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FAILURE OF THE ALL INDIA RADIO GUYED TRANSMITTER MASTS AN ASSESMENT

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Introduction

This paper has, essentially, two objectives, which the authors believe, are of considerable significance to practising engineers. The first is to put on record a brief report on the failures of the two 122m (200ft) guyed transmitter masts of All India Radio of VIJAYWADA during historic cyclone which hit the coastal area of Andhra Pradesh on Nov, 17, 1977. This in itself is of considerable significance to engineering industry. More recently a similar transmitter and a free-standing tower of AIR Kingsway Camp Delhi collapsed in July 1978 due to the passage of a tornado. Each such causality if properly analyzed can throw considerable light on many aspects hitherto neglected. In this country, a code of practice for communication towers is in a preparatory stage and much needs to be done to strengthen its provisions. What is even more important, is the need to highlight the significance and urgency of observing wind records of interest to structural engineers. To-date whatever little gust data of wind is collected by few of the meterological centres is on too crude a scale of no relevance in structural design. Similar to the strong ground motion recorders whose data are of relevance to structural engineers, data of wind should also be recorded to small enough scales of use in structural response calculations. Likewise wind spectra should be developed and recommended by the codes of practice.

The second objective is to present the finding of an approximate but yet useful analysis carried out for the masts in questions. This forms the first stage in a general project of developing more accurate analysis, though has not been able to exactly substantiate the causes of the failure, has indeed given a feel of the internal forces, time periods, dynamic amplifications, etc., and also shown that an analysis can be easily performed as a check on the design. This is expected to provide some safegaurd in the absence of rigorous analysis, which many times may not be readily accessible.

The exact analysis of a guyed mast is beset with many difficult issues. The statical analysis for an equivalent static wind load, is itself a nonlinear one. This nonlinearity enters as a result of beam—column effect, and cable stifness being dependent on the magnitude of cable tension. Thus an incremental procedure is required where in the total load is applied in small increments. At each guy-level there would, in general be six degrees of displacement parameters. The dynamic analysis has two essential difficulties: firstly it demands the transient history of the wind either the actual one if available, or a standardized one recom-

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mended by the standard institution of a country. Secondly since the elastic response itself is nonlinear a direct timewise integration technique has to be employed. If nonlinearity is ignored the concepts of normal modes can be admitted and the mode superposition method can be conveniently used. References (1) to (8) are few of those which discuss the issue of analysis.

This paper first presents brief summary of the failure of the masts. Then an analysis for equivalent static wind load is presented wherein the mast is analysed as a beam accounting for cable stretches. This is followed by a dynamic analysis using mode superposition method. More details of the failure is available in reference (11) and of the analysis, in reference (12).

Towers—Geometrical Dimensions and Sections

The two masts, North mast and the South mast, are identical. Each consist of a latticed mast of 400 ft (121.919m) height guyed at four levels as in Fig. 1(a). The mast section consists of three tubes placed at the vertical of equilateral triangle and braced by angles so that each planar lattice is statically determinate (Fig. 2(b)). The steel tubes are of 76mm external diameter with thickness varying from 5.5mm to 9mm at different heights of mast. The guy wires are hot-galvanized steel cables of ultimate strength 145 kg/mm^2. The sections of cables are 12.5mm dia and 14mm, the first one used only for the shortest (guy/no. 1, Fig. 1(a)). The lattice sections consist of angles $50 \times 50 \times 5$ and $55 \times 55 \times 6$ Fig. 2(a). The angles are welded to the pipes. The lattice units are prebuilt units of 6m length with 25mm flanges welded to the pipes. The mast is a built-up structure with bolted connections between the flanges. At guy-levels heavier horizontal braces (channel section $200 \times 200 \times 10$ approximately) are provided to avoid distortion of the triangular shape of the mast.

The mast has total dead weight of about 11,000 kg is pin-connected at its base. The three tubes rest on the tips of the three cantilever arms of a horizontal frame supported on a porcelain pin which is essentially a cylinder of about 38 cm diameter and 40 cm high.

Each of the guy cables are separated by insulators and butt straps. The cable joints consist of either pressed sleeves, Fig. 3(a) or double clamps, Fig. 3(b). Turn—buckles are provided near foundation blocks. The foundation blocks are heavy concrete blocks appropriately designed for stability.

Failure of the Towers

It was established that the North mast must have collapsed around 6 p.m. on 19th September 1977. The South mast is believed to have collapsed around 10 p.m. when the wind intensity was known to be most severe.

The only nearest records of wind speeds and pressures available were those taken at the Gannavaran aerodrome meteorological office, about 50 km away from Nombur, where the towers were situated. These were peak wind speeds and pressures observed at 3 hour intervals. No gust records were available. The peak wind speed around 6 p.m. must have been about 60 to 70 km/hr and that at 10.00 p.m. around 200 km/hr. The latter is believed to be the peak which caused intense damage in that area. The failure of the masts correspond to

