PHILOSOPHICAL TRANSACTIONS OF THE ROYAL SOCIETY A
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Bridges and other Large Structures

O. A. Kerensky

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Bridges and other large structures

By O. A. Kerensky†
Freeman, Fox & Partners, Consultant Engineers, Abford House, Wilton Road, London, S.W. 1

[Plates 8 and 9]

When invited to give a paper on effects of wind on structures, I did suggest that there are many scientists and experts who knew much more about this subject than I, but was firmly told that an engineer's approach to the problem is required, and I will therefore try to describe the picture as I see it.

Not so many years ago the structural designer's life was relatively simple. All he had to know about wind was that it exerts a static pressure of x lbf/ft^2 ($\approx 50\,\mathrm{N\,m^{-2}}$) on an exposed area which was often taken as merely a projected area of the structures. This happy state of affairs lasted for several decades, maximum design pressures varying from about 3500 to 720 kN m^{-2} (75 to 15 lbf/ft^2) or even less in fully loaded conditions. Buildings were treated in a similar way, assuming constant pressure for the whole height, but, of course, in those days the heights were limited. Historically there were many partial or complete failures of structures caused by wind effects, sometimes by sheer force, like in the case of the Tay Bridge which was designed for a wind pressure of about 500 N m^{-2} (10 lbf/ft^2) and sometimes through aerodynamic instability which destroyed the original Brighton Pier and several early suspension bridges, and only 29 years ago the newly built Tacoma Narrows Suspension Bridge which had shaken itself to destruction in a relatively light wind (figure 1, plate 8).

Although the science of aerodynamics existed long before that date it can be crudely said that its serious application to static structures started with the failure of that bridge.

From then onwards much thought and research has been applied to the problem, but even today, wind effects cannot be evaluated purely theoretically, require a considerable amount of factual local data and often must be determined experimentally by tunnel tests. The recent failure of the Ferrybridge Cooling Towers and of several high transmission towers are a clear indication of the general lack of knowledge, while the bitter controversy that now rages in the profession over the revisions of the U.K. regulations for wind effects on buildings (CP. 3, chap. 5), shows that the research workers have failed to convince the designers of the validity of their conclusions. It is argued by the latter that buildings designed to resist appreciably smaller wind forces than the ones now recommended did not fail in practice. However, as no one knows how near to failure some of these buildings might have been, the argument is not convincing. It should also be appreciated that in the past the simple conventional methods of design seldom allowed for the beneficial effects of filler walls, cladding and for the ductility of the structural frame itself.

A major problem is to 'tailor-in' modern work on wind loading to the long empirical experience of engineers, and in particular to relate predictions made by modern methods to design stresses in order to achieve satisfactory safety. This is arising largely because existing loading specifications do not give the probability of occurrence of load they are seeking to define, so that there is at present probably only poor balance between the probability of failure arising from differing causes, and it is difficult to relate the values of load factors that have conventionally been used to

† Elected F.R.S. March 1970.

the new requirements. The actual risk of failure that is acceptable to the community is in any case ill defined: it is probably rather higher for wind loading (an act of God) than for dead weight or functional loadings. Where wind return periods are mentioned, they are usually equal to the design life, thus giving a load roughly equal to the mean of the probable maxima.

The higher public tolerance of collapse due to wind action is reflected in the '25 % overstress' clause in our Specifications, particularly noting the relatively high probability that the wind load to which they are applied will be exceeded. Of course, an overstress, or reduced load factor, *is* justified when critical condition is a combination of wind with another highly variable load and therefore coexistent maxima are unlikely, but the 25 % and similar provisions have not been rigorously restricted to such cases.

Now that the old-fashioned 'simple steady pressure' wind design is universally recognized as inadequate, the research workers and the designers are involved in the study of wind.

Gusts and dynamic responses of structures situated in different environments

Gust effects can be treated as:

Static when it is mainly a question of correlation of gusts over the structure, particularly if the influence line is not of a simple shape.

Dynamic response—additional strains caused by dynamic magnification of effects of gusts have a duration of the order of half the natural period of the structure, and possibly a reduction of strain due to inertia of structure precluding full response to gusts shorter than half the natural period.

