On the estimation of wind loads for building and structural design

by

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and

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Prof. J. D. Haddon (Retired) wrote that it did not seem many years ago that codes neglected negative pressure on roofs entirely, and they were still not fully satisfactory. The Paper was, therefore, very welcome.

61. Planning authorities should consider the effect of new buildings on the wind pressures upon existing structures. As stated, large suction forces were created on the lee of a bell tower and those were probably covered in design by other conditions of suction. It was the large pressures on the windward side which were not allowed for. For example, in the case of a 30° roof, the lee slope suction changed, at one point, from -0.55 to $+0.9$ on the windward side of a tower when built against the leeward side of the building.

62. Obstructions did not always act as expected. High hedges were used in the south of France to protect buildings from the mistral. Prof. Haddon had found, from models, that a 10-ft-high wall, 60 ft to windward of a 20-ft-high shed with a 30° slope roof, would change the average pressure on the windward slope from about -0.05 to $+0.3$, while with a diagonal wind, the peak suction could be increased to -3.0 .

63. Results vary with different investigators: since velocity profile has a large effect on low slope roofs, height-breadth ratio is important, and if the model is not mounted on the tunnel floor, circulation might take place round the model increasing suction. In Table 10, were the results, which appeared high, given a 'factor of safety', or were the models not mounted upon the floor of a closed wind tunnel?

64. A rectangular prismatic shed with axial wind and side doors open near the windward end might have an internal pressure of -0.8 , while rearward external pressure was only -0.1 , so giving a resultant inward pressure of 0.7 . That was not covered in Table 10 for roof slopes below 45°. The 5° grandstand-type building at C (Table 10) had a resultant outward pressure of about 5.0 .

65. Codes could be improved for multispan roofs where pressures and suctions for diagonal flow could be much higher than for monospan roofs. Had Mr Scruton or Mr Newberry any figures for those?

66. It would be interesting to know the direction of the wind that wrecked the building in Fig. 5 and whether it was in course of construction.

Mr A. G. F. Eddie (Senior Engineer, Rendel, Palmer & Tritton) wrote that the factor of 1.6 by which the values of p' in Table 6 of CP3 exceeded the dynamic pressure due to the mean minute velocity (referred to in § 19 of the Paper) seemed to represent a gust factor of $\sqrt{1.6}$ rather than a shape factor of 1.6 as suggested by the Authors. This could be demonstrated by the velocity profile curves in Fig. 13.

**Proc. Instn civ. Engrs*, vol. 25, June, 1963, pp. 97-126.

68. Curve A showed the velocity profile obtained from equation (9) of the Paper by assuming $\bar{V}_{33}$ to be equal to 67.5 miles/h (corresponding approximately to $\bar{V}_{40}=70$ miles/h), and for category 1 topography, i.e. $K=1.1$, $F_{33}=1.5$, and $\gamma=1/14$.

69. An identical curve could be obtained from equation (8) of the Paper, if $\bar{V}_g=112.5$ miles/h at 900 ft, and according to equation (7), this curve incorporated gust factors of 1.25 at 900 ft and 1.5 at 33 ft. This curve corresponded to a gradient wind of 1 in 100 years' incidence, derived from equation (5a) of the Paper with conditions of $U=77$ and $1/a=1/7.7$. (These two values were used in relation to a particular site, but were average values for England and Wales.)

FIG. 12: DESIGN WIND SPEED : MILES/H

70. Using the rough guide described in § 20 of the Paper, curve A ought to correspond approximately to a mean minute velocity of $70 + 10 = 80$ miles/h at 40 ft. Taking the values of p' for 80 miles/h from Table 6 of CP3, chapter V, and assuming them to be dynamic pressures, curve B was obtained (i.e. $V = \sqrt{(p'/0.00256)}$).

71. This could be seen to approximate quite closely to the Authors' curve for $\bar{V}_{33}=67.5$. If the profile of the mean minute velocity was plotted, assuming this to be 80 miles/h at 40 ft and $V_z/V_{40}=(z/40)^{0.13}$, curve C was obtained. This was seen to be curve B reduced by a factor of 1.25 (or approximately $\sqrt{1.6}$) at all heights.

