DETERMINATION OF STRUCTURAL LOAD EFFECTS FROM WIND-TUNNEL TESTS

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Introduction

Wind-tunnel tests of structures have been routinely carried out for many years, and used in the structural design process. Usually the information provided to designers by wind tunnels is in the form of wind loads or wind pressures, rather than load effects, such as bending moments, axial forces, reactions, etc. Because wind flow and wind pressures are highly unsteady, varying in both time and space, there is usually not a simple relationship between local loads and load effects.

The fluctuations may excite resonant dynamic response if there is a resonant frequency low enough. In cases of very tall or long structures, the resonant response in one or more modes may be dominant, in which cases, the inertial forces distributed spatially according to the mode shapes are the dominant fluctuating loads experienced by the structure. For structures which are not strongly resonant, or those for which the resonant response is negligible, there remains a component of fluctuating wind loading, which affects the structure in a quasi-static way. There is no single spatial distribution for this fluctuating loading. The load distribution which produces the largest value of a structural load effect depends on the structural influence line for that load effect.

The following is intended to clarify the advantages and limitations of current practice, and describes an approach currently in use at CSIRO, and in Germany, which deals with the fluctuating part of wind loading, based on measurement of correlation between wind pressures acting on various parts of the structure.

Current Procedures

The current wind-tunnel practice in Australia has been to assume that structures fall into two categories - quasi-static and dynamic. For the first category, the designer is given local or uniformly-averaged peak (maximum and minimum) pressures acting on small areas of the structure. However, these peak pressures do not occur simultaneously over the whole tributary area of a large structural member or frame. But by applying them simultaneously to determine a critical load effect, e.g. bending moment, reaction, axial force, the designer is making a significant error. The direction of the error depends on whether the sign of the influence coefficient for the load effect all over the tributary area, is the same or not, and also whether the maximum or minimum pressure peak is used. Some examples of this problem are shown in Figures 1, 2 and 3. In Figure 1, a simply supported beam is shown - this may represent the structural system for a flat roof building. In this case, the influence line for the bending moment at centre span does have the same sign for loads applied right across the span, although clearly loads applied in the centre have a bigger relative effect on the bending moment. For this example, local peak wind pressures, with the peaks all taken in the same direction would be conservative, due to the fact that the peaks do not occur simultaneously - the fluctuating pressures are not fully correlated across the span.

The influence line for a bending moment at one point in a roof purlin, which can be regarded as a multi-span beam, is shown in Figure 2. The sign of the influence coefficient changes for loads applied on adjacent spans of the purlins, and maximum wind loads applied to one section should be combined with *minimum* loads acting on the adjacent section. Generally quasi-steady wind loads will be unconservative in this example. A conservative way of dealing with wind loads on roof purlins is to apply a series of load cases, each consisting of a single "patch" load acting on one section, with zero or lower load acting on the rest of the purlin. (A similar problem arises with guyed masts).

Another example of an influence line which changes sign is the arch roof shown in Figure 3. Again if quasi-steady wind loads from a code, or if local simultaneous peak pressures obtained from wind-tunnel tests acting in the same direction, are used, unconservative bending moments will result. A very conservative approach would be to apply the maximum peak pressures over those regions where the influence line is positive, and the minimum peak pressures where the influence coefficient is negative (or vice-versa).

The second wind-tunnel approach that is commonly used is to make a scaled structural model of the structure. Usually a dynamic model, which will reproduce any resonant effects, is used. This approach has several disadvantages :

i) Dynamic models are expensive to make

ii) Modelling at small scales can lead to inaccuracies and errors which cannot be corrected easily.

iii) The model is specific to one structural system, and is not valid if the structural system is changed during the design process.

iv) There may be measurement problems in measuring the required structural effects, such as deflections or bending moments on the model.

However when resonant response is believed to be significant, a dynamic model is usually a good investment. This only applies to those structures with at least one natural frequency well below 1 Hertz. In these cases, the total fluctuating loading is usually assumed to be distributed according to the inertial forces. However this is incorrect for the non-resonant or background part of the fluctuating loading.

A cost effective, flexible and accurate approach to the determination of quasi-static structural load effects is that developed at Ruhr University, Bochum, Germany and CSIRO, in recent years, and described in the following section.

<u>The CSIRO-Bochum Method</u>

The CSIRO-Bochum Method enables 'effective static' pressure distributions, which produce the maximum and minimum load effects (not local pressures) in a structure, to be obtained. The method is based on wind-tunnel pressure measurements in which the correlations between fluctuating pressures acting on the tributary area of the structural system are obtained. These are then processed with the structural influence coefficients to produce peak load distributions. It is only necessary to use distributions for those load effects which are critical, in the structural design process.
Kasperski (1992) made an important contribution when he showed that the peak load distribution for a load effect, $\hat{x}$, is given by :

$$(p_j)_{\hat{x}} = \bar{p}_j + g \cdot \rho_{x,pj} \cdot \sigma_{pj}$$

where $\bar{p}_j$ and σ_{pj} are the mean and r.m.s. pressures at point or panel, j,

$\rho_{x,pj}$ is correlation coefficient between the fluctuating load effect, and the fluctuating pressure at point j. This can easily be determined from the correlation coefficients for the fluctuating pressures at all points on the tributary area, and from the influence coefficients.

g is a peak factor for the response which lies in the range 2.5 to 5. It can be obtained by calibration and adjustment to match the recorded peak factors for the local pressures. Holmes (1992) showed that the application of the above equation can be simplified by using the eigenvalues and eigenvectors of the covariance matrix of the fluctuating pressures. The method has been implemented at CSIRO on this basis.