Effects of turbulence on force coefficient—clearly the fluctuating force may depend on the rate of change of wind speed as well as on the instantaneous value of the speed. There is also a possibility that the whole flow pattern may be changed by the turbulent mixing of the flow, altering the mean drag coefficient.

The static gust problem (perhaps better quasi static) at its simplest is to estimate the maximum value of the total force on the structure and thus arrive at a design average pressure as against the basic design pressure at a point.

There is no doubt that although static concept is convenient it is often incorrect, since it takes no account of the actual structure of the wind or of the dynamic response of the structure to the wind. In the design of tall buildings, long span bridges and high transmission towers, the problem is essentially one of dynamics.

Dynamics of gusts

According to Davenport, the dynamic response of structures to gusts depends on:

(1) size of the resonant gust component which is proportional to

$$\frac{\text{hourly mean wind speed}}{\text{resonant frequency and structure size}}, \quad \text{i.e. } V/nL,$$

(2) strength of the resonant gust component as measured at a point, which is dependent on V/n only;

(3) structural damping.

Damping is not clearly so critical here as in some types of instability problems (vortex shedding particularly). A 50 % error in estimation of damping probably only makes 5 to 10 % error in design load.

The dynamic response in the basic relations above is very dependent on the frequency. Modern trends in design increase working stresses but the modulus of elasticity remains the same and therefore the structure becomes more and more flexible, which is almost certain to result in lower frequency. Also, more 'fully participating' continuous welded structures develop lower damping —hence increasing concern for the behaviour of modern structures.

Except for relatively narrow long span road bridges and single track railway bridges, bridges generally are stiff laterally so that the dynamic effects are small. However, the wide but slender bridge should be watched for the risk of vertical oscillations caused by vertical components of gusts and possible instability.

The very tall slender buildings probably are now reaching a point where oscillation may affect the comfort of occupants. These will, of course, be a resonant case as far as gust response is involved (the slower, almost 'quasi-steady' response to larger gusts, will not cause significant accelerations): but vortex shedding or even more particularly vortex buffeting between the structures, i.e. the risk of oscillation arising due to proximity of other structures as, for example, the two very tall towers of the World Trade Center, New York, high density housing in Hong Kong and perhaps, Ferrybridge Towers, can be significant.

Effect of turbulence on force coefficients

There is some evidence that turbulence reduces the mean force, particularly for 'plates' or slab-type buildings.

Force coefficients in general

Force coefficients in general are now quite well established internationally, although almost all are based on old smooth flow wind tunnel tests and there may yet be some surprises. Curiously there is still a fair variation on the commonest problem; the simple prism building.

Information is also inadequate for box girders with inclined sides—some research work is in progress.

Modern specifications

During the last ten years or so, all major industrial countries have undertaken drastic revisions of their design specifications. The results of this re-thinking are somewhat surprising because the proposed rules differ so widely between themselves, although all accept the basic facts, i.e. that wind pressure is proportional to speed, that speed depends on locality, terrain and height above the ground and that the effects of wind are dynamic as well as static and can be materially influenced by the shape of the structure and its mass inertia.

Specifications from about two dozen countries have been compared, and are commented on below:

Probability of occurrence

Very few specifications give the probability of gust occurrence of maximum wind speed, but those that do, cover a wide range of return periods from one year in France (for normal loading) through 50 years in Denmark, up to maximum speeds ever to be experienced in the Indian and Japanese Codes.

Only four countries give a probability formula similar to the U.K. draft Code for buildings.

Wind speeds

In most specifications the basic value is given at a height of 10 m above ground level in regions of typical ground roughness and topography, which generally corresponds to standard meteorological station observations. However, the gust averaging time and return periods vary considerably as shown in table 1. The basic design pressures vary from 500 to 2000 N m^{-2} (10 to 40 lbf/ft^2) combined with a considerable range of factors increasing and decreasing these pressures, the most common factor being for increases of the basic pressures in the coastal strips up to 10 km wide.

Table 1

country	averaging time	return period (years)
Australia	3 s	50
Czechoslovakia	2 min	5
Denmark	20 s	50
India	3 to 5 min	maximum ever
U.K. (draft CP. 3)	3 to 15 s	50
U.K. (revised BS. 153)	1 h	120

Speed variation with height

Here again there is a great variety of methods, for example, the increase in the design pressures at 150 m above ground as against those at 10 m, varies from ratio of 1.6 to 2.8.