72. Therefore, it would appear that if the values of p' in Table 6 of the Code were dynamic pressures, they would correspond to the mean minute velocity multiplied by a gust factor of 1.25 at all heights.

73. The curve, derived from the Authors' equations for a mean hourly wind corresponding to the mean minute speed in the Code, was only slightly different, and the difference seemed to be due mainly to the variation of gust factor with height proposed by the Authors.

74. Thus it would appear that Table 6 of the Code gave a reasonable approximation to the dynamic pressures, assuming category 1 topography and 1 in 100 years' incidence.

75. The Authors' equations allowed, of course, many more variables to be considered than did Table 6 of the Code, and Table 3 of the Code would seem to be based on too low a gust factor, at least for category 1 topography.

76. The table of shape factors given by the Authors was particularly helpful to the practising engineer, and when used in conjunction with the aspect ratio correction factors in Table 7 of the Paper, seemed to give somewhat lower values than CP3. For a typical circular chimney stack of aspect ratio between 8 and 40, CP3 gave a shape factor of $1.1 \times 0.7 = 0.77$, compared with the Authors' figures of $0.7 \times 0.8 = 0.56$.

Prof. A. G. Davenport (Faculty of Engineering Science, University of Western Ontario) wrote that the Authors were to be congratulated for having ably summarized the many factors which needed to be considered in achieving an integrated view of the problem of wind loading. Expressly, the Paper had been spurred by the need for revision to the British Code and 'to bring into open discussion the means for determining wind loading in relation to structural design'. This raised most difficult issues, which, to answer satisfactorily, required a close scrutiny at the basis of structural design philosophy.

78. A basic question which had been only lightly touched on in the Paper, was: what precisely was a design wind load intended to represent? The answers to this might be varied and probably far from precise. The closest one could get to a definition might be along the lines that it should represent a 'working load' imposed by the wind. This again was far from precise, but it might be interpreted that a working load should represent the 'largest load having a reasonable chance of occurring during the structure's lifetime'. But even this begged the question completely, since it made no suggestion as to what was reasonable.

79. What of the indeterminate factors? Should wind loading be based on a 'conservative' or a 'best' estimate of real loadings leaving the safety factor to take care of the margin of uncertainty? Should the same basic loading be used in estimating collapse conditions as some limiting condition such as yielding or large deflexions? What of statistical uncertainty? Should one design a structure for the wind velocity having a small (say 5%) chance of occurring during the structure's lifetime or for the velocity likely to occur at least once during its lifetime? Few designers would be clear as to which alternative to use for a design wind load (Prof. Davenport included), yet the former was likely to be 50% greater than the latter. Which value should be incorporated in a code of practice? This dilemma of how to rationally assimilate statistical information of the sort presented in the modern statement of wind loading, into the traditional design processes involving working loads, load factors, safety factors and working stresses, was a very real one. Knowledge of wind action had, in a sense, almost outstripped ability to usefully use this knowledge.

80. Against this unsettling background, it was perhaps well to be reminded by the Authors that 'experience has shown that only in occasional and exceptional circumstances are structures completely wrecked by wind and even then, it has been found in general that failure originated locally'. Thus, even if it was not always possible to find a wholly rational solution to the problem it was possible to find empirical engineering solutions which gave satisfactory results. In this sense, each structure put up represented an experiment which tested for safety (but not economy) the complete design process of which the wind loading was one aspect. It could not be stated that this type of experiment verified the wind loading assumptions, for the simple reason that the margin of safety in most civil engineering structures was very comfortable indeed. The serious shortcoming of the empirical approach was that

its effectiveness was only assured for that family of structures in which experience had been accumulated. It would not assure that new types of structure or modes of construction were necessarily resistant to wind; at present day rates of innovation this was the disturbing feature.