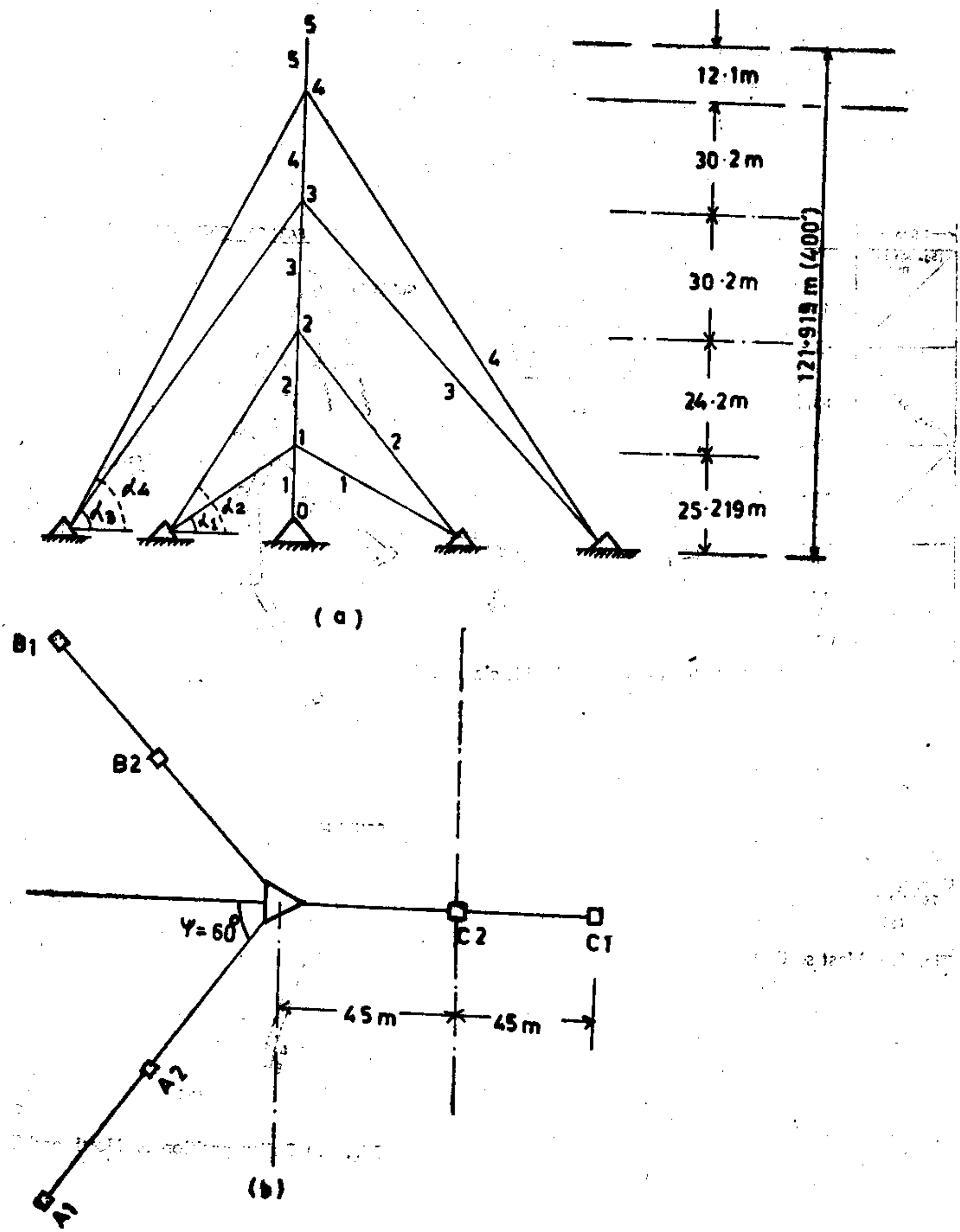

Fig. 1. Latticed mast guyed at four levels.

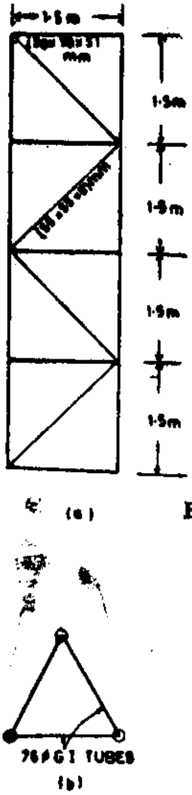

Fig. 2. Mast section.

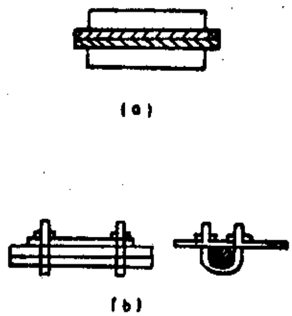

Fig. 3. Cable joints and double clamps.

