Wind effects on structures

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Seminar VI

Wind Effects on Structures

Effets du vent sur les structures

Windeinwirkungen auf Tragwerke

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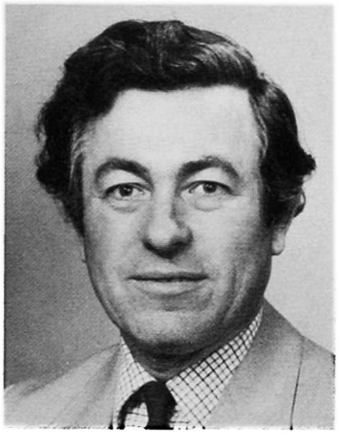

William Melbourne, born in 1934, graduated as an Aeronautical Engineer in 1955 from the University of Sydney and then completed a Ph.D. at Imperial College, London University in 1960. After working for a decade in the aeronautical field he turned to wind engineering research and is currently Chairman of the Department of Mechanical Engineering at Monash University.

SUMMARY
Two concepts, inertial load distributions and use of design wind speeds with direction, both of which are in use by advanced structural designers, are discussed with the objective of illustrating how these concepts can be simply applied and hence incorporated into everyday structural design. The theme is to make progress towards introducing real response and real loading from wind action into the design of structures. The remainder of the paper discusses a number of effects of freestream turbulence on wind loading of structures with the objective of drawing papers dealing with these more complex problems.

RESUME
L'auteur présente deux possibilités pour l'introduction plus réaliste des sollicitations dues au vent: considération des propriétés aéroélastiques de la construction; considération des vitesses effectives en fonction de la direction du vent. Ces deux méthodes sont présentées, l'objectif étant d'illustrer comment elles peuvent être appliquées simplement et par là incorporé dans le travail quotidien de l'ingénieur. Le but est d'introduire la réaction réelle et la charge réelle de l'action du vent dans le projet des structures. L'article discute également un certain nombre d'effets de turbulences et de charges de vent sur les structures, afin de susciter des contributions traitant de ces problèmes plus complexes.

ZUSAMMENFASSUNG
Der Autor zeigt zwei Möglichkeiten zur realistischeren Erfassung der Windbeanspruchung auf: Berücksichtigung der aeroelastischen Eigenschaften des Tragwerkes bei der Einführung der Windlasten; Betrachtung der effektiven Windgeschwindigkeiten in Abhängigkeit der Anströmrichtung. Dabei werden einfache Wege aufgezeigt, um deren Eingang in die tägliche Entwurfsarbeit zu ermöglichen. Ziel ist das effektive Verhalten des Tragwerkes unter den effektiven Windeinwirkungen besser im Entwurf und in der Bemessung zu berücksichtigen. Zum Schluss wird noch eine Anzahl von Einwirkungen infolge Windturbulenzen aufgelistet mit dem Zweck zu diesem komplexeren Problemkreis Beiträge zu erhalten.

1. INTRODUCTION

The last two decades has seen an explosion in knowledge concerning the determination of the response of structures to wind action. By combining work in meteorology, fluid mechanics and structural dynamics, and with the help of probabalistic and spectral mathematics, a description of the real response process has emerged. In spite of this knowledge a great majority of structures, including many major in size and cost, are still designed using wind loads obtained from quasi-steady based wind loading codes. Reliance on such totally artificial load concepts ignores in one direction the economies of scale in reducing global loads and in the other direction the risk of ignoring significant dynamic amplification of response for some wind sensitive structures. This has occurred in a world of structural design where even the smallest structural offices think nothing of using enormously powerful structural packages to analyse member loads. In relative terms it would be a minute effort to add to these programs the ability to output, or use in the calculations, an inertial load distribution related to a given base overturning moment which can now be obtained either from model tests or in many cases analytical techniques for determining response to wind action.

Whilst those of us primarily concerned with wind engineering research have made significant advances in recent years, the impact on the structural design community appears not to have been so significant. It seems that there is still a gap to be bridged to bring the new knowledge and analytical techniques into the world of the practising structural engineer. I will take this opportunity to discuss aspects of new knowledge which could significantly affect the design of structures, in the hope that others at this conference will pick up the theme and make progress towards introducing real response, and real loads, from wind effects on structures into everyday structural design.