Topography and shielding factors

Most countries give reductions for built-up and other sheltered areas. Only France, Germany and the U.K. appear to give increases for narrow valleys and the like. The German specification mentions various categories of low and hilly country together with degrees of angles of approach to the windward of the structure. Generally each increase in grading and slope increases the pressure by some 10 %.

Force and pressure coefficients

The variations here are much less than in other sections of the specifications.

The general method is that used in the draft U.K. CP. 3. As an example, for a 150 m high building on a 50 m × 10 m base, the force coefficient varies from 1.2 in Denmark through 1.45 in U.K., to 1.6 in Romania. However there is a much greater variation for roof suction.

For unclad structures the requirements for design of trusses are very similar, the Belgian specification being the most comprehensive.

For lattice towers, there is a wide variation. For example, on the square tower with 0.15 solidity, the coefficient varies from 2.4 in Canada to 3.55 in the U.K.

Effects of gusts

Several codes include allowances for reductions of pressure with increase of loaded length, by an *area reduction factor*. The Belgian specification, for example, permits a reduction of load by 30 % when the maximum dimensions of the structure exceed 100 m. The draft Australian specification permits a reduction of 33 % when loaded area exceeds 700 m^2. The U.K. draft CP. 3 makes allowances by using gust durations of 3, 5 and 15 s for different sizes of structures.

The theory is based on measurements of the correlation of wind speed between two points: i.e. the ratio of the time average of the product (speed at one point times the speed at the other point) to the mean square speed. The correlation for different elements of the structure can be integrated, provided one makes the 'line like structure' assumption: this is that the force on an element is related simply to the speed at that point (or rather, the speed at that point if the structure were not present). This is reasonably true for a very slender structure, or for a lattice structure composed of slender members, but is not a very good assumption for non-slender buildings and such-like where the pressure at a point will be dependent on the speeds over some significant surrounding region. This assumption is nevertheless the best available at present.

The same method is applicable to calculating an influence within the structure, rather than the total force. For example, the bending moment at mid-span of a beam, because it is more sensitive to gusts striking near mid-span, will reduce slightly less than the total force on the span.

Some of the codes include dynamic allowances for gusts (France, Russia, Romania). But more commonly these are specified only for special structures such as guyed masts or tall towers (Germany, Czechoslovakia). Thus for a 200 m tall mast with a low frequency (say 0.25 Hz) the load would be increased by some 60 % in most specifications that deal with the subject.

Recently much more sophisticated codified presentations have been made, in a form particularly suitable for design of tall buildings. The method presented by Davenport (*Proc. ASCE* St. 6 1967) has been adopted in the Danish Code, and probably will be in the next revision of the Canadian Code. A somewhat similar procedure, derived from a less conservative basis than the 'line like' assumption has been proposed by Velloni & Cohen (*Proc. ASCE* 1968) and is likely to be the basis of the next U.S.A. Code: these recommendations give significantly lower loads than Davenport's.

Notwithstanding genuine efforts at international coordination of knowledge, the design forces on buildings can vary by more than 100 %, thus the total wind force on a building 50 m × 25 m × 150 m high, with a drag coefficient of 1.3, subjected to design wind of 23 m s^{-1} at 10 m above ground, would be 1.1×10^6 kgf (11 MN) in Belgium, 1.3×10^6 kgf (13 MN) in the U.K., and 2.1×10^6 kgf (21 MN) in Romania. Although the figures do not indicate the relative margins of safety because of the different load factors and probabilities of occurrence used by each country, the great disparity in the loadings does not inspire engineers with confidence!

United Kingdom Bridge Code

The revised U.K. specification for bridges, now in the course of preparation, will be based on the concept of limit state design, and the proposed loading clauses have been examined with a view to specifying more precisely the loads likely to be applied during the life of the structure.

A return period of 120 years has been adopted and Mr N. C. Helliwell of the Meteorological Office has produced a map of the U.K. showing isopleths of the mean hourly wind speed at 10 m above ground. The hourly mean speed was selected instead of the more usual 3 s gusts to ensure more accurate estimates of mean speed. Also with loaded lengths of 300 m and above, the design wind speed is nearer the hourly mean than the 3 s gust, and as, in general, only long span bridges are significantly affected by wind, this is a further reason for selecting the more accurate hourly mean as the datum speed.