81. The Authors, referring to the problem of gust action on structures, had quoted Prof. Davenport's approach to this subject (which embodied certain statistical concepts) and also the more familiar 'gust factor' approach. The last factor was applied to the mean hourly velocity and was therefore squared to determine the pressures. In the Paper the term 'gust factor' was used in a restricted sense to refer to the ratio of the 'maximum' gust velocity (or the maximum average gust velocity) to the mean wind velocity. In a broader sense it might be thought of as a factor which took into account the effect of gusts on the structure over and above the effect of the mean wind. To be realistic, the gust factor should take into account the following:

- (i) the higher pressures of gusts;
- (ii) the sequential or time history of gust loading and its dynamic amplification;
- (iii) the spatial extent of gusts.

82. Allowance for all these influences could be made directly using the statistical approach. The principal structural and environmental factors affecting the response were:

- 1. the roughness of the ground and turbulent intensity;
- 2. the height above ground;
- 3. the dominant natural frequencies;
- 4. the size of the structure and its shape;
- 5. the ratio of the superficial area of the structure to its weight (this governed the aerodynamic damping);
- 6. the mechanical damping; and
- 7. the mean wind velocity.

83. Each of these factors could produce very significant variations in the overall effective wind load.

84. In recent years, Prof. Davenport had had the opportunity to examine several structures using the statistical approach with the aim of determining the maximum effects caused by the wind. The results of some of these studies were summarized in Table 11. This gave the effective gust factors as should be applied to the mean hourly wind velocity to obtain the steady wind velocity which would produce the same maximum effect as the gusty wind. None of the values should be taken as necessarily typical of the whole class of similar structures to which they belong, without first comparing the properties listed above. Table 11 indicated the range of gust factors that might be encountered amongst civil engineering structures. The largest was about 3.0 and the smallest close to 1.0. In terms of pressures, this represented a ratio of 9. When compounded with the large variations in mean wind pressure (the greatest might be twenty times the least), one was made aware of the importance of giving careful consideration to wind effects.

85. It was interesting to note that the values of gust factor given in Table 11 straddled the values given in Table 4 of the Paper, although the reasoning behind the two results were different. (The range of values, however, in Table 11 was almost twice as large.) Part of the explanation for the similarity was that for a group of structures the reduction in the effect of gust pressures due to their spatial incoherence more or less balanced the increase in their effectiveness from dynamic causes, and these were the two principal influences absent from the derivation of the gust factors of Table 4. It might also be pointed out that time average velocities such as those used in Table 4 could also be derived directly using the statistical concepts. (Equation (31) of reference 10 contained the appropriate formula.)

TABLE 11: GUST FACTORS COMPUTED FOR VARIOUS STRUCTURES

$$\text{Gust factor} = \frac{\text{effective velocity of wind}}{\text{mean hourly velocity of wind}}$$

Structure	Effect	Open country	City centre
Arc lamp	equivalent static wind load	2.0	2.9
Water tower	equivalent static wind load	1.5	2.1
Suspension bridge (3300-ft span)	lateral bending moment	1.45	—
	lateral shear force	1.55	—
	vertical bending moment	∞	(no effect due to mean wind)
500-ft guyed mast	vertical shear force	∞	—
	bending moment	1.20	1.40
	shear force	1.35	1.25
Long span cable	deflexion	1.08	—
Tower for cable	equivalent static wind load	—	—
140-ft radio telescope (normal to wind)	thrust trunnion bearings	1.08	—
	torque on trunnion axis	1.30	—
100-ft belfry tower	deflexion	2.65	—
400-ft sky-scraper	deflexion	—	1.25

86. It would probably be quite feasible soon to make a broad classification of gust factors along the lines indicated in Table 11. Some highly tentative values might be indicated by Table 12.

87. Some general remarks might be made concerning the nature and form of codes. One of the principal difficulties in the preparation of codes was that they were often scientifically obsolescent as soon as the ink was dry. This had a very real retarding effect on engineering advance which admittedly might, in some instances, be expedient, but in other instances might lead to stagnation. For good or bad, codes exerted a very strong control over the thinking processes and inventiveness of structural designers which frequently commenced with their engineering training.