2. LOAD DISTRIBUTIONS

In respect of wind loading on towers, chimney stacks and buildings, data from wind tunnel aeroelastic model studies and the more advanced analytical techniques generally finish by determining a peak (design) displacement or base overturning moment. This peak response is made up of a mean and fluctuating component of which the latter makes up 70% or more of the total, and the load distribution which actually stresses components of the structure is consequently primarily a "dynamic" inertial load distribution. That is the load distribution is made up of the sum of the mass elements times the maximum acceleration acting on those elements in a given cycle. When modal analysis programs are run to determine the mode shapes and frequencies it would be a simple matter to include an output of the inertial load distribution for each mode. This would hopefully stop the practice of using load distributions based on quasi-steady wind load distributions which are so commonly used. The very large difference between a typical quasi-steady wind load distribution and the real inertial load distribution can be illustrated by considering a tall cantilevered structure with constant mass per unit length and which for simplicity is taken as oscillating in a cross-wind direction such that the mean response is zero and the peak base overturning moment is all due to the fluctuating component. A typical quasi-steady wind load distribution would be one varying with velocity squared and in which velocity might vary with height, $z^{0.2}$ which gives a load distribution.

$$F(z) = \text{Const } z^{0.4} .$$

The inertial load distribution is a function of the mass multiplied by acceleration and for a constant mass per unit length and cantilever mode shape ($y \propto z^{1.5}$) gives a load distribution

$$F(z) = \text{Const } z^{1.5} .$$

A graphic comparison of these two load distributions, to give the same base overturning moment, is given in Figure 1. It is interesting to note that the mid-height bending moment for the incorrect quasi-steady wind load distribution is just over half the base overturning moment and for the correct inertial load distribution is about two thirds of the base overturning moment. For some cases of towers in chemical plant with heavy elevated vessels this difference can be a factor of two.

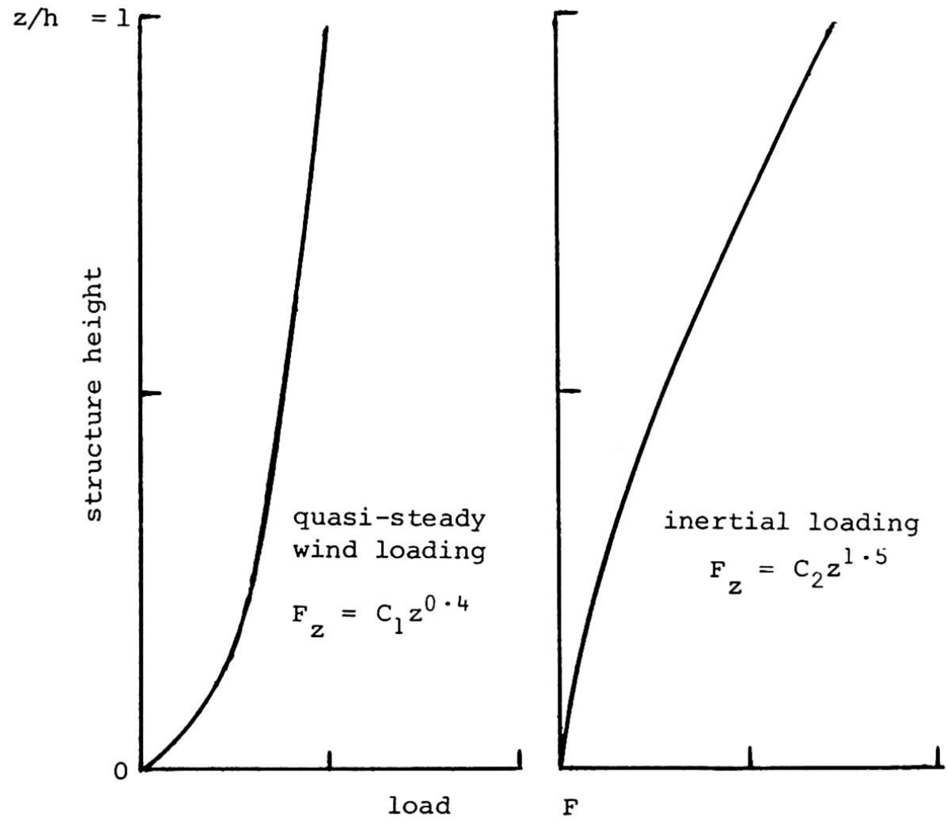

Fig. 1 Comparison of quasi-steady wind load distribution and inertial load distribution for a uniform cantilever tower to give the same base overturning moment