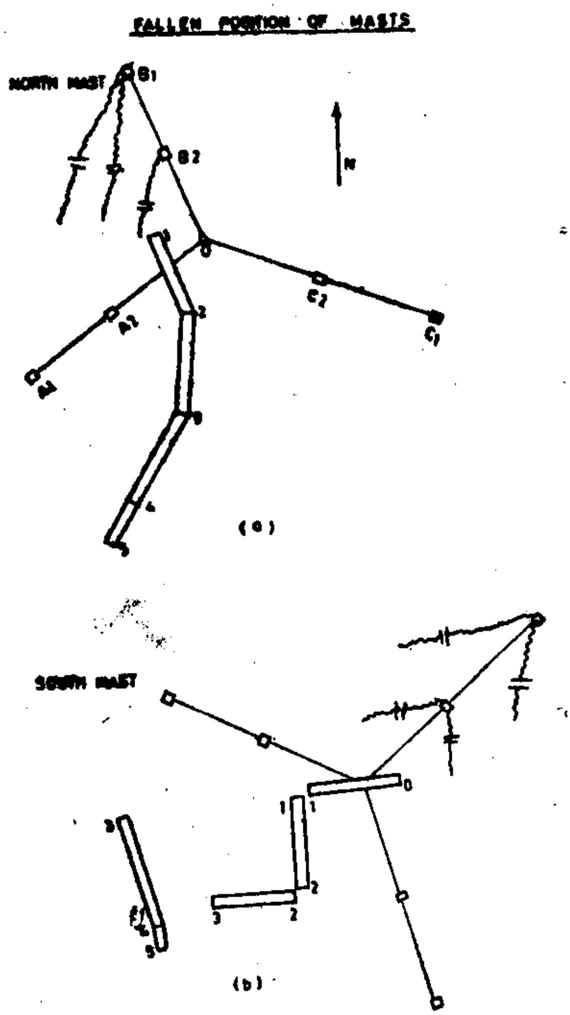

Fig. 4. Fallen position of North and South Most.

these wind intensities. The North mast which failed around 6.00 p.m. has shown lesser damage than the South mast which has been virtually blown over. Fig. 4(a) and (b) show in plan the two fallen masts. Plates 1 to 7 show few photographs of North mast and 8 to 11 those of the South mast.

North Mast

The first span, (span 01, Fig. 1(a)) of 25.219m height was standing intact together with the three guy wires A_3-1, B_2-1, C_2-1, (see plate 1). The upper portion has fallen off as a result of the failure of only the guy cables B_1-4, B_1-3 and B_3-2. Observe that the mast is a determinate structure in space and a failures of a cable renders the other two at its level ineffective. Plates 2 and 3, show the top three spans 2–3, 3–4, 4–5 with clear buckled form. The plan view (Fig. 4(a)) shows that guy wires on the leeward sides are intact. What appears to be true is the possibility that the portion 2–3, 4–5 must have buckled before the snap might have taken place at level 1. This has been confirmed by subsequent analysis which showed excessive axial stresses. Observe that the fall of this portion has not damaged the first level guys. The end 1 of span 1–2 must have passed rather luckily from below the guy A_3-1, to the other side.

It is important to note that two of the three guy wires failed as a result of the failure of the grips. Only one cable Cable B_3-1, showed tensile failure near an insulator. Presumably this must be during the fall of the mast due to failure of top cable, and not a structural failure.

The horizontal sections where the snapping took place (level 1) was examined. The welded joint between the tube and the flange had failed in tension together with few bolts joining the flanges. It is pertinent to emphasize here that joints and grips are the weakest zones where the failure is initiated.

South Mast

The failure of this mast evidently shows that it has been blown over by heavy wind. Fig. 4(b) shows the plan view. All the windward cables were found to have failed. Except cable B_1-3, all others failed as a result of the failure of the grips. The portion [illegible] is separated out from portion 3–2 and blown off badly. It is clear that [illegible] free and piercing the soft ground. The twisted steel shown in plate 4 shows this fact. The remaining portion 0–1–2–3 has suffered right angle bends and levels 1 and 2, which also shows that the wind intensity was too high. Apart from local damages at guy levels the mast has not shown any buckling shapes as the North mast.

Analysis for Equivalent Static Wind

The first effort was to perform an approximate analysis taking an equivalent static wind pressure, though such an analysis is not expected to diognize the cause of the failure. It certainly forms a starting point to monitor the outputs of further dynamic and exact methods of analysis.

The tower is analysed as a beam bending in only one plane. The weakest plane is the

