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Front cover: Terminal building, Glasgow Airport (Photo: Henk Snoek)
Back cover: West Norwood Library lecture hall roof (Photo: Ernie Hills)

The background to the modern design of wind-sensitive structures

Ken Anthony

This article is a slightly modified version of a paper prepared by Ken Anthony for a seminar entitled 'The modern design of wind-sensitive structures' on 18 June 1970 at the Institution of Civil Engineers. The seminar, sponsored by the Construction Industry Research and Information Association (CIRIA), will consist of seven papers in all, together with three worked examples of the application of the wholly statistical approach to design for wind.

CIRIA is an organization supported by the construction industry which sponsors applied research and the dissemination of information on many facets of design and construction.

Introduction

Inevitably, statistics play a large and essential part in what the title calls the 'modern' design of tall wind-sensitive structures. In fact, the method to be described and illustrated is often termed 'the statistical approach' as opposed to the traditional, deterministic treatment with which we are all familiar. While the methods and ideas to be presented have been introduced relatively recently into the field of building structures, they have been employed for many years in other spheres, particularly in the design of aircraft structures. The principles were originally evolved from methods of analysis of electrical and radio signals.

A glance through the literature will reveal very many complicated mathematical expressions. This may give rise to the impression that the method of structural design to be outlined is also complicated and therefore difficult. Most engineers will have received some degree of training in the fundamentals of elementary statistics but, in practice, their experience rarely extends further than calculating the standard deviations of sets of cube results. It is true that the statistical theory behind the design method can be extremely difficult to understand, even when one is given ample time to study it. Practising engineers will not have this time and may therefore reject the statistical approach out of hand. This would be a mistake because the method to be described is not, in fact, difficult to apply and leads to a far better understanding of structural behaviour under the action of wind than any deterministic approach can possibly provide.

On the other hand, the procedures must not be thought of as some magical means of answering all the possible questions one might have about the behaviour of the structure. Engineers will be gratified to know that intuition and judgement are not yet redundant.

It is not *necessary* to be a statistician or a highly qualified mathematician to apply the design method with success. Few, if any, engineers fully understand the mechanics of shear in reinforced concrete beams, but this does not deter them from allowing for it in design. As with most things, however, an understanding of the concepts behind the theory and practice brings about dividends, not only in design competence but also in satisfaction.

This article attempts to provide some background and insight into the concepts and procedures which are described more fully elsewhere. Put simply, there are four questions to be answered.

1 What can this statistical approach achieve?
2 What is the design procedure?
3 What is involved in each step of the procedure?
4 What form do the results take and how are they interpreted?

These questions overlap each other to some extent, as will, inevitably, the answers.

1 What can the statistical approach achieve?

What concerns the engineer when designing wind-sensitive structures is how the structure will behave in the natural wind environment. His interest might range from whether or not the structure will collapse to the average number of windows which may break per year. Provided that the overall structure is reasonably secure against total failure, the centre of interest within the behavioural range depends, among other things, on the type of structure. Designers of microwave towers are very much concerned with keeping deflections within acceptable limits in order that the signals should not be diverted from their targets. They would not worry too much about the sway, although perhaps the maintenance staff would. Sway, however, is of concern when designing a tall hotel block, for in this case, accelerations at or beyond the threshold of perception would severely affect the revenue. At a more mundane, but not unimportant, level, one of the design criteria for a building might be to avoid the cracking of partitions. At all times the engineer is interested in possible values of bending moment, shear and stress. These are essentially questions of safety and reliability. Traditionally, it has been the practice to cover the likelihood of actual loads exceeding assumed loads by the application of arbitrary safety factors. When loads having a high degree of variability are being designed for, the shortcomings of this philosophy are clear. Either the applied factors are so high as to make the structure uneconomic or they are low enough to cause concern that the structure or some important part of it may fail. The choice of load factors is an exercise in the assessment of acceptable risk and has been mainly a matter of personal experience and judgement if it has been considered at all.

There are many factors which ought to be taken into account in the assessment of loads consistent with acceptable risks. These might include the cost of replacement of the structure or component, the consequent loss of life, revenue and prestige, the accuracy of, and the available time for, the design analysis, and the standard of workmanship in the construction. Accounting for all these factors is not, of course, easy. But even if they are not consciously thought about, at least the variability of loading should be positively considered and appropriate criteria determined on the basis of possible risks. No structure is free from the possibility of failure or damage. The loads must be designed to fit the risk.

Deterministic approaches ignore these factors. To design in such a manner to one given loading is often irrational and sometimes even dangerous. It can mean that one has little idea of the performance or safety of the element in question since so much information is ignored in the process. Although the new draft code for concrete design and the draft CP3 Chapter V recognize the variability of loading and strength, most other codes are formulated around a deterministic approach and may not recognize critical loadings. They do not cater for the 'Act of God' which could, without being irreverent, be considered as an event of low probability. Such an event, while not being serious for very small structures, would usually invoke a Court of Inquiry in the case of a structure of some magnitude. Compliance with such codes is, therefore, no guarantee of complete safety. We are, after all, concerned with real structures subject to real loads.

Most loadings and other parameters with which we are concerned in structural design are variable in value as are the strengths of any given material and the sizes of so-called identical units. There cannot be many engineers without experience of tolerance problems. In some cases, two or more different sorts of variation can compound themselves into a third problem. A typical example is the fitting of identical precast cladding panels which may in practice vary from their correct width, between columns which may be inaccurate in their spacing. Another, is the variation in loading on components of varying strengths, such as window panels or floor slabs. Wind is a prime example of the variability of loading. Not only does its general level rise and fall over extended periods of time, but because of its turbulent nature, it fluctuates rapidly over much shorter time intervals. Not only is its magnitude subject to variation, but also is its manner of application to a structure.

All natural wind is turbulent to some extent, even over the smoothest of surfaces. Although such turbulence is caused by many factors, the primary source is the mechanical mixing of the air as it passes over the ground. It is to be expected, then, that although the mean wind speed in a city will be less than that in surrounding countryside, the city régime will be more turbulent.

A structure placed in such a fluctuating airflow will respond in a similar, though not identical, manner. The magnitudes of stress and deflection will, therefore, also fluctuate. The only meaningful sorts of question the designer can ask himself in these circumstances are 'What is the chance of any given stress, deflection or other factor being exceeded during the life of the building?' 'How many times a year is it likely that a given deflection will be exceeded?', or 'What is the chance of fatigue failure in a particular component?' Again, these are questions of safety, reliability and risk. Through the use of probability theory, the statistical approach to design goes a very long way to answering such questions.

2 What is the design procedure?

It has been stated that the responses of the structure will vary in a similar fashion to the applied wind. They will not be identical because the presence of the structure itself in the airstream and the mechanical properties of the structure will modify the responses. Put very simply (Fig. 1), the concept is of a fluctuating load applied to the structure which then exhibits fluctuating responses.

A particularly illustrative example of this concept is that of an aircraft taxi-ing over a rough runway. The roughness of the ground (the input) is transmitted to the undercarriage which has structural properties of stiffness and mass. The undulations in the ground are thus passed through to the wings which again have structural properties of their own. If the wing-tip deflection were the response of interest, it would be observed that the tip also fluctuates (the output). These fluctuations would not be identical to those of the wheels because the entire aircraft structure modifies the input by what might be termed 'transfer functions'. It is clear that these transfer functions can modify the input to such an extent that the amplitudes of the wing-tip fluctuations can be several times larger than the magnitude of the roughness in the runway surface. With the aircraft moving relatively slowly on the ground, only the mechanical properties of the aircraft structure are effective in modifying the input. Had the aircraft been flying through turbulent air, the nature of the fluctuating forces would have been modified by both the aerodynamic and the mechanical properties.

It is the same with buildings, except that the object here is to keep them on the ground. Aerodynamic properties, in this sense, mean the turbulent drag characteristics—how the distribution of gusts over the building surfaces affect the total drag.

It is clear that, in order to describe the responses, it is necessary to describe the input. In between, both the aerodynamic and the mechanical properties must also be described so that modification of the input may be determined. This basically is what the design process consists of.

The applied wind speed is described statistically. This is modified by the aerodynamic characteristics of the structure to yield a description of the resulting force régime. This in turn is modified by the mechanical properties of the structure to produce a description of the response.

There is one fundamental limitation. We are dealing here with *linear* structures only. That is to say that, although the response is a modification of the input, both input and response are describable by the same statistical parameters. Furthermore, the response has a direct relationship to the input. For example, if the input is doubled, then so is the response. Non-linear structures can be dealt with by similar means but the process is far more complicated. Work is currently being carried out to develop an analytical method for these higher order structures suitable for use in design offices.

3 What is involved in each step of the procedure?

There are then, three basic steps to consider.

a The description of the input.

b The description of the properties of the structure.

c The combination of a and b to determine the output.

a *Description of the wind (input)*

We need to describe the nature of the applied wind speed so that the probability of any particular value can be calculated. Furthermore, we need to know something of its energy content and how this is made up so that we may have some knowledge of how the structure will behave. It is useful, for several reasons, to consider wind as having a mean value averaged over a period of one hour, upon which fluctuations in wind speed are superimposed (Fig. 2). The wind can then be described in two parts; the description of the mean wind and the description of the fluctuations.

These descriptions lead respectively to knowledge of the static and dynamic responses.

In practice, the description of both the mean

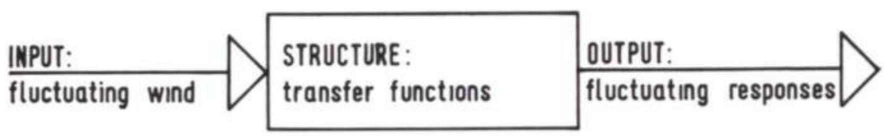


Fig. 1
Concept of design procedure

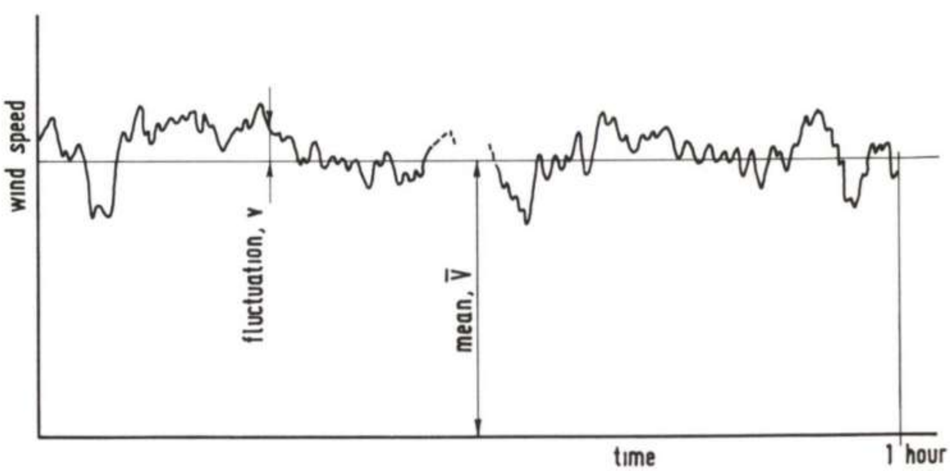


Fig. 2
Fluctuations superimposed on mean

wind speed and the fluctuating component is generally very simple. To determine the mean, a probability is chosen appropriate to the risk considered acceptable. The Meteorological Office will then normally be able to supply the required information derived from one or more nearby recording stations. It may be necessary to make minor adjustments to these figures to relate them to the particular site, but generally this is not difficult. If the site is abroad, where data exists but has not been processed to yield probabilities for wind speeds, then one will have to undertake this analysis oneself. We have computer programs which can handle most of this work for us. The fluctuating part of the wind speed is fully described by its *probability distribution*, its *power spectrum* and its *cross-correlation functions*. The probability distribution enables the chance of any particular value of the gust wind speed being exceeded to be determined. The spectrum describes the time-sequential action of the wind while the correlation functions indicate the way in which the gusts are spatially distributed over the building or structure. It will be seen later that these three parameters are interdependent. Fortunately for the design engineer, expressions for these functions have been advanced and generally accepted. All that is required of the engineer is the substitution into these expressions of constants appropriate to the wind régime, the site and the structure. Almost, that is, for although the manner in which the wind may be thus described is fairly well established, some degree of judgement and intuition is needed in applying it to one's own particular design problem.

It is in this description of the wind that some difficulty may be experienced in understanding some of the concepts and terminology used in the statistical approach. It may be useful, therefore, to provide some background before proceeding further. Firstly, let us look at some of the characteristics of the mean wind.

The mean wind

Why, for instance, should the one hour mean be an appropriate basis to use? Apart from certain meteorological significances, there are structural implications.

If the mean wind speed is to represent the source of the static load on the structure, then the choice of the averaging time to be used for the mean must depend upon several factors:

1 The chosen period should result in a reasonably constant mean speed.

2 The period should be short enough to contain the worst effects of a short duration storm.

3 It should be long enough for the structure to attain steady state conditions.

4 Data for the chosen period should be readily available.

5 The data should conform reasonably to appropriate probability theory.

The ideal averaging period has been given as between 10 and 15 minutes. This is because:

i This period lies well within the meteorological 'spectral gap', meaning that values of wind speed averaged over different periods of time within this range would not be very different from each other.

ii Sources of high winds such as thunderstorms and squalls which usually last from 5 to 10 minutes would be contained within the period.

iii Since the natural periods of structures are usually less than 10 seconds, 10–15 minutes is ample time for steady state conditions to develop.

However, very few meteorological organizations use such a period for recording data, but most do have data on the one hour mean. The above three conditions apply equally well to this period which has statistical reasons for being a most acceptable parameter, not least of which is that its data has been found to fit

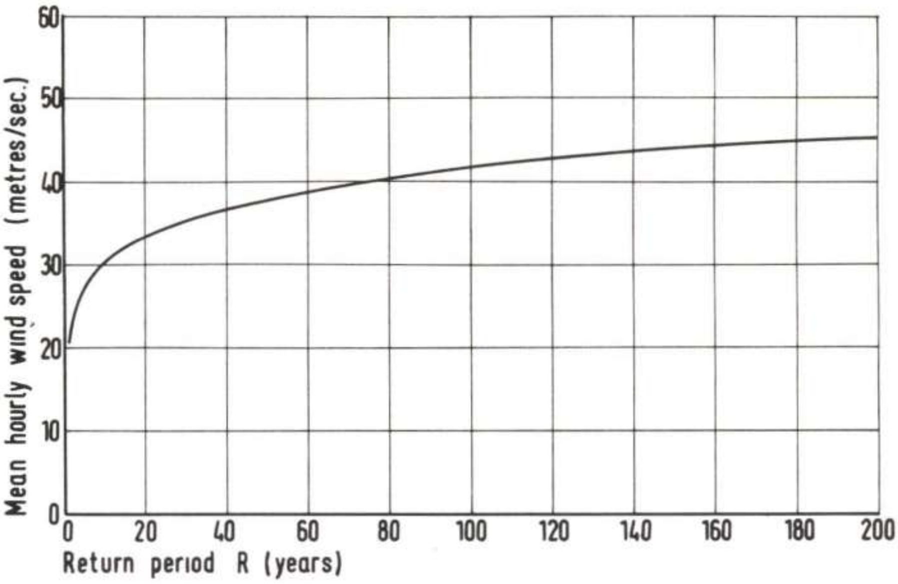


Fig. 3
Wind speed versus return period

probability distributions more than adequately well.

Although the principles described in this article may apply equally well to other averaging durations, the mean hourly wind speed will be used as the basis from which to start.

How is the mean hourly wind speed predicted? The mechanics of determining this have been dealt with in the past by several authors, notably Shellard, Thom and Davenport, but again, it may be useful to outline the concepts here. Basically, the highest mean hourly wind speed occurring each year is recorded at the observation station to provide a set of maximum values which can then be analyzed using extreme value statistics.

A minimum of about 15 annual values is necessary for reasonably accurate estimates to be made. Naturally, the greater the number, the more reliable are the estimates. Extreme values behave statistically somewhat differently from the population from which they are derived. Rather than fitting the Normal distribution, they may conform to a number of other laws such as the Fisher-Tippett Types, I, II (Fréchet) and III (Weibull) probability distributions. These are basically logarithmic transformations of each other.

Whichever distribution is found suitable there are various ways of handling the data. A particularly easy method, attributable to Lieblein, is by means of order statistics, but whichever method is adopted, the final result of interest for any particular location is a set of wind speeds with their associated probabilities (Fig. 3). The concept is rather like that of Olympic Games records. There exists now a record for the long-jump. In some years the record is not broken but when it is, the length jumped is greater than the previous one. Hence as time passes, the record distance can only increase—never decrease. It may be wondered whether there is some physical limit beyond which it is impossible to jump. Similarly, there may be some physical limit to wind speed, but it is clear that even if such a limit should exist, it would be too high to be of interest in structural design. As far as the theory of extremes is concerned there is no upper bound—only lower and lower probability for higher and higher wind speeds.

These considerations apply equally well to other time averages and highlight the fallacy, still existing among some engineers, that the 'highest recorded gust' is a meaningful criterion to adopt.

Mention should be made of a promising development by Thom who has found that extreme wind distributions may well have particular characteristics which enable predictions to be made on the basis of maximum mean monthly wind speeds. The significance of this is that it may not be necessary to accrue many years of annual extreme data. The readily available monthly means may be used in assessing annual probabilities.

So for any particular location where annual extreme data is available, the wind speeds relating to any chosen probability can be determined. If this exercise is carried out for a large number of observation stations throughout a country, a set of contours or isotachs can be drawn for each probability. This was done several years ago by Thom for the USA and more recently in this country by Shellard and Helliwell of the Meteorological Office. The wind map in BRS Digest 99 and the new draft code is an example. In these cases, the map indicates not mean hourly values, but three second gust speeds associated with an annual probability of 0.02. Naturally, as time goes by and more data is collected, these maps need to be revised.

