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Application of Modern Design Techniques to Practical Wind Problems

Application des techniques de projection modernes aux problèmes pratiques posés par le vent

Anwendung der modernen Entwurfstechniken auf praktische Windprobleme

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Introduction

It is clear that the days when wind was considered as a static phenomenon have gone. Gone also in many cases is the simplicity of the quasi-static load case which has normally been assumed as applicable to wind loadings.

The equivalent static load concept is adhered to very strongly. It is clearly valid to express answers as an equivalent static load but it can, if we are not careful, tend to cover up gross inadequacies in the method of calculation and in the assumptions that have been made.

This contribution to the prepared discussion describes attempts to apply the latest concepts and techniques of analysis to dynamic wind loading conditions.

Two major dynamic effects will be considered. There is the problem of aerodynamic instability caused by the formation of regular patterns of vortices in the lee of certain shaped structures requiring the techniques of dynamic analysis of complex structures. There is the problem of gusting as it relates to the more flexible structures involving in addition the techniques of random vibration analysis.

Gusting

Buffeting in a gusty wind occurs largely at random. The random velocity fluctuations are, however, contained within an overall spectrum which defines the amount of wind energy available (on an average) at various frequencies (Prelim. Publication 8th Congress IABSE).

All structures have modes in which they naturally vibrate, the sway modes of vibration being particularly important in the context of wind gusting as they can interact with the wind and accentuate the dynamic effect.

The work done by Davenport (Davenport 1961) was aimed at producing simplifications of the basic techniques of random vibration allowing the engineer to take gusting into consideration. The following is the expression for the effective force which is applied to a structure taking into account its dynamic response.

The effective force = hourly wind pressure x gustiness of wind x structural response x a factor

This last factor is a statistical measure which defines the peak response in which we are interested; e.g. Davenport suggests taking the average of the peak responses which occur within periods of an hour. This factor varies little with the characteristics of the structure. The mean hourly wind pressure and the gustiness of the wind are clearly also independent of the properties of the structures concerned.

The simplest case to consider is that of the lightly damped structures where the response is largely governed by movements which take place at the natural frequency of the structure (fortunately many practical structures respond in this way). In this case Davenport shows that the response is proportional to $\sqrt{\frac{n\,Sn}{\delta}}$ where Sn is the spectral density of the gusting at the natural frequency n of the structure and δ the logarithmic decrement.

It happens that the reduced spectral density in the frequency range from 0.1 Hz upwards is closely given by an algebraic polynomial. Davenport suggests $\frac{x^2}{(1+x^2)^{4/3}}$ and Harris (Harris 1968) suggests $\frac{x}{(2+x^2)^{5/6}}$ where x in each case is given by $1200\,\frac{n}{V}$ where V is the wind velocity in metres/sec. Where x is significantly greater than 1 as it is when n is greater than 0.1, both these polynomials reduce to $x^{-2/3}$.

Hence the effective force is proportional to:

$$n^{-1/3} \quad \delta^{-1/2}.$$

Clearly, therefore, the lower frequency structures are more susceptible to wind gusting as the effective force is greater.

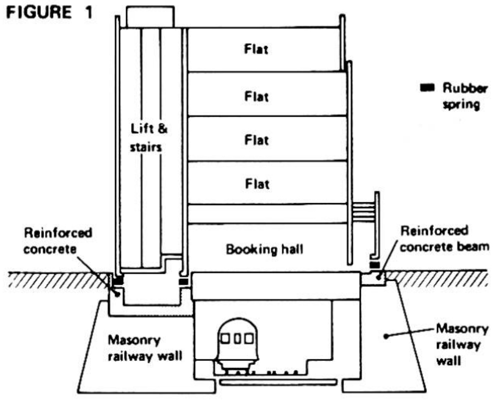
A further effect exists in that the lower frequency gusts tend to be more highly correlated over larger areas than the higher frequency gusts. Conversely, the higher the gust frequency the smaller the effective area over which the gust pressure is applied. There comes a point at which the gusts are so small in relation to the structural size that they do not have a significant effect. Davenport suggests that once the gust frequency exceeds the ratio of the maximum wind velocity to a typical dimension of the building, then the effect of gusting can be ignored.