3. WIND DIRECTIONALITY EFFECTS

Wind directionality, or rather the probability distribution of wind speeds with direction has been used in the design of major structures for many years following the pioneering work of Davenport [1]. However we have been slow to apply the same considerations to low rise structures, probably because the apparent sophistication appears not be be worthwhile. A closer look reveals that in many situations there is a potential for design optimisation, and considerable savings from using different values of design wind speed for different directions. For example, a structure may clearly have suburban roughness approaches for all strong wind directions, but have open country, or be exposed on an escarpment for other wind directions. If wind direction cannot be taken into account the design wind speed for the region for all directions has to be used in combination with the worst loading case, and the end result can be a design load up to twice as high as could be rationally required for a given risk. Similar over-design can occur when having to account for a dominant opening when it is facing directions from which the probability of a given design wind speed is much lower.

The problem from the structural engineer's point of view seems to be that he is reluctant to engage in a full probabalistic design approach. However, in regions where there are significant differences in wind speed probability occurrence for broad bands of wind direction, it is possible to devise conservative design wind speeds with direction which can be used in a deterministic design approach without having to undertake a full integration of directional wind speed data with each load case. If the full integration is not undertaken it is essential to devise conservative design wind speeds with direction to account for (a) the additive effects of lower probabilites of other than the design direction (band) under consideration, and (b) the possible defficiencies in determining the directional wind speed probability distribution. In this latter respect reference is intended to the problems of short period directional data available (less than 20 years for example) and in areas where thunderstorms dominate and the only data available are the maximum daily wind speed and direction which can obscure the possibility of a similar but slightly lower wind speed occurring in a neighbouring directional band.

In Australia a pilot attempt has been made to provide simply usable directional wind speed data, and which is currently being tested against a number of design situations prior to making it available for general use. Examples of this will be given here in relation to a low rise factory building in the hope that other authors will be encouraged to provide comment and other examples of the potential savings and problems (pitfalls) which can accrue from using simplified directional design wind speed data.

