ANALOGIES IN EARTHQUAKE AND WIND RESPONSE OF STRUCTURES

by

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SYNOPSIS

The paper approaches gust effects of wind on tall structures in a deterministic manner in the analogy of earthquake analysis. Equivalent single degree structures are conceived common to earthquake and wind loads for numerical integration, as well as for modal analysis. A specific force response spectrum for wind effects is presented. Results of analytical case study on structures by numerical integration are presented.

INTRODUCTION

Structures are subject by earthquakes to inertia forces which are a function of their masses while wind imposes external forces which are a function of the structural forms. Dynamic wind effects on structures fall under two major categories, viz, self excited vibrations and forced vibrations. Self excited vibrations consist of galloping, torsional flutter and coupled flutter and buildings are not known to be sensitive to these modes of vibrations. The more common and important type of forced vibration on buildings is the "in-line gust excitation" in the direction of the wind. A second type of forced vibration, the Karman vortex excitation, in the direction perpendicular to the wind is not significant in the normal designs of tall structures. Statistical methods [2,4,5] are available for the probabilistic response analysis of the in-line gust effects on structures. The methods assume the mode shapes to be linear and the maximum response as a function of statistical mean response. With these methods, the response effects caused by different patterns of forcing functions remain the same as long as the overall statistical properties of the forcing functions are identical. For a rigorous and physically exact analysis, deterministic methods using time dependent dynamic loading patterns of wind functions and exact dynamic structural properties including mode shapes are to be resorted to. Some work on deterministic analysis using pseudo wind records has been reported [1]. The present work makes use of the analogy of deterministic earthquake response analysis and the actual anemographic records (vide p.31 of Ref.4 for a typical pattern) as forcing functions.

RETURN PERIODS, WIND PROFILE AND FORCES

Extreme probability graphs (vide p.39 of Ref.4) analysed from meteorological wind data are available for U.K. and U.S.A. The graphs aid in the selection of the maximum wind speed that will not be exceeded in a structure's life time within a specified limit of calculated risk. The extreme probability graphs of annual extreme wind speeds analysed in this work from the data of a net work of meteorological stations in South India are given in Fig.1. Wind velocity increases with the height above ground level and the profile follows a power-law depending upon the type of ground surface. Power-law exponent values and gradient heights for three broad groups of surfaces are given in Ref.2. In the building

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represented in Fig.2, the wind velocity V_x at a variable height Z_x is given by: $(V_x / V_1) = (Z_x / Z_1)^a$, where the suffix 1 represents the values at the height at which the anemometer is installed and a, the power law exponent, taking values of 0.16, 0.28 and 0.40 for almost open terrain, residential suburbs or towns and centre of large cities respectively. The wind pressure P_x at height Z_x is $\frac{1}{2} m V_x^2$ where m is the mass density of air. Taking into account C_D, the coefficient of drag in fluctuating wind, the force on the building surface between heights H_u and H_1 by integration is: $C_D\, U\, P_1$ where $U = B(H_u^{1+2a} - H_1^{1+2a}) / (Z_1^{2a}(1 + 2a))$ ...(1). Thus, the wind force acting at each mass level can be determined.

For a SDF system, acted upon by a time dependent forcing function $F_{(Q)}$ with Q as time variable, the response force F_t on the structure at time t is given by: $F_t = C_D\, U\, S_p$ ...(2) where

$$S_p = (p^2 / p_d) \int_0^t P_{1(Q)} e^{-zp(t-Q)} \sin p_d (t-Q)\, dQ \quad ...(3).$$