To the basic datum speed 'V_{10}' two factors are then applied—a funnelling factor of 1.1 where the bridge is in a steep-sided valley or where leewave effects occur, and a combination of height and gust factors. The height factor is obtained by calculating the hourly mean speed V_h at the desired heights using the relationship

$$V_\mathrm{h}/V_{10} = (\tfrac{1}{10}H)^{0.7},$$

where h is in metres, the factor varies from 1.0 at 10 m height to 1.66 at 200 m height.

The gust factor depends on the length of the structure and its height above the ground, the actual increase in gust speed being determined for speeds at 10 m height above ground level and

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remains constant at all heights. The proposed factors are illustrated in table 2. Tests are about to start on the Severn Bridge to determine the relationship of gust speed to length more precisely.

Table 2. Gust factors

height	loaded length				hourly speed
m	20 m	100 m	500 m	1000 m	factor
10	1.57	1.45	1.29	1.21	1.00
20	1.70	1.58	1.41	1.33	1.13
50	1.88	1.77	1.60	1.52	1.31
100	2.05	1.93	1.77	1.69	1.48
200	2.24	2.12	1.95	1.87	1.66

Comprehensive tests are also being carried out at the National Physical Laboratory to determine drag coefficients on some twenty different cross-sections of bridge superstructure, each with various widths of cantilevered deck.

Wind and temperature

Recent research carried out by the Road Research Laboratory and Meteorological Office suggests that maximum winds do not co-exist with extreme temperatures, but what wind to take with what temperature range is still uncertain.

Long span bridges–dynamic wind effects

The adoption of more refined analysis combined with a reduction in the design traffic loadings and an increase in the strength of materials, enables the designers to produce much lighter structures for long span bridges, while continuity over supports and fabrication by welding reduces structural damping. The combined effect of reduced weight and reduced damping has made long span bridges more susceptible to dynamic wind effects. These are of two types, 'single degree of freedom' and 'classical flutter'. The former is caused by vortex shedding and takes the form of vertical or twisting motions occurring separately or simultaneously. Flutter is caused by the variation of the lift force with the angle of incidence of the wind and gives rise to a coupled vertical and twisting motion. The input of energy is at its maximum when the two components are 90° out of phase.

Single degree of freedom motion can be controlled by:

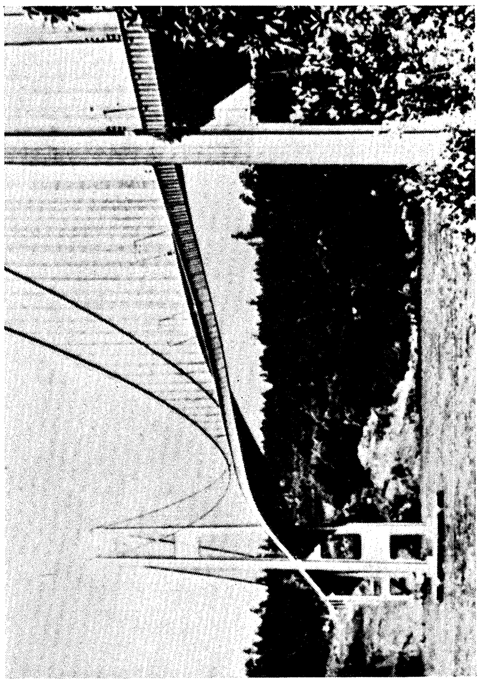
(1) Reducing the size of the vortices and the parts of the structure on which the vortices impinge.

(2) Increasing the stiffness and hence the natural frequency of the structure to such a value that the lowest wind speed at which resonance will occur is higher than the maximum predicted speed.

(3) Increasing natural damping.

In the case of very long span suspension bridges (2) is not economically practical by itself. In the case of the Forth road bridge (figure 2, plate 9) the lattice type stiffening girder and the gaps in the deck reduced the effect of the vortices, while in the Severn bridge the required stability was achieved more economically by streamlining the deck structure into an aerofoil shaped box (figure 3, plate 8), and by using a high hysteresis inclined hanger system which provides additional damping (figure 4*a* and *b*, plates 8 and 9).