We will now consider a few examples of buildings which have been designed and constructed where gusting was of sufficient importance to influence their design.

A Building on Springs

A block of flats (Albany Court) was erected in 1966 over the underground railway in London (Figure 1). The building was supported on a number of laminated rubber springs to isolate it from the vibrations generated by the railway. This is probably the first complete building to be isolated in this way from low frequency ground-borne vibrations (Waller 1966).

FIGURE 1



Clearly the introduction of springs for this purpose significantly alters the natural frequencies of the building. In this case the vertical frequency of the system was designed to be 7 Hz. It is a characteristic of the laminated rubber spring that its horizontal stiffness is two orders less than its vertical stiffness so that in the first instance the designed horizontal frequency of the spring system was about 0.5 Hz. This frequency was sufficiently low for there to be a significant possibility that the building would respond to gusting especially as this frequency is in the range at which eddies would be shed from nearby buildings. It would clearly be unsatisfactory if in eliminating the ground-borne vibrations the building were made significantly sensitive to the wind.

At the low frequencies involved the human sensitivity to vibration can be represented as proportional to acceleration. No rigorous estimate could be made of the likely magnitude of the acceleration induced by wind. It was judged however that there might be a problem at 0.5 Hz as an effective dynamic pressure of 1 lb./ft.^{2} was equivalent to an acceleration of 0.001g for this particular building, a level at which a significant number of people can perceive low frequency vibration.

Now acceleration is proportional to (amplitude)x(frequency)^2 and amplitude is proportional to (effective force)/stiffness. Stiffness in this case is proportional to (mass)x(frequency)^2. Thus acceleration is proportional to:

$$n^{-1/3} \quad \delta^{-1/2} \quad m^{-1}$$

if m is the effective mass of the building.

To reduce this acceleration there are therefore three possibilities: we can increase mass, damping, or natural frequency. In this particular case it was decided that the simplest and cheapest course was to increase the natural frequency. It was found possible to increase the horizontal natural frequency to 2.5 Hz without significantly reducing the attenuation of the ground-borne vibrations.

The building in this case could be considered substantially as a rigid body on a number of springs and the analysis was fairly straightforward. There was little coupling between the various modes although clearly in the vertical planes the horizontal natural frequencies and the frequencies in sway or rock (6 Hz in this case) are coupled together to some extent. Indeed it is this coupling which limits the extent to which the so-called horizontal natural frequency of the spring system can be raised.

Raising the lowest natural frequency to 2.5 Hz eliminated the possibility of interaction with eddies from nearby buildings and kept the effective 'natural' wind energy likely to interact to a minimum.

The effectiveness of the measure can only be judged by experience. Two years have elapsed without significant vibrations being reported.

In the case of the taller and more flexible buildings the flexibility of the structure itself must be taken into account in the analysis. It will often arise that the horizontal frequencies cannot be raised enough to avoid significant gust action. With the taller structures which have natural frequencies in sway of the order of 0.5 Hz it will be impossible to avoid the problem of gust action by increasing the natural frequencies. Here it will be useful to consider the addition of damping into the foundation.

Water Tower

The type of water tower that will be described was the subject of an extensive development programme. The programme was aimed at producing a new form of water tower which would combine the economics of the cheaper form of towers with the appearance of the more expensive. The form of water container finally chosen can be described as an inverted parachute with the fabric made of nylon reinforced rubber and the cables of high tensile steel. These cables are fixed to a stiff annulus which is mounted at the top of the tower leg (Figure 2).

It is well known that the natural frequency of water contained in a tank is low and it was possible that the tower might respond to gusting and to vortex excitation. One twentieth scale wind tunnel tests were commissioned at the National Physical Laboratory (Smith 1964) and indicated that vortex excited instability was unlikely. Unfortunately it is not yet possible to simulate gusting conditions in a wind tunnel and recourse to calculation was necessary in this respect.