p and p_d are undamped and damped natural frequencies of the structure and z is the coefficient of critical damping, zeta. If $P_{1(Q)}$ in equation (3) is replaced by a constant unit force, the resultant value of S_p is defined as SRF, the specific response force. Fig.3 represents graphically the specific response force integral. The algebraic sum of the areas under the Sin curve multiplied by p^2 / p_d will give the value of SRF. The curve consists of the shaded current half period influence and the past influence of all the previous half periods whose net area will be negative. Neglecting ordinates less than .003 of the curve in Fig.3, the significant past influence time for zeta values .005, .01, .02, .05, .10, .20, .30 and .40 are respectively 755, 365, 181, 97, 57, 53, 17 and 9 quarter periods of the structure. Calculations reveal that the SRF value is not dependent on the period but is influenced by the zeta value. The current half period SRF values for the above zeta values are respectively 1.984, 1.969, 1.939, 1.854, 1.730, 1.525, 1.362 and 1.240 respectively and for zero damping, the value is 2.0. The respective past influence SRF are -0.984, -0.969, -0.939, -0.854, -0.730, -0.525, -0.362, -0.240 and -1.0. The SRF spectrum over the range of zeta from 0.to 0.4 has been presented in Fig.4. The spectrum serves as an influence line. The error involved in adopting a less duration of past influence than the significant duration has also been plotted in the spectrum. As an illustration, the anemogram for Madras Forecasting office for 20 - 11 - 1960 is considered for a structure whose z = .02 & fundamental period = 4 secs and consequently the significant past influence duration = 181 secs. The maximum wind pressure is 104 kg/m^2 and it occurred at 15.06 Hrs. Positioning the peak pressure (P_c) on current half period and the portion immediately preceding the peak over the past influence (vide Fig.3), the average value of the immediately preceding wind pattern for 181 secs (P_p) = 8 kg/m^2. From SRF spectrum, current response = 1.939 and past influence = -0.939. Hence S_p = 104 (1.939) - 8(0.939) = 197.03 kg/m^2. The dynamic magnification is 197.03/104 = 1.89. The maximum wind pressures (kg/m^2) from anemograms of South Indian stations are Madras F.O., 20-11-1960, 104 (P_c), 8(P_p), 25.0 m(Z_1), 1952 (records date back to); Madras Harbour, 25-6-1974, 127, 1.5, 33.0, 1962; Tiruchirapalli, 28-5-1967, 106, 9.0, 17.2, 1965; Begumpet, 15-5-1971, 120, 2, 18.5, 1954; Bangalore, 2-2-1959, 53, 4, 19.2, 1953; Mangalore, 9-12-1965, 56, 2, 16.0, 1964; Visagapatanam, 3-1-1966, 89, 11, 9.1, 1943. Substitution of the value of S_p in Eqn.(2) will yield the maximum response force. The design force may be determined considering the return period factor and the permissible load factor.

MULTIDEGREE OF FREEDOM SYSTEMS

Multistoreyed building masses are lumped at the floor and roof levels. The corresponding input forces at the mass levels are determined from Eqn. 1. Equivalent single degree structures (ESDS) are conceived common to Earthquake and Wind analysis. The expressions for effective mass (M_e), effective stiffness (K_e) and radius of participation (R_e) remain the same as in Eqns. 2,3 & 7 of Ref.(3). The relevant notations used may also be seen in Ref.(3). The effective force input $f_e = \underline{f}^T \underline{A} / R_e$ where $\underline{f}$ is the input force vector. ...(4).

Fig.6 indicates the ingredients of the deterministic approach for wind response analysis of structures. Aero interaction and aerodynamic magnification are taken care of by C_D in Eqn.1. Dynamic magnification is taken care of by the rigorous numerical integration performed on the ESDS of all the modes or by the dynamic magnification factor (DMF) in the response spectrum analysis. With the SRF spectrum technique, $K_e\, x_e = (DMF)\, C_D\, f_e$ ...(5). The displacement x_i at floor level i is obtained from $x_i / x_e = A_i / R_e$ ...(6). The spectrum response values of all modes are then combined by the method of square root of sum of squares. Fig.5 illustrates a typical wind forcing function adopted in numerical integration technique. The overall procedure remains the same as for earthquake [3], except for minor modifications outlined here. It is idealised that the forcing function shapes are identical at all mass levels but the values vary as per power law for wind velocities. At mass levels, the force input takes normalised form corresponding to unit pressure at reference point (anemometer installation). The wind pressure $P_{1(Q)}$ at the anemometer installation is adopted as the forcing function. At every time step, the input normalised effective force is multiplied with the pressure at the reference point and coefficient of drag. A hypothetical ground acceleration is defined as the negative value of forcing function divided by the effective mass of ESDS. With these amendments the computations are done adopting the earthquake response analysis programme using equivalent single degree structures [3]. For a case study, the TV tower, Madras (176 m) and the Central Govt. office building, Calcutta (22 storeyed) were analysed for the Madras harbour wind of 25-6-1974 using C_D values of 3.35 and 1.25 respectively. The Incometax building, Madras (9 storeys) was analysed for Madras F.O. wind of 20-11-1960 with C_D value of 0.9. Alternate calculations as per I.S. Code 875-1974 were also done. The rigorous numerical integration was also compared with response values as per SRF spectrum.