Photographs of North Mast

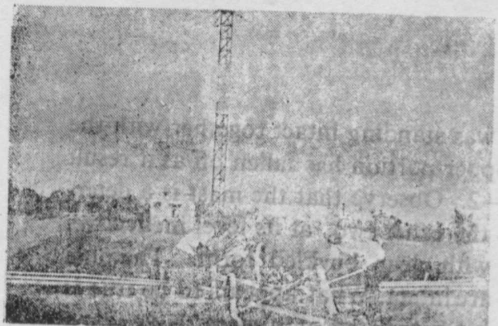

Plate 1

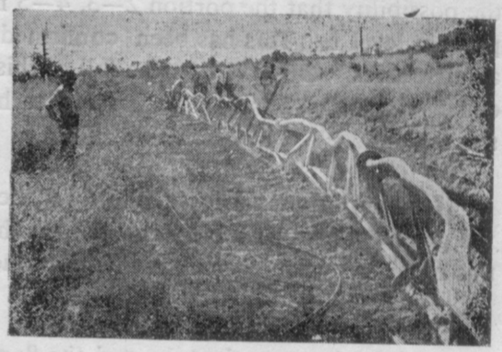

Plate 2

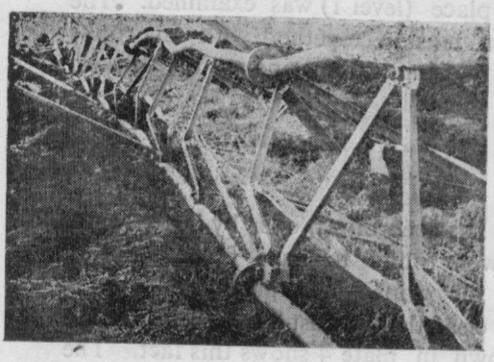

Plate 3

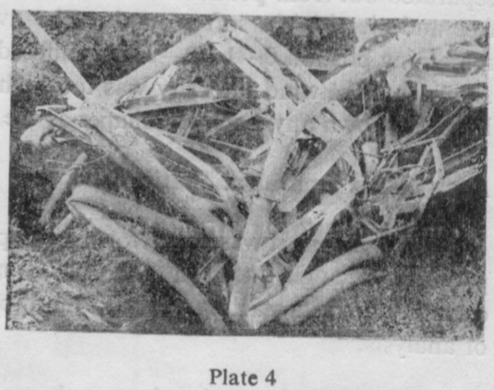

Plate 4

Photographs of North Mast

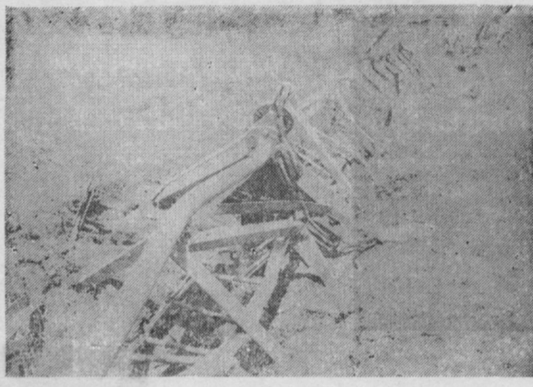

Plate 5

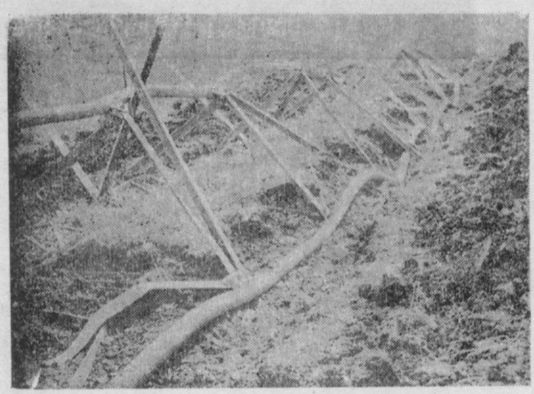

Plate 6

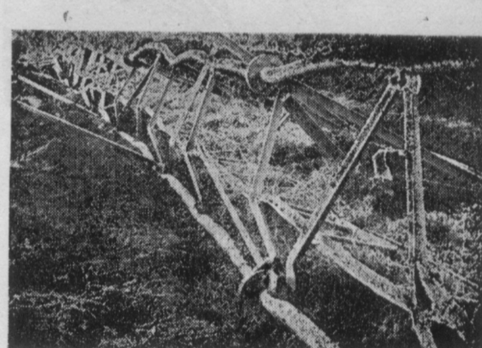

Plate 7

Photographs of South Mast

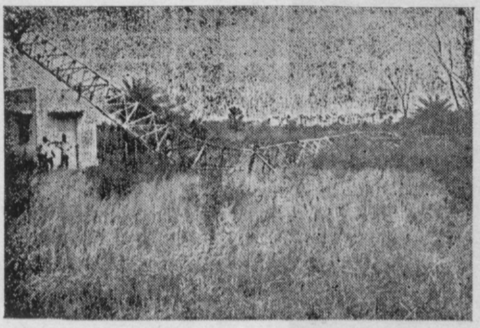

Plate 8

Plate 9

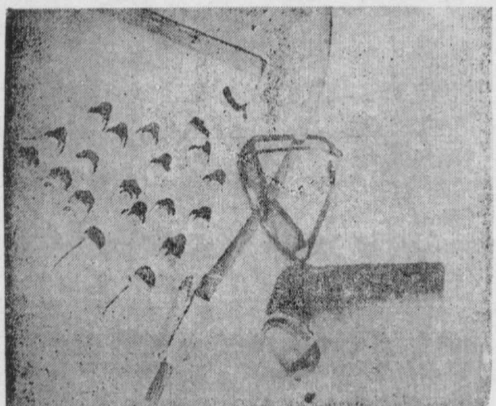

Plate 10

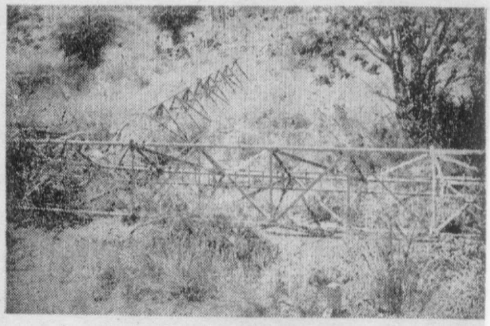

Plate 11

plane parallel to one of the sides of the triangle [Fig. 5(a)]. In this case the cables on the leeward side and those perpendicular to the direction of the wind are ineffective since the cables are not prestressed. Thus the inclined components of the cables on the windward side alone support the mast against bending in the plane of the wind.

Here the bending only in the plane of the wind is considered, the lateral bending and torsion being ignored. In general, two way bending and twisting have to be considered simultaneously.

The horizontal component R of a cable and the cable tension T are related simply as [Fig. 5(b)].

$$R = R' \sin \psi = T \cos \alpha \sin \psi \qquad (1)$$

The cable elongation e and the horizontal displacement v of the mast of the cable connection point (along y) can be related similarly as :

$$v = e/\cos \alpha \sin \psi \qquad (2)$$

This is verified from Fig. 5(b).