It is common practice to use the terms *recurrence interval* or *return period* when talking of annual probabilities. The relationship is simple. If $F(\bar{V})$ is the probability of the extreme wind NOT exceeding the value $\bar{V}$, then $1 - F(\bar{V})$ is the probability of it being exceeded at least once in any given year. The return period, expressed in years is then,

$$R = \frac{1}{1 - F(\bar{V})} \quad \ldots\ldots\ldots (1)$$

There still exists some misunderstanding of the meaning of the return period. In effect, it is the average interval of time between occurrences of a particular wind speed being reached or exceeded at least once during an infinite period of time. There is no periodicity implicit in this definition. One cannot say that a particular value will occur every 50 years, 100 years, or whatever. One cannot say *when* it will occur. It could be today, it could be entirely outside the return period, or it could happen any number of times within the period.

Although it may not always be consciously considered, buildings have an anticipated lifetime. Some structures are designed to a specific length of service. In New York, this is often as low as 30 years. The only really meaningful statement that can be made is that a given wind speed has a certain chance of occurring or being exceeded within a specified number of years. It is clear that this chance must become greater as the specified number of years is increased.

As this principle is of the utmost importance to a probabilistic approach, it is worthwhile looking at it a little closer. The question a designer may ask himself is 'What return period, R, should I use in designing my structure which has an anticipated life of n years if I am willing to accept a risk, P_n, of a wind speed, $\overline{V}$, being exceeded during the n years?' The risk or probability, P_n, may be given by

$$P_n = 1 - [F(\overline{V})]^n \quad \ldots.(2)$$
$$= 1 - [1 - 1/R]^n \quad \ldots.(3)$$
$$= 1 - e^{-n/R} \text{ (approx.)} \quad \ldots.(4)$$
$$= n/R \text{ (approx. for small values of n)} \quad \ldots.(5)$$

from which R and hence the design wind speed may be calculated or read off an appropriate graph such as Fig. 3.

It is often thought that the probability of, say, a 50 year return period wind occurring within any given interval of 50 years must be unity. Application of equation (3) will reveal that the chance is, in fact, 0.63. This is what is meant by the 0.63 probability level in the BRS Digest 101 and new draft CP3 Chapter V in choosing the S_3 factor. Whether or not this 2 in 3 chance can be considered as an acceptable risk is a question of load factors.

Equations (3) and (5) are also useful when determining suitable loadings for components as opposed to entire structures. It might be reasonable, for instance, to consider replacing a few damaged windows or light cladding panels every few years rather than overdesign all of them. In assessing the acceptable risk, the consequences of failure call for considerable thought. The principle of acceptable risk and length of service is valid for a multitude of problems associated with variable loadings and strengths. The portion of the wind loading which gives rise to the static response of the structure can thus be dealt with, having chosen a probability suitable to the risk one wishes to take for the design of the structure or element in question.

Let us now turn our attention to the fluctuating part of the wind load.

The fluctuations (gusts)

It was stated earlier that the fluctuating part of the wind régime could be fully described by the probability distribution, the power spectrum and the cross-correlation functions. Before an insight into the meaning of these terms may be gained, it is necessary firstly to make a number of simplifications in order that the ensuing analysis may yield results without too much difficulty. Secondly, it is necessary to consider a few other parameters which lead naturally to the spectrum and cross-correlation functions.

Referring again to Fig. 2, the variable which constitutes the fluctuating wind speed about its mean value is termed a *continuous time series*. Such a series, describable only through its statistical probability properties, is known as a *stochastic* process. The most important of the simplifications to be made are that a, the process is *stationary* and b, the process may be described adequately by the more fundamental properties of its probability distribution. To expand on this,

a

A stationary random process is one in which the average properties of the variable remain unchanged even though the value of the variable behaves in a random manner. In other words, no 'trends' are displayed during the period under consideration. The significance of the term 'stationary' can be appreciated if it is remembered that our basic physical model of a high wind is a mean flow stirred mechanically by the roughness of the ground over which it passes. A laboratory analogue of this model is a wind tunnel with a roughened floor. In such a tunnel, the mean flow is determined by the fan setting and the turbulent fluctuations superimposed on the mean are generated by the floor roughness. Suppose that in such a tunnel, the fan setting and the floor roughness were maintained and a number of records were obtained in the form of traces of wind speed against time, all the records would be different but simple averages such as the standard deviation and the root mean square would be almost identical for all the records. It follows then that the average properties determined from one record yield information about the average properties of the entire collection of records. In statistical terminology, any one of the records is called a *sample* or *realization* taken from an infinite collection or statistical *ensemble* of records. Such an ensemble is stationary if the external controlling factors, the fan setting and the floor roughness, remain the same.

In the case of our model of atmospheric turbulence, it is assumed that a record, obtained in high wind conditions with given mean hourly wind speed and ground roughness, is one sample from the stationary ensemble of records which could be obtained at the site. Deductions about the average properties of gusts contained in the sample are assumed, therefore, to be generally applicable to all such records.

b

It has already been inferred that a structure will be responsive to the time sequential manner in which the fluctuating load is applied and that the pulsating nature of gusts can give rise to magnifications in levels of stress and deflection, etc. It was also implied that, should the structure be large, long or high, then because of the turbulent nature of the wind, gusts will not occur simultaneously over the face, length or height of the structure. That is, the gusts will not be fully *correlated* over the structure.

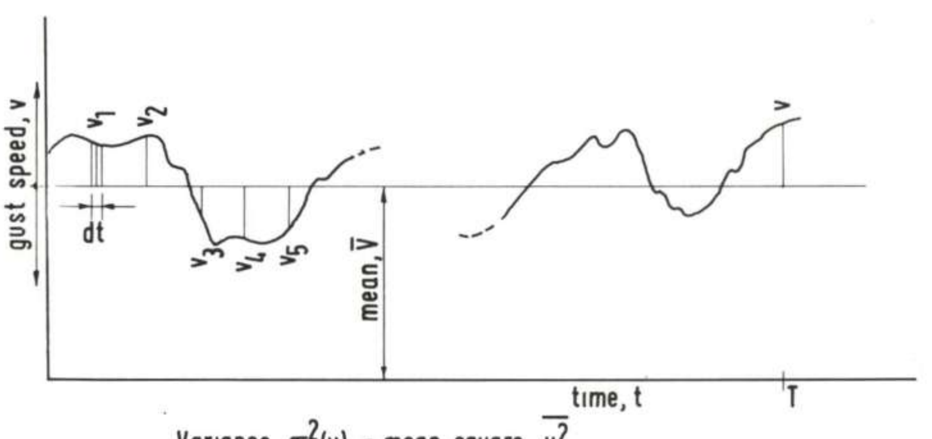
We must then, have some knowledge of the power spectrum and correlation functions in order to describe these sequential and spatial actions of the wind. It is well known that however random a continuous time series may be, it is theoretically possible by Fourier analysis to break down the series into separate sine or cosine forms of differing frequencies and amplitudes. One way of visualizing the power spectrum is to think of it as a means of describing the manner in which the total wind energy is distributed over the entire range of these frequencies contained in the fluctuations. In order to arrive at a fuller understanding of the meaning of the spectrum, it is convenient to consider the fundamental properties referred to previously. In fact, if the power spectrum were to be estimated theoretically, the step by step procedure would be to make separate estimates of these fundamental parameters.

It has already been explained that the structural engineer need not concern himself with the theoretical determination of these parameters to design his structure. The following brief descriptions of the statistical properties of the stationary process which constitutes the fluctuations in wind speed are offered as an aid to the understanding of the concepts and terminology commonly used.

To simplify the explanations, a single wind record will be considered. This restricts us to the study of the properties of the fluctuations in the time sense and in the direction of the mean wind, but the concepts apply equally well to the other wind speed vectors implicit in the turbulence. The considerations are illustrated in Figs. 4 to 11 where the mathematical expressions will also be found for each parameter. It should also be made clear that, as the statistics of the mean wind speed have already been dealt with, we are concerned now with fluctuations only. That is, the random variable now in question is the wind speed relative to the mean—not the absolute value.

The variance, $\sigma^2(v)$

The variance of the random function may be physically derived from a wind record by summing the squares of a large number of gust speeds and dividing by the number of values taken. The result is the *mean square*, $\overline{v^2}$, of the gust speed, Fig. 4.



$$\text{Variance, } \sigma^2(v) = \text{mean square, } \overline{v^2}$$
$$= \frac{1}{N}(v_1^2 + v_2^2 + \cdots + v_N^2) \qquad [N \text{ large}]$$
$$\text{or more precisely} \quad = \frac{1}{T}\int_0^T v.dt \qquad [T \text{ large}]$$

Fig. 4
Representation of variance

The term *standard deviation* is familiar to most engineers. The variance is the square of the standard deviation and represents a measure of the spread or dispersion of the gust speeds about their mean. It is the variance which is minimized by the method of least squares used when fitting lines to experimental data. The object is to find the line which produces the least scatter. The standard deviation is more often referred to in this context as the *root mean square*, $\sigma(v)$, of the fluctuations. The 'gustiness' of the wind is represented by the term *intensity of turbulence* which is measured by the ratio of the root mean square of the gust speeds to the mean value of the wind speed.

The autocovariance function, $C^v(\tau)$

This is a most important function in the analysis of continuous random series because it not only leads directly to the power spectrum but it also leads to a measure of the time scale, or correlation in the time sense, of the random process.

If one were to undertake the tedious operation of calculating values of the autocovariance function by hand from a long continuous record of a gusty wind, one would note a large number of instantaneous wind speeds and then note the values at a given interval of time or *lag* later. The products of the paired values would then be summed and the mean found by dividing by the number of pairs considered. Naturally, the greater the number of pairs of values considered, the better would be the estimate. The result of this operation is a value of the autocovariance function for the one particular lag chosen. Fig. 5 illustrates one way of doing this by shifting the record trace in time by an amount equal to the lag.

Since the process under study is stationary, the value of the autocovariance will be independent of time and a function only of the time lag. This means that the autocovariance depends only on the length of time between the two points, not *where* they are chosen.

If the same procedure were carried out for several different lags, one could plot the autocovariance values against the various time lags (Fig. 6). It is to be expected that as the time lag is increased, the autocovariance function will tend to zero since, as the process is random in character, one would not expect any correlation to be revealed between values separated by extended periods of time. Furthermore, the autocovariance must be an even function. In other words, the above procedure could be carried out by considering negative values of lag with the same results, except that in this case, a mirror image would be obtained.

It is interesting and useful to note that if the lag is given the value zero, the autocovariance, $C^v(0)$, becomes equal to the variance or the mean square value of the fluctuations.

The autocorrelation function, $\rho^v(\tau)$

It is often convenient when comparing time series with different scales of measurement to 'normalize' the autocovariance, by dividing by the variance. The ratio is commonly termed the autocorrelation function. Expressed graphically (Fig. 7), it displays a decay form in exactly the same manner as the autocovariance except that the ordinate scale is reduced. Since the autocovariance at zero lag equals the variance, the autocorrelation function must have unity value at zero lag. Put another way, there must be 100% correlation when the lag is zero.

Clearly, the autocorrelation function is also even and constitutes a measure of the time scale of the random process. That is, it is a measure of the time distance over which there exists a dependency between mean values of the fluctuations. To be more precise, the time scale is defined as the area under the autocorrelation curve.

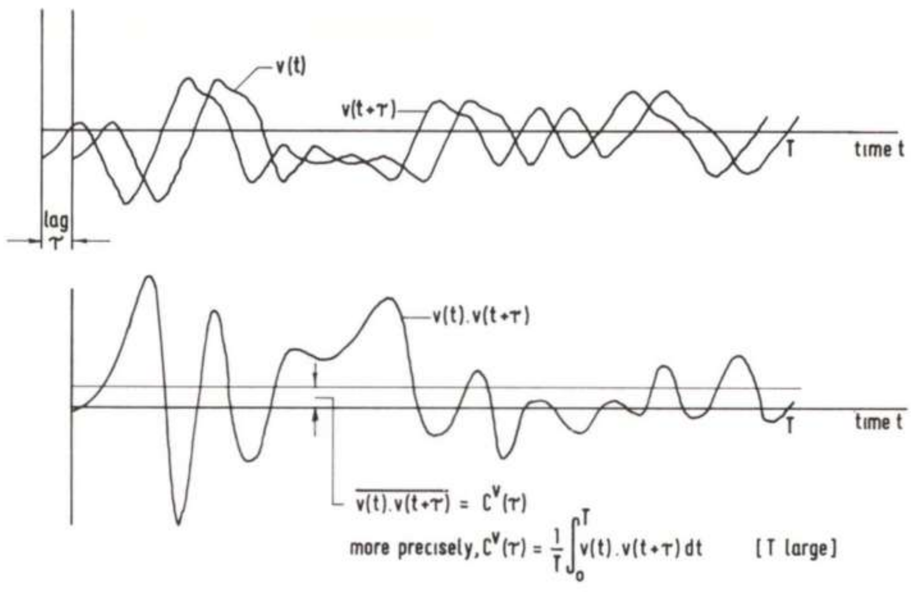


Fig. 5
Graphical representation of autocovariance for lag τ

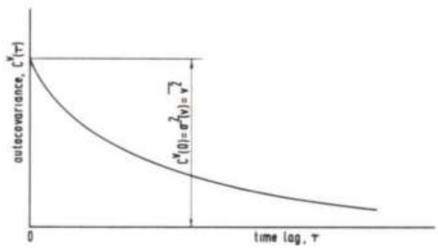


Fig. 6
Decay of autocovariance function

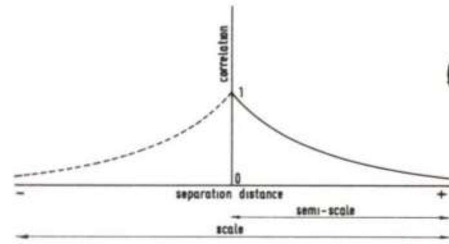


Fig. 8
Spatial correlation

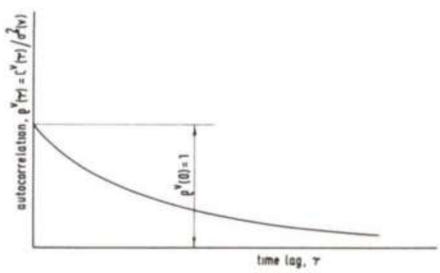


Fig. 7
Decay of autocorrelation function

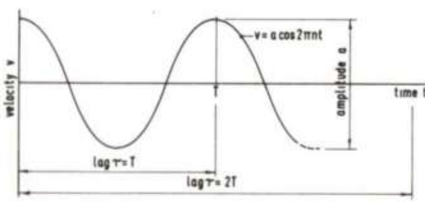


Fig. 9
Autocovariance of a cosine function

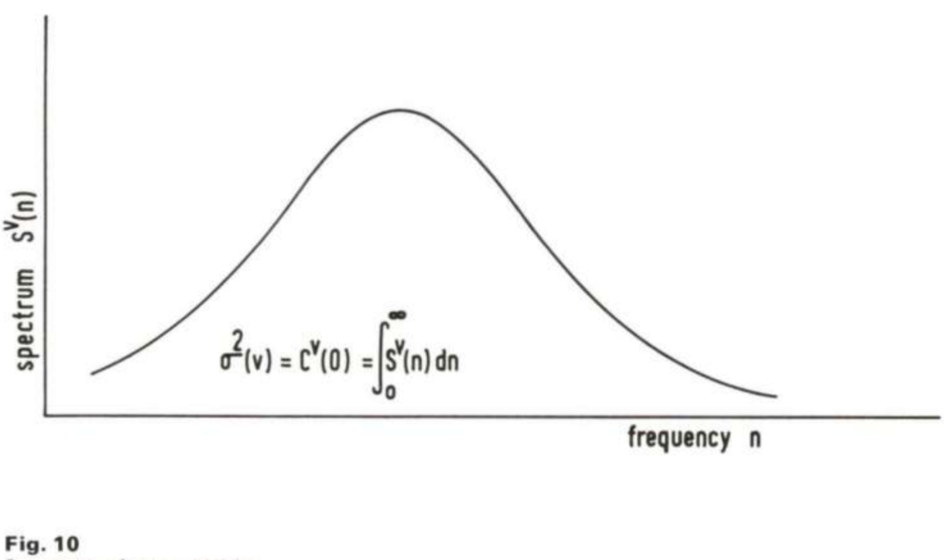


Fig. 10
Spectrum of gust velocity

Space correlations

All that has been described above has, for simplicity, been restricted to considerations of correlations in time only. This has had to be so because only a wind trace which might be obtained from a single anemometer in space has been studied. This is also why the prefix 'auto' has been attached to the terms 'covariance' and 'correlation'. The operations described have been carried out on the single random process itself. It is clear, however, that similar operations may be carried out on the fluctuations occurring at two different points in space. In this case, the pairs of values are formed from the two separate records. Correlations, then, may be defined in space as well as time. It is convenient to mention space correlations at this juncture, before turning to the power spectrum, because the concepts follow directly from those previously discussed.

Instead of considering, as we have done, the time correlation between a down-wind (longitudinal) component of wind speed with respect to time lags, one could evaluate the correlation between the same components recorded at two different points in space but with respect to the distance between the two points. One could visualize doing this in practice by setting up one anemometer in a fixed position and another capable of being moved to any location in space. Simultaneous gust speeds could then be recorded at each anemometer. If the two instruments were set up at the same height and the simultaneous longitudinal components of the gust speeds were recorded as the mobile anemometer was moved in steps transversely to the mean wind direction, the across-wind correlation of the longitudinal component could be obtained. Similarly, the vertical correlation could be evaluated by locating the mobile anemometer at various heights above the fixed one.