Fig.7 depicts the effective masses and effective forces of the two buildings. While about 80% of the effective mass is concentrated on the first mode, over 90% of the effective force is concentrated on the first mode. The sum of the effective masses is equal to the total storey masses and the sum of the effective forces is equal to the total input forces. The effective mass is a measure of the earthquake energy absorbed by each mode and effective force, the wind energy absorbed. They therefore indicate the significant modes for earthquake response and wind response respectively.

Constant damping coefficients was adopted in numerical integration for a direct comparison with the SRF spectrum method. Fig.8 indicates the response of the three structures by numerical integration. Most of the response displacement is concentrated on the fundamental mode. As regards response forces, the adoption of the fundamental mode value alone will not result in any significant error in the total responses for use in structural designs unlike earthquake designs where the first few modes of response forces are significant [3] . Fig. 9 would indicate that the displacements and response forces as per dynamic analysis are far more than I.S. code provisions (875-1974). For I.T. office building, wind response as per IS code is nil. The case study indicated that there is a good tally of response values between the SRF spectrum method and numerical integration using stepped functions of average wind forces. These results are also in fair agreement with numerical integration using fluctuating short period waves idealised from the anemogram patterns. The agreement bwetween the two methods establishes that the SRF spectrum method is an effective tool for dynamic response analysis against wind forces. For wind, higher mode contributions are insignificant and hence the response spectrum technique is more potential than for earthquakes.

CONCLUSIONS

Dynamic properties of a structure remain unaltered, irrespective of the type of external forcing function. The equivalent single degree structure conceived in the paper preserves the physical scales and provides meaningful values. A classical mode superposition approach is possible for wind analysis by numerical integration and also by response spectrum as in the case of earthquake analysis. Effective masses for earthquake and effective forces for wind indicate the significant modes. For wind, first mode mostly contributes the response forces. The I.S. code provisions for wind analysis would require a review. It is recommended that the code be revised in a rational manner with provisions for coefficient of drag, velocity variations with height, return period factor and computations of dynamic effects.

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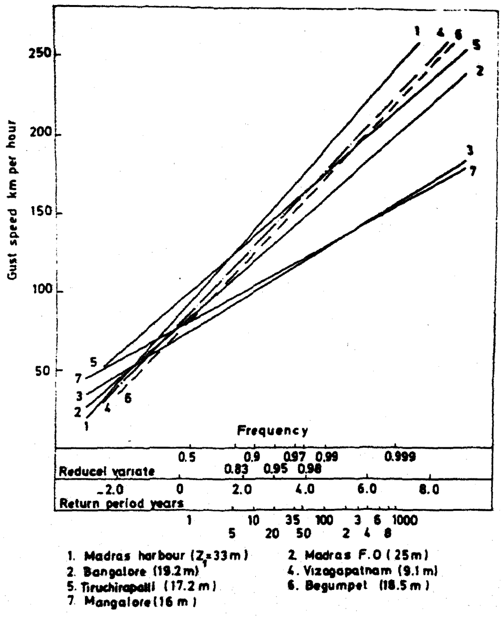


FIG. 1 Extreme probability chart of annual maximum wind speeds at south Indian stations 1946 - 75