TABLE 12: VELOCITY GUST FACTORS

Type of structure	Terrain category		
	Open country	Wooded terrain, suburbs	City
Small, flexible, ponderous structures	2.0	2.5	3.0
Structures of intermediate size, stiffness and damping	1.5	1.8	2.1
Large, stiff, light structures, heavily damped	1.1	1.2	1.3

It might be possible to avoid some of the less desirable features of codes by attention to the form in which they were written. One possible approach might be to give greater emphasis to aims rather than to means, since it was usually in the latter area that the greatest uncertainties existed, the greatest changes took place, and there was the least justification for being dogmatic. An example of where this approach appeared to have been taken was in the Civil Engineering Code of Practice

on Foundations (published by the Institution). Prof. Davenport believed that another notable advance was made in the wind loading section of the National Building Code of Canada, 1960. In this, in contrast to previous editions, the section giving detailed pressure coefficients (in fact the excellent tabulation prepared by Prof. Ackeret) was removed from the body of the code and issued as a separate supplement. In this form it could be reviewed and modified far more frequently than the parent document, and the body of the code could indicate general requirements without involving technicalities. This line of thinking could probably be extended to other factors. The Authors' comments were invited.

88. The Authors had clearly indicated the value of wind-tunnel tests for many structures. It might also be suggested that the value of site investigation of the wind conditions was no less worthwhile. The two principal features of terrain controlling wind structure were its roughness and surface contours. These governed not only the climate of the wind but also its velocity variation with height and the gustiness. All three quantities could be evaluated simply and inexpensively. The necessary field measurements could be conducted simultaneously with other geophysical explorations below ground (such as soil sampling), would cost no more and the design implications could be equally or more significant.

89. Summarizing, Prof. Davenport congratulated the Authors on their able summary of wind loading. Although much more was now known concerning how the wind acts on structures, the model that fitted the facts best was a statistical one, and it was still unclear as to how to embody this information in a rational design process: to bridge this gap, some dependence must still be placed on the empirical methods and the intuitive judgement of the engineer. While the velocity gust factors suggested in the Paper reflected many desirable features, their range of values was still not wide enough and did not take into account several influential factors: alternative values were indicated. The diversity of factors influencing the effective wind loading suggested the need for modifying the format of code specifications. The method of tabulation of shape factors given in the Paper was commendable. Attention was drawn to the benefits to be derived from the site investigation and competent analysis of wind conditions.

The Authors, in reply, were pleased to receive a contribution to the discussion from Prof. Haddon, who had carried out many investigations of wind pressure on buildings in the wind tunnels at the Royal College of Military Science, Shrivenham.

91. They agreed that further information was required on the effects of nearby buildings, and of protuberances such as mullions, chimneys, and towers on the wind pressures experienced over the walls and roofs of buildings.

92. Prof. Haddon's remarks on the influence of screening were of interest but it should be noted that the effect of wind-breaks depended crucially on their permeability and on their position relative to the building.

93. The values given in Table 10 were obtained from considerations of the most severe combination of internal and external pressure due to wind from any quarter. The data came from a number of sources, but were mostly based on tests with models mounted on the floor of the tunnel. In his example of the rectangular shed with axial wind and side doors, Prof. Haddon had found a case not covered adequately in Table 10, which should be amended to cover such a condition. The Authors were grateful to Prof. Haddon for pointing this out, and also the outward pressure coefficient of -5.0 for the corner regions of a grandstand roof. The fact that such cases could be missed, even after careful consideration of the problem, emphasized the value to the designer of a reference in the form of Table 10 giving pressure coefficients for the worst conditions that might arise.

94. Mr Eddie suggested that the value of the wind pressure p' for unclad structures, etc., as given in Table 6 of the present Code, was a dynamic pressure based on a gust factor of 1.25 applied to the minute wind speed. However, the Authors' information

was that the 1.6 factor introduced into the Table was in the nature of a shape coefficient and that the dynamic pressures on which Table 6 was based were derived from the minute wind speed. It might be noted that a similar shape coefficient of 1.6 was given in Table 9 for frames or trusses of solidity $\phi = 0.3-0.6$. This range of solidity was typical of that found in practice, allowing for the additional solidity due to ice coatings.

