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Contents

The cover, front and back shows main parade ground at Wellington Barracks, Photo: Axel Poignant

Temporary stabling at Wellington Barracks

Melvyn Grant

The keynote to this job is in the first word of the title - temporary.

KNIGHTSBRIDGE
The Minister of Public Building and Works decided that the picturesque but somewhat dilapidated Cavalry Barracks at Knightsbridge, which have stood guard on Hyde Park for close on a hundred years, needed to be completely rebuilt. Knightsbridge was the home of the Household Cavalry squadrons who traditionally mount the Sovereign's Escort on all state occasions.

BUCKINGHAM PALACE
It was considered necessary for the Cavalry to remain in London, so a search began to find a location with a reasonably clear atmosphere that was not too far from Buckingham Palace. Wellington Barracks was the obvious choice, although the decision did not please the Footguards who have had to cede their quarters to the 'mounties' whilst the £3 million reconstruction at Knightsbridge is in progress. Wellington overlooks St. James's Park and is so close to the Palace that when the wind is in the right direction you can hear the corgis barking. However, proximity to the 'Big House' immediately presented us with a problem. The buildings should in no way offend Her Majesty, or, to be more precise, should not offend the sensibilities of those who claim to know what would cause displeasure to our royal neighbours.
A further factor to which consideration had to be given was that the main parade ground was used to form up for the changing of the guard ceremony, as well as the staging area for most state occasions. One could always be sure of meeting a guards officer in anything from sports jacket and bowler hat to the full ceremonial regalia.

SIR BASIL SPENCE, O. M., R. A.
The architect appointed by the Ministry for Knightsbridge, and so for Wellington, was Sir Basil Spence. Compared to Knightsbridge we had a pretty meagre purse. The total cost was £135,000 of which £73,500 was structural cost.

THE SITE
The site is bounded by Birdcage Walk to the north and Petty France to the south. Queen Anne's Gate is to the east and the Palace to the west. Existing buildings retained were married quarters, barrack rooms, offices and officers' mess. Various Nissen hut type structures such as a mess hall, gymnasium, garages, etc. were demolished as well as a rather fine old circus building.

DEMOLISHING THE CIRCUS BUILDING
The circus building, probably the oldest of its kind in London, was the subject of a rather belated appeal by Astragal of The Architects' Journal. On 16 October 1963 he asked, 'How many eagle-eyed architects passing along Petty France have wondered what that mysterious circular building beside Queen Anne's Mansions is? It is a Victorian circus - date unknown, about 1850 suspected - which was converted for use as a garage years ago, and is now being reconverted (sic) by Sir Basil Spence and Ove Arup & Partners to accommodate all the Queen's horses from Knightsbridge Barracks during rebuilding. The impressive cast iron roof span is about 125 ft., and

Fig.1
Petty France Circus, Westminster, April 1964. Elevation
Photo: S.W. Newbery
Reproduced with the permission of the National Monuments Record

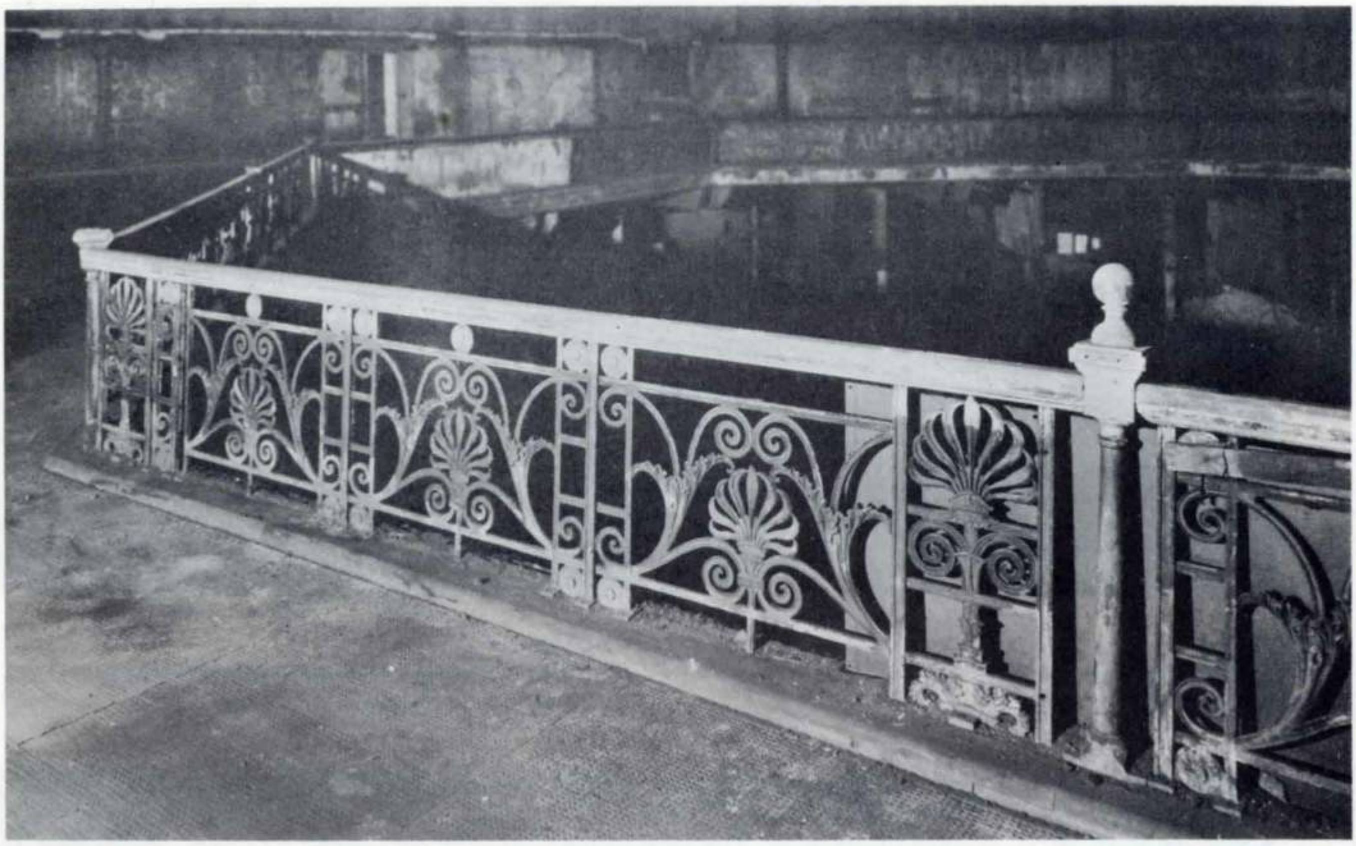

Fig.2
Petty France Circus, Westminster, April 1964.
Balustrade detail
Photo: S.W. Newbery
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the National Monuments Record

Fig.3
Petty France Circus, Westminster, April 1964.
Inside the circus
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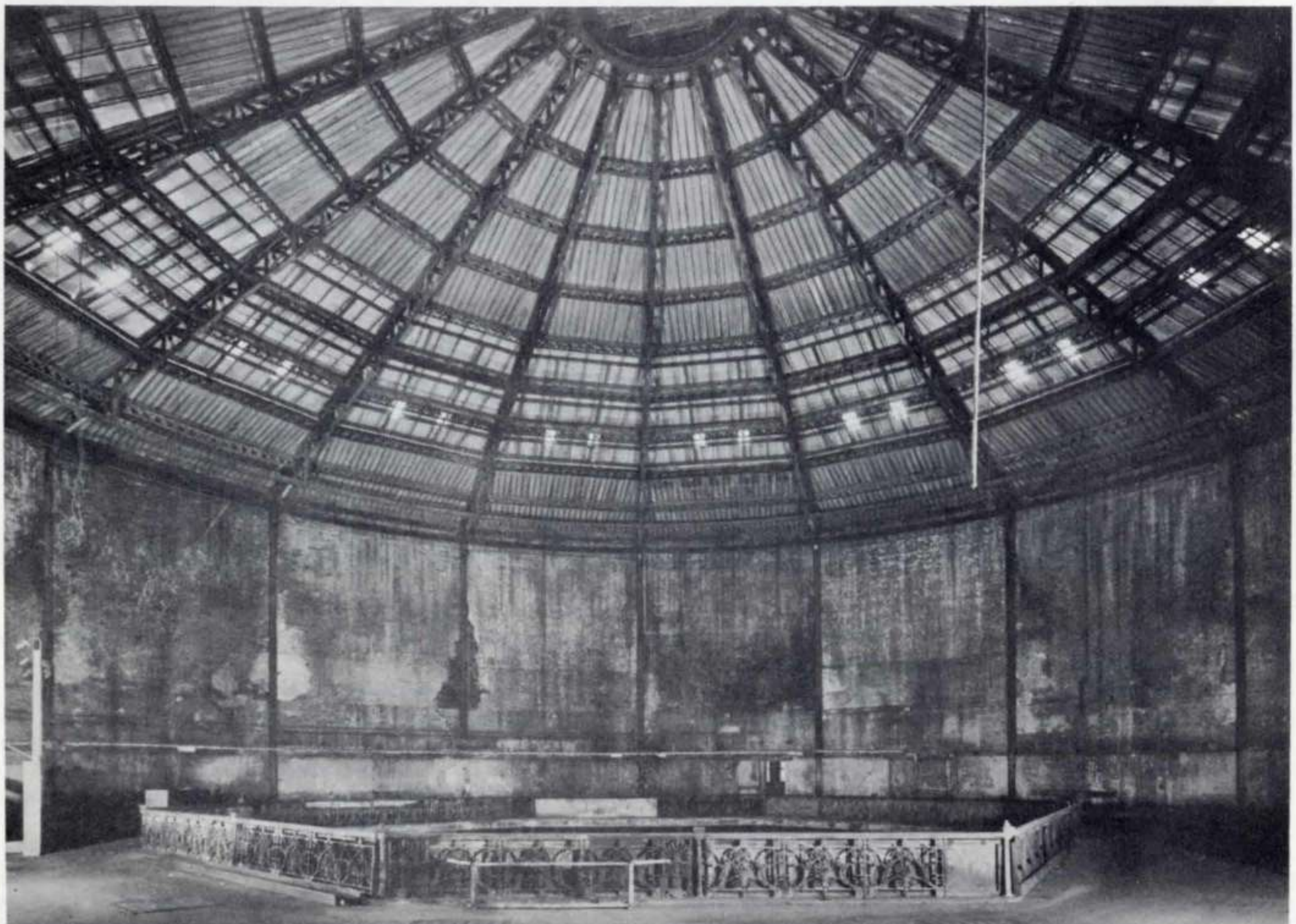


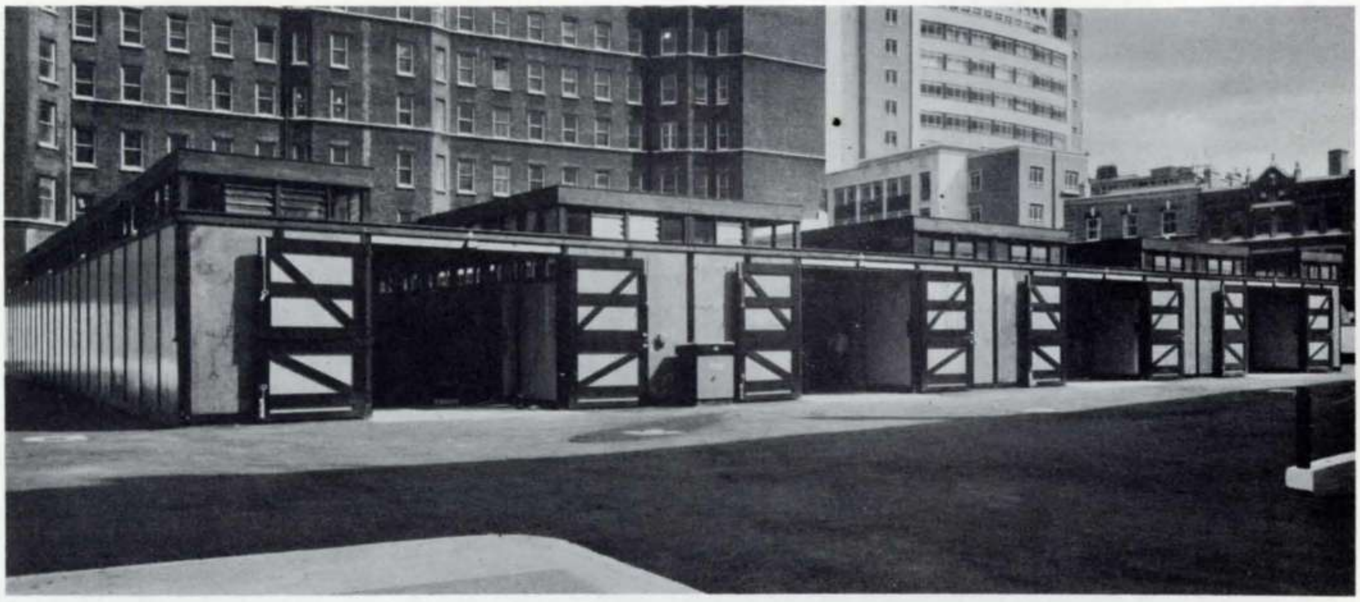


Fig. 4
Details of shoeing shop
Photo: Axel Poignant

Fig. 5
Life Guards and London District troop lines
Photo: Axel Poignant

if relieved of depressing accretions the building is worthy of the attention of the Victorian Society and others.'
On 12 August 1964 he sadly recalls, 'Last October Astragal was asking how many eagle-eyed architects had noted the Victorian circus building in Petty France: now he must inquire how many of the same have noted that the building has been demolished. The intention was to convert it to temporary use as stabling for the horses from Knightsbridge Barracks, while the new stables are built. However, when one of the Arup brigade went on the roof of the big top he was so frightened by the state of the wrought ironwork that he came down again like a mouse on a clock, and demolition was decreed forthwith.' The Arupian, who shall be nameless, is one of Ted Happold's crew.
At the eastern end of the site is the Guards Chapel. The original chapel received a direct bomb hit during World War II. Some 140 Guards officers and their families were wiped out. All that remained was the eastern end of the chapel which has been incorporated into the new building and is still visible from Birdcage Walk.