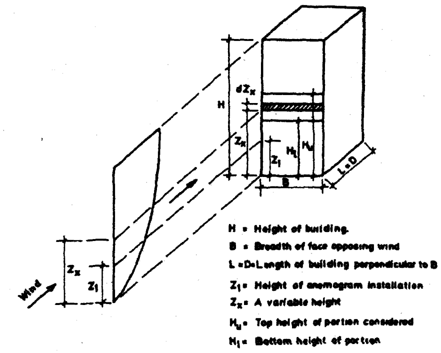


FIG. 2 Wind pressure variation on building face

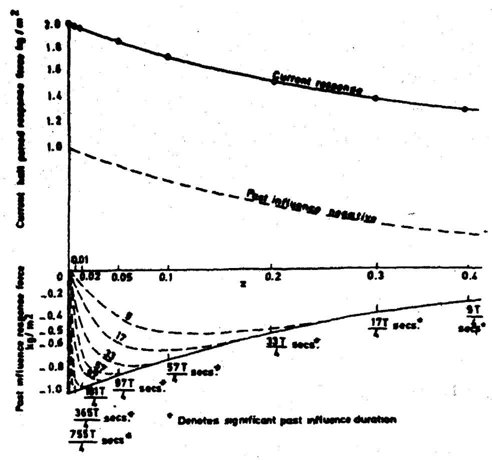


FIG. 4 . Specific response force spectrum

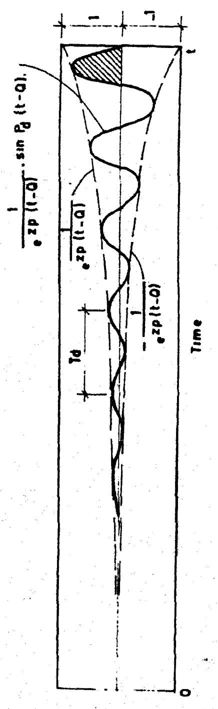


FIG. 3 . Specific response integral represented graphically

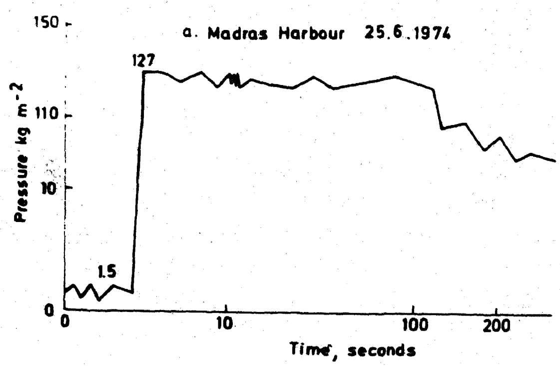


FIG. 5. Wind forcing functions

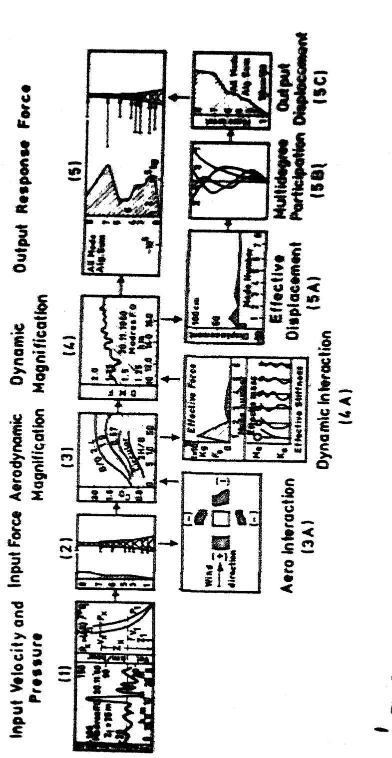
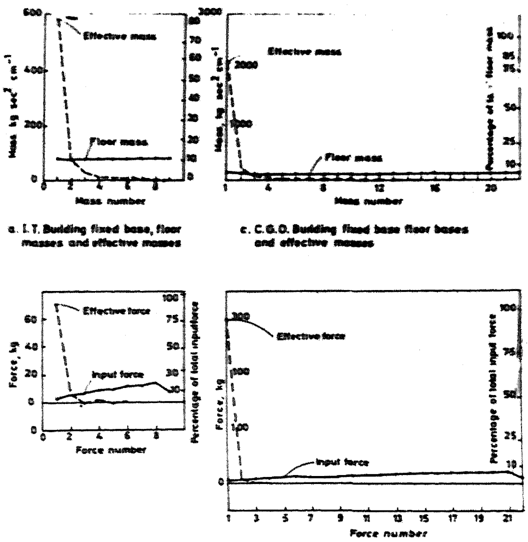