Consideration of the longitudinal component in the longitudinal direction (at constant height) brings us back, in effect, to the autocorrelation function since the spatial separations are equivalent to time lags consistent with the mean flow velocity. In a turbulent airstream having a mean speed of, say, 40 m/second (131 ft./second), a spatial separation of 40 m is equivalent to a time lag of one second. The turbulent wind régime is assumed to be carried down-wind at the mean wind velocity. This assumption, known as Taylor's Hypothesis, does not apply vertically, because mean wind speeds increase generally with height.

The correlations being discussed are known as *cross-correlations*. More precisely, as simultaneous gust speeds are being described, the correlations are *cross-correlations at zero lag.* As might be expected, the spatial correlations, like the time correlations, exhibit a decay form and provide a measure of the *scale* or *eddy size* of the turbulence (Fig. 8). The concept may be likened to that of a street of houses. Next-door neighbours often influence each other in some respects, but as the separation distance increases, any such influence decreases until the distance is so great that no effect is felt at all.

Extending the concept further, it is possible, between two points in space, to evaluate spatial correlations between different components of the gust velocities. Furthermore, one is not restricted to cross-correlations at zero time lag. One could correlate one component at one point with another component at another point at various intervals of time later. In all, as there are three components at each of the two points in space, there must be nine different cross-correlations that could be performed. Which of these is of interest to a structural designer depends on the geometry of the structure and on the mode of behaviour under consideration. The designer of a tall slender building would be interested in the vertical correlations of longitudinal and transverse components because he would be concerned with the down-wind and across-wind responses of the structure. A suspension bridge designer would study the horizontal correlations of the longitudinal and vertical components.

Although, strictly speaking, there are nine different cross correlation functions, the acceptance of two basic assumptions can reduce the number to two. The first assumption is that of Taylor's Hypothesis already mentioned. The second is that the turbulence of natural wind is homogenous and isotropic. These assumptions have been found experimentally to have validity. The overall effect is to simplify the considerations, and therefore the design procedure, greatly.

It will be recalled that the statistical method of design being described is concerned with both time and spatial correlations. The first enables us to take account of the repetitive nature of gust loading and the second to allow for the non-uniform manner in which wind acts over a tall or extended structure and/or across the faces of a large building. Both forms of correlation have been discussed but the autocorrelation function, while being a suitable parameter to describe the time correlation at a single point in space (a structural example of which is a small bank of arc lights on a single slender column), it is not the most convenient parameter for use with other structures. A better approach is to use the power spectrum in conjunction with the spatial cross-correlations.

The power spectrum, $S^v(n)$

To gain an insight into the meaning of the power spectrum, let us return briefly to the autocovariance function. Let us also consider a very simple fluctuating velocity function (Fig. 9), of the cosine form,

$$v = a \cos 2\pi nt \quad \ldots\ldots\ldots (6)$$

where a is the amplitude, n is the frequency, t is time and T (=1/n) is the period. If the autocovariance were to be calculated by taking N pairs of values of v separated by a time lag $\tau = T$, the result would be

$$C^v(T) = \frac{a^2}{2} \quad \ldots\ldots\ldots\ldots (7)$$

Since the values of v are equal to a for $\tau = T$, 2T, 3T, etc., it is clear that $v = a$ at $\tau = 0$. It will be remembered that for zero lag, the autocovariance becomes equal to the variance, which in this case of a cosine form will also be equal to $\frac{a^2}{2}$.

This is clearly a measure of the kinetic energy or average power contained in the wind fluctuations.

In nature, however, wind speed does not display a cosine form; it is random in character. Fourier tells us that no matter how irregular a function may be, it can be broken down into separate cosine forms of differing frequencies and amplitudes. For such a train of waves, the variance becomes

$$\sigma^2(v) = C^v(0) = \sum_{i=1}^{N} \frac{a_i^2}{2} \quad \ldots\ldots (8)$$

where N is the number of cosine waves present. The variance of a time series containing a mixture of cosine waves can, therefore, be regarded as being made up of components of the average power or variance at the various individual frequencies. All the frequencies contribute to make up the whole spectrum of variance or power. There will be so many individual frequencies in a random function that one can talk of there being a continuous range of frequencies. The *power spectrum* may, then, be defined by

$$\sigma^2(v) = C^v(0) = \int_0^\infty S^v(n)\, dn \ldots (9)$$

where $S^v(n)$ is the spectrum of gust velocity at frequencies n (Fig. 10). The area under the spectrum equals the variance.

For those more mathematically inclined, the autocovariance function and the power spectrum constitute a Fourier Transform pair—each is a Fourier Transform of the other. Hence, knowledge of the autocovariance function means that the spectrum may be determined. A simple analogy to illustrate the physical meaning of the spectrum is that of a set of tuning forks, arranged in ascending order of their natural frequencies, subjected to random noise. Each tuning fork will vibrate sympathetically with that frequency in the noise which coincides with its own. The energy absorbed by a particular fork is a measure of its particular contribution to the total energy or power.

If the values of these individual energies were plotted against their individual frequencies, a graph (Fig. 11) of the manner in which the energy was distributed over the frequency

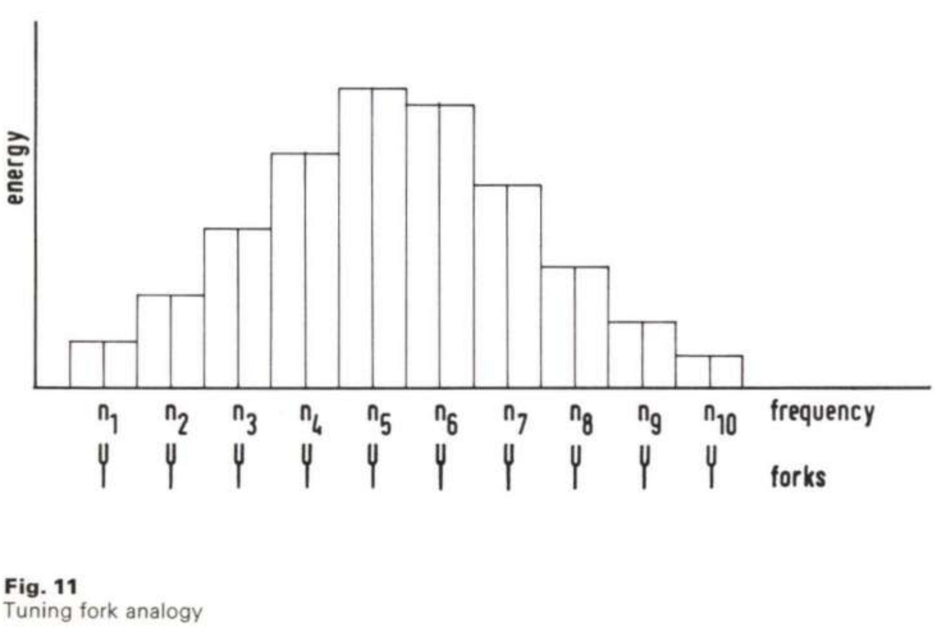


Fig. 11
Tuning fork analogy

range would be obtained. The only difference between this illustration and the concept of the power spectrum lies in the fact that tuning forks have discrete frequencies of their own, whereas the power spectrum is a *continuous* function over a *continuous* range of frequencies.

As was the case with the autocovariance function, it is sometimes convenient (for example, when comparing time series having different scales of measurement), to 'normalize' the spectrum by dividing by the variance.

The resulting expression, $\frac{S^v(n)}{\sigma^2(v)}$, is often termed the *spectral density function* or *normalized spectrum*, which, with the autocorrelation function, forms a Fourier Transform pair.

In exactly the same way that autocorrelations can be found for the three vector components of the gust speeds, so there are spectra for each component. Furthermore, cross-spectra can also be formed between different components recorded at different points in space. Detailed discussions of these functions and their uses will be found in the general literature.

All that has gone before has been an attempt to provide a background to the development and concept of power spectra and their associated correlation functions. It is worth repeating that it is not necessary for the designer to analyze records of wind speed from first principles. Expressions defining all the functions in terms of simple parameters are available in a suitable form for use in the design of many types of structure.

To recapitulate, the mean wind is described by the probability distribution of the annual extreme values of the hourly average. The gusts are described by the probability distribution, the cross-correlation functions and the power spectrum of the fluctuations. Now what of the structure itself?

b *Description of the structural properties*

Now that the properties of the wind have been briefly discussed, attention can be turned to the properties of the structure which modify the wind input to yield the responses of interest to the designer. These responses, it will be recalled, are usually those of stress and deflection but may also include acceleration and the fatigue effects of recycling of loads. Earlier in the paper, the modifying properties of the structure were referred to as 'transfer functions'. This concept is directly analogous to that of a helical spring of stiffness k, to which a static axial load, P, is applied. The deflection, d, of the spring would be given by

$$d = k \times P \quad \ldots\ldots\ldots\ldots (10)$$

The stiffness, or spring constant is a transfer function and

$$\text{Output} = \text{transfer function} \times \text{input} \quad . (11)$$

If a series of experiments were performed on this spring in which the loads, instead of being statically applied, fluctuated in simple harmonic form at one given but different frequency in each experiment, it would soon become apparent that the dynamic manner in which the spring deflected depended upon the frequency of the applied force. In other words the transfer function was frequency dependent. A particular and obvious case of this dependency is when the frequency of the applied force coincides with the natural frequency of the spring.

Natural wind, however, is not simple harmonic in character but, as we have seen, can be considered to contain a whole spectrum of frequencies. If the structure is considered elastic and its transfer functions frequency dependent, then its responses will be subject to frequency also. Just as the wind is describable through its mean, probability distribution, and spectrum, so are the structural responses. The mean response is obtained from the mean wind in the usual traditional manner. The probability distribution is defined by the mean and the standard deviation, which is the square root of the variance. The variance is the area under the spectrum curve. However, before the response can be determined, the transfer functions need to be described. There are, essentially, two transfer functions to consider, namely,

a the mechanical 'admittance', and

b the aerodynamic 'admittance'.

The term 'admittance' is really a legacy from the original developments of the methods of analysis of electrical signals but has found general use also in the structural field, with respect to the input frequency. In essence, the two functions each represent the ratio of the variances (or mean square values) of the amplitudes of the output and input of a system to which the applied input is sinusoidal in character. In the process of structural design, the order in which the two functions are considered is the reverse of that shown above, but as the mechanical function is the more easily understood, it will be explained first.

Mechanical admittance, $|\chi_m(n)|^2$

Reverting to the spring analogy, if the force were to be applied as a simple harmonic (sinusoidal) function, the variances of the amplitudes of the deflection and load could be determined. The ratio between these values would represent the mechanical admittance at that particular input frequency. Repetition of this experiment at different input frequencies would result in a graph of mechanical admittance plotted against frequency. This graph would reveal two properties of the function, namely its frequency dependence and its expected tendency to 'peak' at the spring's natural frequency.

A typical result for a structure having a well-defined natural frequency is given in Fig. 12. It is clear that the structural properties of mass distribution, stiffness and damping characteristics have a profound influence on the mechanical admittance. Therefore, reasonably accurate assessments of these properties are called for. It follows that the response is very much dependent on these structural properties.

Following the concept of equation (11), since in the case of a structure placed in a turbulent wind, the applied force and the structural responses are random in nature and can be defined by their individual spectra, their relationship can be expressed as

Output spectrum=
transfer function x input spectrum . . . . (12)

or, for example,

Deflection spectrum=
mechanical admittance x force spectrum(13)

$$\text{i.e.}\ \frac{S^v(n)}{Y^2} = |\chi_m(n)|^2 \cdot \frac{S^f(n)}{F^2} \quad \ldots (14)$$

The spectra are normalized because they are measuring different physical quantities. Equation (14), as shall be seen, represents a step in the overall procedure.

Aerodynamic admittance, $|\chi_a(n)|^2$

The aerodynamic admittance is a rather more difficult function to describe and evaluate, but in essence, it is a measure of the effect that turbulence has on the drag forces in relation to those which occur in steady flow conditions. It basically describes two principle effects; the manner in which the applied forces vary spatially over the structure and the effect on the force coefficients arising from the fluctuating nature of the applied wind. As such, it is part of the 'transfer function' between wind velocity and the resulting pressures or forces on the structure.

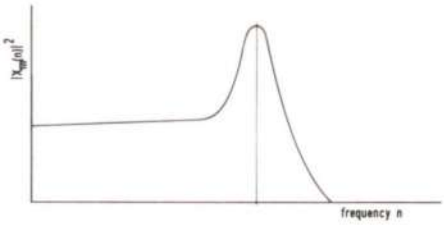


Fig. 12
Mechanical admittance

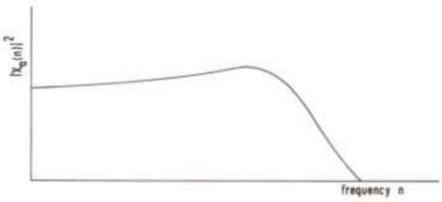


Fig. 13
Aerodynamic admittance

The physical characteristics of airflow around a building depend to some extent on the scale of turbulence. It is well known that the width of the wake to leeward of a building in steady flow conditions is a measure of the consequent drag. Turbulence introduced into the flow can have the effect of causing reattachment of the flow. This in turn affects the wake width and hence the drag. It is to be expected that the aerodynamic admittance is dependent upon the form of the structure. A lattice tower will have a value different from that of a clad structure. It is also not surprising that the function is frequency dependent. Experiments on flat plates in turbulent flow have indicated, for example, that the drag coefficient tends to rise as the frequency implicit to the turbulence is increased. This is offset to some extent, however, by the decrease in the correlation of velocities over the face of the structure as the frequency rises. At zero, or low frequencies, that is in steady or near steady flow conditions, the velocities can be said to be fully correlated. In such a case, the gusts have wave lengths in excess of the structure's dimensions. The structure will then respond to these low frequency fluctuations. On the other hand, very high frequency gusts lead to low correlation. The structure fails to respond to these, thus reducing the value of the admittance. It is not certain which tendency predominates, but it is fairly clear that the admittance falls to zero at very high frequencies (Fig. 13). Research is being undertaken to clarify this point for different types of structure.

One way of visualizing the aerodynamic admittance, then, is to consider it as the ratio between the force coefficients in turbulent and steady flow. Expressed mathematically, this may be written

$$|\chi_a(n)|^2 = \frac{C_F(\xi)}{C_F(0)} \quad \ldots\ldots (15)$$

Where ξ is referred to as the reduced frequency $\frac{nD}{V}$, D being an appropriate structural dimension. $C_F(0)$ is, of course, the force coefficient applicable to steady flow.

Because of the tendency, outlined above, for one effect to cancel the other as the frequency rises, designers often assign the value of unity to the aerodynamic admittance. This usually has some validity as the rapid falling off normally occurs at frequencies higher than the structure's natural frequency around which most interest lies.

It was stated earlier that the aerodynamic admittance was *part* of the 'transfer function' between wind velocity and pressure or force on the structure. If one were to consider the relationship between force and velocity at a *point* in fluctuating flow, it could be shown that their normalized spectra are related by

$$\frac{S^f(n)}{\bar{F}^2} = 4\,\frac{S^v(n)}{\bar{V}^2} \quad \ldots\ldots(16)$$

The full relationship for a structure of finite size then becomes

$$\frac{S^f(n)}{\bar{F}^2} = 4|\chi_a(n)|^2\,\frac{S^v(n)}{\bar{V}^2} \quad \ldots(17)$$

Equation (17) represents the remaining step in arriving at the spectrum of the response.

c *The determination of the output*

It is now possible to combine all the preceding concepts and descriptions into an overall design procedure which results in the output spectrum of response being determined. Following the principle of response spectrum = transfer functions x input spectrum, the entire design procedure can now be summarized by the expression

$$\frac{S^y(n)}{\bar{Y}^2} = 4|\chi_a(n)|^2|\chi_m(n)|^2.\frac{S^v(n)}{\bar{V}^2}\cdot \quad (18)$$

where y is the response of interest.

A graphical representation of this procedure is given in Fig. 14. It would, of course, be very tedious to actually undertake this procedure graphically by hand. The method lends itself ideally to computer techniques. It is in fact possible for some computers to obtain the response in a fraction of a second.

4 What form do the results take and how are they to be interpreted

It will be recalled that the area under the spectral curve of a function is equal to the variance of that function. The variance of the response is therefore found from the area under the response spectrum. This is equivalent to integrating equation (18). Knowledge of this variance provides a complete description of the probability distribution of the response. It is possible, therefore, to determine the magnitude of the response associated with any chosen probability. A usual method is to define the chosen probability or acceptable risk in terms of a number of standard deviations from the mean value. Most engineers will be interested in probable peak values of the total response. This standard deviation approach can be expressed as

$$Y_{\text{peak}} = Y + k\,.\,\sigma(y) \quad \ldots\ldots(19)$$

where k is the number of standard deviations chosen. The standard deviation, $\sigma(y)$, is directly calculated from the variance and is represented by the square root of the area under the response spectrum. The response associated with acceptable risk can thus be evaluated by means of simple probability theory.

The form of the response curve is fairly self-evident. Comparing it with the form of the force spectrum, one would expect the shape of the response curve to follow that of the force at the low frequency (near steady-state) end. However, as the frequency approaches the natural frequency of the structure, the response rises sharply to a peak. This means that if peak responses are of interest, and they usually are, then these will normally be found close to the natural frequency of the structure. The height of the peak depends upon the amount of damping present. It is often found that, although the actual values of response are random and are defined by the normal distribution, there is a strong periodicity in the response (in the time sense), the frequency of which is close to the natural frequency of the structure.