THE BRIEF
Our brief was to provide a self-contained unit, to house all the horses and ancillary equipment for the Household Cavalry. Basically, the buildings had to be aesthetically

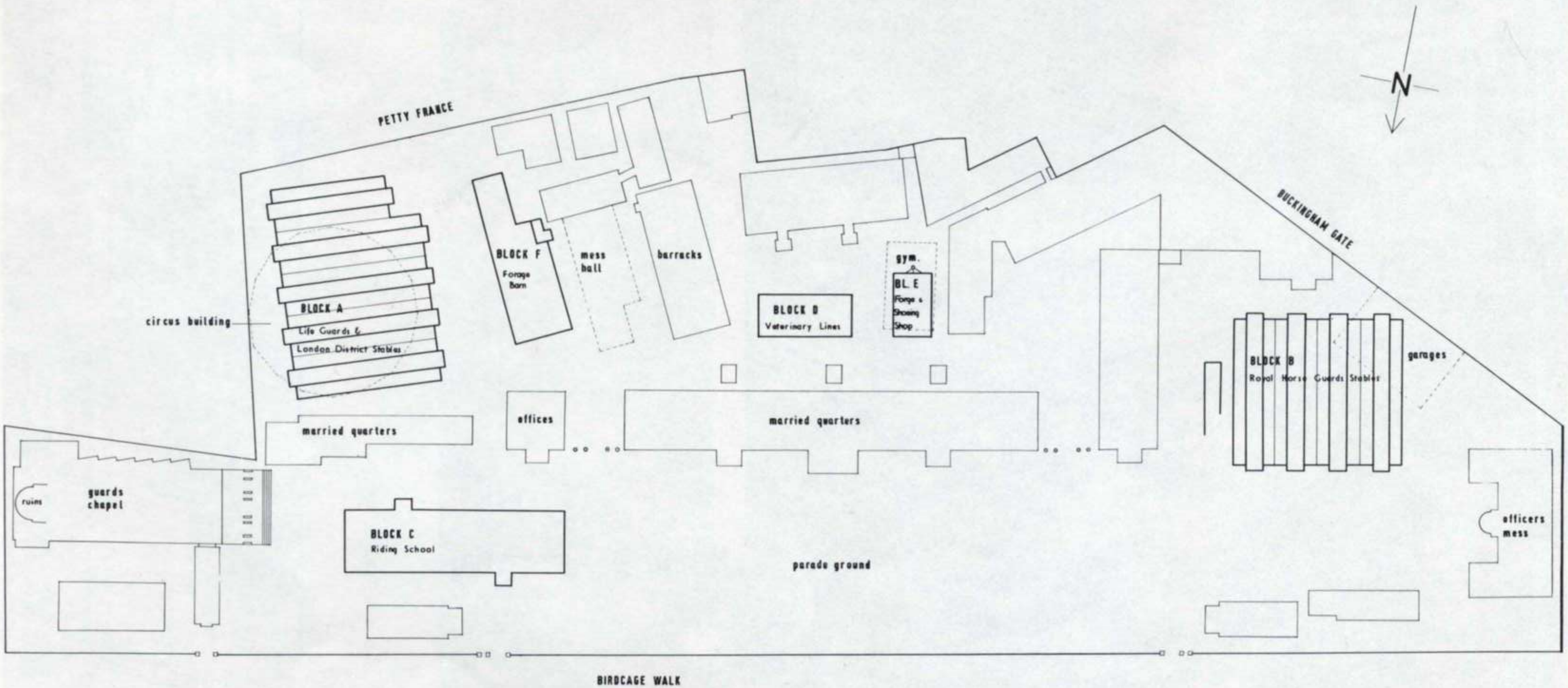


Fig. 6
Site plan.

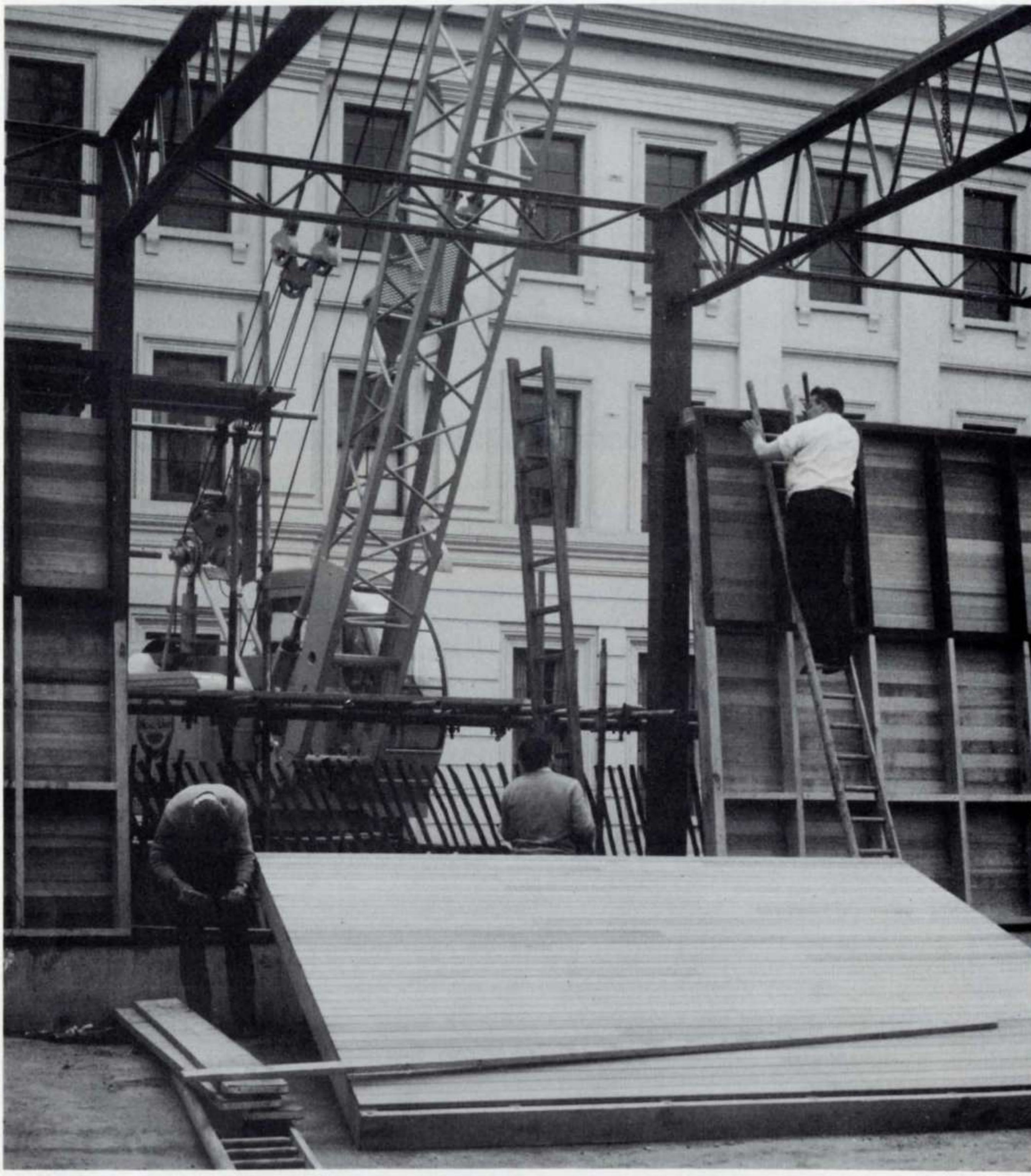


Fig. 7
Erection of wall panels in Riding School
Photo: Axel Poignant

pleasing, durable for the anticipated period of occupation and lastly, temporary, i.e. easily demountable, so that they can be removed when no longer required, possibly for resale. I understand that there have already been enquiries from prospective purchasers.

THE ANSWER

To fulfil all the requirements of the brief we decided to use a timber frame construction with timber infill panels to provide both cladding and overall lateral stability. The superstructure sits on a 6 in. mesh reinforced concrete slab with all kerbs, falls and drainage channels formed integrally with the slab.

The two stable blocks, block A which houses the 133 horses, 11 chargers and one drum horse of the Life Guards Squadron and Headquarters London District, and block B, 102 horses, 8 chargers and one drum horse of the Royal Horse Guards (the Blues), are both built to this plan. A charger is an officer's mount and so merits a loose box twice as large as the tethering space afforded to a trooper's horse, whilst the drum horse, which is the enormous brute that carries two kettle drums in parades, merits a space as large again. It was necessary to design all columns for the possible impact of either the rump or hoof of this largest horse, which may weigh over a ton.