Fig. 6 Ingredients of Deterministic Approach to Buffeting Wind Loading on Structures



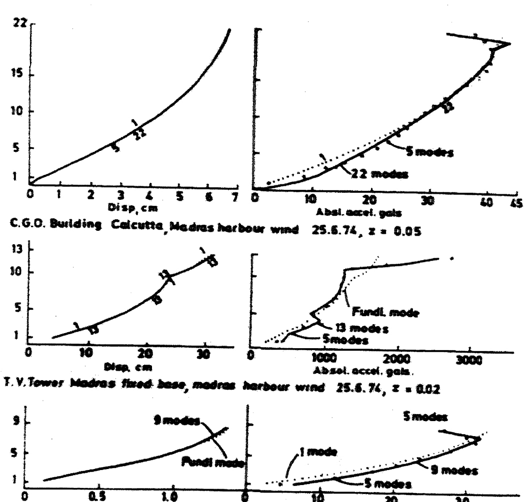
a. I.T. Building fixed base, floor masses and effective masses

c. C.G.O. Building fixed base floor bases and effective masses

b. I.T. Building fixed base input and effective forces of madras harbour 20.11.60 Pref = 104

d. C.G.O. Building fixed base input and effective forces of Madras harbour wind 25.6.74 Pref = 108.7

FIG. 7 Effective masses and effective forces

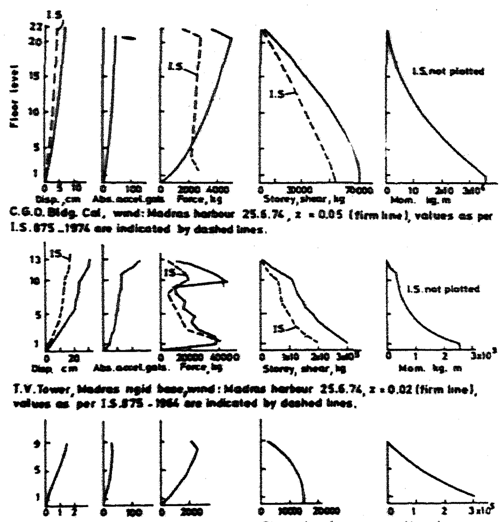


C.G.O. Building Calcutta, Madras harbour wind 25.6.74, z = 0.05

T.V. Tower Madras fixed base, madras harbour wind 25.6.74, z = 0.02

I.T.O. Madras fixed base, madras F.O. wind 20.11.60, z = 0.05

FIG 8 . Wind response : Mode contributions



C.G.O. Bldg. Cal, wind: Madras harbour 25.6.74, z = 0.05 (firm line), values as per I.S. 875 - 1974 are indicated by dashed lines.

T.V. Tower, Madras rigid base, wind: Madras harbour 25.6.74, z = 0.02 (firm line), values as per I.S. 875 - 1964 are indicated by dashed lines.

I.T.O. Building, Madras, wind,: Madras F.O. 20.11.60, z = 0.05

FIG. 9 Wind response of structures

DISCUSSION

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The unsteady component of wind force in wind direction is caused by turbulence in wind and by a cyclone due to pressure gradient in it. It is customary to include these effects in the computation of the total wind force acting on the structure while the lateral vibrations caused by vortex shedding are important in dynamic analysis. The reported failures of towers and bridges in the past support this approach rather than the 'in-line excitation' considered by the authors as more important than the lateral vibrations. The authors may clarify this point.

Authors' Closure

Not received.