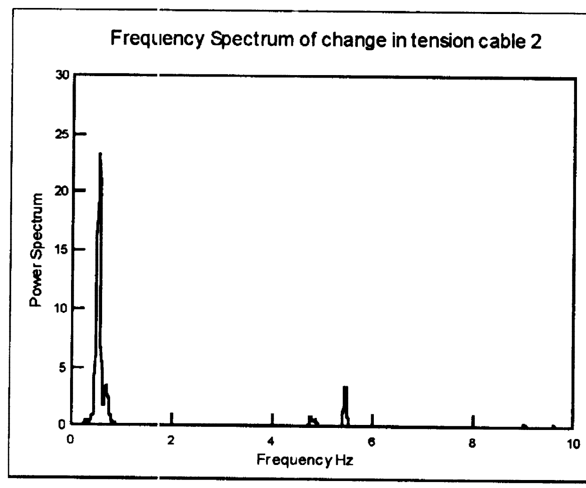

Fig. 5b. Frequency spectrum of change in tension.

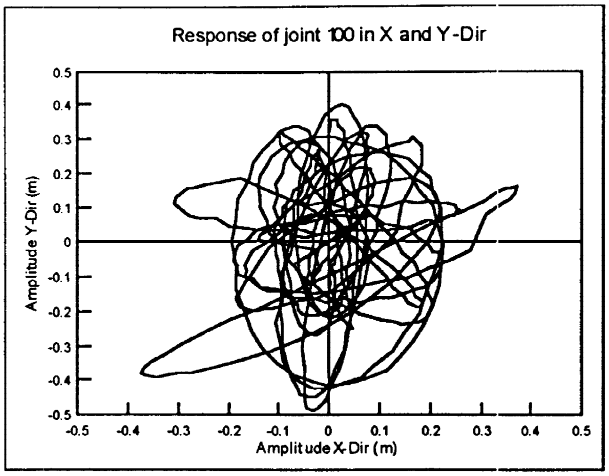

Fig. 6. Movement of joint 100 in the x-y plane.

To investigate the effect of earthquake loading on overall design forces of the mast, a comparative listing of maximum forces due to self weight, design wind load and the earthquake loading is made in Table 8. The table shows the maximum force in the base of the mast, the maximum bending moment of the mast shaft and the maximum tension at each stay level.

Table 8. Maximum force, bending moment and tension of the mast.

	Max. Force (kN)	Max. Moment (kN)	Max. Tension (kN) in Stay Level				
	Shaft Base	Mast Shaft	1	2	3	4	5
Static Self Weight	3560	N/A	205.0	179.0	140.0	77.0	70.0
Static Design Wind	4521	1175	407.0	375.0	310.0	250.0	232.0
Dynamic Earthquake	5900	3164	327.0	278.0	311.0	210.0	237.0

CONCLUSIONS

The results of time domain dynamic response analysis, unlike their static or frequency domain analysis counterparts, are complex and require much effort to extract its multifold layers of information. Although a record was kept of maximum deflections and forces within the time duration of the analysis, these values are not appropriate for the design of structures, since the maximums do not occur at the same time. In fact if these values are used for design purposes, the structure will be over designed. The way to overcome this problem is to keep a record of forces in all members when a maximum axial force, shear force or bending moment develops in any member. In other words, to freeze the structure at that time and use its instantaneous forces for design to a particular force. However, presentation of such results for a large structure will be rather impractical. Time domain analyses provide understanding of the behaviour of the structure under dynamic loading, rather than simple listing of the results. This, in many cases, can be achieved by animation of the movement of the structure and its various modes of vibration.

During the course of this work, many computer runs were made for different parameters of time history generation and properties of the structure such as damping. The summary of maximum amplitudes and forces are given in Tables 7. and 8. Table 7 shows that, in general, the amplitudes calculated by frequency domain analysis are larger than those calculated by time domain analysis. In the time domain analysis, the cable stays restrict the movement of the mast shaft at stay levels but the cantilever vibrates with a large amplitude. In Table 8. the effect of vibration of cables on the forces in the mast shaft can be seen, although the tension in cables due to earthquake loading, shown in Table 8., are smaller than those of design wind load, they relate to lateral deflections of about one third of those produced by wind loading. This means, if the amplitudes produced by dynamic excitation reach those of design wind, the tension in cables would be much larger.

Although the stiffness matrix constructed for the mass-spring model provided good correlation between the static deflections of the guyed mast and the small model, the results obtained from the linear dynamic analyses of the mass-spring model are not comprehensive enough to give good comparison with the results of time domain analyses. In particular, the six degrees of freedom of the small model related to lateral movements in one direction only were not sufficient. To achieve more conclusive results, the degree of freedom, and not the number of nodes, must be increased for the mass-spring model to contain movements in the other two directions. This require the construction of much larger stiffness matrix by the method described in the Static Analyses section.

REFERENCES

Buchholdt H. A. Moossavi Nejad S. E. and Iannuzzi A. (1986). Nonlinear dynamic analysis of guyed masts subjected to wind and guy rupture. *Proceedings of the Institution of Civil Engineers,* **Paper 9015, Part 2.**

Clough R. W. and Penzien J. (1993). *Dynamics of Structures.* McGraw-Hill.

Moossavi Nejad S. E. (1985). Implementation of wind loading in time domain dynamic response analysis by minimisation of total potential work. *Proceedings of Asia Pacific Symposium on Wind Engineering.* University of Roorkee, Roorkee, India

Newmark N. M. (1959). A method of computation for structural dynamics. *Journal of Engineering Mechanics, Division ASCE,* **Vol. 85, No EM3.**