95. Mr Eddie's comments illustrated the difficulties which had been repeatedly shown to exist in the interpretation of the present Code and supported the Authors' approach to the wind-loading problem in separating considerations of the design wind speeds from those of the shape and pressure coefficients.

96. With regard to the comparison of the shape factors made by Mr Eddie in his final paragraph, it was evident from Table 5 of the present Code that its compilers adopted the shape coefficient for the circular section at sub-critical values of the Reynolds number, i.e. $DU_x < 50$. This was confirmed by the procedure recommended in the 1958 Amendment No. 1, where, allowing for the shape coefficient of 1.6 incorporated in the pressure p' of Table 6, the shape coefficient for the circular section became $1.6 \times 1.1 \times 0.7 = 1.2$, approximately. The corresponding value from the Authors' Paper was $1.2 \times 0.8 = 1.0$, approximately, for $DU_x < 50$; but only $0.7 \times 0.8 = 0.6$ approximately, for super-critical values of Reynolds numbers, i.e. for $DU_x > 50$.

97. The Authors believed that their treatment enabled a more complete assessment of the wind load to be made to suit a wider range of conditions. Finally, when a comparison was made between the wind loads deduced from the Code and from the Paper, account should be taken of the different bases of assessing the design wind speed.

98. The Authors were largely in agreement with Prof. Davenport on all the interesting issues which he raised. With regard to the basic definition of the design wind speed, the Authors would accept the definition he proposed, but would attempt to specify the chance of the design wind load being exceeded during the lifetime of the structure. Prof. Davenport's remarks with regard to statistical uncertainty, and to the effect on design wind loadings of a specified risk that this wind loading would be exceeded during the lifetime of the structure, might require some clarification. The use of the anticipated lifetime (n years) as the return period r (see equations (5) and (5a)) implied the acceptance of a 100% risk. To reduce this risk to $x\%$, the return period became $r = (100/x)n$, so that for a 5% risk of the design speed being exceeded in a lifetime of 50 years, the return period required was 1000 years. In this example an increase in design loads of about 35% resulted from the reduction in risk from 100 to 5%. The chance acceptable would vary with the usage of the building; and would be greater the cheaper the construction and the less the involvement of danger to persons. The aim of the Paper was to produce data from which the most accurate assessment possible of wind loads could be made in the present state of knowledge. In themselves, these wind loads contained no factor of safety. Safety factors must be applied, and in dealing with types of structure of which there was no previous experience, the safety factor would naturally be appropriately adjusted.

99. In selecting gust durations, the Authors had taken some account of the variation in the mean speed of the gust with the gust duration, and of the spatial extent of gusts; they had not felt it practical to introduce the time history of the gust loading and its effect in producing a dynamic amplification. The dynamic amplification was probably significant only for structures of low frequency and low structural damping, and the application was considered to be too complex for incorporation in a code.

100. Prof. Davenport's suggestion that it might soon be feasible to make a classification of gust factors as in Table 12 was of considerable interest and it was hoped that he would develop it further. At present the structural categories of

Table 12 were not sufficiently well defined and the qualities assigned to them appeared to be somewhat in contradiction to those of real structures.

101. As regards the form of presentation of the Code, the Authors were fully in agreement with Prof. Davenport that all possible steps should be taken to prevent the Code from becoming quickly obsolescent. They supported the view that numerical data which were subject to revision from time to time should be issued as a supplement to the main text which should deal primarily with methods.

CORRIGENDA

In the original Paper, on p. 111, equation (9) should read

$$U_z = F_{33} K V_{33} \left(\frac{Z}{33} \right)^{1/7}$$

On p. 123, the last sentence of the footnote should read:

'An approximate value for the frictional coefficient $C_f = 0.05$, and the total frictional drag on the roof becomes $f = 0.05 q a b$.'