Conclusion

This paper has attempted to provide a background to the wholly statistical approach to design, applicable to those structures for which wind is a predominant factor in their loading. It is hoped that the concepts have been explained in a manner understandable by those not familiar with the principles of the design procedures, but who, nevertheless, approach the subject with an open mind. Of necessity, much has been simplified and much has been omitted. In particular, no mention has been made of vortex shedding and related phenomena. Similar, but modified techniques are available. The techniques used for assessing the probability of fatigue failure have not been discussed. These and other features of structural behaviour are, however, important and may be the controlling factors in the design of some structures.

It is not claimed that the statistical method of design has yet been fully developed. There are short-comings in several respects. Much research is yet to be done in confirming the mathematical model of the wind in city environments where most major buildings are constructed. More work is needed in investigating the admittance function. Nor can it be said that the method is suitable for use in the majority of design offices. Heavy reliance is placed on the computer to perform the otherwise very tedious calculations.

Nevertheless, the techniques available at this time represent a considerable advance in the attempts of design engineers to rationalize their approach to the problem of realistically describing structural behaviour under the action of wind.

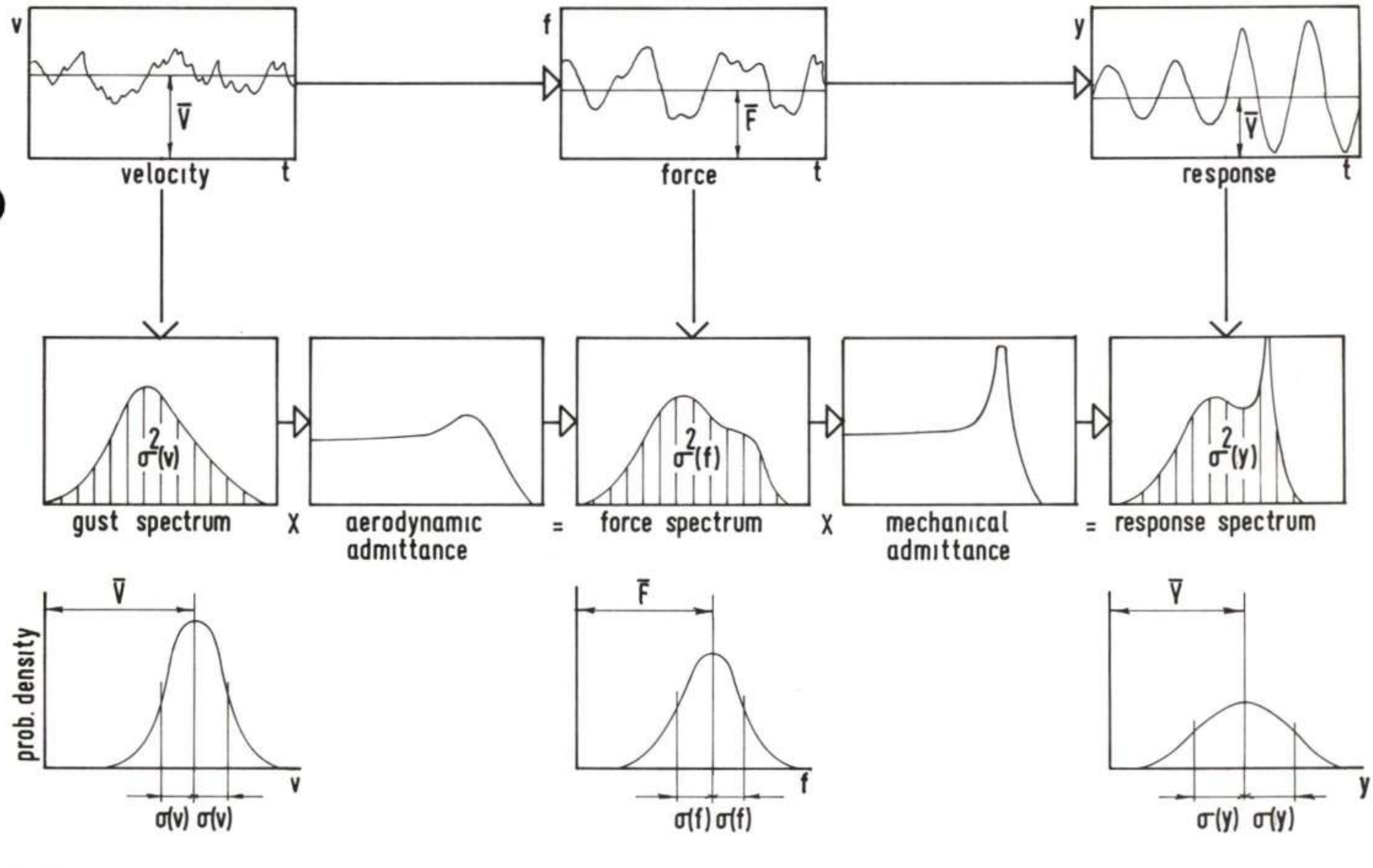


Fig. 14
Graphical representation of the design procedure (after Davenport)

Glasgow Airport Development

Hamish Stears

Early developments

The job title *Glasgow Airport* has appeared with a certain amount of regularity since its first occurrence on the new jobs list in 1962.

Our involvement at this time was with the new Terminal Building complex (job no. 1582) designed by Sir Basil Spence, Glover & Ferguson. As you will probably know this building was completed in May 1966, the first passenger flight being an inbound chartered *Viscount* carrying members of the design team staff from Edinburgh. This building was designed to a brief prepared by the Board of Trade, which anticipated that a load of 1.6m. passengers per year would be reached in 1970, and, to accommodate this, 17 aircraft stands were required. Of these, eight were exclusively for the use of domestic BEA flights in what is known as the West Pier and nine available for domestic or international flights in the East Pier. The buildings and apron were planned to deal with medium-sized aircraft, a 130-seat *Vanguard* for example.

During the course of construction of the Terminal Building, etc., it was suddenly realized that it would be most inconvenient for the Airport Engineer and his staff, together with the motor transport maintenance depot, to remain at Renfrew, some 8 km (5 miles) away, the site of the then existing airport. The same design team was therefore commissioned (job no. 2522) to carry out the refurbishing of an existing hangar, together with the construction of associated office accommodation, to be completed in time for the opening of the airport. The contract for this was an extension of the Terminal Building contract carried out by John Laing Construction Ltd., and the target date was almost met, the Motor Transport Depot opening about two weeks after the transfer of air traffic which took place on the programmed date.

Almost immediately after the opening of the airport came the criticisms, to be expected with a new public building, and, to do them justice, Glasgow Corporation set about putting in hand measures to insert all the extras which had been cut from the brief as economy measures. It was obviously not possible to do anything which would radically change the basis of the design, but before the end of the maintenance period, we had set about installing an escalator (job no. 2735).

If you have ever tried installing an escalator in an existing building you will know the difficulties, but to do this in a building which operates 24 hours a day with a limit set on amount of noise, and more important, dust, increases one's chances of getting ulcers. The circumstances were not eased by the fact that the lower floor was a suspended concrete flat slab, and the upper floor required a new floor to span from the end of an existing concrete cantilever. This, coupled with a requirement to start after New Year and be operational before Easter (which in 1967 was in March), gave a number of headaches, but all were eventually overcome. The delivery of the escalator unit was made a week before Good Friday by British Rail to a position literally 1.5 m (5 ft.) horizontally from its final position. This was achieved by various means, including removing a door and its frames from the building, handrails from the escalator, and air from tyres.

Traffic growth problems

Towards the end of 1967 it became apparent that traffic figures had been undercalculated and the figure for that year was over the 1.6m. passengers predicted for 1970. One of the major squeeze areas was in the bar and buffet and it was therefore decided to extend the area of new floor provided for the escalator, and completely resite the bar facility, allowing the buffet area to expand into that formerly occupied by the bar (job no. 2945).

At the same time, continuing their policy of improving the standard of passenger handling facilities, Glasgow Corporation decided to install passenger loading gangways (job no. 2829). In addition to providing covered level access from building to aircraft, this decision has made it necessary for aircraft to be parked nose-in to the building, thus gaining one aircraft bay increase for each three existing bays and thereby gaining two stands on the existing West Pier without a major extension.

Both these latter developments were carried out in the early part of 1968.

Major extensions

In the spring of 1968 the architect was approached to prepare a preliminary report on outline proposals for the development of the airport for passenger traffic into the 1970's. Subsequent to this report, submitted in August of that year, the original design team was again appointed to prepare detailed pro posals up to sketch plan stage for this development (job no. 3105). These proposals, given in the form of an A4 book of some 100 pages, together with approximately 80 drawings, were submitted in March 1969, and put forward a phased development as shown in Fig. 1.

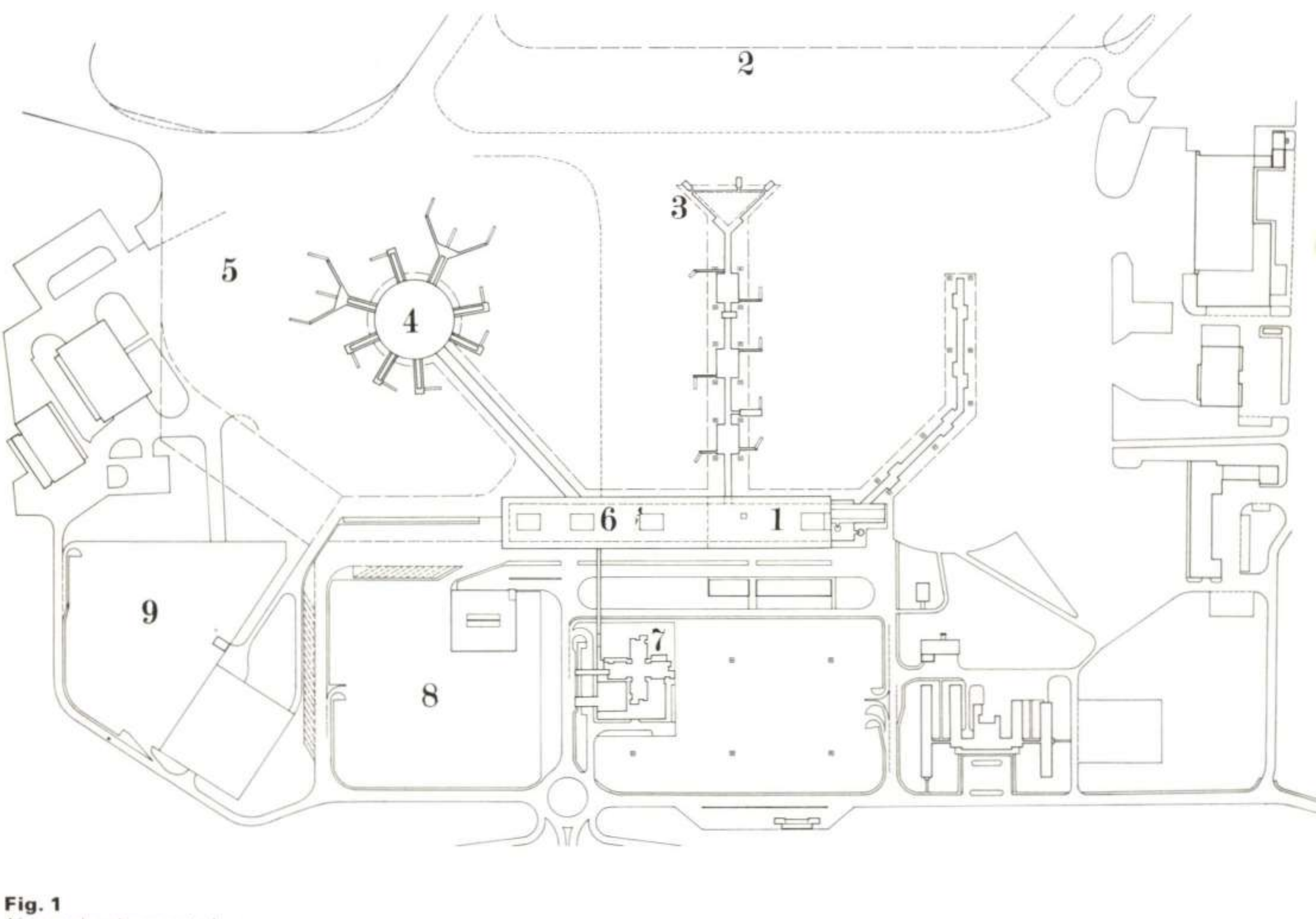


Fig. 1
Airport development plan

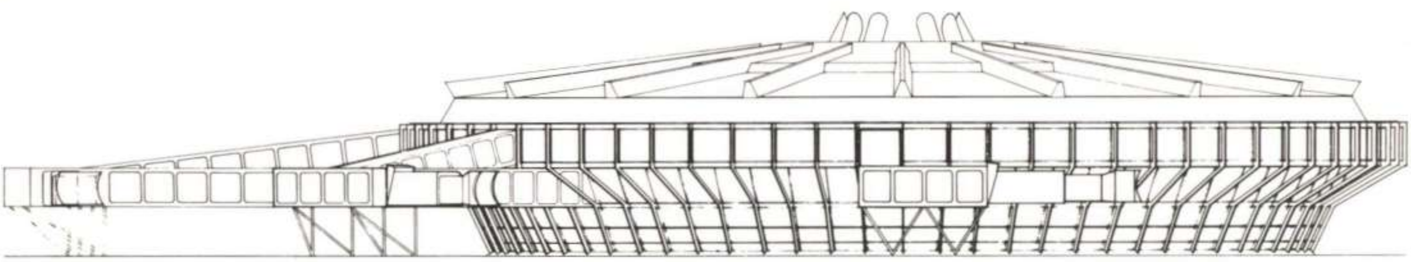


Fig. 2
Section/elevation of proposed satellite building

Phase I

Due to the requirements of inclusive tour traffic, an early (summer 1970) requirement was to provide additional aircraft bays in close proximity to the existing international pier. It was decided to extend the existing apron to the north and divert the main east-west taxiway onto this extension (job no. 3415). This provided additional parking bays for aircraft in positions which could be fed from the end of the East Pier without crossing aircraft lanes.

The apron extension completed, work could then commence on the construction of a major extension to the West Pier (job no. 3416). This was required to be ready by the spring of 1971 in time to accept the larger (160–170 seat) aircraft, which are expected to be introduced by BEA at that time. The original principle of forward waiting areas and check-in for passengers, was amended here in light of the obviously inefficient use of this type of space which had become apparent in the existing buildings. The amendment consisted of amalgamating this type of space for the five new aircraft stances into one larger area with toilet facilities, etc., the resultant space and volume giving much more pleasant conditions for the waiting passengers. The design which finally emerged produced a triangular building with the roof support at the perimeter and on the central vertical duct only, the shorter edges of the triangle being 36.6 m (120 ft.). The existing pier is of exposed structural steelwork, and it was therefore decided to proceed with this type of construction. To reduce foundation costs, the existing concrete apron has been maintained and column loads are placed on this, using concrete strip or pad-type foundations. The first floor is supported on columns on an approximately 6.1 m (20 ft.) grid, and is of composite structural steel-concrete construction, precast prestressed planks spanning 3 m (10 ft.) being used as a permanent shutter between the beams. The requirements of the roof have resulted in the decision to use a two-way spanning grid on an approximately 3 m (10 ft.) module. The construction takes the form of trusses fabricated out of rectangular hollow sections with the exception of the lower boom, which consists of two angles placed toe to toe, the lighting units being placed in the resultant channel. This continues a feature of the existing building where the

Fig. 3
The terminal building (Photo: Henk Snoek)

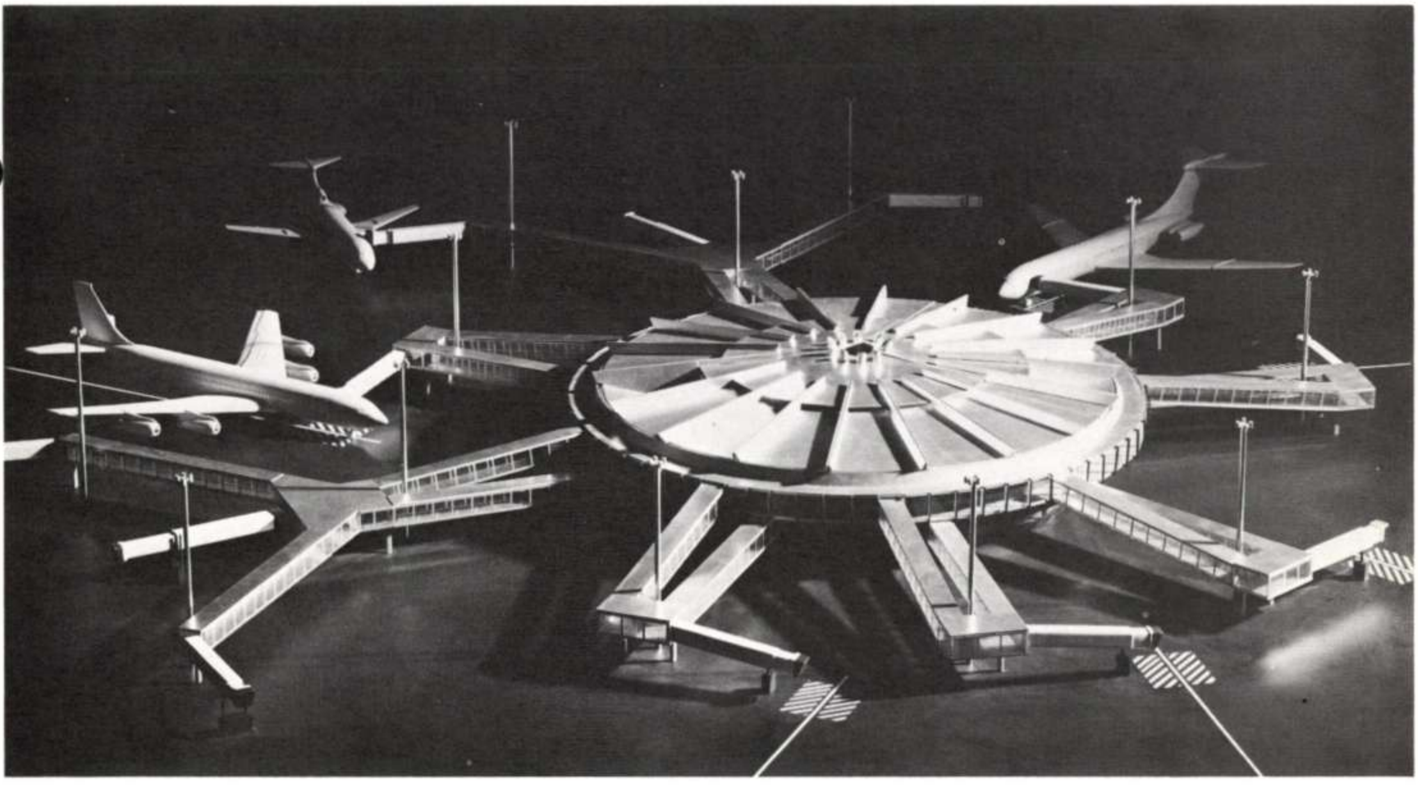


Fig. 4
Model of satellite building
(Photo: A. L. Hunter of Edinburgh)

lighting is contained between the twin booms of the prestressed concrete trusses. This form of construction also caters easily for the overhang required for solar shielding purposes. The lower boom and all vertical supporting steelwork are exposed and the external walls are glazed at the passenger level.