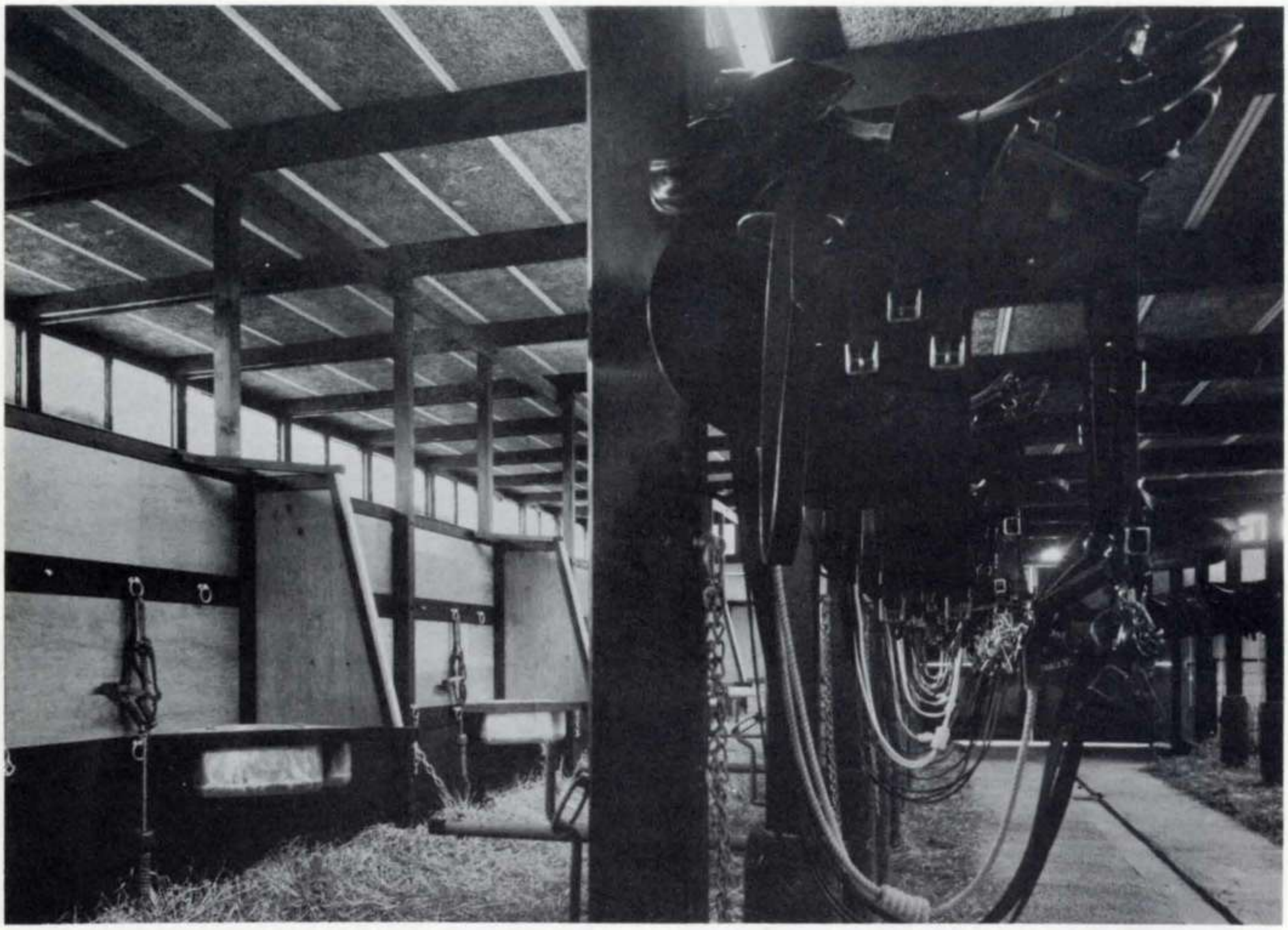


Fig. 8 above
Troop line interior
Photo: Axel Poignant

Fig. 9 below
The forge, the 'doctored' anvil is on the left
Photo: Axel Poignant

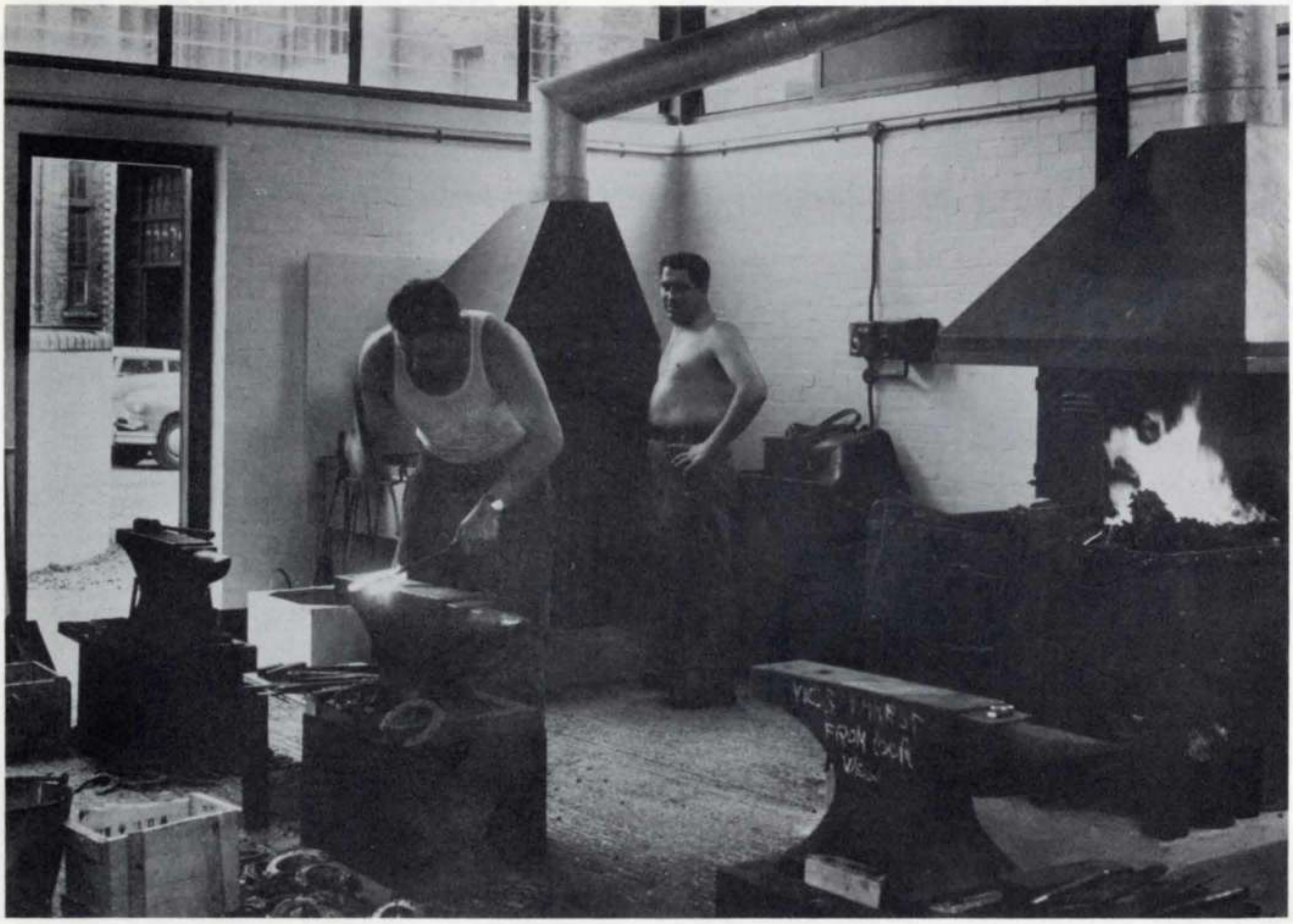
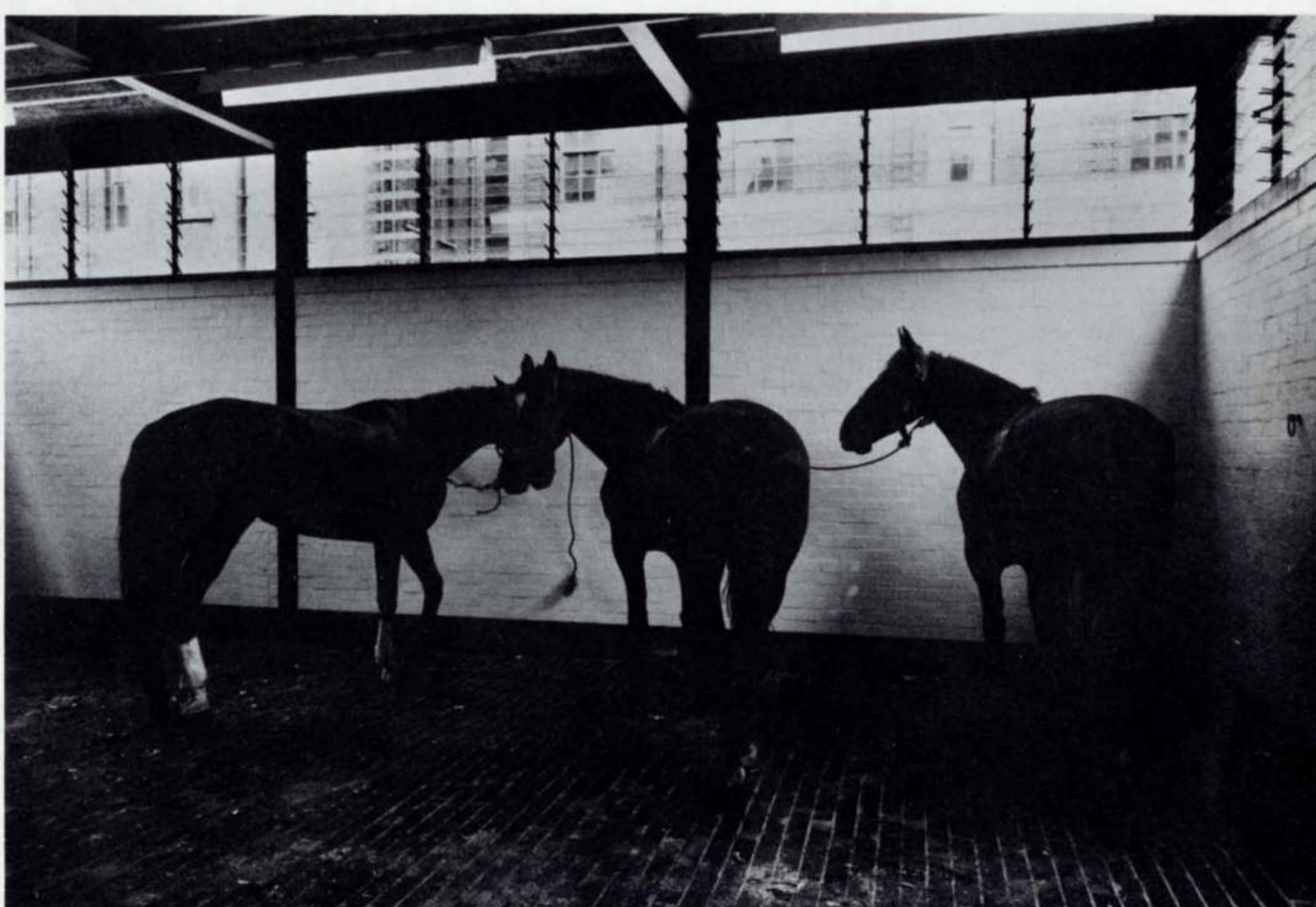


Fig. 10 above
Inside the Riding School
Photo: Axel Poignant

Fig. 11 below
The shoeing shop
Photo: Axel Poignant



Block D, the Veterinary Lines, has a similar form of construction as the stable blocks, with closed-in office accommodation at one end and loose boxes for the rest. In the examination room we installed an additional twin beam capable of carrying a central point load of a ton, as sick horses are hoisted right off their feet for examination to prevent their kicking. Once the beam was installed it looked so slender that the military began to cast doubts on its capacity. So we organized a beam test. On the appointed day, surrounded by riding masters, veterinary officers and the like, we gravely loaded the beam, via block and tackle, with 56 lb. weights to a maximum of $1\frac{1}{4}$ tons and recorded deflections. We were extremely relieved when the maximum deflection of 0.07 ft. coincided with the $\frac{3}{4}$ in. we had rashly predicted a week earlier.
Block F, the Forage Barn, was an existing barrack block about 50 years old with shooting ranges in the basement. Although the ranges were no longer in use, we didn't want all the fodder and suchlike disappearing into the basement. So we cut some holes in the floor to determine its construction. We found a 14 in. x 6 in. (approximately) metal section, repeating on an 11 ft. module. The floor slab was 6 in. thick with a 6 in. square mesh reinforcement draped between the joists which were also cased in concrete. Assuming composite action, we determined that the slab oould possibly carry a superload of 120 pounds per sq.ft., so applying Cook's Law we gave a maximum design load of 60 pounds per sq.ft. But since it is not too easy to measure the density of fodder, we are keeping our fingers crossed.
Block E, the Forge and Shoeing Shop, by its very nature needs to be a more substantial structure, so we used a universal beam and column frame with cavity wall brick infill panels. In the forge are three anvils and three forges, which are served by a 60 ft. high aluminium clad steel chimney. The anvils are part of the equipment transferred from Knightsbridge. The Army specified that they had to be set on a timber anvil block 1 ft. 9 in. x 1 ft. 6 in. x 3 ft. 2 in. high, sitting on a 9 in. bed of graded hardcore. This was to provide some rebound or resilience for the smith when striking the anvil, otherwise he would dislocate his shoulder. We discovered that in the new Knightsbridge Barracks the proposed forge was not at ground level. How was one to fulfil the specification in that case? We suggested that we try out an anvil sitting on a sawn-off block on a NEOPRENE pad. When it came to a practical demonstration the smith could not tell which anvil had been doctored.
Our 'pièce de résistance' was block C, the Riding School. Here we had to take great pains, as we are permanently on view from Birdcage Walk. We adopted a portal frame construction, on a 12 ft. module, with 26 in. deep METSEC trusses exposed internally and 8 in. square rectangular hollow section columns exposed internally. The walls are preformed timber units which sit on a concrete kerb and are bolted to the columns. The walls themselves rake outwards at a slope of one in five. This is functional, in that it prevents riders' legs from being injured and is also aesthetically pleasing. The floor of the Riding School, which is retained by the kerb, is a 12 in. layer of compressed tan especially transported from Knightsbridge. Tan would appear to have the following composition - 50% sawdust, 40% sand and 10% horse manure.
Probably our most ridiculous problem was to find a tarmac surface for the parade ground, suitable for horses to walk on comfortably while still allowing foot soldiers to march in formation. Everybody from the general down seemed to have different ideas of what a horse liked. Eventually, we looked at the existing surface, which contained a smooth, large size aggregate, and which horses like, and then produced a specification to match this as closely as possible.
Work started on site in January 1965. The contractors were Walter Lawrence and Sons Ltd. The structure was sensibly complete by the end of June and the horses in habitation by the beginning of August.

WIND; FACTS, FICTION AND FIGURES

These papers will be discussed at the April Technical Staff Meeting. Everyone is welcome to attend.

The nature of wind

John Martin

It is valuable for an engineer to have a good feeling for the characteristics of wind even if it is not yet possible to put numbers to some of them. To relate what is known, in a very general way, will not take long, because although large quantities of writing exist on the subject and it has become a mathematician's playground, in fact most of the work on the nature of wind, as opposed to the analysis of its effects on structure, consists of attempts to fit what little is known of its behaviour into mathematical equations.

RECORDS
Let us imagine, to begin with, that you are making a record of the strength of the wind at one place. It is at ground level, and it might as well be pleasant, because you have to spend several years there. If you plotted a graph of wind speed and studied the variations very carefully, you might notice that the windy and calm periods recur at intervals which tend to follow a number of patterns. There are actually several quite separate patterns or cycles of windier and calmer periods but all these patterns are superimposed, of course, and whenever a particularly strong wind or gust recurs, it is due to the additive effect of the peaks of separate cycles acting simultaneously. There are large-scale patterns, where the peak periods of stronger winds are more than an hour or so apart, and small-scale patterns, where the cycles of gusts recur at less than 5 minute intervals. It is interesting to observe that there is no pattern with a frequency between 5 minutes and an hour. This is very useful to the engineer because in different countries it is the practice to record the average wind speeds for different periods, which range between 5 minutes and one hour, and since these averages are virtually the same, the results can be used in the same way in our design.
The cycles which have a large time scale are all due to meteorological effects, and they range from the very large low frequency patterns, sun-spot effects and yearly seasonal changes, to the smaller scale cycles of four day low pressure systems and the homely half-daily changes from tranquillity at dawn and dusk to windy periods at midday and midnight. So far as the design of our structures is concerned, these long cycles produce only steady winds. The periods are too big to concern us. They are of interest

only in our estimation of the likely maximum steady winds for which we must cater.

GUSTS

The short-period fluctuations of wind, gusts in fact, are caused by the nature of the ground surface, and these are very important to us. Gusts are measured in durations which vary from 1 second which is the shortest most anemometers will record, up to five minutes. Clearly the shorter the period, the higher is the maximum gust speed which can last that long. Also, since gustiness is caused by the roughness of the ground surface, it is possible to relate it to types of terrain. So it is gustier in large cities, where most of our tall structures are, than it is in gently rolling countryside.

It is not that winds are stronger in a built-up area, on the contrary. If you think of the rough surface of a city as offering friction to the wind, you will see how it at the same time slows down the wind and also causes turbulences and gusts. In certain circumstances the gusts can be a more serious problem than a steady strong wind.