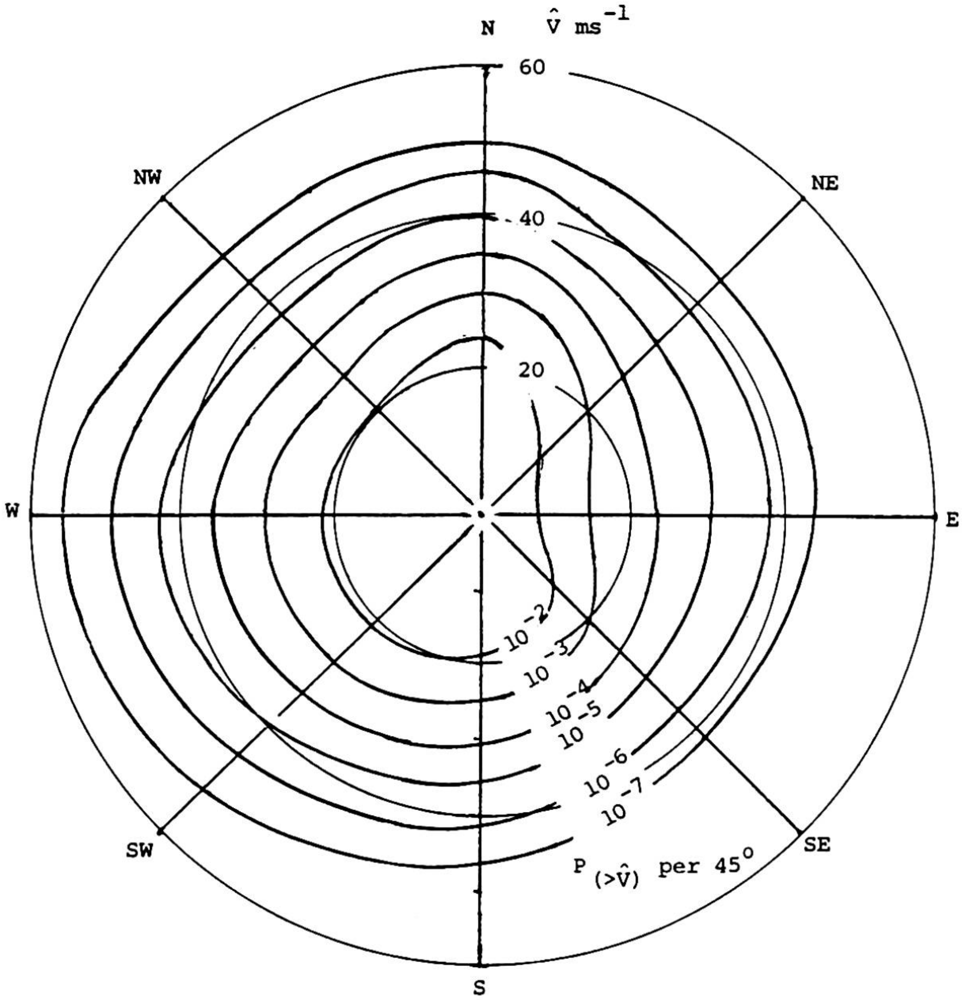

Fig. 2 Probability distribution of 3-second mean maximum wind speeds based on daily maximum data referenced to a height of 10 m in open country terrain (z_o=0.020 m) for the City of Melbourne

In Figure 2 the probability distribution for the 3-second mean maximum wind speeds (gust wind speeds) in the City of Melbourne are given. These data were derived from a composite of 114 years of daily maximum gust wind speeds with direction, from five anemometers spread some 50 km apart (largest single record 32 years). These data have been corrected for approach fetch and anemometer position error for 16 wind directions and given with reference to a 10 m height in open country terrain (roughness length z_o = 0.020 m), which is typical of the airfield sites in which most of the anemometers were located. The probability for a 50 year return period is $P(>v) = 1/50 \cdot 365 = 55 \times 10^{-6}$. A full integration for all wind directions gives the 50 year return period wind speed to be 40 ms^{-1}. On evaluation of a number of factors the author has proposed (at least initially) that conservative design directional wind speed data, for use in a simple deterministic design application, be derived for eight wind directions along a contour with half the 50 year return period wind speed probability (in Australia the code requires 50 year return period wind speed to be used in the current factored load design approach). Such an approach results in the design wind speeds with direction given in Table 1. Examples of the application of these design wind speeds, using all directions design wind speed compared with taking direction into account, will be given using pressure coefficients and wind speed profiles for two surface roughness conditions as specified by the Australian Standards Association AS 1170 Part Two, Wind Loads 1981.

Direction	NE	E	SE	S	SW	W	NW	N	All Directions
Wind Velocity	29	27	29	34	38	40	36	38	40

Table 1 REGIONAL BASIC DESIGN WIND VELOCITIES FOR 45° DIRECTIONAL SECTORS FOR THE CITY OF MELBOURNE (3 second mean maximum gust wind speed in ms^{-1} at 10 m height in open terrain, z_o = 0.020 m, relating to a 50 year return period)

Consider a low pitch roof on a building, 5 m in height and with a dominant opening in the east wall. From AS1170, th external pressure coefficient, C_p = - 0.9, and the internal pressure coefficient, C_p = 0.8, both acting upwards; and in open country terrain the design gust velocity at 5 m is 0.93 times that at 10 m, and in suburban terrain it is 0.65 that at 10 m in open country terrain (i.e. as referenced in Table 1).

(i) For open country approaches in all directions

(a) For All Direction Design Wind Speed

Design Wind Speed = 40 . 0.93 = 37.2 ms^{-1}

Design Roof Pressure = 1.7 . 0.6 . 37.2^2 = 1412 Nm^{-2}

(b) For Directional Design Wind Speed

Design Wind Speed, Max from N, S, and W = 40 . 0.93 = 37.2 ms^{-1}

Design Wind Speed, Max from NE, E and SE = 29 . 0.93 = 27.0 ms^{-1}

Design Roof Pressure = 0.9 . 0.6 . 37.2^2 = 747 Nm^{-2}

or = 1.7 . 0.6 . 27.0^2 = 744 Nm^{-2}

The former being the design case a reduction of 47% is achieved by taking design wind speed with direction into account.