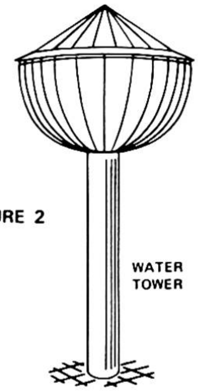


FIGURE 2

The natural frequencies for the tower were difficult to calculate. The mode of particular importance can perhaps be best described as the 'sloshing' mode. Formulae do exist for the 'sloshing' of water within rigid containers (Housner 1963), but the nature of the present container was such that no previous solution could be found that was applicable and it was decided that scale models would be a better method of establishing the natural frequencies than a theoretical exercise. The other mode of vibration of relevance is the vibration of the stalk itself with the water playing little part. It is easy to show that this frequency is much higher than the 'sloshing' frequency of the water in the tank. From the scale model used in the wind tunnel tests the natural frequencies were measured with various amounts of water in the tank and the lowest natural frequency occurred with the full tank. Further it was found that the system was essentially linear.

% Capacity	Natural frequency Hz
5	3.78
40	0.82
100	0.81

The ability to scale models for this purpose is of considerable advantage. Analysis demonstrated that the system could be considered, for scaling purposes, as a compound pendulum for which natural frequency is proportional to (linear scale)$^{-\frac{1}{2}}$. Pressure is proportional to (linear scale) and force is proportional to (linear scale)^3 when velocity is proportional to (linear scale)$^{\frac{1}{2}}$. The model was made to represent as closely as possible the full scale situation; there remained many uncertainties; however the logarithmic decrement of 0.025 measured in still air conditions was taken to represent a full scale tank. The effect of wind was to increase this damping slightly. It was also considered that the likely effect of foundation damping, joints in the structure, etc. would be to increase it again and that this figure would be on the safe side.

One other significant factor had to be established in relation to the behaviour of the tank before its full scale behaviour could be predicted. Not all the wind force is modified by the dynamic behaviour of the tank. A proportion of it is effectively applied directly to the top of the tank and stalk which is more rigid. The remaining part of the wind force can be considered as acting upon the flexible portion of the tank and therefore modified by the tank's dynamic characteristics. The distribution of the wind force was measured by displacing the tank in a steady wind stream until it was completely stationary. When the displacing wind force was suddenly removed the tank and water were set into a state

TABLE

Steady Wind Overturning Moment

Mean hourly wind speed $= 27 \text{ m s}^{-1}$

Equivalent model speed $= 27 \sqrt{\frac{1}{20}}$

$= 6 \text{ m s}^{-1}$

Model overturning moment = 0.7 kg m from test (Smith 1964)

Full scale overturning moment $= 0.7 \times 20^4$ kg m

Full scale mean hourly overturning moment = 110,000 kg m

Additional Moment Due to Gusting

Additional moment = (mean hourly moment)x(gustiness)x(response)x(factor)
(Davenport 1961)

(i) Mean hourly moment as above.

(ii) Gustiness.

Gustiness $= 2.45\sqrt{K} \, (\frac{z}{10})^{-\alpha}$ = 0.145 for an open site

When K = surface drag = 0.005
α = power law exponent = 0.16
z = height of centre of pressure = 30m

(iii) Response.

The additional moment due to gusting can be divided into two components, one third being modified by the dynamic response and two thirds acting on a 'rigid' structure.

$$\text{Response} = 2\sqrt{\frac{\text{velocity spectrum area}}{\text{response spectrum area}}}$$

Unmodified response = 2

$$\text{Modified response} = 2\sqrt{\frac{0.82}{\delta} \quad \frac{nSn}{K\,V_{10}^{2}}} = 10.6 \text{ (ignoring aerodynamic magnification)}$$

$$\text{for } \frac{n}{v} = .008 \text{ m}^{-1}$$

$$\delta = .025$$

(iv) Factor.

Factor for unmodified component = 4.1
Factor for modified component = 3.8

Total Overturning Moment

$= \text{mean hourly moment} \times (1 + \frac{2}{3} \times 0.145 \times 2 \times 4.1 + \frac{1}{3} \times 0.145 \times 10.6 \times 3.8)$

$= 110{,}000 \times (1 + 0.8 + 1.9)$ kg m

= 400,000 kg m*

* Using Davenport 1967 Total overturning moment = 360,000 kg m.

of motion. The motion took place in various modes simultaneously but predominantly in the two modes associated with the 'sloshing' of the tank and contents and the sway of the stalk. The motion of the stalk decayed quite rapidly but the 'sloshing' of the water continued for some time afterwards. By monitoring the motion of the tank by sensing the displacement of the stalk and extrapolating the behaviour of the water back to zero time, it was possible to show (Figure 3) that the wind load was distributed approximately two-thirds on to the stalk and one-third on to the flexible tank and water. It is this third of the force which is potentially magnified by the dynamic response of the water and the tank.