Figure 4 shows examples of peak-load distributions obtained, using the method, for an arch-roof (reproduced from Kasperski, 1992). The dashed and dotted lines show the load distributions for a support reaction at the right-hand end, and for a bending moment in the arch. These distributions are compared with the mean wind load distribution and the maximum and minimum pressure distributions. The latter form an envelope for the other distributions. It should be noted that the distribution for the bending moment, (which is at a position similar to point C in Figure 3) includes a region of positive pressure, unlike the mean pressure distribution. These load distributions will give accurate predictions of the design load effects required by structural designers.

Applications

The main features and advantages of the approach to structural wind loads outlined in this paper are as follows :

i) It is based on based on wind-tunnel pressure measurements from rigid models. The measurements do not require the structural system to have been established before the wind-tunnel tests, only the geometric shape of the structure. The model manufacturing cost is significantly cheaper than that for a structural or dynamic model, and the measurement procedures are more-straightforward.

ii) Calculation of the peak load distributions is carried out relatively simply on a personal computer using spreadsheet software.

iii) The method is based on sound aerodynamic and structural principles, and errors in the application of aerodynamic data to structural design are minimised.

The method has recently been successfully applied to several steel-framed low-rise portal frame buildings; in these cases, significant reductions in the critical load effects governing the sizing of the frame section were found compared to the values obtained using loads derived from the Australian Standard. In Germany, the method has been applied to an arched roof (as shown in Figure 4), and for several other structures.

References

J.D. Holmes (1992). Optimised peak load distributions. *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 41, pp 267-276.

M. Kasperski (1992). Extreme wind load distributions for linear and nonlinear design. *Engineering Structures*. Vol. 14, pp 27-34.

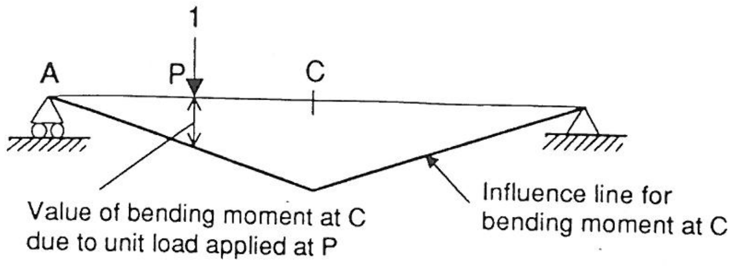

Figure 1. Influence line for bending moment on a simply-supported beam

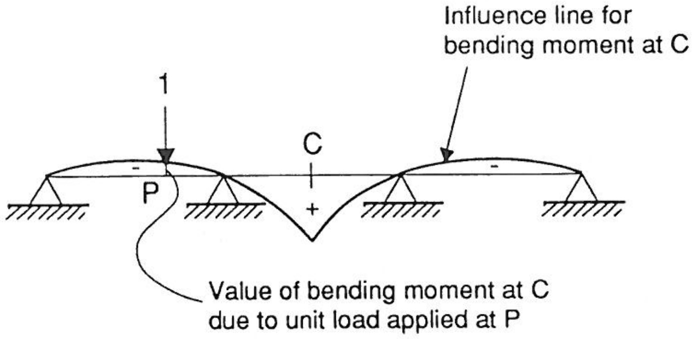

Figure 2. Influence line for the bending moment at the centre of a three-span beam

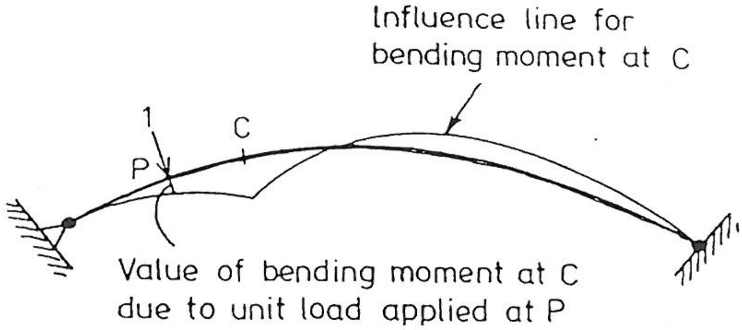

Figure 3. Influence line for an arch.

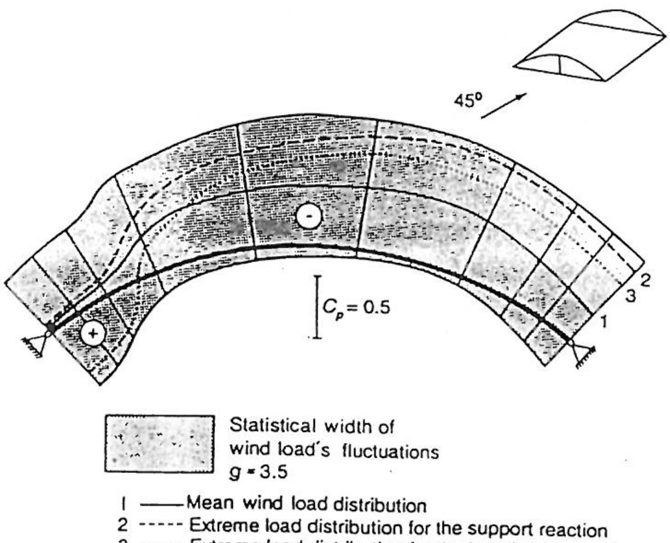

Figure 4. Peak load distributions for load effects on an arched roof (from Kasperski (1992))