(ii) For suburban terrain approaches from S, W and N and open country terrain from the E

(a) For All Direction Design Wind Speed

Design Wind Speed $= 40 \,.\, 0.93 = 37.2\ ms^{-1}$

Design Roof Pressure $= 1.7 \,.\, 0.6 \,.\, 37.2^2 = 1412\ Nm^{-2}$

(b) For Directional Design Wind Speeds

Design Wind Speed, Max from N, S and W $= 40 \,.\, 0.65 = 26.0\ ms^{-1}$

Design Wind Speed, Max from NE, E and SE - $29 \,.\, 0.93 = 27.0\ ms^{-1}$

Design Roof Pressure $= 0.9 \,.\, 0.6 \,.\, 26.0^2 = 365\ Nm^{-2}$

or $= 1.7 \,.\, 0.6 \,.\, 27.0^2 = 743\ Nm^{-2}$

The latter being the design case, a reduction of 47% is achieved by taking design wind speed with direction into account.

If the opening was centrally located in the east wall it would have been reasonable to use the Basic Design Wind Speed for the E Wind Direction of 27 ms^{-1} instead of the 29 ms^{-1} for the NE and SE directions, and this would have resulted in a reduction of 54% by taking wind direction into account.

Obviously there are many examples of the savings in structure which can be achieved by taking wind direction into account. Whilst in the past it has only seemed worthwhile going to the extra trouble for large expensive structures it is hoped that these examples show also how easy it is to achieve significant reductions for a simple, relatively inexpensive structure, but one for which savings are often significant.

4. WIND TURBULENCE EFFECTS

One of the author's main research interests has been in the fundamental aspects of the effect of freestream turbulence on bluff body aerodynamics, and in particular the application of these effects to the design of structures to withstand wind action. To draw on papers which will address these problems it is proposed to highlight a number of recent examples where turbulence effects were very significant in determining design wind loads.

4.1 Cladding loads

There are a number of examples of cladding failure where turbulence plays a major part; the most graphic in recent times have probably been the glass failure on the Boston Hancock Building and the mass failure of roofs in Darwin during the passage of Cyclone Tracy. In both cases there were structural inadequacies which contributed to the scale of the disasters, but in which there were major contributions from the very large enhancement of the high negative pressures under the re-attaching shear layers occasioned by the presence of relatively high turbulence in the incident air stream. It is noted that in low turbulence (smooth) air flows very high intermittent negative pressures near a separating leading edge do not occur at all; it is the presence of the freestream turbulence which causes the shear layer to re-attach so rapidly on the streamwise surfaces with attendant high negative pressures.

From model tests and limited full scale experience, the highest negative pressures in this respect seem to occur where there is an edge discontinuity (an intersection with a lower stage building or just a change in edge shape) in a region of high freestream turbulence, which, of course, occurs at lower heights near the tops of surrounding buildings or roughness elements. Time and again the highest peak pressures established in model studies come from the lower part of the building, not the higher part, where the mean pressures are always highest. This would not occur if the turbulence characteristics, in particular magnitude (turbulence intensity), were not modelled correctly.

A recent study of cladding pressures on a proposed building in Hong Kong, which had lower edge discontinuities and very high intermittent negative pressures, prompted the author to look carefully at wind characteristics in tropical cyclones, Melbourne and Blackman [2]. This resulted in a major study of tropical cyclone wind data collected at the Hong Kong Royal Observatory and at Waglan Island, including modelling the position errors for both of these anemometer sites. The conclusion was that in extreme tropical cyclone events the lower part of the atmospheric boundary layer has turbulence characteristics similar to flow expected over suburban roughness (i.e. z_o = 0.20 m). This is contrary to many earlier findings in respect of wind characteristics over the sea. Because these tubulence characteristics are so important to the determination of cladding pressures, it is worth emphasising that the effective surface roughness of the sea in extreme tropical cyclone conditions is much higher than previously thought, and this also is relevant to wind effects on offshore as well as onshore structures.

4.2 Grandstand roofs, bridge decks

A phenomenon allied to that in §4.1 is the wind loading on cantilevered grandstand roofs. Again it is the high levels of incident turbulence through the shear layer re-attachment system which can cause very high response of these roofs, much higher than predicted by quasi-steady codes. This phenomenon was illustrated by Melbourne [3,4] and a method of reducing the wind loading was put forward. This entailed using a slot behind the leading edge which bled flow into the re-attaching shear layer bubble and which in turn prevented the development of very high loads. This suggestion has recently been taken up for a grandstand roof constructed by BASC Contracts Ltd, Cook [5].