Now, having studied the wind of one point, let us move at right angles to its direction and see what changes there are. You find that gusts are often quite narrow things. In fact, you can imagine that a plan of wind speeds over an area at a given moment would show great variations all over. One could draw a 'contour' map of the wind speeds over the area. Figure 1a indicates what this might look like. In fact this figure really shows a record of wind speeds at one second intervals taken on a series of stations 60 ft. apart roughly perpendicular to the direction of the wind. The implication of this is that a wide building is unlikely to be as troubled by a strong gust as a narrow building, because the peak of the gust will hit different points on its face at different times.

Let us travel upwards and see how the behaviour of the wind changes with height. First, you find that gust speeds vary 'in elevation' as much as they do 'in plan'. This is shown in Figure 1b. Horizontal and vertical sections at B-B are shown in Figure 1c. Also, since gustiness is caused by roughness of the ground surface, it is to be expected that the higher you go above the ground the less gusty does it

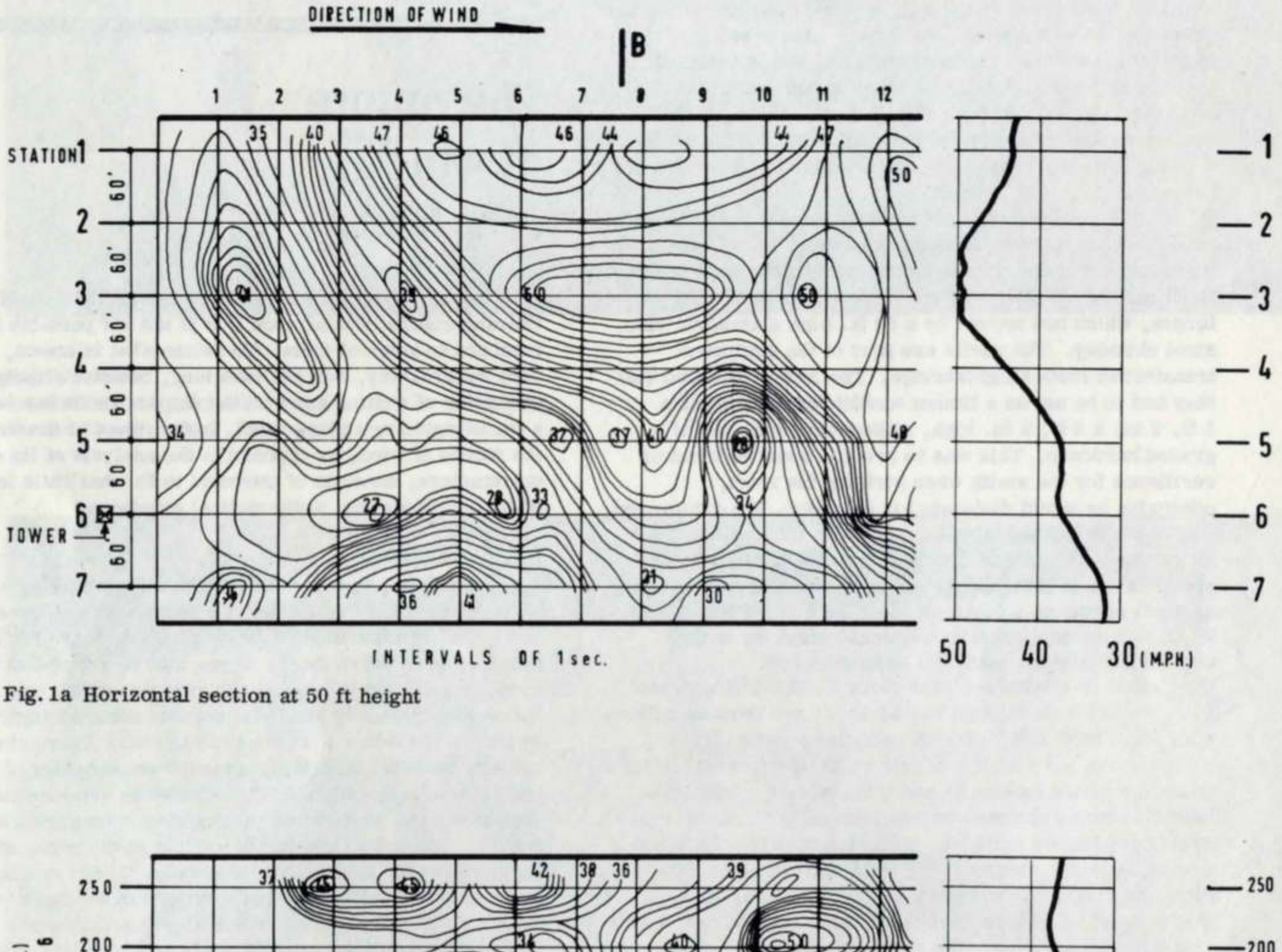


Fig. 1a Horizontal section at 50 ft height

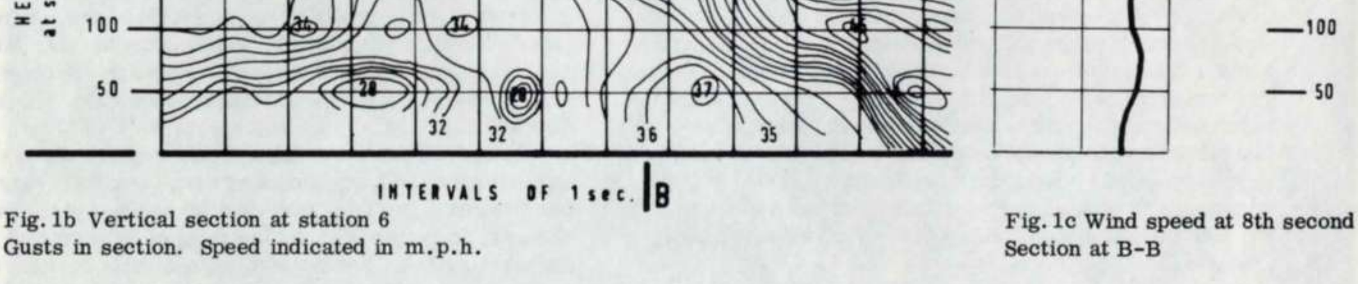


Fig. 1b Vertical section at station 6
Gusts in section. Speed indicated in m.p.h.

Fig. 1c Wind speed at 8th second
Section at B-B

Figs. 1a to 1c Wind speeds

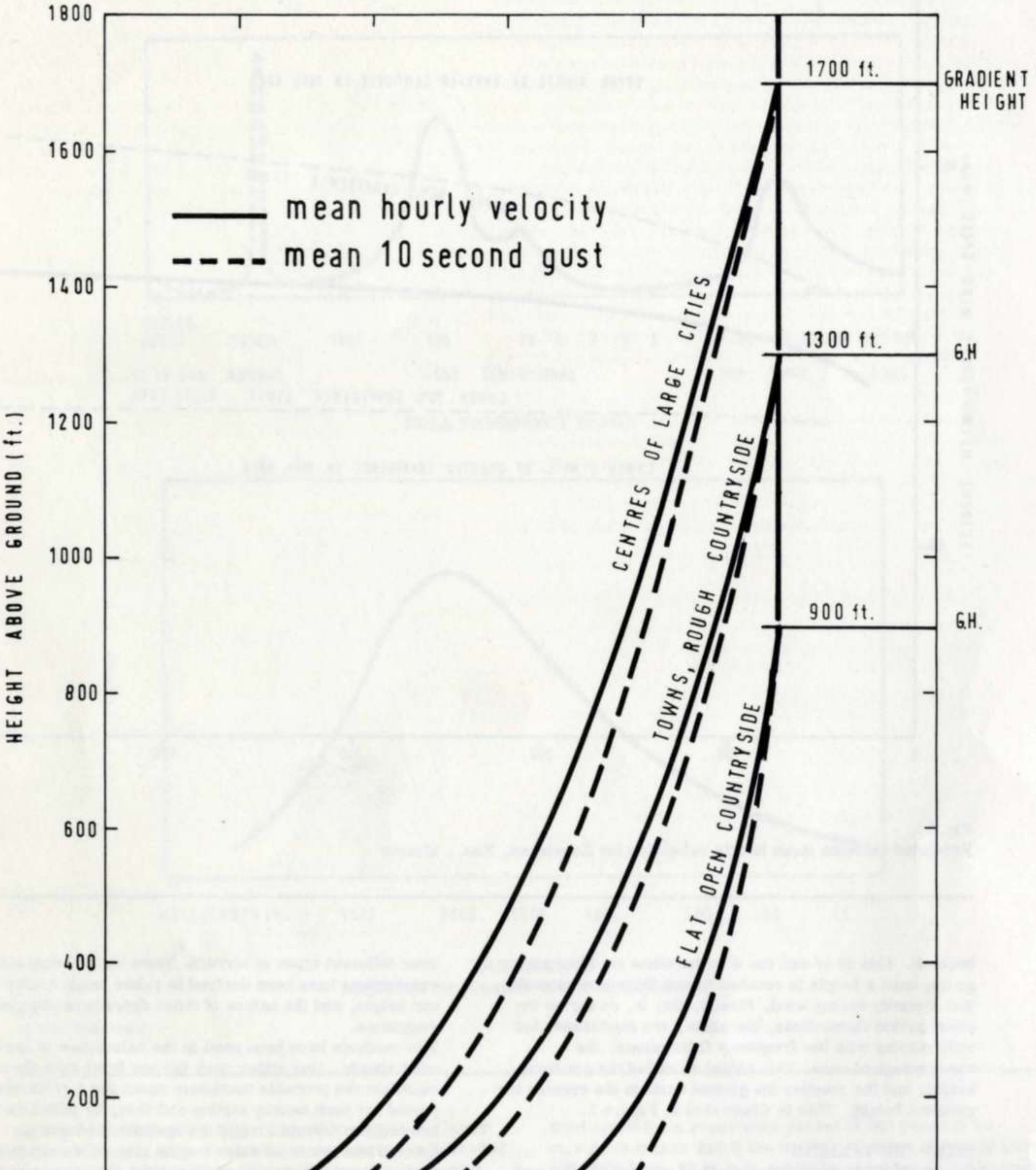


Fig. 2 Escalation of wind with height

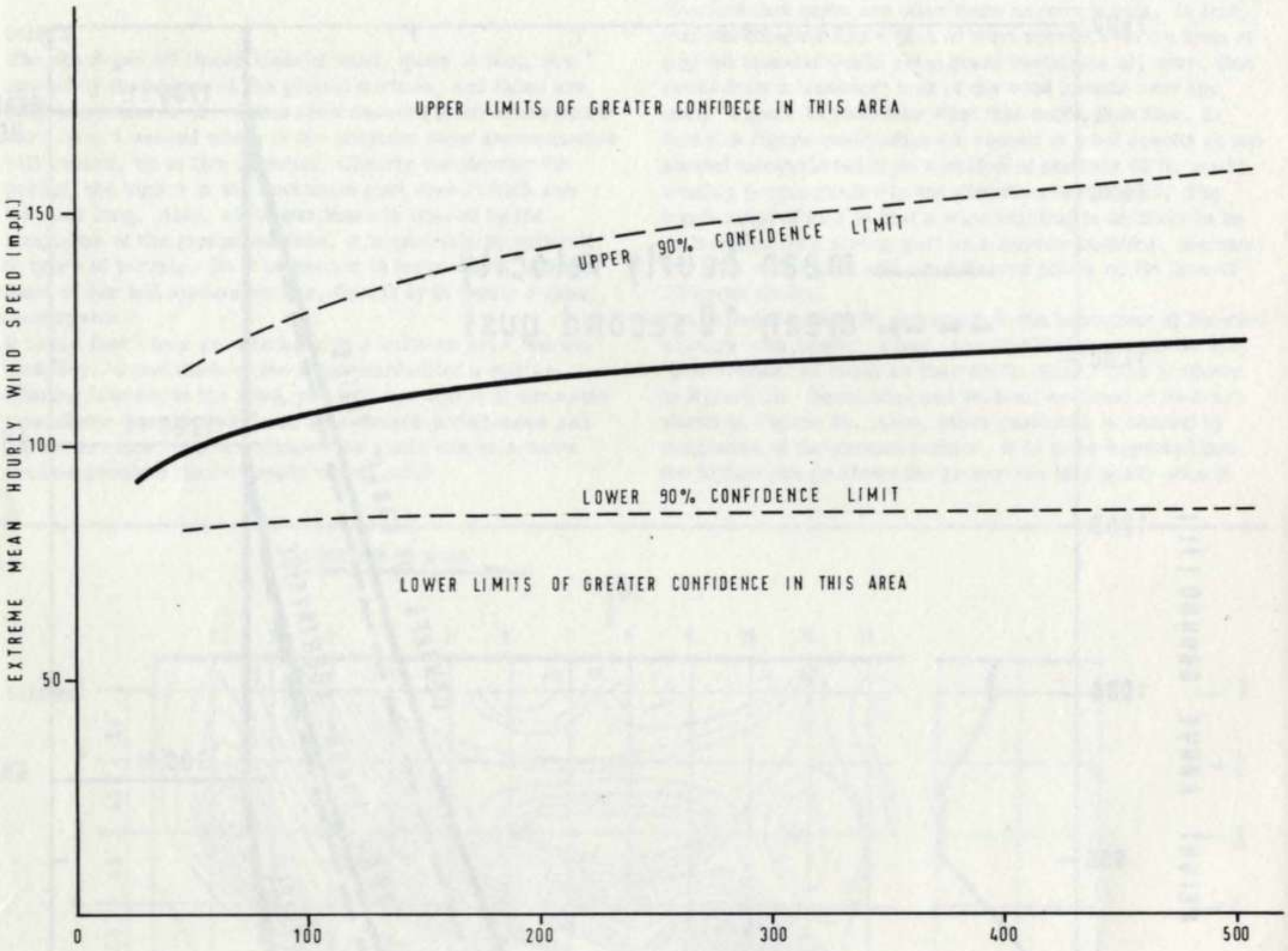
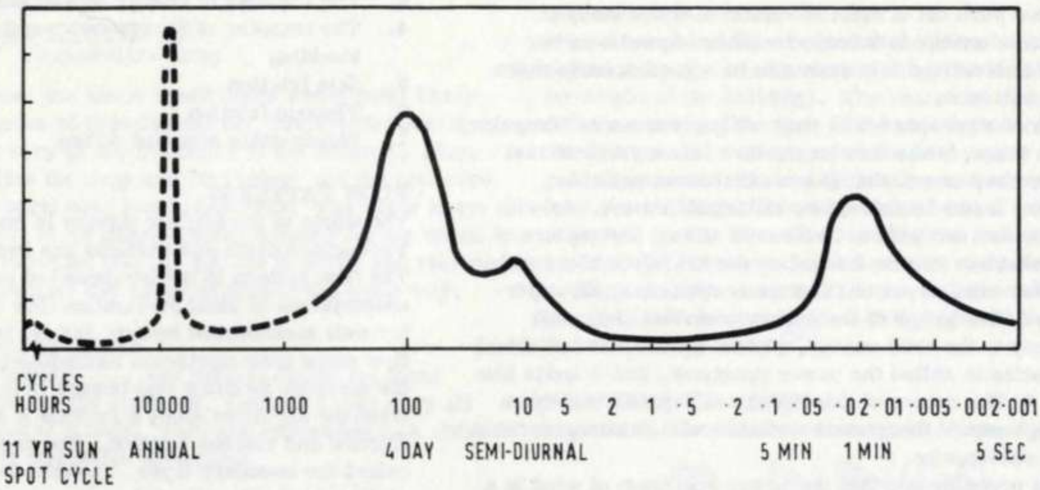