As the building is completely sealed against noise, mechanical ventilation is required and the ducts are carried from the ground floor plant rooms vertically through the central duct, and then distributed by threading their way through the roof grid trusses. In addition, further extract facilities are available within the roof space only, to discharge air through sound filters housed in the roof overhangs. The contract for the whole of phase I (job nos. 3415–6) went out to tender in September 1969, and was awarded to John Laing Construction Ltd., in November. Phase Ia was substantially complete in May 1970. At the same time as the apron construction, further nose loader bridges and waiting areas were constructed in order that eight aircraft bays would be available to BEA during construction of the West Pier extension.

Phase II

Unfortunately, the time of submission of the development plan and instruction to proceed with phase I came at the same time as the financial report on the airport, which included such factors as a major industrial dispute and the collapse of British Eagle, so we were not instructed to proceed with working drawings of phase II. This instruction has not yet been given and is not unconnected with the delay in decision on an extension of the main runway. The extension is, I understand, necessary, in the view of the airlines, to allow them to operate the larger aircraft (*Trident 3*) intended for the major trunk routes in 1971. Recent parliamentary answers indicate the runway length required to operate this aircraft in compliance with Board of Trade safety regulations to be 2,500 m (8,200 ft.), and another airport operator is understood to require a runway length of that order before considering to operate an airport some 64 km (40 miles) to the east. The proposals for the main terminal extension (job no. 3417) are generally to provide space to deal with an annual capacity of up to 5m. passengers. The original philosophy of design, a 1½ level building concept, is maintained, but the earlier disadvantages of the economy-enforced, narrow check-in hall has been overcome by providing a check-in hall the full depth of the building, giving space for a total of 50 desks. The ground floor of the existing building will be revamped, and the customs area, together with the bulk of baggage reclaim, will be resited in the extension. The extension is approximately 165 m (540 ft.) long and the appearance and construction will be generally as the major part of the existing building. The foundation problem is as before, but with the added problem of larger and deeper basements. This will entail taking precautions, both during and after construction, to prevent flotation, and such proposals as permanent sub-soil drainage, tension piles, and dead weight at basement slab level are being considered. Again, due to soil conditions, it will be necessary to design basement and ground floor slabs as suspended.

The basic planning of the building results in columns at 6.1 m (20 ft.) centres along four longitudinal lines giving rise to floor spans of 9 m (30 ft.) and 15 m (50 ft.) with cantilevers up to 4.6 m (15 ft.) on the north side at first floor level to provide cover for the baggage unloading bays. The structural depth at first floor is restricted to that adopted in the original building which is 610 mm (2 ft.) overall, and it has been decided that the most economic solution will be, therefore, to continue with the same form of construction, i.e. a ribbed floor, spanning across the building on to short span beams on the column lines. This will be interrupted in certain areas where there will exist a mezzanine walkway access to the baggage reclaim areas. Where this occurs, a 250 mm (10 in.) solid slab will be substituted, together with a subsidiary row of column ties, to prevent excessive deflection of the adjacent cantilevers. In order to achieve maximum economy of use, the same ribbed floor construction will be adopted for the ground floor over the basement, and in those areas of the second floor where suitable. To provide a visual unity to the complete building, the second floor office structure used in the original building will be extended to the section of offices in the new part, and to the southern bay at the west end.

It is proposed to continue with the same form of double skin construction for the roof, using precast concrete arch units spanning between trusses. The original form of concrete truss is anticipated but alternative proposals for structural steelwork have been prepared.

In the new satellite pier (job no. 3418) the principle of amalgamating forward assembly areas has been carried to the ultimate. In this section of the project, 12 aircraft bays have been positioned round a single 61 m (200 ft.) diameter building, thus giving minimum passenger walking distance on final call, and minimum apron extent. The segregation of arriving and departing passengers has been obtained by the provision of a second floor perimeter gangway connected by ramps to the various aircraft bays.

The requirement at the perimeter of the building to provide future flexibility of aircraft dispersal will necessitate support at close centres to accept the loading from the aircraft access bridges on an approximately 2.1 m (7 ft.) module. This will be provided for by the use of structural steel bents tied to the concrete floor slabs at the requisite centres in order to carry the loads directly to ground level.

The ancillary work in connection with the satellite will consist of the high-level link to the main building and the aircraft access bridges. This work will generally be carried out as structural steel trusses of spans in the order of 24–30 m (80–100 ft.) to provide for future flexibility and to keep the apron as clear as possible for service vehicle movement.

In addition to these new buildings, phase II includes the provision of some 10 hectares (25 acres) of concrete to the west of the existing apron (job no. 3419). The design is based on the requirements of the known aircraft with a small amount of reserve for any future possibilities.

The very low bearing strength of the subgrade on this site necessitates the use of a rigid pavement to minimize both depth of construction and cost. For the pavement construction proposed this gives a Load Classification Number of 90. The pavement required on grassed areas will be 300 mm (12 in.) 'pavement quality' concrete, on 100 mm (4 in.) 'dry lean' concrete, on a minimum of 610 mm (2 ft.) of compacted infill. The 300 mm (12 in.) concrete slab will be dowelled. Where there is existing concrete immediately under the new pavement, the specification for the latter may be reduced according to results from bearing tests.

Longitudinal and transverse falls at the aircraft bays will be, in no instance, greater than 0.9% and 0.4% respectively. Collection of surface water will be by continuous gutter on the perimeter of the satellite pier aircraft bays.

Phase III

The third phase of the development (job no. 3420) consists of providing large areas of car parking and re-routing service roads on the landside of the airport, and this will probably take place as and when required.

This completes the known picture of the developments at Glasgow Airport with the exception of the hotel which is at present on site. This is a joint Fortes/BEA project and is being constructed by MacAlpines. The remaining future developments are very hazy, but could well include cargo-handling facilities.

The problem with airport development has always been the rate of growth. In the past the actual growth rate has always been in excess of that predicted and present planners state that 12% is the minimum, but any figure up to 20% could easily occur at isolated points. Adopting 12%, it would appear that the 5 m. passengers per annum, which we have designed for, would be reached by 1979. The present programme for the development provides that, with immediate approval to proceed to the next stage (phase II), the building complex, etc., would be ready by mid-1974, by which time the building would be required to cope with well over 2½m. passengers per annum. We can only hope that these estimates are not as far out as the original, which was three years, or we may have to consider the development of the development by the time the development is complete.

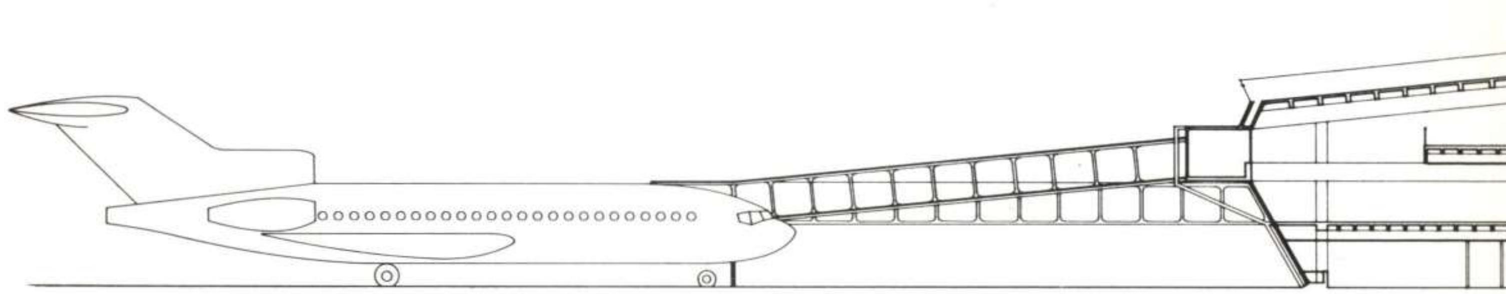
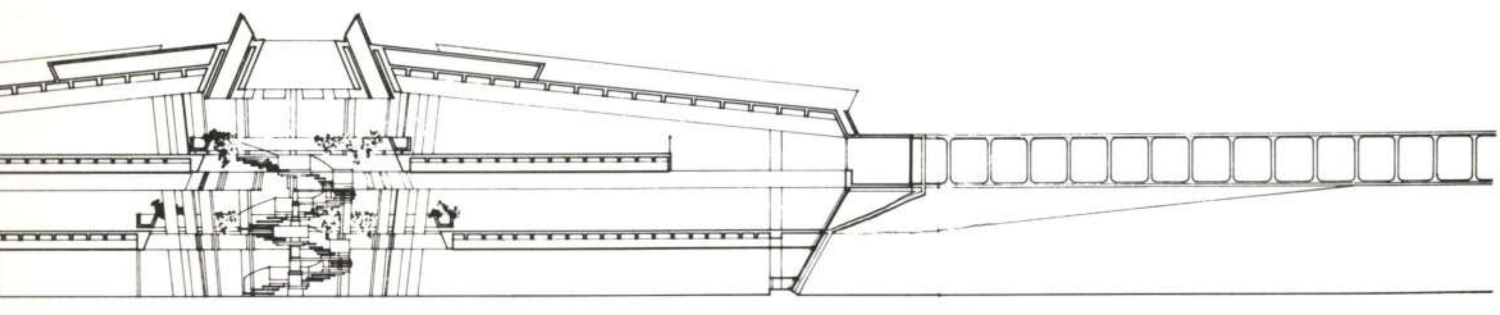


Fig. 5
Bridge to nose-loaders to provide direct covered access from aircraft to the building

Fig. 7
External stair to
public viewing gallery
(Photo: Henk Snoek)

Fig. 6
The semi-circular staircase from the concourse to the restaurant and lounge bar
(Photo: Henk Snoek)

Fig. 8
Check-in hall
(Photo: Henk Snoek)



Sonic booms and ancient buildings

Poul Beckmann

Introduction

In an age when money is becoming the yardstick for nearly everything, it is perhaps not surprising that property often commands more respect than people. We should therefore not be surprised that the opponents of regular overflights of supersonic airliners have adopted as one of their arguments the damage which the sonic boom, which accompanies such overflights, might inflict on cathedrals and other historical buildings.

Any extra noise added to our environment will be harmful to the health and well-being of some people, but it is difficult to assess this damage in terms of cash. It appears much easier to describe the damage to buildings in terms of money (especially if they are 'priceless'). Unfortunately, there is not much evidence to show that scheduled supersonic overflights will cause any major damage to structures, whether ancient or new.

The effect of a sonic boom on the hearing is violent, but so are many performances of music and there is as yet no evidence of any pop group having 'brought down the house' literally. It is therefore important not to confuse the emotional reaction to the sound with the real physical damage to buildings.

The nature of the boom

The origins of the sonic boom are the shock waves created by the nose and tail of the aircraft travelling at speeds above that of sound. These shock waves form cones with their apices at the nose and tail and would, in section, look very similar to the bow and stern waves created by a boat being pulled along in water. The angle between the shock front and the axis of the aircraft depends upon the speed of the aircraft relative to the velocity of sound.

The shock wave is propagated in directions at right angles to the shock front and produces a very sharp pressure rise, then a gradual decrease in pressure to below the ambient value followed by a sharp rise back to normal. The overpressure from steady flight at about 12,000 m (40,000 ft.) altitude is between 50–150 N/m^2 (1 and 3 lb./ft.^{2}), depending on the size, and weight of the aircraft. The duration between the positive and negative peak is in the order of $\frac{1}{10}$ to $\frac{1}{3}$ of a second depending on the length and speed of the aircraft. This so-called 'N wave' is what the ear registers as a double bang.

This fairly simple picture is modified by a number of factors. Because of the increase in the speed of sound with reduced height, the shock front, the so-called 'Mach Cone', tends to flare out towards the ground and, in certain instances, becomes a vertical plane. As the shock front progresses at right angles to itself it means that in this case it travels horizontally and if the flaring out to a vertical plane occurs above ground level, the shock wave never reaches the ground.[1]

Under certain atmospheric conditions, known as inversion, the situation is reversed so that the shock-wave is focused on the ground instead of being dispersed and an increased overpressure results.

From the fact that the angle between shock wave and aircraft generally depends on relative speeds only, it follows that, for the same speed, a climbing aircraft is more likely to achieve horizontal propagation of the shock wave above ground level, and, conversely, the steeper an aircraft descends, the smaller the angle between shock front and ground will be, and hence the pressure intensity will increase substantially.

The shock waves are reflected from the ground and from any hard surfaces they hit and, for this reason, the effect, even from level overflights, can be magnified through a reflected wave from one point arriving simultaneously with the direct wave originating from a point further on the flight path.

Probably the most significant fact behind the majority of documented cases of building damage caused by sonic booms from military aircraft is what is known as acceleration focusing. As supersonic aircraft accelerate, the angle between the shock front and aircraft axis decreases and the shock front travels towards the ground at an increased angle with the horizontal. This leads to the situation where the shock waves originating at successive points on the flight path arrive simultaneously at the same point at ground level. The overpressures from the various shock waves are added together and the net effect is that of an up to 5, and even 10, fold multiplication of the overpressure.

Fighter aircraft are built to be capable of supersonic speeds at ground level. It is uneconomic and pointless to try and build an airliner to do the same. When talking about building damage one should therefore distinguish clearly between the two categories. An example of massive damage which comes to mind is that of the fighter pilot making a low-level flight over Ottawa airport, doing an unauthorized turn and accelerating 'out of trouble'. The result was $300,000 worth of damage on a nearly completed air terminal building.

The cabin to the control tower was glazed with 10 mm (⅜ in.) thick glass. All the panes facing the oncoming aircraft were broken.[2] This suggests overpressures in the 2500 N/m^2 (50 lb./ft.^{2}) range. It is quite clear that supersonic airliners at regulation height producing overpressures between 50–150 N/m^2 (1 and 3 lb./ft.^{2}) cannot, by any stretch of the imagination, be considered in the same light.

The effect of sonic booms on building elements

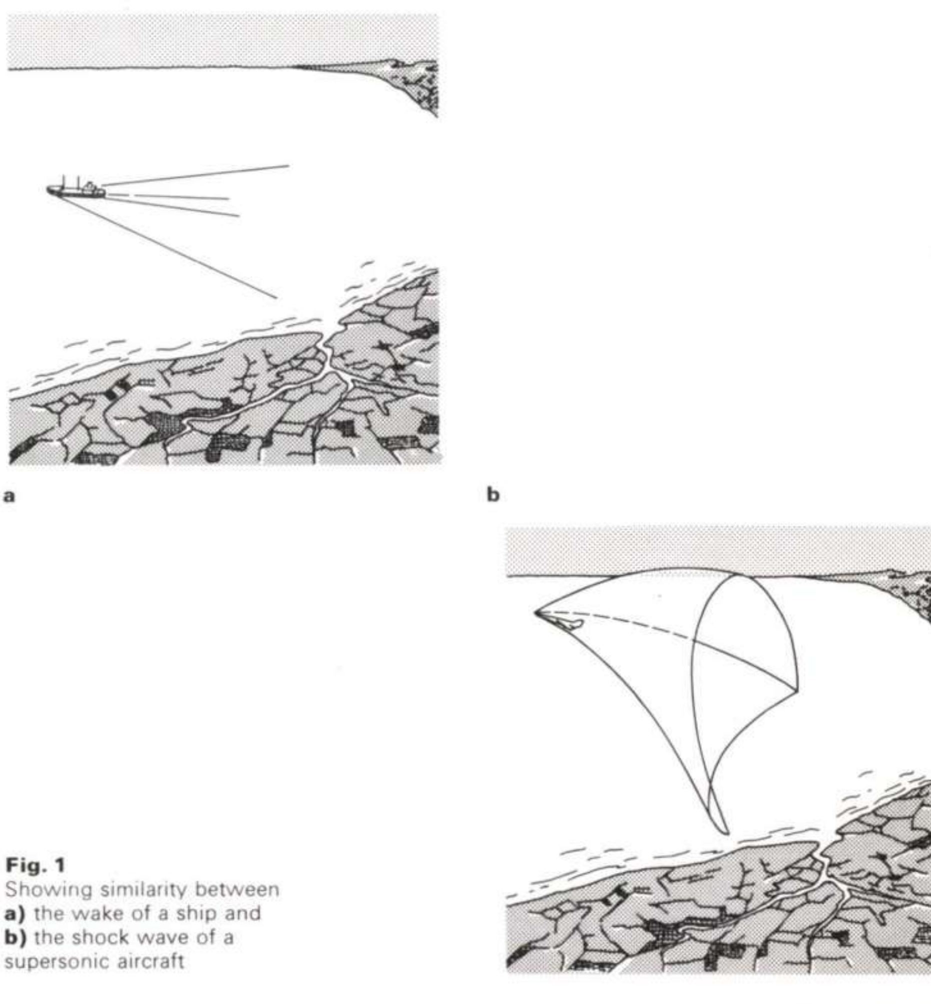
From the previous paragraph it follows that overflights at steady supersonic speed at regulation height will not do any damage to sound structures and well fixed finishes, because the effects of the simple boom correspond to static pressures far below those created by normal wind effects.