The response of large span box girder bridge decks has been shown to be much more dependent on the magnitude of incident turbulence, Melbourne [6], than was predicted using quasi-steady assumptions and strip theory. Full scale and model studies on the cable stayed box girder West Gate Bridge (centre span 336 m) in the City of Melbourne concluded that vertical deck response was increasing approximately with turbulence squared. Again the mechanism is related to flow phenomena near the leading edge which are very dependent on the incident freestream turbulence, which, it is suggested, can be reduced by permitting bleed flow through a porous or slotted leading edge configuration in much the same way as for the grandstand roof.

These examples of the major influence of turbulence and the way in which leading edge pressures in high turbulence flows may be controlled by bleeding flow into the re-attaching shear layer region perhaps have other applications, even to industrial buildings which could be designed with slotted leading edge roof configurations.

4.3 Interference effects

Many authors have instanced examples of where the response of structures in the interference wake flow of other structures has been greatly increased. This has included full scale examples involving buildings, chimney stacks and bundled electrical conductors. With respect to the latter two, work by Wardlaw [7] and Ruscheweyh [8] have indicated just how complex these problems can be with the involvement of many excitation mechanisms. However, with buildings the problem seems much simpler, that is simpler to understand and simpler to predict.

The major enhancement of building response in the wake flow of other stuctures seems to be attributable to the increase of incident turbulence caused by the wake flow. For example, Melbourne and Sharp [9] showed that increase in interference effects was relatively much greater when the buildings were in low surface roughness terrain. In this situation the increase in turbulence level caused by an upstream building is relatively much more than when the original approach turbulence is due to city building type roughness. The danger, in a design sense, is to design for the response of a tall building in a shore front or open country exposure when there is any possibility of similar buildings being built upstream (on reclaimed land in respect of the former situation). A single upstream building in these situations can produce much greater response on a downstream building than if the building were in a generally rougher terrain (more turbulence) because when the interference occurs at the edge of the building wake we have the worst combination of high mean wind speed, due to the small roughness approach terrain, and the high turbulence at the edge of the wake. There is a case for saying that all tall buildings designed initially for open country or shore front exposure should be checked (designed) for both along-wind and cross-wind response with city centre type turbulence levels superimposed on the same open exposure mean wind speeds. Such a procedure would go a long way towards eliminating the interference response problems for tall buildings.

4.4 Reynolds number/turbulence effects

To illustrate that not all increased turbulence effects produce higher wind loading and the effect of Reynolds number, we can consider structures with curved surfaces, in particular circular structures like chimney stacks.

It has been known for some time that predictions of circular chimney cross-wind response based on data obtained from aeroelastic model tests tend to overestimate the full scale experience. Fortunately most reinforced concrete stacks are such that the design is dictated by the along-wind response; only for very large, stiff stacks (such as during the erection phase) will the cross-wind critical wind speed be high enough in the wind speed range for cross-wind response to dominate design considerations.

To quantify the effect of turbulence on circular cylinders over a range of Reynolds number from sub-critical to super-critical, a major study has been undertaken at Monash University. Some of these results have been reported by Cheung and Melbourne [10] and the effects of turbulence can be seen to be acting in two ways. In Figure 3 an example has been given of the variation of fluctuating lift with turbulence intensity and Reynolds number.

At sub-critical Reynolds numbers the effect of increasing turbulence is to decrease the fluctuating lift. As the organised shedding disappears in the critical flow regime, the fluctuating lift drops to a lower value. At supercritical and transcritical Reynolds numbers increasing turbulence causes the fluctuating lift to increase. Similar characteristics are shown with respect to the fluctuating drag component.

Prediction of the response of structures with curved surfaces can be seen to be very difficult, and wind tunnel tests conducted at Reynolds numbers below 2×10^5, even with the correct turbulence, are quite meaningless in terms of providing data relevant to full scale structures.