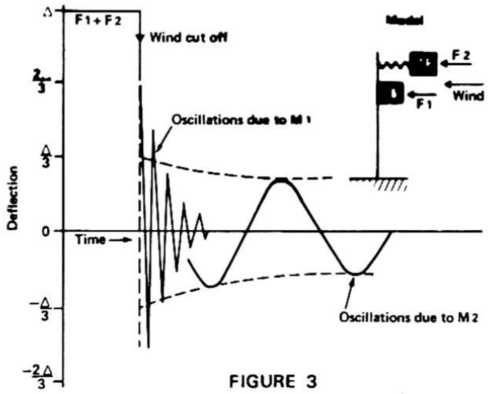


FIGURE 3

It is instructive perhaps now to consider the wind loads on a typical full scale tower. Taking a capacity of 500 cubic metres of water the diameter the tank would be about 12.2 m. The calculations appropriate to such a tower are scheduled in the facing table. The wind speed and surface roughness coefficients used have been estimated for an exposed site.

The total calculated wind loads are roughly double those which would have been assumed, taking an averaging period of 1 minute as suggested by British Standard Code of Practice CP3, Chapter V, or 70 percent greater than the loads on a rigid structure taking account of gust loading.

Drax Chimney

When completed this will probably be the world's largest multi-flue chimney. It will have a height of 260m and will have a constant outside diameter of 26m. The outer shell is made of reinforced concrete whose thickness varies from 1.5m at the base to .37m at the top; the three flues are also of reinforced concrete and they are elliptical in section, having major and minor axes of 13.7m and 9.2m respectively.

A circular cross-section was chosen for the chimney because it is this shape which has the lowest level of vortex excitation. Nonetheless there is still considerable doubt as to the behaviour of tall flexible cylindrical structures, and it was necessary to carry out a number of studies and analyses of the potential behaviour, both under gusting conditions and under potential vortex excitation. As the flues are in 22m lengths and are carried on expansion bearings (Figure 4) in order to prevent thermal effects from inducing unacceptable stresses in the concrete shell, the dynamic behaviour of the chimney as a whole is extremely complex. Too complex in fact to contemplate a wind tunnel model. A computational model was therefore built up and one element of it is shown in Figure 5.

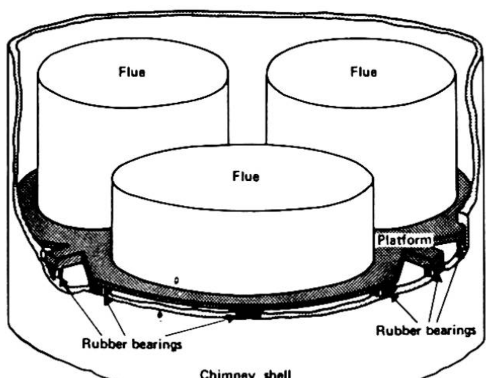


FIGURE 4

The flues are represented as a mass with rotational inertia supported on the bearings which had a finite stiffness in the horizontal direction but which have been regarded as infinitely stiff in the vertical (as above, the vertical stiffness is 100 times the horizontal). The shell is broken up into an equal number of elements (i.e. 11) to correspond with the flue sections and here again rotational inertia has been allowed for. Hysteretic damping has been included both in the bearings and in the shell. The complete system therefore has been represented by twenty-two masses with associated spring and damping systems.