When we consider historical buildings which have been structurally weakened by abuse and neglect through centuries, or which in themselves are so slender that no one would dare to build them today in their original material, the situation is not nearly so clear.

Let us consider what happens when a sonic boom hits a building element.

As the sonic boom produces an overpressure and an underpressure in quick succession it will exert a push-pull action on anything that it impinges upon. The effect of this will depend not only on the static strength of the building element in question, but also on its vibration characteristics.

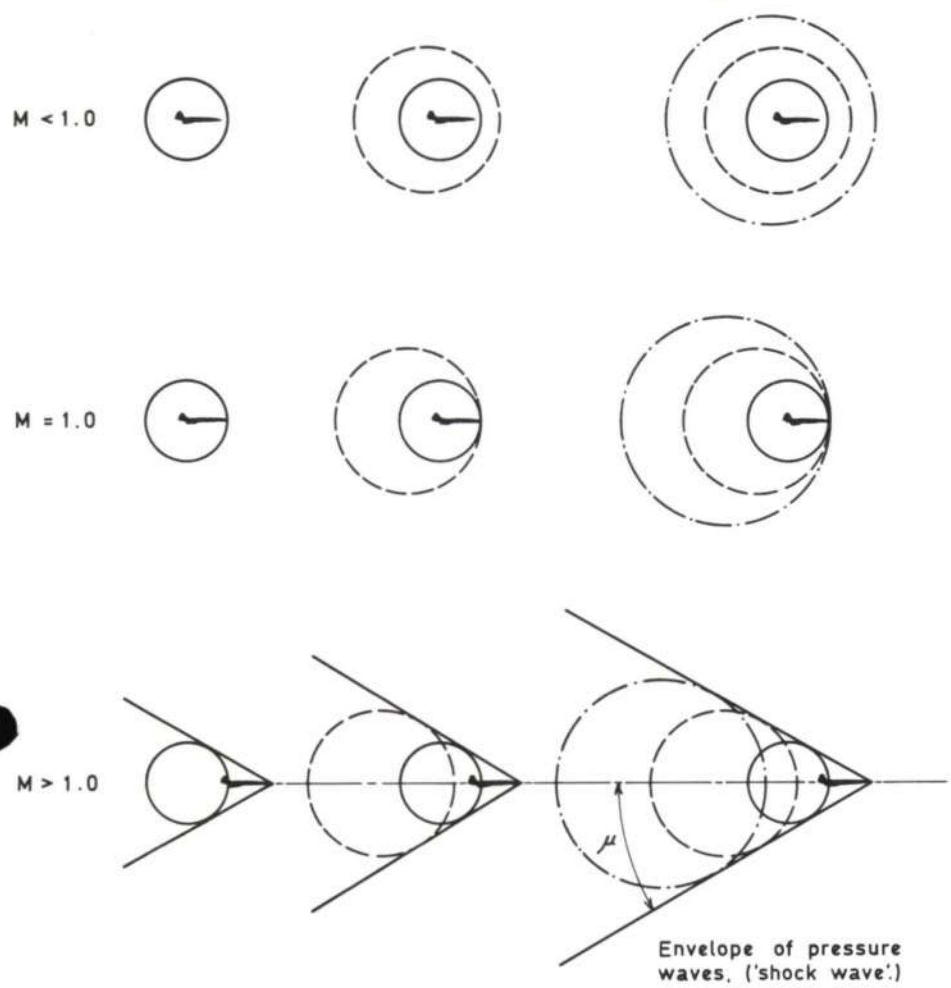
Every single building element, such as a roof rafter, a vault, a wall or a window, has, in principle, its own frequency at which it will oscillate if disturbed and subsequently left to itself. If one tries to impress a vibration on such an element the amplitude of the resulting oscillation will depend on the relation between the frequency of the imposed vibration and the element's own frequency. The nearer the two are together, the larger the effect, and, neglecting damping, an infinite amplitude of oscillation results when the frequencies coincide, regardless of the strength of the



a

b

Fig. 1
Showing similarity between
a) the wake of a ship and
b) the shock wave of a
supersonic aircraft



$$M = \frac{\text{Velocity of aircraft}}{\text{Speed of sound}} = \frac{V}{a}$$

$$\sin \mu = \frac{a}{V} = \frac{1}{M}$$

Fig. 2
Propagation of pressure waves from aircraft travelling at different speeds

imposed vibration. The fact that such resonances do not always produce disasters is due to the damping effect of the internal friction in the material and of the aerodynamic resistance to the oscillations.

When the damping is substantial, the effect of the forced vibration becomes dependent on the energy input, as well as the degree of resonance.

If elements are built together they interfere with each other's resonance frequencies and as a result, any element found in a real building will have a large number of frequencies at which it will respond to outside excitation.

Whilst the sustained vibration can therefore produce a large amplification of the effect which would normally result from a simple applied force of the same intensity, a sonic boom represents a mixture of two cases; it is not a single force nor a sustained oscillation, but something in between the two, and, whilst the oscillations considered in the previous paragraphs are generally thought of in terms of 'sine waves', the sonic boom has a sharp 'N' outline. This shape can, however, be broken down into a number of sine waves of different frequencies but, the net result is that we are considering a system subjected to a short-term imposition of vibrations at a number of frequencies which may or may not correspond to the system's own frequencies. As if this were not enough, the damping characteristics of most building materials and assemblies are imperfectly known, to put it mildly, so that an analytical approach to the problem is generally impractical, and one is therefore thrown back on to the experimental approach.

If it were possible to produce a 'synthetic sonic boom' producing the same shape of pressure wave but at reduced intensity, it might be possible to measure the effect of this and arrive at the effect of real sonic booms by means of multiplying in proportion to the ratio between the pressures.

Whilst measuring the effect of such a synthetic boom, one would naturally also measure the response of such imposed vibrations as constitute part of the normal environment of the building, i.e., in the instance of cathedrals, the ringing of bells and the playing of the organ; and for any historical building in a built-up area, the effect of traffic, and, in the case of old slender pinnacles, the effect of wind, to see whether the sonic booms would produce effects of similar or bigger magnitude than those which the building appears to sustain successfully at present.

The perfect simulation of a sonic boom has unfortunately not been invented yet, at least not in a portable form such as would be needed if the responses of a lot of buildings were to be measured. The nearest approach so far is the 'snifter', as developed by the Royal Aircraft Establishment, for this particular purpose. This is in simple terms, a large fireworks 'banger', set in a metal tube which is shaped in such a way as to absorb some of the higher frequencies created by the bang and thus produce a frequency spectrum more akin to the sonic boom.

Vibration responses in cathedrals

In October 1968, a team from Southampton University's Institute of Sound and Vibration Research, headed by Dr. Crawford, toured a total of 13 cathedrals in this country and, by means of accelerometers and seismometers linked to an electronic device, measured the response of typical cathedral elements to ambient vibration and to 'snifter' bangs in those positions where these bangs would not be complicated by reflections from nearby buildings so as to make them too unrepresentative of simple sonic booms. The 'snifter' responses were measured at five cathedrals.

The results of this investigation revealed a number of factors. Not surprisingly, walls in and near bell towers had by far their worst vibrations imposed by bell ringing. Organ playing generally produced oscillations of these walls a hundred times less intense and the effect of the 'snifters' varied considerably from being about $^1/_5$ of the worst of the bell vibrations to less than $^1/_{1000}$ thereof. To put this in perspective one should remember that bell ringing is notoriously damaging to church structures. (Four towers in Norfolk collapsed within 20 years.)

On elements more remote from the bell towers the effect of bell ringing was obviously less and on the lighter elements, such as timber roofs, the influence of the 'snifters' approached that of the bells.

All these measurements would still not tell us very much without a correlation between the effect of a 'snifter' and that of a real sonic boom, and to this end measurements were made in Germany, on two churches situated where frequent sonic booms occurred from military aircraft, and which were acknowledged to be in a very fragile state in parts.

As the churches are baroque, and thus entirely different in construction from British cathedrals, the measurements taken on them are not wholly representative of the response of Gothic structures but they do, however, give a lead. Based on these measurements, with the appropriate corrections for the differences in the frequency spectra, calculations have been made predicting the response to a 120 N/m^2 (2.5 $lb./ft.^2$) 'N wave' such as would result from a scheduled supersonic overflight.

Generally speaking, taking the average of the measurements made on the various cathedrals, the calculations show that the effect of a sonic boom of this magnitude on walls, roofs and vaults, will generally be comparable to, or less, than that produced by bell ringing, organ playing, etc., and, incidentally, the measurements also show that traffic vibration generally has an insignificant effect.

The situation is, however, not nearly so clear when we turn to stained glass windows. It was quite clear that the effects of the 'snifters' were such as to produce accelerations between 5 and 10 times as large as those produced by organ playing. As the predicted response for the 120 N/m^2 (2.5 $lb./ft.^2$) 'N wave' was generally well above that measured from the 'snifters', it would appear that sonic booms would subject stained glass windows to substantially greater vibration than anything normally occurring as part of cathedral environment. Whilst generally speaking, this would still not result in wholesale shattering, there remains the unknown effect of repeated vibrations and the brittleness which seems to attack lead with age.

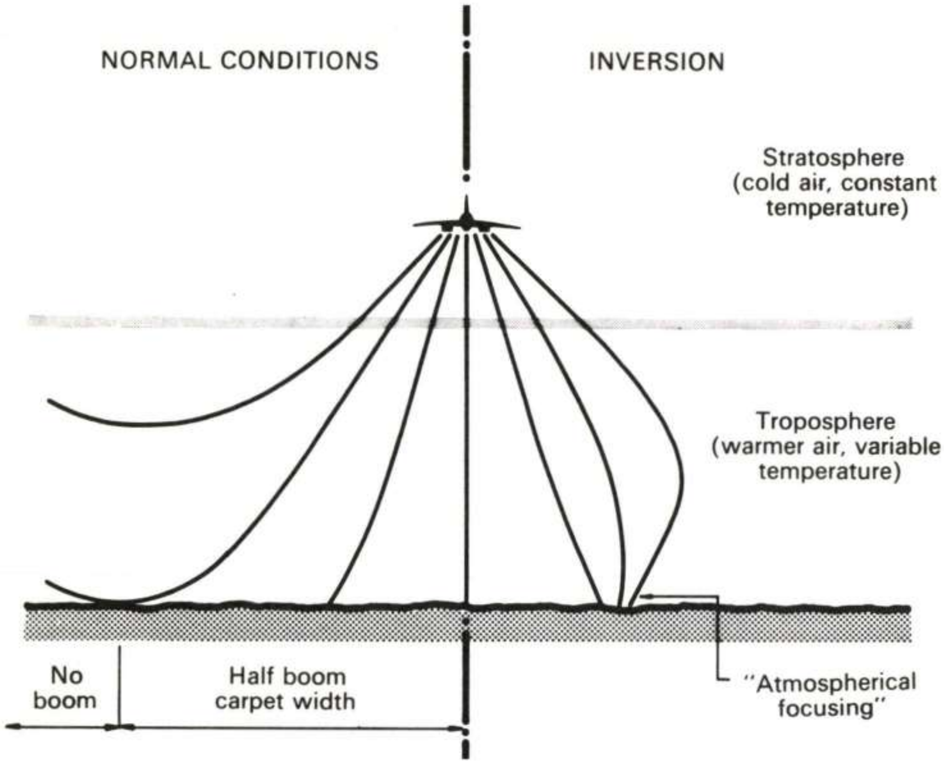
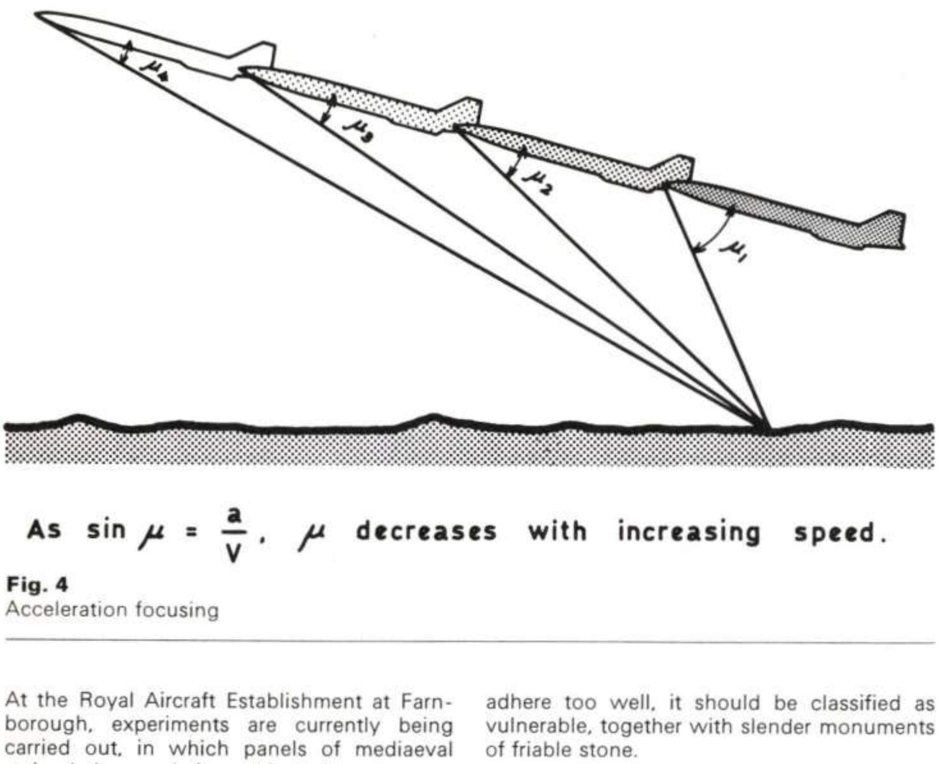


Fig. 3
Normal dispersion of shock waves as opposed to atmospherical focusing



As $\sin \mu = \frac{a}{V}$**,** μ **decreases with increasing speed.**

Fig. 4
Acceleration focusing

At the Royal Aircraft Establishment at Farnborough, experiments are currently being carried out, in which panels of mediaeval stained glass are being subjected to a succession of 'synthetic organ music' and 'N-waves' produced by a laboratory device. The object of these experiments is to compare the behaviour of panels influenced by 'N-waves' and the panels subject to 'organ music' only. This could be described as an accelerated fatigue and ageing test. The results to date indicate that exposure to several thousand 'N-waves' of peak intensities, well above what is predicted for supersonic airliners, does not result in any damage or significant permanent deformation.

Some measurements were made in Germany of the response of false vaults and pilasters of lath-and-plaster construction. As these were situated inside heavy baroque structures, their surprisingly small responses may not be typical and, as decorated plaster tends not to adhere too well, it should be classified as vulnerable, together with slender monuments of friable stone.

One last question to which at present there is no clear answer is due to the fact that, in the core of many mediaeval walls, the mortar has deteriorated to such an extent that by now it is practically dry, loose sand and dust. Instances have been quoted of larger voids being found in walls in the immediate vicinity of organs than elsewhere in similar walls. If in fact, acoustic vibration from organ music can dislodge mortar particles to the extent that they drop down through the interstices of the masonry, what will the effect be of the sonic boom? Bearing in mind that the total energy input from a few sustained deep organ notes is probably equivalent to 50 to 100 sonic booms, one is tempted to think: not very much, but, as the peak pressure of a boom may exceed that of the organ note, it could mean that booms would succeed where organs have failed and thus start deterioration where there was none before. Some experimental work on this would seem to be indicated, as cumulative damage of this nature might be significant.

Further problems

Whilst some progress has undoubtedly been made towards better assessment of the effect of sonic booms on ancient buildings there are still many unanswered questions such as:

'What manoeuvres are possible at supersonic speeds for Concorde-type aircraft?' Those which are possible will be done at some time or other, even if they are against the regulations and, in that case, 'What will be the focusing effect?' 'How frequently will climatic conditions with cold air on top of warm lead to "atmospheric focusing" and what pressure magnification is likely to result?' And, to look at it another way, 'Is it practical to fly supersonic transport planes at such high altitudes that the Mach-Cone will always flare out to vertical above ground level and therefore the shock wave will not hit the ground?'

These problems should be fully investigated before overland supersonic flight is permitted. After all, the old buildings were here first, and they give pleasure to many, so the onus must be on the advocates of speed for the few, to prove that no damage will be caused.

This summer will see the beginning of a series of 50 test flights planned to extend over three years to try the Concorde under normal and slightly abnormal operating conditions. These flights will take place over the Irish Channel in the main, but will pass over Oban, St. David's and Truro Cathedrals. It is planned to make measurements, at these cathedrals, of responses of various elements to sonic booms. At St. David's, measurements will be taken of responses to wind excitations which will be correlated with gust speeds measured at the same time. In addition to measurements of dynamic response, a certain amount of instrumentation will be provided to try and detect any permanent deformation caused by these flights. The small number of flights, coupled with the fact that in very few instances have measurements been taken of behaviour before the test flights, make it unlikely that anything substantial will emerge from these tests, the more so as they will all be confined to as near as possible fair weather conditions.