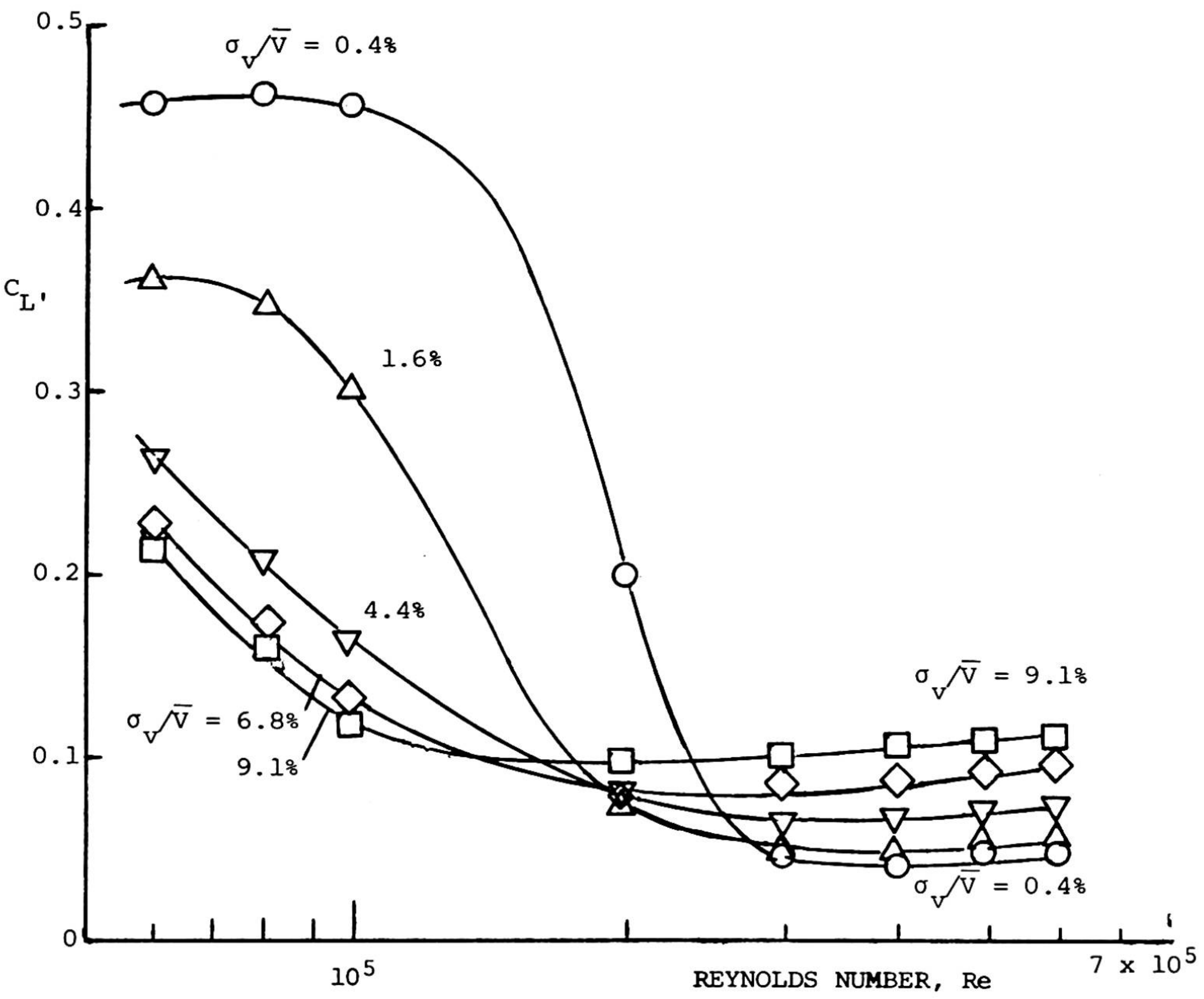

Fig. 3 Fluctuating lift coefficients on a circular cylinder as a function of Reynolds number for different turbulence intensities.

CONCLUSIONS

This introductory paper has discussed two aspects of wind loading on structures which are common knowledge amongst wind engineering researchers and used by some of the leading structural designers. The discussion of inertial load distribution, and the use of design wind speeds with direction has been aimed at exciting interest in bridging the gap between advanced knowledge and general practice by emphasising that both of these real wind loading concepts can be simply incorporated into everyday structural design.

The remainder of the paper has discussed a number of effects of freestream turbulence on the wind loading of structures with the objective of drawing papers dealing with these more complex problems.

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