Fig. 3
Predicted extreme mean hourly velocities for Zacatecas, Zac., Mexico

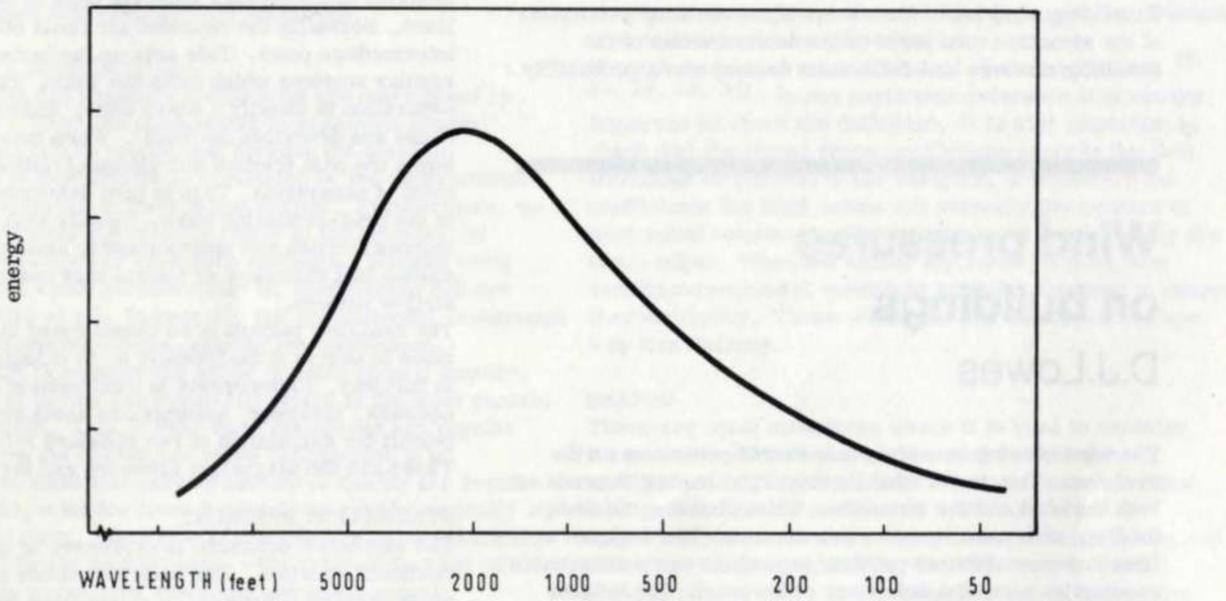
become. This is so and the wind becomes steadier as you go up, until a height is reached where there are no gusts, just a steady strong wind. Steady, that is, so far as the short period fluctuations, the gusts, are concerned, but still varying with low frequency fluctuations, the meteorological ones. This height is called the gradient height, and the rougher the ground surface the greater the gradient height. This is illustrated in Figure 2.

HILLS AND VALLEYS
One must at least mention the effect on wind of hills and valleys. This is a different matter from surface roughness, which is mainly a question of smaller scale obstructions to the wind, such as buildings. Here one is thinking of such matters as the speeding up of wind as it passes over a ridge, channelling along valleys, and lee waves. It should not be thought that if you are standing in the lee of a hill you are necessarily sheltered. There can be considerable turbulence and lee waves in such places and severe gale damage at Sheffield in 1962 was ascribed to this cause. This can present a real danger and increases of 10-20% in wind speed are suggested to allow for turbulence increases in the lee of hills.

WIND SPEEDS
The basic information consists of mainly low level records of wind speed and direction on strip charts kept at weather stations, and some high level readings which serve to provide a guide to the variation of wind speed with height over different types of terrain. Some simple mathematical expressions have been derived to relate mean hourly velocity and height, and the nature of these depends on the ground roughness.
Two methods have been used in the calculation of basic design wind speeds. One either uses the low level records to calculate the probable maximum speed for a given return period for each nearby station and then, by judicious averaging knowing the terrain around the stations, chooses an appropriate low level value for the site. This can then be extended upwards by calculation using the relationship which suits the ground roughness. Alternatively, one can calculate the probable maximum speed for the required return period at gradient height above each weather station, using this information to compile a map of gradient wind speeds for the area. You then use as a basis the gradient wind speed over your site and, reversing the previous process, use the appropriate ground roughness to calculate the wind speed at any lower level.
These two methods should come to the same thing in the end, but because of the rather shaky nature of the mathematics normally employed to relate low level and gradient height wind speeds, it can be more accurate to adopt the first. We have used it in our studies of wind speeds in Mexico and, incidentally, we have produced our own computer programme to calculate maximum speeds for given return periods from station readings. In practice, it seems best to use this approach for important problems which justify more investigation, and to use the latter, the gradient height



FULL FREQUENCY RANGE



GUST RANGE
(important for dynamic effects on structures)

Fig. 4
Power spectrum

approach, for everyday work. The advantage of the latter is that we have a map of extreme mean hourly gradient wind speed for the British Isles. Using this map, and knowing the nature of the terrain round your site, it is easy to calculate gust speeds at any height, although the answers obtained near ground level may have to be carefully adjusted. This latter method is that used in Technical Note 26.* There are both statistical and engineering grounds for using the mean hourly velocity as the basic value for design, gusting then being considered as a fluctuating velocity superimposed on this basic wind speed. From the values of maximum annual mean hourly velocities thus recorded at a particular place, it is possible to calculate the probability of any particular speed, or conversely and more usually, to find the speed for any given probability. This probability is often referred to in terms of the 'recurrence' or 'return' period. As an example, if the wind speed for a recurrence period of 100 years is 90 m.p.h. it means that if the number of years of record were infinitely long, 90 m.p.h. would be exceeded on the average every 100 years. Plainly, the longer the records the better the prediction. But they will never be long enough for an exact prediction to be made; it will always lie between certain 'confidence limits', the range of which will depend on the quality of the data and the recurrence period chosen. Thus, naturally enough, the greater the confidence required the higher the upper limit will be. Figure 3 shows this diagrammatically. From the given wind records of a weather station, our computer programme can be used to determine both the predicted wind speed for any recurrence period and also the upper and lower limits of wind speed for any required confidence level.

POWER SPECTRUM

It is traditional to assume that the wind pressure acts statically, but we are beginning to build tall structures which might be troubled by the dynamic effects of wind, and account must be taken of the period of gusting.

* Ove Arup & Partners. Technical note no. 26 - Wind pressure on buildings and towers, by J.G. Nutt. 1961.

To do justice to a dynamic approach to design would be outside our scope here, but to round off this short description of the nature of wind, the concept of the power spectrum should be introduced. Its application cannot be developed here but it must be understood if a study of dynamic behaviour is later to be taken beyond a rather empirical level and this seems to be a good point to make its acquaintance.

A graph of wind speed with time will appear as an irregular, random trace, without order, rather like a graph of test cube results, except that it is a continuous variable. However, it can be defined by statistical means. As with other random variables, like waves at sea, the pattern of their behaviour can be defined as the net effect of a number of regular sine waves of different frequencies, all superimposed. The graph of the energy contributed at each frequency to the total energy, plotted against the individual frequencies is called the power spectrum, and it looks like Figure 4. The nature of this spectrum depends mainly on the roughness of the ground surface and it is characteristic for the chosen site.

You can probably see that the power spectrum of wind is a most important factor in determining the dynamic response of a structure because the spectrum can define the energy of the various frequencies put into the structure by the fluctuating wind load. Knowledge of the dynamic properties of the structure then leads to the determination of the resulting stresses and deflections for any given probability.

Wind pressures on buildings

D.J.Lowes

The wind blowing on a structure exerts pressures on the surfaces. This is the wind loading. The loading depends on both the wind and the structure. When choosing the design loading, both must be taken into account. The designer's first and most difficult problem is to make some reasonable assumption about the behaviour of the wind. The subject of this article is the second and easier problem, which is to estimate the loading due to the assumed wind.

FLUID DYNAMICS

Air is a fluid. The science devoted to the study of fluids in motion is called fluid dynamics. The basic physical phenomenon involved is that defined by Newton's Second Law of Motion: the rate of change of momentum is proportional to the force applied and takes place in the direction of that force. All fluid dynamic pressure calculations depend on this law. However, fluid dynamics is mainly an experimental science and a great many of the results depend on experimental observations[13].

FORCES

The total force is the resultant of all the surface pressures. Generally the direction of the force is different from that of the air stream. The exception is when the flow is symmetrical. When the flow is two-dimensional, the force is represented by its 'drag' and 'lift' components. These terms come from aerodynamics. (Structural engineers sometimes prefer the words 'normal' and 'tangential'). The drag force acts in the direction of the air stream and the lift force perpendicular to that direction. Both forces act in the plane in which the airflow is taking place. In flow around tall buildings, drag and lift act horizontally. The concept of the wind losing momentum helps identify the causes of drag. According to Newton II, the force acting on the wind is the only reason for the reduction in momentum. The force in question is the reaction to the drag. The momentum is reduced by the following:

1. Deceleration of the air in front of the building
2. Transverse acceleration of some of the air
3. The transfer of energy into turbulent motion
4. The transfer of energy into regular systems of vortex shedding
5. Skin friction
6. Viscous friction
7. Steady-state attached eddies

FLOW PATTERNS

The whole of the airflow pattern is involved in drag. Turbulent motion and vortices are particularly important. The flow pattern is not composed of parallel filaments. The assumptions of ideal streamline flow are approximately true for well streamlined bodies, but not for building. Separate flow paths from upstream immediately lose their identity in the complex swirling that occurs in the air as it passes by. When the air flows along a surface it is retarded by skin friction and viscous friction. The zone of retarded air is called the boundary layer. Outside the boundary layer the airflow is unaffected by the surface. It is as if the body had a streamlined shape like the outside curves of its boundary layer. Unfortunately the boundary layer only remains attached to the surface when the body is very well streamlined. Normally the retarded air peels off at some intermediate point. This sets up the turbulent eddies or regular vortices which form the wake. The point of separation is usually a sharp edge. Bodies with sharp edges are described as 'bluff'. When there are no sharp edges the skin friction and viscous friction determine the point of separation. This in turn determines the formation of the drag-producing wake. Usually skin friction and viscous friction are unimportant to buildings, but on rounded shapes like chimneys or towers they control the whole airflow pattern.

The real flow pattern is so complicated that no attempt is made to derive it analytically or to calculate the drag force in that way. Experiments in wind tunnels provide the answers. However, assumptions about the airflow pattern permit the calculation of two standard reference figures. These are the stagnation pressure and the hypothetical drag.

STAGNATION PRESSURE

The stagnation pressure is expressed by a form of the Bernoulli equation. The theory assumes an ideal fluid flowing along parallel filaments. If at one point in a stream tube all the kinetic energy is converted to pressure energy, then:

$$p_s = \tfrac{1}{2}\rho v^2$$

where p_s is the stagnation pressure (also called 'dynamic' or 'velocity' pressure)
ρ is the mass density
v is the velocity

If the velocity is in ft/sec and the mass density in slugs/cu.ft. (1 slug = 32.2 lb. mass) then the stagnation pressure is in lbf/sq.ft.

HYPOTHETICAL DRAG

The hypothetical drag is calculated assuming that the stagnation pressure acts on an elevation with area A. The elevation is usually, but not always, the windward elevation.

$$F_{HD} = \tfrac{1}{2}\rho v^2 A$$

The implied assumption is that all the air flowing towards the building is brought to a dead stop on the windward surface ... and then disappears. The air to the sides and behind the building is assumed to be unaffected by this occurrence.