A far more promising approach would seem to be the 'Blunderbuss.' This is a large conical tube in which a sudden release of air pressure, by transmission through a long conical funnel, will, at the wide end, produce a very good imitation of the 'N-wave' to any desired intensity. This was what was used for the stained glass experiments at Farnborough. A larger version is currently being built at the old R101 Hangar at the RAF Station at Cardington and it may be possible there to carry out experiments to establish whether or not cumulative damage in masonry is going to be a problem!

Meanwhile guardians of old buildings may find it prudent to measure the responses to general vibration of the more delicate treasures in their care and to provide protective stiffening to cope with the odd 240 N/m² (5 lb./ft.²) boom from erring commercial craft, bearing in mind that even if overflights were to be banned, mistakes will be made and accidents will happen.

Even if it were to be proved that supersonic overflights were not damaging to buildings, it doesn't, however, follow that they should be lightly permitted over land. People matter more than property and to cut the flying time from Heathrow to Kennedy by a few hours does not justify the infliction of the misery of the sonic boom on millions of people, especially as the same effect on the overall travelling time could be achieved by rail links between the city centres and their airports.

West Norwood Library and Public Hall

John Morrison

The original West Norwood Library is now over 80 years old and it had been apparent for a number of years that the building had reached the end of its functional life. Lambeth Borough Council, therefore, decided that a new library should be built on a site nearby which had been donated by the Nettlefold family, and they made up their minds that the opportunity should be taken to provide a public hall which could be used for dances, stage shows, films, lectures and concerts.

The site was not exactly in a desirable location, being surrounded on three sides by a cemetery (see Fig. 1) and having an area of only 0.27 hectares (0.67 acres). This situation presented a severe challenge to Lambeth Architects Department, who dealt with the problem admirably by designing the library as a complement to the dominant focal point of St. Luke's Church, which is just across the road. St. Luke's is one of the Waterloo churches, so called because they were built to celebrate the Iron Duke's victory.

The brief called for the provision of the following departments:

1 Adult library
2 Young adult library
3 Junior library
4 Record and music library
5 Administration block
6 All-purpose hall to seat 250 people.

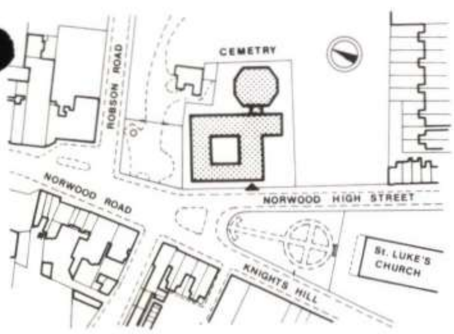
The general arrangement of the scheme is shown in Fig. 2.

Because the site was surrounded on three sides by a cemetery and on the fourth by the main street, it was decided that the library should be inward looking and it was, therefore, planned around an open courtyard. This courtyard provided a quiet area away from the street noises and also supplied a valuable additional source of light to the interior of the building. The high brick wall on the outer face of the building shut out the view of the tombstones, but in order to provide daylight for the areas behind these faces, we had to adopt a clerestory light system for the adult and young adult libraries.

People of all ages are encouraged to join the library and young readers will progress from the junior library, to the young adult library and finally, the adult section.

The library entrance (see Fig. 4) faces Norwood High Street, and the junior library and its control area are on the left as one enters. The next department beyond this section, and around the courtyard in an anti-clockwise direction, is the young adult library. This is aimed at the young teenager, usually a difficult group to attract to books. To avoid the stuffy institutional atmosphere that many libraries

Fig. 1
Site plan



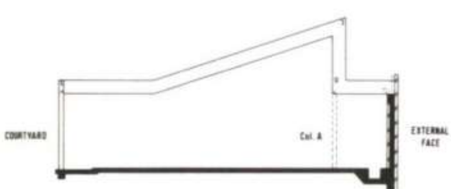


Fig. 3
Section through adult library

Fig. 2
Ground floor plan (library) and first floor plan (public hall)

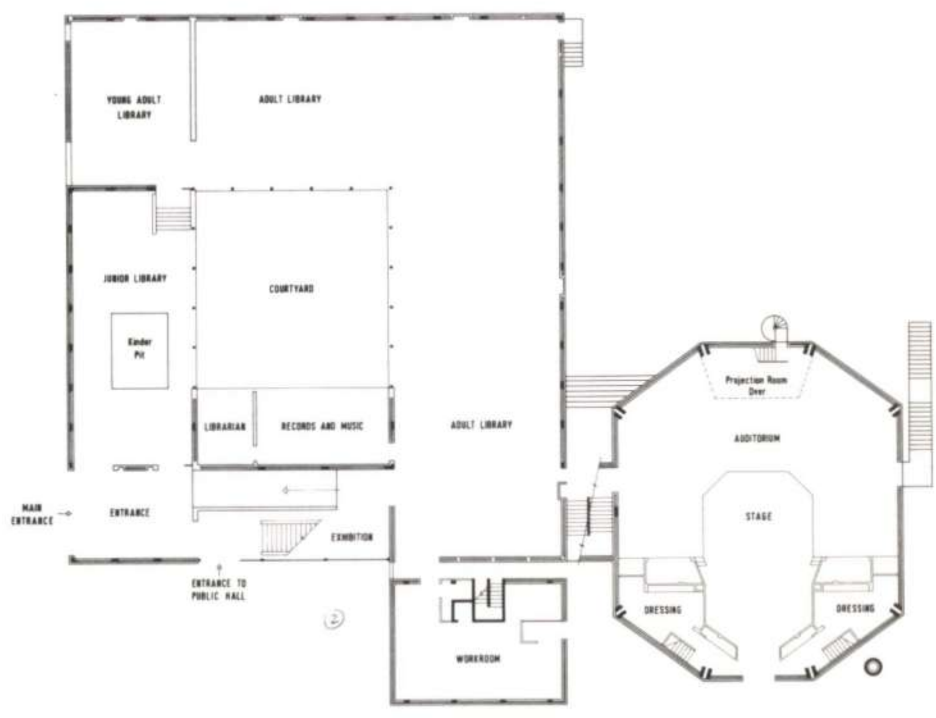


Fig. 4
Library main entrance (Photo: Ernie Hills)

have, this area, like the whole library, has been furnished with subdued luxury by providing coffee tables and Mines & West chairs (see Fig. 5). A window was built in the outer wall so that passers-by would be able to look into the young adult library; it is hoped that this view will attract the younger people. Two further sides of the courtyard are formed by the adult library and, in the summer, sliding windows will be opened so that the borrowers will be able to sit out and browse *al fresco*. The main control area is located between the administration block and the adult library. People visiting the latter enter by the main entrance and walk straight ahead, passing the main staircase which leads up to the public hall. The area below this main stair will be used as an exhibition area (see Fig. 6).

The public hall is an octagonal building behind the library, the hall proper being at first floor level and designed to seat 250 people. At ground level there are a restaurant, kitchen, bar and a small room available for hire to clubs, etc., and below this is the basement boiler room. For evening functions an alternative main entrance will be used, so that the main section of the library can be closed to the general public (see Fig. 2).

Fig. 5
The young adult library
(Photo: Ernie Hills)

Fig. 6
The entrance staircase
(Photo: Ernie Hills)

Foundations

A soil survey of the site had shown that there was a small amount of made ground over 8.2 m (27 ft.) of shrinkable brown clay, followed by blue London Clay. The high perimeter to floor ratio of the building resulted in fairly light foundation loads and this fact, coupled with the presence of shrinkable clay, ruled out continuous strip foundations. We were left with the choice of pad footings or short bored piles. Two schemes were prepared showing the alternatives and the difference in cost between the two was found to be negligible. We decided that piled foundations should be adopted due to the presence of some Black Poplar trees on the site boundary. These trees have a rather notorious reputation and there have been several recorded instances of settlement when they have been near shallow foundations and shrinkable clay. The administration building and the public hall were designed to be supported on pad footings, as we felt that the greater weight and deeper foundations of these blocks would make them less susceptible to the movements of shrinkable clay.

Waterproof construction

The basement of the public hall is made up of a boiler room, electrical intake room, transformer, oil storage area, ventilation equipment, kitchen, toilets and welfare room. All of these rooms had floor ducts and as these would have led to complicated tanking details, we decided that waterproof concrete should be used for the slabs and retaining walls.

The soil report had not revealed any evidence of a water table, although slight traces of water had been encountered during boring. Due to the low permeability of the brown London Clay, we felt that any surface water finding its way into the backfill around the retaining walls would be trapped. Therefore, although no ground water was known to be present, we felt it desirable to form a waterproof basement.

Having once decided that there is water and a need for waterproofing, the next problem is to decide the standard of protection that is necessary, commensurate with the cost. The combinations are innumerable, ranging from habitable spaces protected from water pressure in gravel soils to uninhabited areas protected from surface water seepage.

For the boiler house basement wall we adopted the following standard:

1 Minimum wall thickness was 230 mm (9 in.) to allow for ease of placing concrete. Cover to external reinforcement was 50 mm (2 in.) and internally 25 mm (1 in.); so that a 100 mm (4 in.) gap was maintained between the two layers of reinforcement.

2 The steel stresses used were reduced by 20% to reduce cracking. This did not lower the steel stresses to those in CP 2003, but this was justified by virtue of the lower order of problem that we were dealing with.

3 A minimum distribution steel of 0.3% using deformed bars was used to control shrinkage cracking. This was slightly above the standard laid down in CP 2003 as the use of the deformed bars was specified. These bars produced smaller crack widths with a better distribution.

4 150 mm (6 in.) waterbars were used in all construction joints and all shutter ties were to be waterproofed with a baffle plate.

5 The position of all construction joints was shown on the drawing and the maximum length was limited to 6.1 m (20 ft.).

6 For waterproof concrete the water/cement ratio was specified as being 0.5 to ensure a dense concrete and to control shrinkage. Having specified a water/cement ratio, it is important to ensure that the specified design strength is compatible. For example, had a low design strength been specified, then it would have been possible to form a mix with low density and high shrinkage, whilst still complying with the other aspects of the specification. We finally settled for a specification of 4,500 psi.

7 At the foot of the wall a 100 mm (4 in.) diameter agricultural drain, surrounded with gravel rejects, was to be constructed. This was the first line of defence and is usually the most important single contribution to any structure that is built above a known water table.

The architects were very keen to use a bitumastic membrane on the external faces of the retaining wall and under the slab and, in view of the glowing guarantees that were offered by the manufacturer, we were prevailed upon to omit the 100 mm (4 in.) drain.

We now feel that this was a mistake, not so much from the waterproofing aspect but from the problems associated with the application. The concrete surfaces must be clean and dry, an almost impossible task in the depth of winter at the bottom of a muddy hole. The contractor went on record, in a moment of desperation, as saying that he was still prepared to guarantee a waterproof basement if only we would admit the bitumastic paint.

Alternative designs for public hall roof

We arrived at the final design for the hall roof after a series of solutions had been investigated. We feel that the way in which the design developed is of interest and the stages through which it passed are set out below

Development of the roof design

Design criteria	*Solution*
1 The plan shape was to be octagonal and the roof pitched. The roof was to be visually interesting internally and structural timber was suggested.	Triangulated timber trusses were used (Fig. 7).

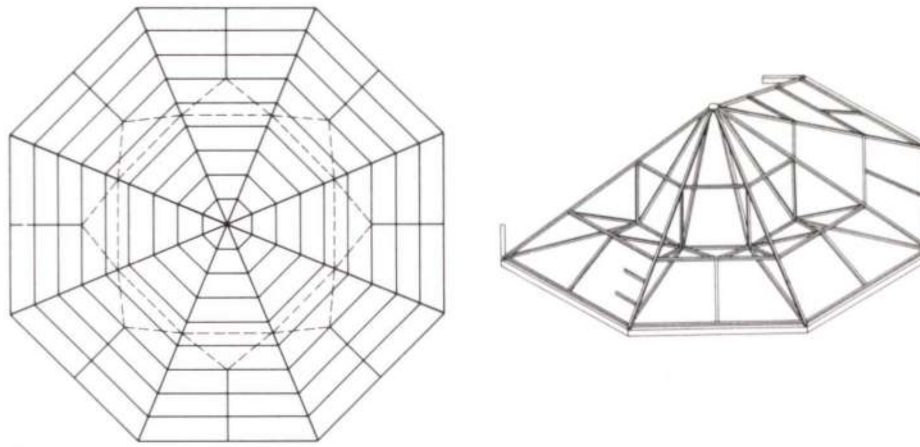


Fig. 7

2 The planning requirement was such that the octagon had to be stretched, i.e. two opposite sides made longer.	The first scheme was amended. The second scheme used stressed skin plywood folded plates. A concrete ring beam was introduced and supported on concrete columns (Fig. 8).

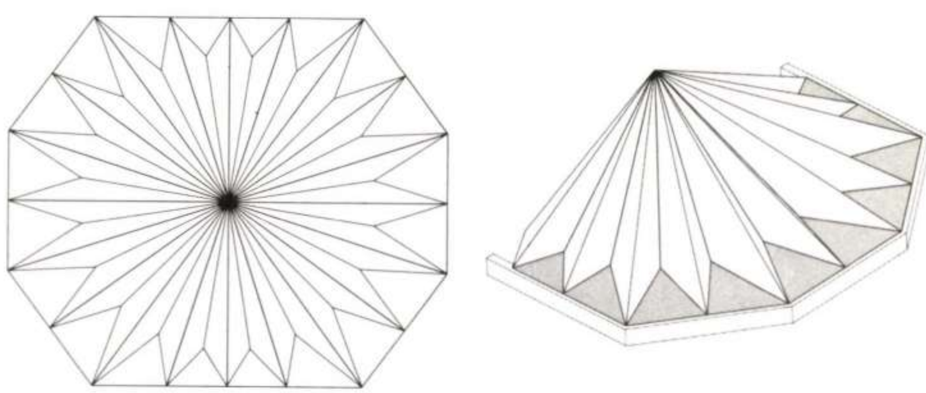


Fig. 8

3 The services were to be included in the roof space and the stage lighting was to be hung from the roof. Rainwater gutters were to be introduced. The outer roof covering was to be copper which, because of its cost, necessitated the utilization of the smallest possible surface area.	A pitched roof with two-way spanning timber trusses was developed. The ring beam was shaped to act as a gutter and also designed to resist horizontal and vertical forces (Fig. 9).

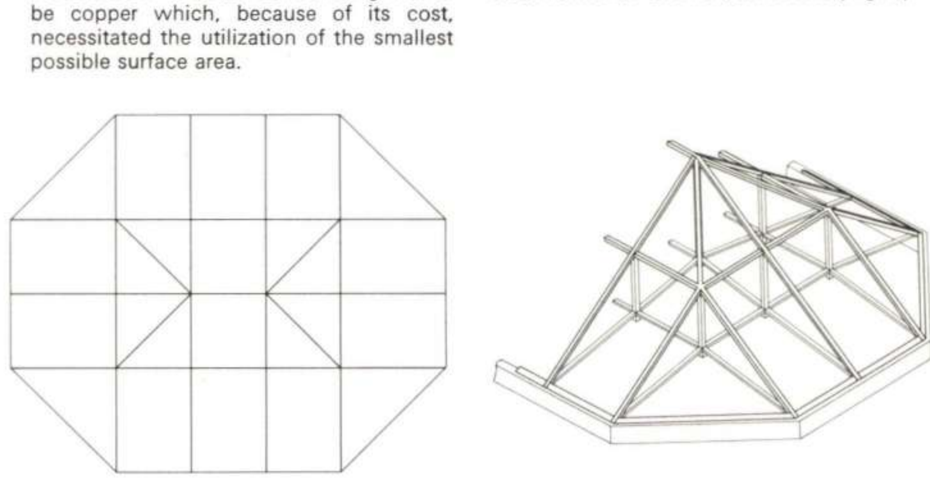


Fig. 9

4 An attempt was to be made to produce a more 'shaped' internal surface and one above which all services could be hidden.

A low pitched roof was used with large, hollow, boat-like structures, which were, in fact, stressed plywood skins on a structural framework (Fig. 10).

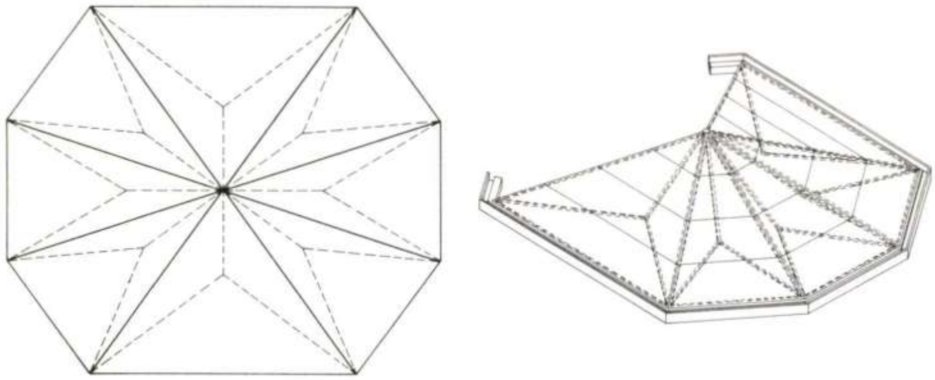


Fig. 10

5 The licensing authority required a two hour fire rating or the use of non-combustible materials. The stage lighting was required at a lower level. The projection room was to be cantilevered from the concrete columns. The ventilation ductwork, drainage and electrical service requirements were known in detail.

Triangulated steel trusses of rectangular hollow sections with fish bellied purlins were used. A separate stage lighting support structure was developed. In view of the large size of ducting required, twin columns were used with duct space between. Forces from the cantilever projection room were taken in the ring beam and column structure (Fig. 11).