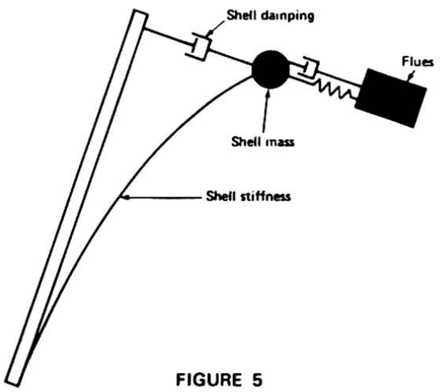
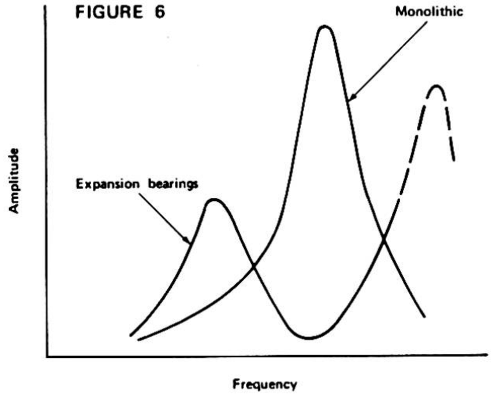


FIGURE 5

Unit sinusoidal forces were applied in turn to each mass element of the shell and the response of the whole system calculated. The computer programme centres round the 88 square matrix which was condensed to a diagonal matrix rather than inverted. The total effect is obtained by summing the effects of the loads on each shell element. This is done for various frequencies, and the interval between the frequencies was chosen depending on the sensitivity of the response of the chimney to frequency.

The first calculations were for a chimney with normal laminated rubber expansion bearings supporting the flues (Figure 6). For comparison, the case without expansion bearings was calculated and is represented by a curve labelled monolithic. It can be seen that the behaviour of the two systems is entirely different. It was a relatively simple step from knowing the response characteristics of the chimney to calculate its behaviour under gust conditions. This was done by integrating the response spectrum obtained using these frequency characteristics, and the net result showed that the chimney with flexible expansion bearings had in fact a dynamic response of only about half that of the monolithic chimney. This was based on the assumption that the damping in the shell structure, i.e. the quadrature component due to damping, was 2 per cent of that due to stiffness. The bearing damping was taken as 12 per cent. In either case the significance of gusting was small in relation to the total load, and particularly so when account was taken of the probable lack of correlation of wind pressure over the height of the chimney. It was estimated that a gust lasting for at least 15 seconds was required in order to envelope the chimney, and this compared with the period for the structure of between 1 and 2 seconds. Clearly there is unlikely to be much dynamic response under these conditions.

FIGURE 6


Vortex excitation

The second type of dynamic problem discussed in this contribution is that of vortex excitation. Indeed it is this vortex excitation which is the main problem with structures like the Drax chimney. The difficulty is that little data exists on the behaviour of structures of this size. It is also virtually impossible to carry out model tests in a wind tunnel because of the very high Reynold's number (approximately 10^8) which prevents the correct scaling of the dynamic effects. What data there is suggests that the cross-wind lift coefficient can have any value from 0.7 down to 0.1, and indeed recent papers at an International Symposium (Wootton 1968) suggested that the force coefficient might be even less. Whilst it is difficult therefore to calculate in advance the magnitude of the loads and movements of the chimney, the computer model enabled comparisons to be made between the various possibilities. The previous Figure indicating the response of the chimney to unit forces (as a function of frequency) is now modified to take account of the variation of wind force with velocity (a function of frequency via a Strouhal number of 0.27). Figure 7 then represents the non-dimensionalised response of the chimney to winds of varying velocity. It was clear that the use of the expansion bearings had significantly modified the behaviour of the chimney compared to the monolithic case. Further, the vibration amplitudes were reduced. This suggested that if the properties of the expansion bearings were chosen with the dynamic behaviour in mind then the overall maximum amplitude of the chimney could be further reduced. This was done by decreasing the shear stiffness of the bearings until the amplitude at the top of the chimney was less than one tenth of that in monolithic case. There are two peaks in the amplitude curve. The first is at relatively low wind speed where the frequency of vortex shedding corresponds with the natural frequency of vibration in the horizontal mode of the flue segments. The magnitude of their response is limited by the damping in the bearings. At higher wind velocities and frequencies the shell itself is playing the major part in the mode of vibration, and its amplitude increases as the wind speed approaches that corresponding to its natural frequency giving a second amplitude peak. However, the maximum wind speed likely to occur is somewhat lower than that necessary to produce a resonant condition here.