The flexibility of the cable accounted for the change in the sag is generally expressed as :

$$\Delta e = \frac{L}{\eta AE} \Delta T \qquad (3)$$

where η is a factor which depends on the tension in the cable at which the increments are being considered. This factor (reference 9) is,

$$\eta = \frac{1}{\left(1 + \frac{W^2 AE}{12 T^3} \cos^2 \alpha\right)} \qquad (4)$$

where W is the total weight of cable and T the tension in the cable. This factor tends to unity when the tension in the cable tends to be large. A correct analysis has therefore got to be an incremental one. In such a process the total design load is applied in small increments. At each increment the cable flexibilities are computed according to the level of the initial tension. Herein such a procedure has not been followed since only an estimate of the order of internal forces was being investigated. An average value of η corresponding to two limiting cases has been used. One is the initial unloaded stage and the other corresponding to the tension computed.

The procedure adopted is a simple, in which the mast is first analysed assuming cables to be of infinite stiffness. The corrections due to the cable stretches are then determined as secondary effects. These corrections are refined by iterations.

Wind Load

The analysis is carried out using an equivalent static wind pressure of 55 lb/ft² (268.5 kg/m²).

This is expected to include in a crude way, the dynamic effects of wind of about 120 miles per hour. ISI DOC SMBDC7 (2005) P2 (Code of practice for use of steel in microwave antena towers Part I (reference 10) suggests to determine wind load W on tower surface as

$$W = C_f . P . A_e \tag{5}$$

where, C_f = overall force coefficient

P = dynamic pressure of wind (kg/m²)

A_e = surface area on which wind impinges (m²)

C_f depends on solidity ratio, cross sectional shape of mast and sectional shape of structural members.

Solidity ratio (ϕ) is defined as $\phi = A_e/A_n$

where, A_n is the gross area of surface enclosed by boundary of the frame normal to wind.

For the mast in question, this ratio works out 0.1947, and the coefficient C_f as 2.72. Presently equation (5) gives the normal load as 213.4 kg per meter height, and the component parallel to sides therefore works out to 184.8 kg/m.

Analysis

The analysis procedure is an iterative one. First the mast is analysed as a beam on unyielding supports. The resulting cable extensions are computed and a secondary analysis is made for the corresponding movements of the masts at guy levels. In computing cable elongations an average value of η is used in employing equation (4); the value of T to be used is the average of the initial tension and the tension computed from the beam analysis. On superimposing the set of secondary moments, a new sets of cable tensions, cable flexibilities and cable elongations are computed. Presently three such iterations converged to the set of final mast moment as shown in Fig. 6. The cable stresses and other quantities of interest are as given in Table 1.

Stresses in Mast Sections

It is observed from Fig. 6 that the maximum moment (23.15 tm) in the mast occurs at the third cable point (top most but one). The corresponding thrust due to the cable tension is 5.74t. Next point of maximum moment is the first cable point. Here the moment is 18.96 tm, but the thrust is higher due to self weight as well as cable tensions. Due to cable tensions alone the thrust is 18.14 t. The maximum shear is found to occur just below cable point 3, the maximum shear force is found to be 3.04 t.

The stresses in the tube sections are next determined (reference 11, 12). The axial stresses in the tubes due to dead weights cable thrusts and bending are superimposed. The dead load causes only direct compressive stresses; the cable thrust will involve bending also. Stresses at level 3 and level 1 were investigated. It has been observed that at level 3 maximum compressive and tensile stresses in the mast tube are 1763 kg/cm² and 1237 kg/cm² respectively the permissible compressive stress (L/r=61) is only 1190 kg/cm². This analysis in a crude way

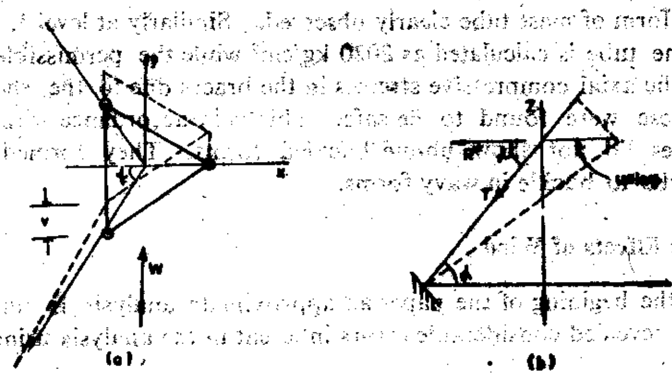

Fig. 5. Weaker plane, parallel to one side of triangle.

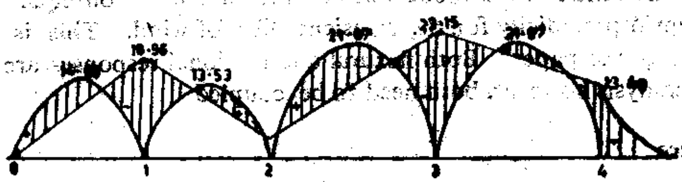

Fig. 6. Final mast moments.