SHAPE COEFFICIENTS

Real drag and lift forces are related to hypothetical forces by shape coefficients. They are based on wind tunnel measurements. They depend on:

1. The external shape of the object
2. The direction of the wind
3. Sometimes the velocity of the wind

The drag coefficient is

$$C_D = \frac{\text{measured drag}}{\text{hypothetical drag}}$$

and the lift coefficient

$$C_L = \frac{\text{measured lift}}{\text{hypothetical drag}}$$

(In aerodynamics, the shape coefficients would most likely be defined in terms of hypothetical lift, not hypothetical drag. The definitions vary in the literature of the subject). Shape coefficients define the drag and lift forces, not the pressure distribution or particular local pressures. The same shape coefficients apply to all geometrically similar bodies which differ in size, provided that the airflow patterns are also geometrically similar and differ in size in the same way.

PRESSURE COEFFICIENTS

Pressure coefficients, C_P, are used to define the local pressures normal to the surfaces. Naturally they vary all over the surfaces of a body. They are expressed as a ratio of measured and stagnation pressures

$$C_P = \frac{\text{measured local normal pressure}}{\text{stagnation pressure}}$$

Like shape coefficients, they are the same for geometrically similar bodies with similar flow conditions.

EMPIRICAL BACKGROUND

All drag, lift and pressure coefficients are determined by experiment. They are often stated in somewhat general terms. This can be misleading because the sensible application of coefficients is restricted by the experimental conditions involved in their measurement. Furthermore, the shape coefficients specifically relate to the hypothetical drag on one particular reference elevation. Without being certain about which elevation this is, coefficients will not mean anything at all. In general, the experimental background must be considered before applying any coefficient.

In an attempt to broaden the scope of experimental enquiry, the shapes of bodies are ordinarily stated in the most general terms possible. This is easy when the shapes are regular solids. When the shapes are more complicated it is necessary to introduce various factors to qualify the results. For example, a lattice truss bridge is an aerodynamically complicated body. The shape of the truss is aerodynamically generalized by the solidity ratio - the ratio of the area of the elevation to the area contained within the outside boundaries of the elevation. When two trusses are spaced apart to make a bridge then the leeward is shielded by the windward. The spacing ratio allows for this effect - the ratio of the interval between trusses to the depth of the truss. The angle of incidence of the wind is always important but it is the custom to calculate the hypothetical drag on only the simplest normal elevation. The shape coefficients for a lattice bridge are therefore a function of the solidity ratio, the spacing ratio and the angle of incidence. As might be expected, the shape coefficients of complicated bodies are often expressed as formulae composed of several shape-descriptive general parameters.

Drag and lift coefficients apply to a body as a whole, not its component parts. The drag forces on individual members of a lattice truss, calculated as if they were alone in the windstream, are satisfactory for checking bending of individual members, but the strut and tie forces depend on the drag exerted on the structure as a whole. It is not possible to calculate the drag on a lattice structure by adding together drag forces appropriate to individual members. Likewise, on a body of any shape, the drag for a particular wind direction depends on the measured drag coefficient in that direction. It is not the vector sum of drag forces that could be attributed to the vector components of the wind velocity.

The shape coefficients of a body of constant cross section vary with its length. When the body is very long the flow is practically two-dimensional. When it is short the flow is three-dimensional because some air flows around the ends. This property of shape is described by the aspect ratio - the ratio of the length of an object to its width. In buildings the aspect ratio is twice the height divided by the width, because the ground represents a plane of symmetry in the air flow pattern. Shape coefficients must be adjusted for different aspect ratios [8, 17].

Two-dimensional flow cannot occur if the velocity of the approaching air stream varies along the length of the object (or height of the building). The reason is that pressure differences along the face of the object promote flow in the third dimension. In nature wind speed varies from the ground upwards. The result is often a downwash on the face of a building. This may be unpleasant for pedestrians at ground level. The effect on drag has not been extensively investigated. However, some results show that the drag coefficient is the same as that in a uniform velocity field, provided that the hypothetical drag is based on the root mean square velocity [16].

FORCE CALCULATIONS

In general, the wind forces are as follows:

$$\text{drag} \quad F_D = C_D \cdot \tfrac{1}{2} \rho v^2 . A$$
$$\text{lift} \quad F_L = C_L \cdot \tfrac{1}{2} \rho v^2 . A$$

It is usually reasonable and convenient to assume that the forces are applied as uniformly distributed loads, but this is only an assumption. It may be dangerous if the structure is sensitive to the load distribution.

Several references contain lists of shape coefficients [8, 15, 18, 19, 34]. In any particular reference it is always important to check the definition. It is also important to check that the stated shape coefficients apply to the flow conditions of interest to the designer. Fortunately the coefficients for bluff bodies are virtually independent of wind speed because the flow patterns are controlled by the sharp edges. When the bodies are more rounded then certain experimental conditions must be observed to ensure flow similarity. These conditions are discussed in Paper 4 by Ken Anthony.

SHAPES

There are some structures where it is vital to examine the pressure distribution as revealed by pressure coefficients. Cylindrical shapes are one example. Equal and opposite lateral suctions of more than twice the stagnation pressure act on the cylinder without making any difference to the drag or lift. On more conventional building shapes there are frequently local high pressure zones, for example the ridges of roofs [8].

AIR DENSITY

All the pressure calculations include a term for the mass density of air. This property varies with temperature and pressure [14, 18]. In the frequently quoted formula for the stagnation pressure

$$p_s = 0.00256\ V^2 \quad \text{lbf/sq.ft.},$$

where V is in m. p. h., the constant allows for $\rho = 0.00238$ slug/cu.ft. at 15°C. and 760 mm. mercury. At an altitude of 6,000 ft., say in Johannesburg, the air density and stagnation pressure would be about 20% less. Humidity makes very little difference, 1% at the most [14].

CP3 CHAPTER V

The most familiar reference for wind loading is CP3 Chapter V [33]. The figures in the tables are based on assumptions about the wind and measurements in wind tunnels. The assumptions involve informed guesses about the natural wind speed profile, the physical constants of the atmosphere and the response of the building. The experimental results were obtained from pressure measurements on small models of average buildings. The Code figures are therefore somewhat removed from first principles. They are not directly comparable with other literature on the subject and they do not lend themselves to extrapolation in unusual situations.

In the Code, the pressures in the Table of Basic Wind Pressures 'p' are not stagnation pressures. They are in fact stagnation pressures multiplied by a drag coefficient. The Code does not state the value of the drag coefficient but it is probably about 1.3 to 1.6 depending on what it is assumed was the velocity profile used in the calculations. These are reasonable values for fairly low rectangular buildings. The Table of Design Wind Pressure on Roofs and the Table of Shape Factors both refer to 'p' and therefore also allow for the included drag coefficient. The tables are therefore not tables of pressure coefficients or shape coefficients as defined above.

READY REFERENCES

Nearly all standard shapes have by now been tested in wind tunnels. The results have permeated the literature of the subject and found their way into codes of practice. Fortunately the majority of buildings may be safely designed in accordance with the provisions of CP3 Chapter V. However, the Code itself warns that its recommendations do not apply to exceptionally large buildings, or to those with unusual shapes or with large openings in the walls. Nevertheless, special wind tunnel tests are seldom necessary. The standard references do not contain all the answers, but they usually prove sufficient for most problems.

Vibration, damping and dynamic loads

John Blanchard

When determining the effect of dynamic loading on a structure the most important quantity to calculate is the natural period of vibration of the structure. Indeed, if the dynamic loading is wind this is almost the only calculation that can be made.

When a structure is disturbed from its equilibrium position it will vibrate in a most confused way. This motion can be broken down, however, into the sum of a number of oscillations of different frequency; corresponding to each frequency will be a different deflected shape known as a vibration mode. The lowest three frequencies and their corresponding modes for a simple cantilever, free-ended beam and ring [24a] are shown in Figure 5.

There are theoretically an infinite number of natural frequencies and modes, but in engineering applications usually only the lowest or fundamental frequency is of any importance. The formulae illustrate the typical fact that to increase the fundamental frequency one should increase the stiffness or decrease the weight or span.

Exact theoretical calculations of natural frequencies can only be made for very simple uniform structures. For more complicated structures approximate methods have been developed which are sufficiently accurate but extremely tedious [24b, 24c, 25a]. Computer programmes have been written based on these but are not available on our own machine.

DAMPING

In an actual structure vibrating after an initial displacement the amplitudes of the oscillations will steadily decrease due to effects such as internal friction and hysteresis losses, which are lumped together under the term damping. For practical purposes the ratio of successive amplitudes is constant for a particular structure, so damping is measured by a quantity known as the logarithmic decrement which is the natural logarithm of that ratio. Thus, if each amplitude is 110% of the succeeding one, the logarithmic decrement would be $\log_e 1.1$, i.e. 0.095. Little is known about structural damping but for reinforced concrete or welded steel construction it is only about one tenth of that for traditional bolted steel or masonry structures. Rocket

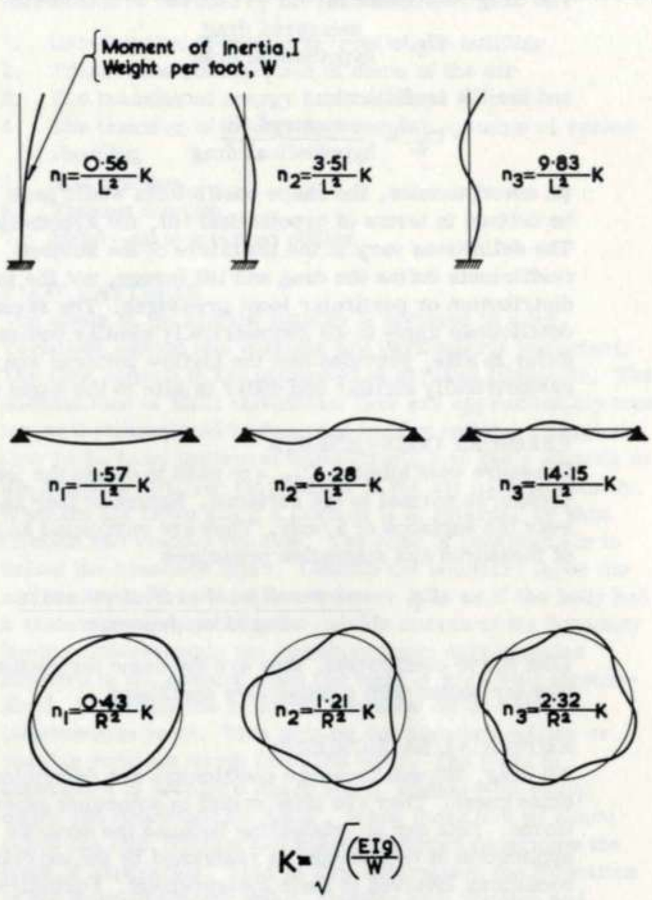


Fig. 5
Natural frequencies

experiments on actual chimney stacks (necessarily with low amplitudes) have suggested values for the logarithmic decrement in welded steel stacks of between 0.03 and 0.06 with slightly higher values for reinforced concrete stacks (26a).

DYNAMIC LOAD FACTOR

When an impulsive force is applied to a structure the movement is complex, but when the force has attained a steady value the structure vibrates in its natural modes, chiefly in the fundamental mode. The quantity of interest to the engineer is the dynamic load factor (d.l.f.), that is, the ratio of the highest stress to the stress that would occur if the maximum load were exerted as a steady static load. The d.l.f. depends on the shape of the pulse and on the duration of the pulse compared with the natural period [25b]. It is, of course, 2 for a sharp-edged front, i.e. for a load applied instantaneously and maintained at a steady value.

When a force oscillating with a regular frequency is applied, the structure vibrates with the frequency of the applied force. The amplitude of this vibration depends on how close the force frequency is to a natural frequency of the structure and also on the damping. If the frequencies actually coincide then the amplitudes can be very large and are in fact infinite if there is no damping present.

QUASI-STATIC APPROACH

In turbulent winds the stresses caused by the impulsive load of a gust striking a structure will differ from those calculated by the quasi-static approach. This approach assumes that the maximum pressure in the gust is applied as a steady static load, i.e. the d.l.f. is one. The true value of the d.l.f. is immensely difficult to determine [22]. It depends on the shape of the impulsive load, the extent of the gust, the natural frequency and the damping of the structure. It is complicated by inertial effects of the accelerating wind and by the finite time that elapses between the arrival of a gust and the application of pressure. Furthermore, the distribution of wind speed in the gust is complex, and the angle of attack of the wind varies as the gust passes.

If yield stress is reached in the structure the analysis becomes even more complex. The deformations will be greater than if the structure had remained elastic but, depending on the duration of the load pulse, they may still be small enough to be acceptable. As an example, M.R. Horne (30) made calculations for a steel portal frame (natural frequency 1 c.p.s.) loaded horizontally by a sharp-edged rectangular pulse. He estimated that the portal would withstand a load of 3, 1.5 or 1.1 times the static collapse load provided that it was not applied for more than 0.5, 1.4 or 37 seconds respectively. He assumed that the structure would remain serviceable if the deflections were not more than ten times their value when yield stress was first reached. Yield stress, ultimate stress and modulus of elasticity all increase as the rate of strain increases and allowance was made for this.

Fortunately the researches of Davenport have indicated that for structures with natural frequencies greater than .5 c.p.s. the true d.l.f. is less than one and the quasi-static approach is satisfactory (8). This would apply to all traditional and most modern structures. For more flexible structures the design pressures should be increased to allow for dynamic effects but there is little information available on what the allowance should be.

Methods not related to the quasi-static approach have been used to analyze the effect of wind turbulence but have not yet been developed sufficiently for general use. One method which involves breaking down the wind variation into sine-wave forms has been used for the design of aeroplane wings (22). More promising is the statistical treatment of the random nature of gusts so as to derive the probability of failure in a given period (22, 28); this involves a knowledge of the gust energy spectra for the site as described by John Martin.