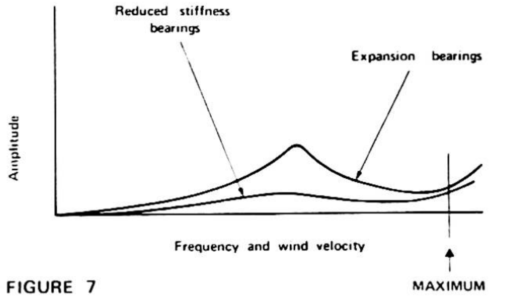


FIGURE 7

It is worth commenting that there are three basic ways of reducing vibration amplitudes due to vortex excitation in a structure. Firstly there is changing the natural frequency to avoid resonance; secondly increasing the damping to keep the amplitude to a reasonable level; and thirdly the prevention of the formation of the vortices by changing the shape of the chimney. In this case the cheapest solution involved changing the natural frequency and damping simultaneously to give a better performance. The possibility of eliminating the vortices at source was also considered and in parallel with the above computations a test programme was commissioned on behalf of the Central Electricity Generating Board (Walshe & Bearman 1967). Several methods of preventing the formation of vortices were considered, including the use of helical strakes and a perforated shroud. The tests demonstrated that the helical strakes produced a marked increase in the overturning moment due to wind whilst the perforated shroud did not. Although the effectiveness aerodynamically of the shroud was not as great as that of the strakes it was considered in the event of a chimney being subject to vortex excited oscillations that the shroud represented a more reasonable repair scheme. It reduced the vortex excitation quite significantly but did not introduce an increase in the wind drag load.

The behaviour of the Drax chimney will be monitored during construction so that the calculations can be compared with actuality, when in the unlikely event of the effects being underestimated the chimney can be modified accordingly.

Whilst the main theme of this paper has been the use of the more advanced techniques of analysis one lesson which could be drawn from the examples quoted is the need to monitor the non-standard structure during its erection and immediate post-erection period to determine the likelihood or otherwise of untoward behaviour.

It is also clear from this paper that whilst modern techniques are being used the amount of data available and the quality of the data are poor. This has been the subject of another paper by one of the authors (Waller 1968) in which the general conclusion was that more of the research efforts should be directed towards the full scale interaction of the wind and structures so that the more sophisticated design techniques can be utilised with confidence.

Notation

α	wind power law exponent.
δ	logarithmic decrement.
g	acceleration due to gravity
K	surface drag coefficient.
m	effective mass.
n	natural frequency.
V	mean hourly wind speed.
x	$1200\frac{n}{V}$.
z	height above ground.
$\frac{n\,Sn}{K\,V_{10}^{2}}$	reduced gust velocity spectrum.

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SUMMARY

Several structures are described which are novel in that dynamic wind excitation was a significant design parameter. Gust excited vibration is considered in terms of occupant comfort in a block of flats and the structural integrity of a water tower. The reduction of vortex excited vibration of a 260 m chimney is described. The quality of wind data is poor and consequently the need to monitor the behaviour of such structures during and following erection is emphasised.

RÉSUMÉ

On parle de plusieurs structures qui sont nouvelles en ce sens que l'excitation dynamique par le vent a été un paramètre important de leur dessin. Les vibrations causées par des coups de vent sont considérées en relation au confort des locataires d'un immeuble et à l'intègrité structurelle d'un château d'eau. On décrit la diminution des vibrations causées par tourbillons d'une cheminée de 260 m de hauteur. Les data donnés pour le vent ne sont pas très sûrs et on souligne donc la nécessité de surveiller le comportement de telles structures pendant et après l'érection.

ZUSAMMENFASSUNG

Verschiedene Strukturen werden beschrieben, welche insofern neu sind, als die dynamische Erregung durch Wind ein massgebender Konstruktionsparameter war. Durch Windstösse verursachte Schwingungen werden mit Hinblick auf den Komfort der Bewohner eines Wohnblocks und auf die strukturelle Integrität eines Wasserturms bewertet. Ferner wird das Nachlassen der durch Wirbel verursachten Schwingungen an einem 260 m hohen Schornstein beschrieben. Die auf Wind bezüglichen Daten sind nicht sehr zuverlässig, und es wird daher auf die Notwendigkeit hingewiesen, das Verhalten solcher Strukturen während des Aufbaus und danach zu überwachen.