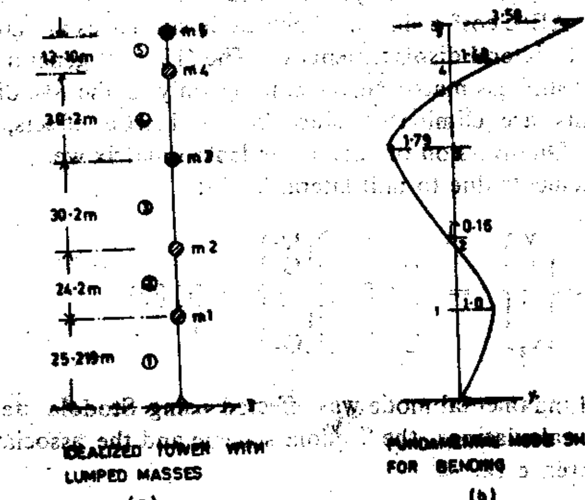

Fig. 7. Lumped mass system.

explains the buckled form of mast tube clearly observed. Similarly at level 1, the actual compressive stress in the tube is calculated as 2020 kg/cm^2 while the permissible are (L/r=63) is only 1110 kg/cm^2 The axial compressive stresses in the braces due to the shear forces were also computed. These were found to be safe. This is in accordance with the observation that most of the braces did not show undue buckled forms. They formed strong enough struts for the mast tube to buckle in wavy forms.

Analysis for Dynamic Effects of Wind

As discussed in the begining of the paper an approximate analysis is undertaken. Even such a procedure has revealed considerable errors inherent in the analysis using an equivalent static wind load.

This paper summarizes the findings of the popular mode superposition technique, using the fundamental mode only. Since the structure is essentially nonlinear as a result of nonlinear cable stiffness, beam-column effects, and above all, possible plasticity effects, such a mode superposition procedure would not be of true significance if higher modes are superimposed. Thus only the fundamental mode was considered and a consequent timewise integration using an assumed periodicity for the transient effect of wind. This is consistent with the limited objective of the paper. Both flexural and torsional responses are examined separately. In an exact analysis however, both need to be coupled.

a. Analysis for flexure

The same assumptions are made as in the static analysis. Fig. 7(a) shows the lumping of the masses. At each mass point we have two degrees of freedom the displacement v along y-axis (parallel to the side of triangle) and a rotation θ in vertical plane passing through y-axis. A complete stiffness matrix for the mast is assembled, using the flexural stiffness matrices for mast members, and axial stiffnesses for cables. In using eq. (3) the factor η has been here taken as unity or else the approximate analysis becomes intractable. Equations (1) and (2) tend to the cable stiffness as $AE/L \cos^2 \alpha \sin^2 \psi$. Such cable stiffnesses are added to the diagonal locations corresponding to lateral displacements v. The (12×12) general stiffness matrix is next condensed to a (5×5) stiffness matrix corresponding only to the five displacements v_1 to v_5. The five rotation terms are eliminated since rotary inertia effects, are, as generally assumed, considered small. On inversion of such a condensed matrix we obtain the flexibility matrix [f] giving the displacements due to unit lateral loads:

$$\{v\} = \begin{Bmatrix} v_1 \\ v_2 \\ \vdots \\ v_5 \end{Bmatrix} = \begin{bmatrix} & & \\ & f & \\ & & \end{bmatrix} \begin{Bmatrix} Q_1 \\ Q_2 \\ \vdots \\ Q_5 \end{Bmatrix} = [f]\{Q\} \tag{6}$$

The calculation of the fundamental mode was effected using Stodola iteration technique. The details of the above calculations, the Stodola scheme and the associated programming aspects are available in reference 12.

The lowest circular frequency was found as p=3.50 rad/sec. This then gives the fundamental period as 1.79 sec. The normalized mode shape is as sketched in Fig. 7(b).

The next step is to carry out the dynamic response of the structure for a given periodicity of the wind. Since no records of gusts are available a typical form was assumed as shown in the inset of Fig. 8. Since the order of magnitude of t_d of the gust was unknown, the analysis was carried out for over a practical range of interest to the problem at hand. The peaks have been assumed to be 20 percent over the mean. A damping ratio $\zeta = 0.02$ has been used, a value close enough for a steel structure.

Herein the Newmark type timewise integration program has been used. The essentials of the procedure are given in Appendix B. The response calculations were made for ratio of td/T ranging from 0.25 to 2.0, where T is the fundamental period of tower determined. Each of these were carried out for two cases, with and without damping. Fig. 8 shows the response curves for ratios of td/T=0.8, 0.1 and 1.2. Fig. 9 shows the summary, which is of direct interest, namely, the dynamic load factor (DLF) for different gust frequencies. This is the response spectrum for structure. In fact availability of such curves for realistic wind data will greatly help the designers. This approach is similar to the one recommended in earthquake resistant design.

Results

Table 2 and 3 show the comparison of displacements and cable stresses with these obtained by equivalent static wind load. It is seen that some of the magnitudes as well as signs are widely different. Note that the mode shapes in each case are different.

The dynamic analysis performed here is quite handy and can be always used in checking the preliminary design which can begin with the procedure of using an equivalent static wind load. Such an approximate analysis can however be superseeded by more elaborate dynamic analysis. But what is least essential is an approximate dynamic analysis.

b. Analysis for torsional vibrations

An analysis was carried out to estimate the natural period in torsion vis-a-vis that in flexure.