OSCILLATIONS

A structure might be expected to oscillate in a turbulent wind if the wind speed fluctuated regularly with a frequency near the natural frequency of the structure. In practice, because of the random fluctuations of the natural wind, this does not seem to occur, at least not long enough for the oscillations to build up significantly. Such oscillations may, however, become possible in modern structures, with their low frequencies and damping.

A Danish code of practice recommends that the natural frequency of structures should not be less than 0.4 c.p.s., and this is presumably to prevent oscillations of this type.

The more important oscillations are those that occur in steady wind conditions. They can be galloping or vortex-excited oscillations.

GALLOPING

Galloping is typified by the large amplitude (15 ft.) low frequency oscillations of suspended cables that sometimes occur. It is attributed to the lift forces generated by the change of angle of attack of the wind as the cable moves across the wind and can occur with any cross-section except a smooth circular one. The wind speed at which galloping will occur is predictable for rectangular cross-sections (26b) and is too great for the phenomenon to be of importance for present day buildings. Galloping can occur with very slender angle bracing-members and has resulted in failures by fatigue of the end connections.

STALL-FLUTTER

Phenomena related to the above and with similar causes are the stall-flutter of aeroplane wings and the oscillations of suspension bridges, but these do not occur in buildings or stacks. All the above oscillations are characterized by the fact that the forces involved act only when the member is in motion and that increase of structural damping will not prevent the oscillation occurring but will merely increase the wind speed at which oscillation takes place.

EDDIES

Vortex-excited oscillations do not have these characteristics. If a solid member or structure is placed in a steady wind-stream, then a regular pulsating force acts on it at right angles to the wind. This force is associated with the shedding of vortices (or eddies) into the wake on alternate sides of the bar, and acts whether the bar is held stationary or allowed to oscillate. The frequency of the oscillating transverse force is equal to SV/D, where S is the Strouhal Number which depends on the shape of the cross-section and equals 0.15 for a square and 0.19 for a dodecagon. For a circle (26c) the value of S depends on Reynolds Number being equal to 0.2 for R less than 3×10^5 and about 0.26 for R greater than 3.5×10^6. For intermediate values of R, vortex-shedding tends to be random and no value of S can be quoted.

If the wind blows steadily with a speed whose associated vortex-shedding frequency is near the natural frequency of the structure acting as a cantilever from the foundations, then swaying oscillations will occur. For most chimneys and for some tall flexible buildings (especially those on slender piloti) this critical speed is within the range of wind speeds likely to occur, and oscillations of many actual structures have been reported. Fortunately the structural damping of the structure (except for welded steel chimneys) is usually large enough to prevent these oscillations building up to an amplitude large enough to cause damage.

AMPLITUDES OF OSCILLATIONS

The problem is then not to determine whether oscillations can occur (this can be done with fair accuracy) but to calculate the likely amplitude of the oscillations. There is not, at present, enough experimental evidence for such a calculation to represent more than an act of faith except perhaps for circular cylinders with a small Reynolds Number, for which values of the lift force coefficient have been found (26d). From these coefficients it is relatively simple to calculate the excitation factor which is defined similarly to the logarithmic decrement of structural damping but represents a tendency for amplitudes to increase. For most wind speeds this excitation factor is negative so that the structure is aerodynamically damped and will not oscillate even without structural damping. For wind speeds at or slightly above the critical speed the excitation factor is large and positive and inversely proportional to the amplitude. Assuming a value for structural damping, it is thus possible to calculate an amplitude at which the aerodynamic excitation is exactly equal to the logarithmic decrement. This then is the maximum amplitude that will occur. In fact, the structural damping will tend to increase at large amplitudes due to yield of the structure or bolt slip, a fact which has probably prevented failures in the past. Values of lift coefficients for higher Reynolds Numbers and other shapes are becoming available as more experimental work is done.

It might be thought that oscillations of a tapered stack would not occur as the changing diameter would imply different critical wind speeds up the stack. In practice, however, it is found that vortex shedding occurs along the whole height at a frequency appropriate to the diameter near the top (26e).

Vortex-excited oscillations of actual structures can be cured by introducing artificial structural damping or by attaching some form of aerodynamic spoiler to the structure. For circular chimneys a system of helical strakes of a particular shape and pitch seem the most efficient (26f, 29).

'BREATHING'

A ring bending oscillation as in Fig.1 known as ovalling or 'breathing' sometimes occurs near the top of thin-walled circular towers. The critical frequency is equal to $\frac{1}{2}$ SV/D since it is related to the shedding of individual vortices, not of pairs of opposite vortices. The effect of damping is unpredictable so ovalling is best prevented or cured by adding stiffening rings to increase the critical speed to a safe value.

WAKE BUFFETING

Obviously the regular formation of eddies mentioned above means that a structure in the wake of another structure will be subjected to regularly fluctuating pressures even in a steady wind. These fluctuating pressures may cause oscillation either along or across the wind stream. This phenomenon known as wake buffeting was first described after the Meopham air disaster in 1930, where the tail-plane failed because of oscillations caused by eddies shed off the wing. It occurs rarely with structures but has been observed in model tests of adjacent bridges and with actual groups of chimney stacks. One example was a group of four welded steel stacks placed in line 4D apart [27]. With the wind blowing along the line the transverse oscillations of the leeward stack had an amplitude three times that of the windward stack, resulting in fatigue failure of a welded joint near the bottom.

Addition of helical strakes did not affect the leeward chimneys although it completely stopped the oscillation of the windward one (this is characteristic of wake-buffeting) and the only cure lay in providing artificial damping. The critical wind speed also may be modified by adjacent structures.

SITING OF STRUCTURES

Obviously the siting of structures in close neighbourhood will not only produce oscillation effects but will affect the pressures exerted and their distribution.

This is known as the group effect and was shown up in the wind tunnel tests made of the Ferrybridge cooling towers after their collapse [12].

Under a steady wind the pressures fluctuated heavily and the pressure distribution differed completely from that on an isolated tower. The total time averaged or static force on a leeward tower (that failed) was slightly less than for an isolated tower, but the total fluctuating force was more than twice this. Surprisingly, the stresses due to this fluctuating force calculated by a quasi-static approach were not greatly above the permissible and group effect could not be said to be the cause of failure.

No theoretical machinery exists for predicting wake-buffeting or the magnitude of group effect. Wind tunnel tests on models of the actual arrangement are the only source of information. If group effects seem likely a substantial allowance should be made for the uncertainty of the stress-distribution and the effect of this in relation to the ultimate load capacity of the structure should be considered.

Wind tunnel tests; their uses and limitations

K.C.Anthony

When we are faced with the wind design of a structure which, because of its configuration or location relative to particular surrounding objects, cannot be adequately treated by building codes or theoretical analysis, then we have either to design over-safely or resort to wind tunnel tests.

It is not necessarily an easy decision to make. Nowadays, it may be several months before a tunnel is available and none of us wishes the design process to be unduly extended. Moreover, tunnel tests are not cheap and must be considered in relation to the value of the data obtained. Wisely used, tunnel tests can often save clients' money since the results can lead to a more economical structure. Again, quite minor changes in shape can sometimes bring about reductions in the overall wind loading.

USES

How can tunnel tests help us? Roughly, tests on structures fall into three main categories which determine:

1. Quasi-static loading arising from pressure distributions on buildings of sufficient mass and stiffness to be free of wind induced oscillations.
2. Dynamic effects arising from wind induced oscillations in such structures as tall shafts, bridge decks, large span canopies, etc.
3. Air flow around buildings to investigate problems of smoke dispersion, ventilation, rain penetration, snow accumulation and the effects of wind velocities on the comfort of pedestrians.

Most of the available data on pressure and drag coefficients, and oscillation effects have been derived from tunnel tests on models, and while the full scale conditions cannot be reproduced exactly, test results are usually reliable enough to apply to the final structure.

PROBLEMS AND MODELS

What about the problems inherent in tunnel testing?

To begin with, there is the question of model size. Naturally, the larger the model (which must be in direct linear scale) the more accurate the results, provided that a sufficiently large tunnel is available. If the model takes up more than 5% of the tunnel cross-sectional area, then it accelerates the airflow and unpredictable inaccuracies occur. A good working rule is to limit the model area to about 2-2½% of the test section area.

Then there is the question of the Reynolds Number effect. This is a ratio defined by the product of wind speed, air density and a typical external dimension, all divided by the viscosity of the air. Ideally the Reynolds Number should be the same for both full size structure and model. If one were testing a one-hundredth scale model, for instance, then to obtain Reynolds Number similarity, either the air velocity or the air density would need to be multiplied one hundred-fold. Tests can be run at several hundred miles per hour but there comes a limit when shock wave effects intrude.

It is also possible in closed circuit tunnels, to increase the density of the air by compressing it, even up to 25 atmospheres but above this considerable inaccuracies can occur.

Sharp edged shapes such as the rectilinear buildings we usually have to deal with under category 1 are little affected by differences in Reynolds Number. These can be tested at low air speeds and at atmospheric pressure, in comparatively inexpensive tunnels and the results applied directly to the full scale. Unfortunately these are the buildings which rarely require tunnel tests. Those that do are buildings of cylindrical or complex curvilinear form requiring accurate Reynolds Number similarity with the attendant high velocities or air pressures. No cheap tunnel can deal with these.

Models of some structures, particularly those in category 2 require similarity, not only of geometry and Reynolds Number, but also to an appropriate scale, of their dynamic characteristics such as mass and damping distribution. Such models are termed aerolastic and are used to investigate not only their drag and lift properties but also their aerodynamic instability characteristics. The testing of such models is far more complex than that already described. In fact, ideally there are five non-dimensional parameters, involving eight physical properties, which should be satisfied. In practice, however, a compromise must be made which while introducing uncertainties into the interpretation of results, does allow useful data to be obtained.

The type of problem being investigated will dictate the choice of similarity parameter. Further, there is a choice of model type. A 'full model' can be made to a linear scale and in a suitable material or it may be constructed in some different manner to obtain the required distribution of mass and stiffness. Alternatively a spring mounted 'sectional model' may be made of a typical part of the structure (such

as a lattice mast) when a linear 'full model' would be too small for accurate results. Both methods can achieve the appropriate similarity properties.
Literature is obtainable which sets out in detail all these similarity requirements.
For investigations of problems in the third category, low speed open tunnels are suitable and relatively small models may be used. In fact they have to be small in order to get all the surrounding obstructions into the working section. Smoke plumes are used to illustrate the flow which is then photographed. Velocities are measured with sensitive pilot tubes or hot-wire anemometers. The most useful information we can derive from these tests is the magnitude of velocities occurring at various points at low level. These can be as much as 30% higher than the velocities existing over the top of tall slab blocks nearby.
Low speed open tunnels are also useful for investigating the effects of surrounding hills, valleys, etc., although only a general idea of the airflow can be determined because of the small model scale. In such cases, too low a Reynolds Number must be avoided.
A particularly useful facility is to have the model on a turntable so that winds from all directions can be studied.

SIMULATION OF NATURAL CONDITIONS

A great deal of thought has been given recently to the simulation of natural wind in tunnels. An effective means of producing velocity gradients is a screen of horizontal slats at various spacings a short way upstream of the working section. Pressure distributions and airflow around buildings can be quite strongly affected by the shape of the applied wind profile.
The simulation of the natural gustiness of wind is a far more difficult problem because of scale effects. Although the turbulence in a wind tunnel can have the same general properties as that of natural wind, there is a very great difference in the size of eddies. In a wind tunnel, these are about the same size as, or smaller than, the model, while in natural conditions they are far larger than full size buildings. This is an important point to be borne in mind when considering tests on structures which are particularly susceptible to dynamic loading.
Tunnel testing can never be a complete answer to our problems until its accuracy can be gauged by full scale tests upon completed buildings. The GPO Tower is probably the only building in the country on which this correlation is to be attempted.
Perhaps Ferrybridge will initiate the setting up of further sorely needed tunnel testing facilities in this country. If not, then our engineering intuition will be somewhat stretched for many years to come.

Wind on tall buildings; design procedure

M.J.Barclay

If engineers built only what they could perfectly analyse they would not have built the Pont du Gard or the Sydney Opera House, and if they took no risks they would still be building pyramids on rock foundations and never have attempted Stonehenge. On the other hand, they would not have built those Ferrybridge cooling towers.
This series of articles has dealt with the nature of wind and many of its effects. It remains to suggest a practical method of choosing wind loads for the design of tall buildings which is both safe and economical.
The four preceding articles have developed the quasi-static design procedure which might be summarized in the following way. The random wind at the site, described by John Martin, is statistically predictable and it can be defined by simple parameters on the basis of meteorological records and probability theory. The engineer chooses return periods and gust durations in order to deduce suitable design wind speed profiles which represent the most severe wind loading conditions as short but steady gusts. The corresponding static pressures on the building itself then follow according to the laws explained by David Lowes and these pressures may have to be multiplied by a dynamic load factor to allow for any additional response due to the fluctuations of the natural wind and the possible structural vibrations discussed by John Blanchard. Finally, or concurrently, model tests may be made to check the theory in the light of Ken Anthony's advice.
It is fortunate that the wind loading on many tall buildings does not need rigorous investigation and that even CP 3 Chapter V with all its pitfalls can be a useful guide as well as a statutory check. The engineer's problem however is to recognise the buildings which do need more careful consideration.
This concluding article deals mainly with the choice of design wind profiles, the anticipation of dynamic risks, and the classification of tall buildings according to the depth of research into wind loading which they are likely to need.