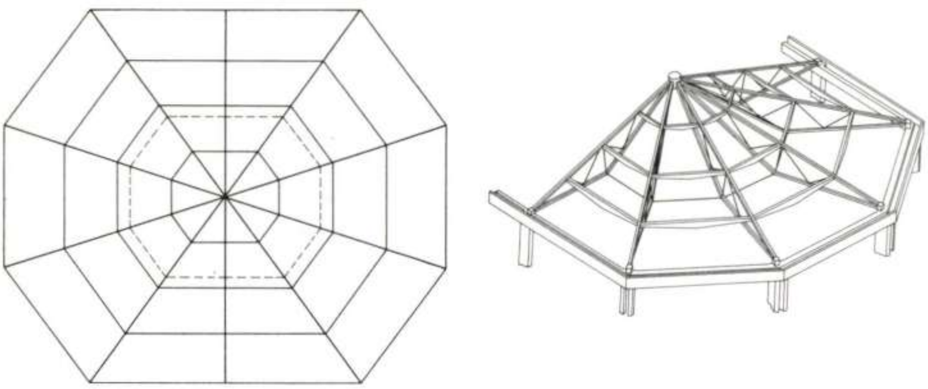


Fig. 11

6 Concealed lighting was required on the ring beam. Additional electrical services were required for the main hall roof.

The ring beam sizes and roof geometry at the support were modified. Provision was made for services to be fed through rectangular hollow sections and then into a channel section which supported the light fittings. Fig. 12 shows the completed scheme and details of one of the trusses.

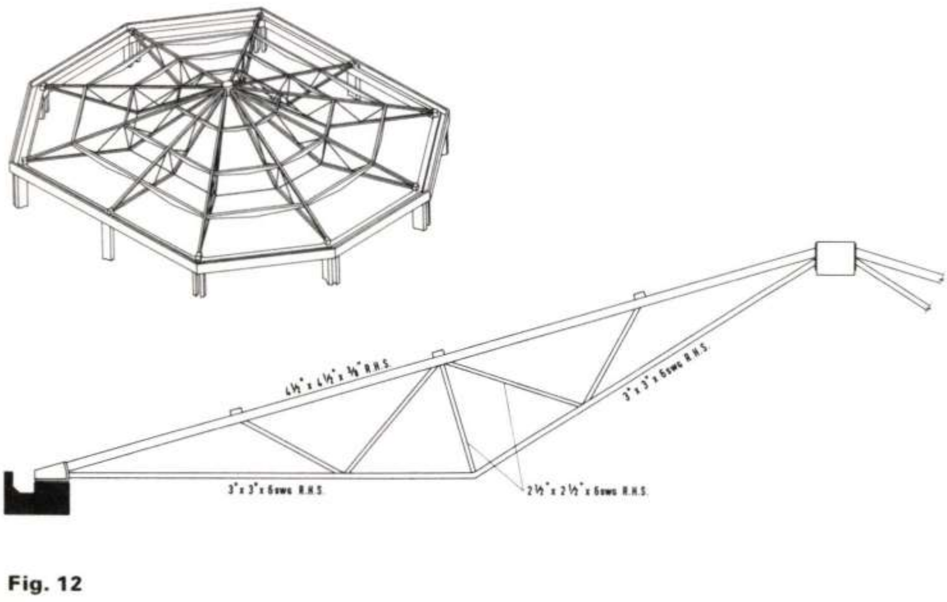


Fig. 12

In retrospect it is interesting to note how the very production of a solution to meet a certain set of criteria can lead to the need for modification of that design criteria. Thus you have a situation where not only the design is changing but the basic requirements are also changing, the final solution being one in which any required change in performance can be accommodated within the existing framework of design with the minimum of complication.

Library roof

The architects' original scheme called for a roof configuration and support system as shown in Fig. 3.

At our suggestion it was decided that column A should be omitted to provide a clear span structure. The steel roof was to be supported on concrete padstones on a 340 mm (13½ in.) cavity wall on the outer faces and carried on steel columns around the inner courtyard. Although the horizontal force on the top of the brick wall was small, it was found that, due to the height of the wall, its method of support, the cavity construction and the small vertical loads, tension was induced in the brickwork. One possible solution would have been to build a steel column into the wall, but the architect wished to maintain the clean external face of brickwork. We were also unhappy about the maintenance problem that would have occurred with this steelwork.

It was finally decided that 460 mm (1 ft. 6 in.) x 150 mm (6 in.) reinforced concrete columns should be built into the walls and the top of the column widened to 230 mm (9 in.) to make it look like a padstone, resting on the brickwork. We agreed that this was structural cheating but the end result is quite effective.

Thomas and Edge Ltd. were the main contractors and work started on site in May 1967, the contract being completed in April 1969 for a total cost of £257,000. The library and public hall were opened to the public by Her Royal Highness Princess Margaret.

Fig. 13
Library seen from the cemetery
(Photo: Ernie Hills)

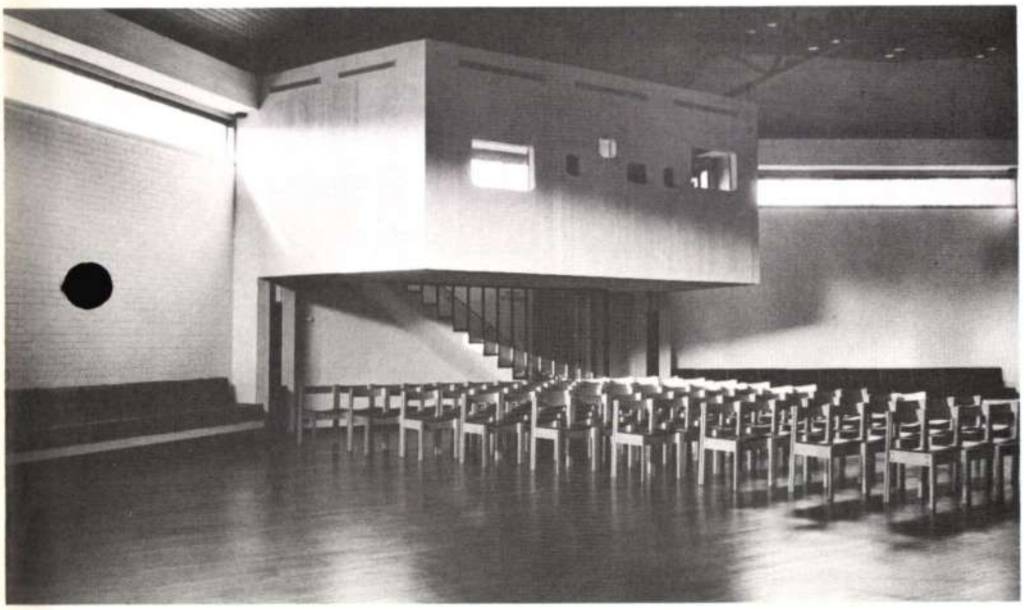


Fig. 14
The projection room
(Photo: Ernie Hills)

Fig. 15
The courtyard
(Photo: Ernie Hills)

Surface Finishes

Turlogh O'Brien

This paper was originally commissioned by the Institute of Building and was published in their journal Building Technology and Management *April 1970. It is reprinted here with the Institute's permission.*

The quest for variety

Modern buildings are often said to be dull and repetitive, but more careful observation reveals that, at least as far as surface finishes are concerned, variety is highly valued. The finishes used within one scheme may be dull and repetitive, but variety may be seen by comparing one with another. Certain fashions may be found at any one time, ribbed concrete being an example at present, but, in general, the approach may be summed up by the cliché 'variety is the spice of life'.

In response to this demand, building materials' producers have expended considerable effort in providing a wide range of surface finishes on their products. Even humble products like bricks have received their share of surface improvement. Concrete provides the most spectacular example of a material that has suffered all manner of ingenious treatments in the cause of variety. The showrooms of companies specializing in precast concrete cladding are filled with examples of successful and unsuccessful attempts to enliven the often dull surface of concrete. The results of this experimentation as seen on recent buildings, in some cases called sculpture, may not please everybody, but it should not be long before the 'real' surface appearance of concrete can no longer be positively identified.

In the last decade, plastics have finally established a position for themselves in the repertoire of external surface finishes. They are often, however, made to take on the appearance of something else, usually wood, because many people consider that the 'true' appearance of plastics is unattractive. Plastics weatherboarding and wood-grained effects are common. In other cases the surface is masked by using stone chippings to provide a texture. Plastics are sometimes offered as 'maintenance-free' paints, but discussions with the paint industry would show that, even if this claim could be substantiated, a wide range of colours would be required. Also, changing the colour after some years can be a positive advantage. Paints exemplify one extreme of the quest for variety.

Even metals have been made to conform to the new ethic of variety. The range of coatings for steel has increased considerably, particularly with the addition of plastic coatings. Aluminium and stainless steels have been 'improved' (you can now get aluminium clad with stainless steel) and copper can now be prevented from weathering to its normal green by the use of clear lacquers.

The search for attractive finishes can take some surprising turns. Who would have thought 10 years ago that rusty steel could be an acceptable decorative material? Yet the slow rusting 'weathering' steels have been used on some impressive buildings in the USA and are slowly catching on here.

It seems curious that, alongside the requirements for standardization for economical mass production of building components, there should be this trend towards diversity. Perhaps the latter trend is a direct result of the former. The answer is standardized variety, but it is not clear what this means.

The quest for quality

Quality is a difficult attribute to specify. In the context of most building work there is no such thing as 'absolute' quality. There are relative levels of quality and these are usually easy to compare, at least in a qualitative way. The trouble is that, with surface finishes, it is often difficult to describe precisely the standards required. In some cases it is possible to have a mock-up made which is then used as the standard for comparing performance on the job. In most cases, specifications must rely on rather general descriptions of uniformity, flatness, freedom from defects, etc. These problems may often lead to assumptions being made about 'currently accepted good practice'. The designer assumes that the contractor will have the same standards of quality as himself.

Under relatively static conditions this would not be expected to cause trouble, but the quest for variety brings the problem of lack of experience, both of operatives and supervisors. Surface finishes applied in a factory can be practiced and a high standard of uniformity may be obtained. On site, the situation is different. Even with skilled tradesmen, the conditions under which they work may limit the quality that can be achieved. When new finishes are required, site staff do not often have the chance to carry out a sufficient number of trial samples to get the technique right. They must learn on the job.

In addition there is the necessity of ensuring that expensive surface finishes are not damaged during subsequent building operations. Too often, stains or mechanical damage occur which must be made good. With new materials it is sometimes difficult to remove the traces of the defects and the overall quality of the job is lowered. Some of these problems stem from a lack of understanding on the part of the operatives of the amount of money that may have been invested in achieving particular finishes. It also arises from the experience that in traditional building, most of the usual damage may be cleared up or made good relatively easily by time-honoured methods. Today, materials that find increasing use on sites are very difficult to remove (crane oil and epoxy resins, for example) and traditional stains (e.g. rust) are difficult to remove from new surface finishes.

Clay bricks have proved themselves to be durable materials for external finishes, and few defects usually appear after the building is completed. With other materials, and particularly with concrete, some defects may not become apparent for some years after completion. These failures result in expensive repairs, usually involving alterations to the appearance, and in unpleasant arguments. Their avoidance through better quality control is easier to specify than to achieve. Quality checks generally take the form of inspections of materials arriving on site, but by this stage the ability to reject defective components is severely restricted by the problems of consequent delays to the construction programme. Whereas the responsibility for the cost of replacing defective components is quite clear, it is usually much more difficult to recover consequential costs, which may be much larger.

Under present conditions, good quality, durable surface finishes can only be achieved if everyone concerned is really interested in the result.

The quest for eternal youth

The cleaning and restoration of the external surfaces of many important buildings seems to be part of a general fashion for a clean, fresh look. Whilst this is generally welcome, it is necessary to examine the implications if this approach is carried too far. The results of the Clean Air Act are now being observed, and the problems of the environment are receiving considerable attention, at least at a political level. However, freedom from atmospheric pollution will not come quickly and buildings must be designed with this in mind.

The main visual objection to effects of weathering is non-uniform staining. Few people mind about a general uniform darkening of buildings, provided that large sums of money have not been spent on light coloured finishes (e.g. white concrete). The formation of unplanned dirty streaks is generally considered to be undesirable.

The trouble is that it is extremely difficult to design external surfaces in such a way that this does not happen. The quest for variety of colour, texture and profile in finishes means that each new proposal has to be considered on its merits, without recourse to years of past experience. A few general principles may be used for this, but a look at many recent buildings suggests that if these have been used at all, they have not been fully understood.

In addition to weathering defects, failures and loss of youth may arise through incorrect selection of materials and poor detailing of the junctions between them.

Green staining from copper flashings, streaks of bitumen at parapets, rust from partly concealed steel components which cannot be maintained, bleeding of oils or other constituents from mastics, migration of salts from brickwork to stone and leaching of lime from concrete to form stalactites may all be seen on recent buildings. Many defects are attributable to a combination of design and workmanship faults. Examples are the loss of adhesion of mosaic and tiling and of slip bricks on boot lintels, or the inadequate provision of compression joints in cladding, particularly stone, in order to prevent load being transferred from the concrete frame.

Defects of this type call attention to the fact that the cladding must be seen as a part of the overall building. The surface finishes may be affected by a number of components which in themselves do not necessarily contribute to the initial appearance. The difficulties of the designer have been emphasized already. He is the man responsible for the performance of the whole system, but his detailed knowledge of the behaviour of the parts will not be as complete as that of the materials' producers. The integration of a wide range of increasingly specialized technical factors is a task of great complexity, and this is often not fully appreciated by the suppliers on the one hand and the contractors on the other. At the same time, the designer often expects the component manufacturer to have a range of understanding of building problems that close examination would reveal to be unreasonable.

When put in this way, it is clear that the problems of avoiding obvious defects in design of surface finishes are formidable. So much effort must go into this that little is left for solving the other weathering problems. When an attempt is made, it is often too late to alter major details because these have been settled at an earlier stage.

It is small wonder, therefore, that, instead of controlling the weathering of buildings, attempts are often made to prevent it. As this fits in with the quest for eternal youth, considerable effort is being expended to achieve it. But some building materials just are not weather resistant in the right way. Invisible surface treatments are needed to enable the appearance to remain unchanged. Experience has shown that these are difficult to develop, and those available now have limited durability. The most familiar example is that of clear varnishes for timber. These require maintenance at 2–3 year intervals. Lacquers have been developed for preventing the weathering of copper, but they must be renewed after a period of 5–8 years. The poor weathering of concrete has boosted the use of silicone water-repellent treatments, but as with the controversial rejuvenation drug procaine, silicones only alleviate the symptoms of ageing, they do not prevent. Regular courses of treatment are required and these can be costly.

These thoughts on surface finishes may be summed up by the following statements:

1 Variety is achieved at the expense of quality.
2 Quality comes with repetition, or at least with the ability to discard the first attempts.
3 Eternal youth requires costly medication, and may, in the end, be illusory.

Fig. 1
The detailing of surface finishes occasionally overlooks some important aspects of the structural system of the building
(Photo by courtesy of the Institute of Building)

Fig. 2
Non-uniformity of weathering can produce interesting and unexpected results.
Unfortunately this is the exception rather than the rule
(Photo by courtesy of the Institute of Building)

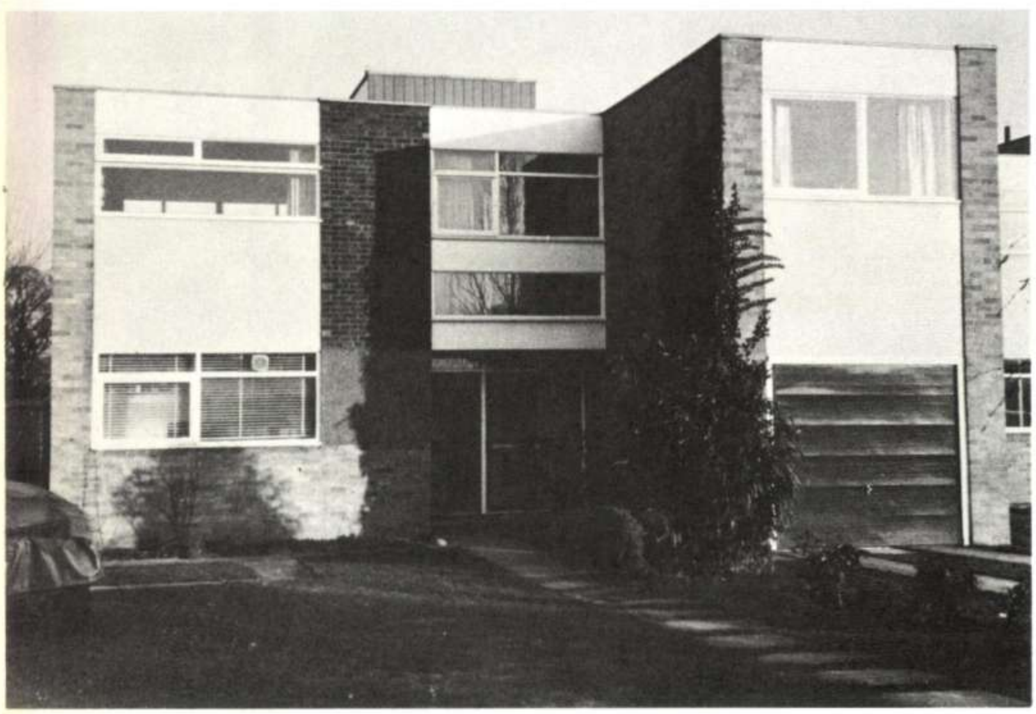


Fig. 3
Variety of surface finishes is apparent in all types of building. This modern house has two colours of brick (dark brown, grey-brown), a panel of dark blue mosaic, light blue painted panels below first floor windows, dark blue painted garage door, drab-green painted timber cladding to water tank, and white paint elsewhere
(Photo by courtesy of the Institute of Building)