It has been shown in reference (12) that the torsional rigidity of the mast per unit length is $AE/4\sqrt{(2)}$, where AE is the extensional rigidity of one inclined brace [Fig. 2(a)]. This is derived by noting the compatibility of brace elongations with the displacements of the tube due to torsion. The mass moment of inertia (Σmr^2) has been lumped as shown in Fig. 10(a) (Appendix A). The mast, being hinged at its base will have the antisymmetric mode as the fundamental mode. In such a mode the torsional rotation will be zero at the centre of the mast. Thus only half the portion with three masses is analyzed. A procedure similar to the one described in the previous case (Stodola iteration using flexibility matrix) has given the fundamental frequency 22.0 rad/sec (compared) to 3.5 in flexure) and a time period of 0.286 (compared to 1.79 sec). The mode shape is as shown in Fig. 10(b). As expected, the mast is quite stiff torsionaly. The participation of the torsional mode is thus not significant.

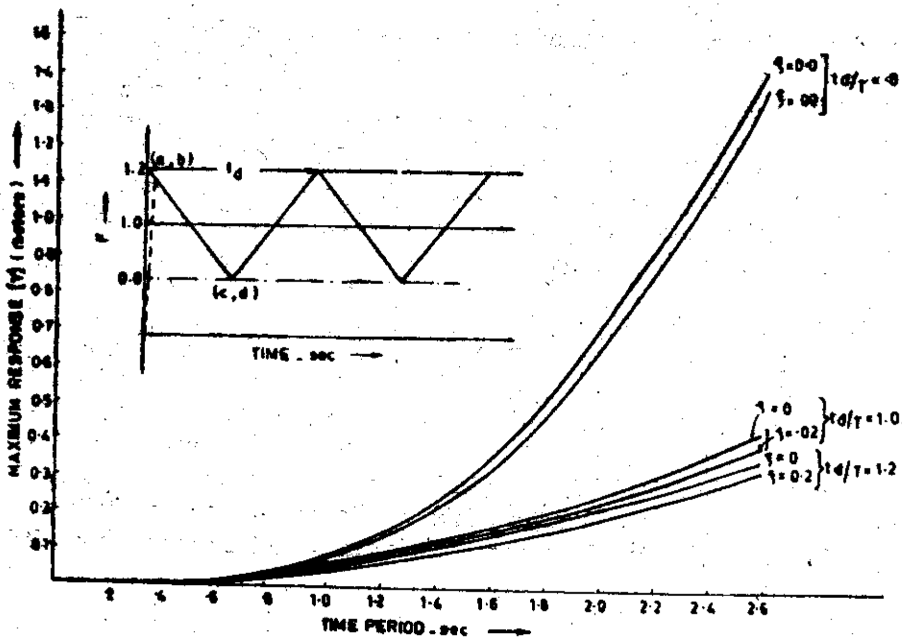

Fig. 8. Displacement response curves.

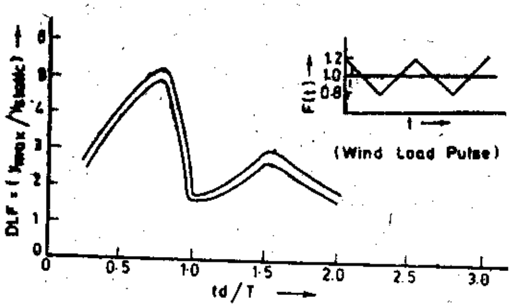

Fig. 9. Dynamic load factor vs td/T.

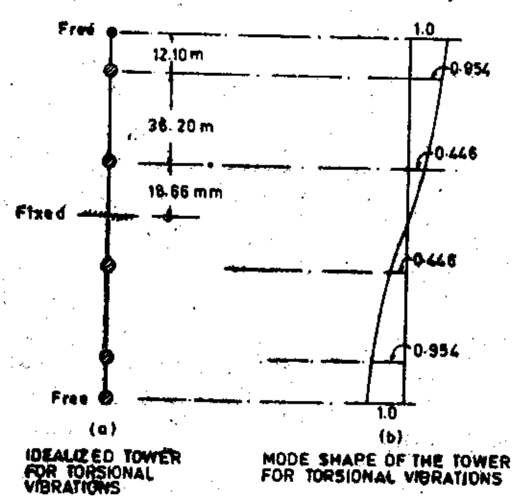

Fig. 10. Mathematical model for torsional vibration.

However we cannot conclude about the significance until an exact analysis is performed taking flexural-torsional coupling into account.

Conclusions

Occurences of structural failures such as the one reported in this paper should be regarded as special occasions for people in engineering profession to pause and think not to much about the postmortem of the failures, but of the future steps to be taken regarding improvement of design codes and where necessary organize planned effort to collect data such as, wind, earthquake and similar loads. This paper has demonstrated a relatively simple procedure of assesing dynamic effects of wind. The response curves such as Fig. 9 will be a set of useful curves for design. For a given pattern of observed gusts, one can examine the natural period of the structure and can either account for the dynamic amplifications or change the structure properties to yield safe natural periods. The approximate procedure will involve only simple computer programming considerations easily implementable on small computers. At any cost the code of practice can certainly insist on an approximate dynamic analysis as a prerequisite for approving the design.

The considerations of stability effects is equally important. The approximate analysis has identified high compressive stresses. A more rational analysis may reveal redistribution of these. As the loads are increased plasticity effects would also control the behaviour. Exact analysis procedures should give a complete load deflection picture till collapse, taking into account dynamic effects on a structure with geometric and material nonlinear characteristics. And this is, by no means easy. In their absence approximate methods continue to serve as good friends.

Acknowledgement

The authors gratefully thank Chief Engineer and Regional Engineer, Andhra Pradesh of All India Radio, for their interest shown in the technical assessment of the failure, and the permission to publish the findings.