WIND SPEED PROFILES

The power law model $V = V_g \left(\frac{H}{H_g}\right)^x$ for the wind speed profile up to gradient height H_g is simple and near enough to the truth. The real profile is of course a very ragged curve, continuously changing its shape, but impossible to represent mathematically. V is the maximum mean gust wind speed at height H and V_g is the maximum mean gradient velocity which depends on the exposure of the site and the chosen return period, while the index x depends on the roughness of the terrain and the chosen gust duration. If V_g and x can be estimated from meteorological records and the roughness of the terrain for, say, the maximum mean hourly wind speeds likely to occur on average once a year, then the adjusted values for any longer period and any shorter gust duration can be taken from the following tables.

Return period (years)	1	5	50	500	5000
	1.0	1.15	1.39	1.62	1.85

Terrain	Open	Medium	Built-up
Typical gradient height (ft.)	900	1100	1500
Typical values of 'x'			
1-hour mean	0.12	0.20	0.33
1-minute gust	0.09	0.14	0.22
10-second gust	0.07	0.10	0.15
2-second gust	0.04	0.06	0.08

The procedure is fully described in references [8a] and [9] but there are two points to watch. The first is that wind strength records have only been kept for a few years, geophysically speaking, and we are extrapolating boldly in estimating a 5000-year maximum. The second is that the three terrain types listed are not sufficient to describe all sites, which range from open sea to Wall Street.

GOING UP AND GOING DOWN

The Meteorological Office at Bracknell prepare on request maximum wind speed predictions for any return period and gust duration for any site in Britain, taking account of local topography. They are a synthesis from the weighted observations of neighbouring recording stations which have been made over differing periods and at varying heights around 40 feet above ground level. The power-law indices used to obtain the profiles are based on limited evidence

TABLE I - POSSIBLE CHOICE OF DESIGN WIND PROFILES FOR A BLOCK OF FLATS 400 FT. HIGH IN BIRMINGHAM

	Purpose of calculation	Limit state	Acceptable risk of one 'failure' in 50-yr. life	Design profiles: Return period (yrs)	Design profiles: Gust Duration (secs)	Load factor ø on static wind pressure only (excluding dynamic load factors)	Mean design pressure with shape factor 1.6 (lb/ft^2)
1.	To establish ultimate safety	Overturning of block	Practically nil	5,000	10	1.2	57
2.	To calculate wind reinforcement	0.85% yield stress in steel	10%	500	10	1.1	40*
3.	To limit partition cracking	Excessive sway amplitude	25%	50	10	1.4	39*
4.	To limit discomfort to residents	Excessive jerk (rate of change of acceleration)	100%	50	2 (plus impulse)	Not applicable	Not applicable
5.	To prevent upper windows breaking	Glass fracture	1%	5,000	2	1.1	67
6.	Local authority requirements	CP 114 working stresses	?	Exposure 'C'		nil	26

* These design pressures are virtually the same and the two different approaches to them are shown for comparison.

ø This load factor is applied to the wind pressure derived from the design profile in order to obtain the basic design wind pressure itself. It is necessary in order to allow for the following factors, taking account of the acceptable risk of failure and the chosen return period.

a. Uncertainty of original wind parameters, i.e. width of confidence limits in predicting long-term maximum gales
b. Possibility of extreme gale occurring
c. Consequences of failure and level of insurance against failure
d. Redistribution of pressures due to vertical air currents on face of building
e. Uncertainty of structural analysis
f. Future alterations in local wind conditions
g. Any other relevant factors for the particular building

The values shown in the table are subject to adjustment at the engineer's discretion.

from similar terrain elsewhere and they are sensitive to the judgement used in estimating the effective height of the original observations. This information is summarized in H.C. Shellard's contour maps of 50-year maximum mean hourly and maximum 2-second gust speeds at 40 ft. over the British Isles (4).

The complementary approach starts from a gradient wind contour map such as A.G. Davenport's map of the British Isles (5). This method should always be used as a check on the upper end of profiles based on low-level readings. Conversely, profiles based on gradient winds should be checked against actual observations near ground level. Overseas this basic information may not be so readily available and the engineer must establish the local values of the parameters as best he may, bearing in mind that ignorance is expensive in conservative safety factors or unsafe buildings.

The chosen return period will not be the expected life of the building. It is usually much longer. This is because a gale that is likely to be exceeded only once in 500 years has a 1 in 10 chance of happening in the first 50. It is convenient to choose a return period for each limit state separately and to use different wind load factors which take the consequences of failure into account. A 5000-year gale might be a good check on ultimate stability, for example, while a 50-year gale might be a more useful guide to the control of partition cracks. This is illustrated by an example in Table I.

The choice of gust duration is easier. The design gust 'rugby ball' is the smallest that can produce the effect considered. For overturning, the design ball should be 8 - 10 times the width of the building, for example. A 10-second ball is about 1200 ft. long in an 80 m.p.h. gale and the 2-second ball within it is only about 240 ft. long. This consideration is also illustrated in Table I.

The resulting profile should be scaled up at this stage to allow for any local acceleration of the wind stream by venturi effects, through neighbouring buildings for example, or by small-scale topography not previously allowed for in the profile.

ODIOUS COMPARISONS

The design wind profiles chosen for the isolated 400 ft. high block of flats in the example can be compared with each other and 'Exposure C' of CP 3 Chapter V on the basis that the building is, say, near the centre of Birmingham. This comparison is shown in Fig. 6. Although this is not stated explicitly, the code tabulates pressures which include a 1.6 shape factor and are related to 50-year maximum 10-second gusts. The difference between the profiles will be further exaggerated, of course, by the subsequent

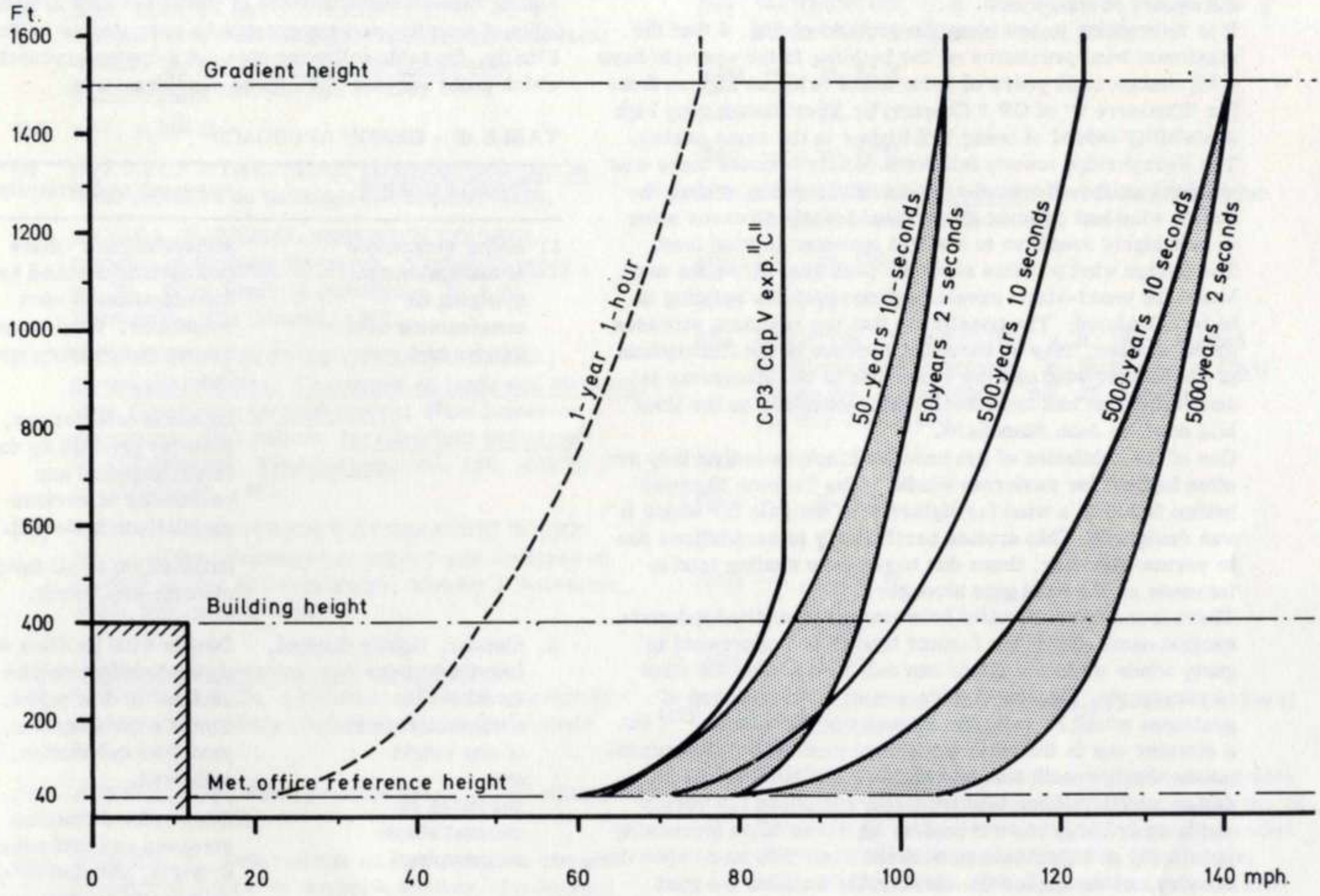


Fig. 6 Comparative maximum wind speed profiles for Birmingham, England

TABLE II

Warning parameter			'Green'	'Amber'	'Red'
Logarithmic damping			> 0.1	0.1 - 0.5	< 0.05
<u>Sway Oscillations</u>					
Natural frequency	(sec^{-1})	n	> 1.0	1.0 - 0.5	< 0.5
(Vortex excited)					
Reduced frequency	(number)	$\frac{nD}{V}$	< S	> S	S
Gust factor	(number)	$\frac{V_{gust}}{V}$	> 1.2	1.2 - 1.1	< 1.1
(Gust excited)					
Natural wavelength	(ft.)	$\frac{V}{n}$	< 50	50 - 150	> 150
Gust factor	(number)	$\frac{V_{gust}}{V}$	< 1.1	1.1 - 1.2	> 1.2
<u>Breathing Oscillations</u>					
Reduced frequency	(number)	$\frac{nD}{V}$	< S/2	> S/2	S/2
<u>Wake Buffeting</u>					
Spacing of buildings	(ft.)	p	> 20D	20D - 5D	< 5D
<u>Group Effect</u>					
Spacing of several buildings in each direction	(ft.)	p	> 10D	10D - 3 D	< 3D

D = Diameter or width of building (ft.) S = Strouhal number V = Mean wind speed (ft/sec) conversion to design pressures which are proportional to the square of wind speed.

It is interesting to see from the profiles of Fig. 6 that the maximum wind pressures on the building in the example have a 1% chance in 50 years of being about twice as high as those for 'Exposure C' of CP 3 Chapter V. They have a very high probability indeed of being 10% higher in the same period. The Ferrybridge towers fell down chiefly because there was no check on the effects of a wind slightly stronger than the design wind and because the critical tensile stresses were in fact highly sensitive to a slight increase in wind load.

The design wind profiles show the peak gust strengths and hence the quasi-static pressures for which the building is to be calculated. The possibility that the resulting stresses or deflections, may be exceeded because of the fluctuations of the natural wind and the vibrations of the structures is something that has to be dealt with separately on the lines laid down by John Blanchard.

One of the subtleties of dynamic load factors is that they are often highest for moderate winds. (The Tacoma Narrows bridge failed in a wind far lighter than the gale for which it was designed). This applies particularly to oscillations due to vortex-shedding, those due to periodic gusting tend to increase as the wind gets stronger.

There is another distinction between vortex-excited and gust-excited oscillations: the former tend to be suppressed in gusty winds while the latter can only arise when the wind is very gusty. Davenport uses a statistical measure of gustiness which he calls the intensity of turbulence [21] but a simpler one is the ratio (maximum gust speed)/(maximum mean-hourly speed) for the height of the building and the design profile. When this ratio exceeds about 1.2 vortex shedding is likely to be irregular but there is an increasing possibility of significant gust-excitation. The more open the country and the higher the building the smaller the gust factor.

GUST SPECTRUM

At this point we return to the gust spectrum (Fig.4) which is based on the important and fairly good assumption that natural turbulence has a pattern in space which is independent of the wind speed for any particular terrain. This means that the gust energy is concentrated in waves, typically between 100 and 10,000 ft. long, whose peaks reach the building at a rate proportional to the mean wind speed. Therefore, in strong gusty winds there is likely to be substantial energy in gusts which are resonant with the natural vibration of a low-frequency building.

Adding these factors to those of David Lowes's article, a table of warning parameters can be compiled as in Table II. Finally, the table following sets out a design approach which takes account of all these considerations.

TABLE III - DESIGN APPROACH

Type of building	Research and Analysis
1. Major structures of unusual shape, grouping or construction over 200 ft. high	Meteorological Office predictions checked by site investigation of wind conditions. Wind tunnel testing in boundary layer flow. Dynamic calculations, possibly checked by damping investigation if any possibility of serious oscillations indicated. Satisfaction of all design criteria separately.
2. Slender, lightly damped, low-frequency or otherwise suspect structures of any height and Buildings of unusual shape or grouping	Design wind profiles with dynamic effect checks as outlined in this paper. Further investigation, by model or calculation, if indicated. Conventional working stresses and deflection criteria, checked by ultimate load factor on gale having 1% risk of occurring in life time. Further limit state checks if necessary.
3. Conventional or familiar structures over 200 ft. high	As 2. above but using established dynamic load factors and other short cuts based on experience.
4. Other tall buildings up to 200 ft. high	CP 3 Chapter V and relevant byelaws, with ultimate load factor check.

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