Table 1 Final Values of Cables Stresses and Displacements

Cable	T Cable tension (t)	Area of cable (cm^2)	Cable Stresses (kg/cm^2)	e Elongation (m)	v Displacement (m)	Axial thrust in mast due to T V (t)
1	6.66	1.226	5432	0.228	0.30	5.81
2	8.41	1.538	5468	0.369	0.62	5.71
3	8.93	1.538	5806	0.710	1.09	6.69
4	9.041	1.538	5878	0.849	1.55	5.74

Table 2 Displacements

Support No.	Displacements by approximate analysis for equivalent static wind loads (m)	Displacements by dynamic analysis (m)	Ratio
1	0.30	+1.87	6.23
2	0.62	−0.29	−0.47
3	1.09	−3.33	−3.06
4	1.55	+2.66	1.72

Table 3 Stresses in Cables

Cable No.	Cable stresses by approximate analysis for equivalent static wind loads (kg/cm^2)	Cable stresses by dynamic analysis (kg/cm^2)	Ratio
1	5432	5661	1.04
2	5468	1021	0.19
3	5806	11512	1.98
4	5878	7597	1.29

APPENDIX A

GEOMETRICAL DATA

Cable Data

Cable No.	Length m	α	$\cos \alpha$	$\cos \alpha \sin \psi$	Initial tension to (kg)	Dia-meter (mm)	Area cm^2	Self Weight (t)
1	51.56	29°−13′	0.489	0.7558	624	12.5	1.226	0.050
2	66.25	47°−13′	0.740	0.5882	612	24.0	1.538	0.065
3	120.17	41°−30′	0.664	0.6485	1160	14.0	1.538	0.118
4	141.84	50°−37′	0.773	0.5495	1160	14.0	1.538	0.139

Mast Data

Span	Tube dia (mm)	Tube Thick-ness (mm)	Area cm^2	M.I. of mast about X axis cm^4	r	L/r (L=150 cm)
1	76	9.0	18.94	213442	2.39	63
2	76	6.0	15.17	170980	2.45	61
3	76	6.0	15.17	170980	2.45	61
4	76	5.5	12.18	137270	2.5	60
5	76	9.0	18.94	213442	2.39	62

Weighted moment of inertia $(I_w)=\dfrac{I_1R_1+I_2R_2+I_3R_3}{R_1+R_2+R_3}$

$$I_w=175627 \text{ cm}^4$$

Value of E for Steel $=2.98\times10^6$ kg/cm^2

$$(AE)=1510.77 \text{ tons;} \quad EI_w=36530.5 \text{ tm}^2$$

Mass Moment of Inertia (Fig. 10)

Mass No.	Lumped mass (m) t-sec^2/m	Mass moment of inertia (I) (t-m-sec^2)
1	0.075	0.1684
2	0.065	0.1459
3	0.018	0.0404

APPENDIX B

NEWMARK'S INTEGRATION TECHNIQUE

In general to solve any differential, equation of the form,

$$M\ddot{Y}+C\dot{Y}+KY=F(t) \tag{1}$$

Newmark has suggested the following procedure

Find out at time t

$$\Delta y(t)=\frac{K^*}{F^*(t)} \tag{2}$$

where,

$$K^*=K+\frac{M}{\beta.\Delta t^2}+\frac{\gamma.C}{\beta.\Delta t} \tag{3}$$

$$F^*(t)=F(t+\Delta t)-M\left(\ddot{Y}(t)-\frac{\dot{Y}(t)}{\beta.\Delta t}-\frac{Y(t)}{2.\beta}\right)$$

$$+C\left(\frac{\gamma}{\beta}\dot{Y}(t)+\frac{\gamma}{2\beta}\Delta t\ddot{Y}(t)-\Delta t\ddot{Y}(t)-\dot{Y}(t)\right)-KY(t) \tag{4}$$

Substitute value of $\Delta Y(t)$ in the following equation to get displacement at time $(t+\Delta t)$ where Δt is the time interval.

$$\ddot{Y}(t+\Delta t)=\frac{\Delta Y(t)}{\beta.\Delta t^2}-\frac{\dot{Y}(t)}{\beta.\Delta t}-\frac{\ddot{Y}(t)}{2.\beta}+\ddot{Y}(t) \tag{5}$$

$$\dot{Y}(t+\Delta t)=\dot{Y}(t)+\gamma.\Delta t\ddot{Y}(t+\Delta t)+(1-\gamma)\Delta t\ddot{Y} \tag{6}$$

$$Y(t+\Delta t)=Y(t)+\Delta tY(t)+\beta.\Delta t^2-Y(t+\Delta t)+(0.5-\beta)\Delta t^2\ddot{Y}(t) \tag{7}$$

After getting ΔY, all the equations from 5 to 7 are solved and thus displacement at next interval is found. This procedure is repeated again and again, thus values of Y are obtained.

In the above equations β and γ are two parameters which affect the period elongation and amplitude decay respectively. If $\gamma=0.5$, amplitude decay will be zero for displacement function varying sinusoidally. The value of β affects the rate of convergence within each step, the stability of analysis and the accuracy. Here the value of β is taken to be 0.25 which corresponds to acceleration which remains constant at an average value of $(\ddot{y}_t+\ddot{y}_{t+\Delta t})/2$. The net effect of β is to change the form of the variation of acceleration during time interval Δt. The value of γ is taken to be 0.5. This combination of parameter has the most desirable accuracy characteristics.

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