'Classical flutter' cannot be controlled by damping or by reducing the vortices effects. It has to be tackled by raising the critical 'flutter' speed well above the maximum predicted actual



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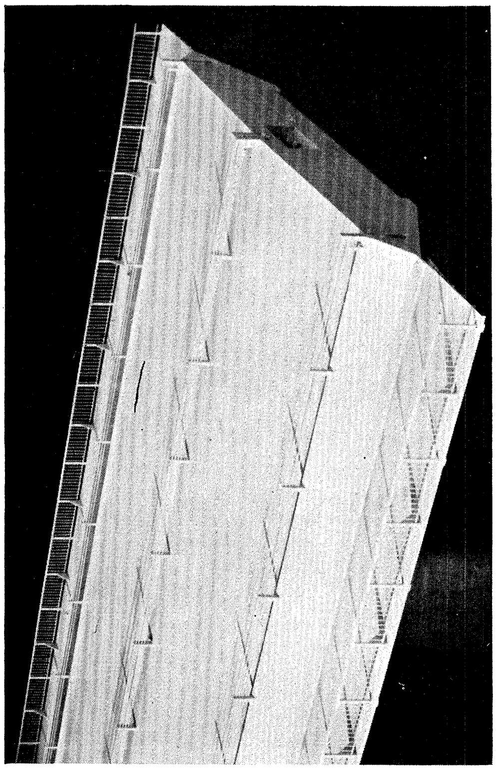

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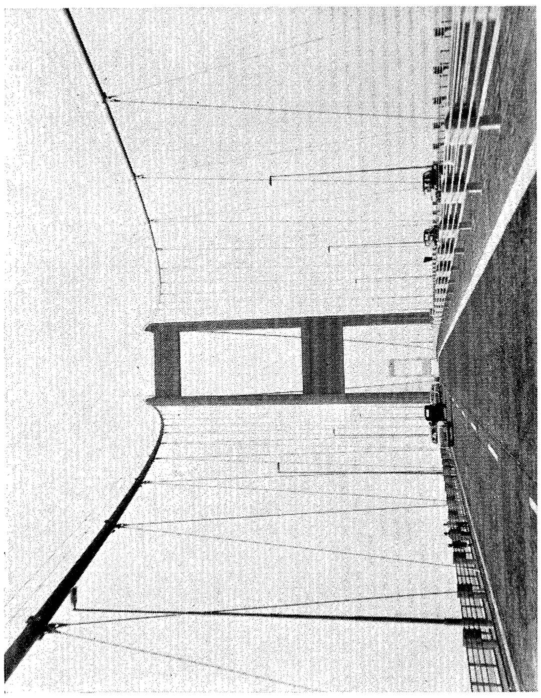

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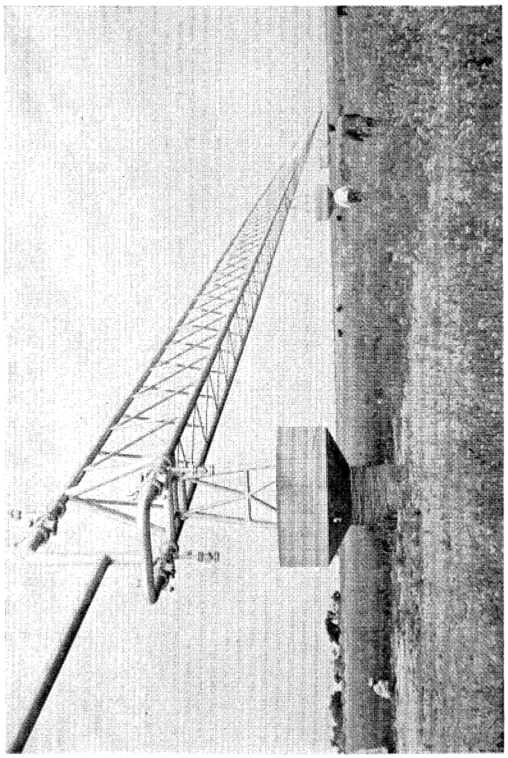

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FIGURE 1. Tacoma Narrows bridge.
FIGURE 3. Model of the Severn Bridge box girder.
FIGURE 4*a*. View of the Severn Bridge to show inclined hangers.
FIGURE 5. Sutlej Bridge.

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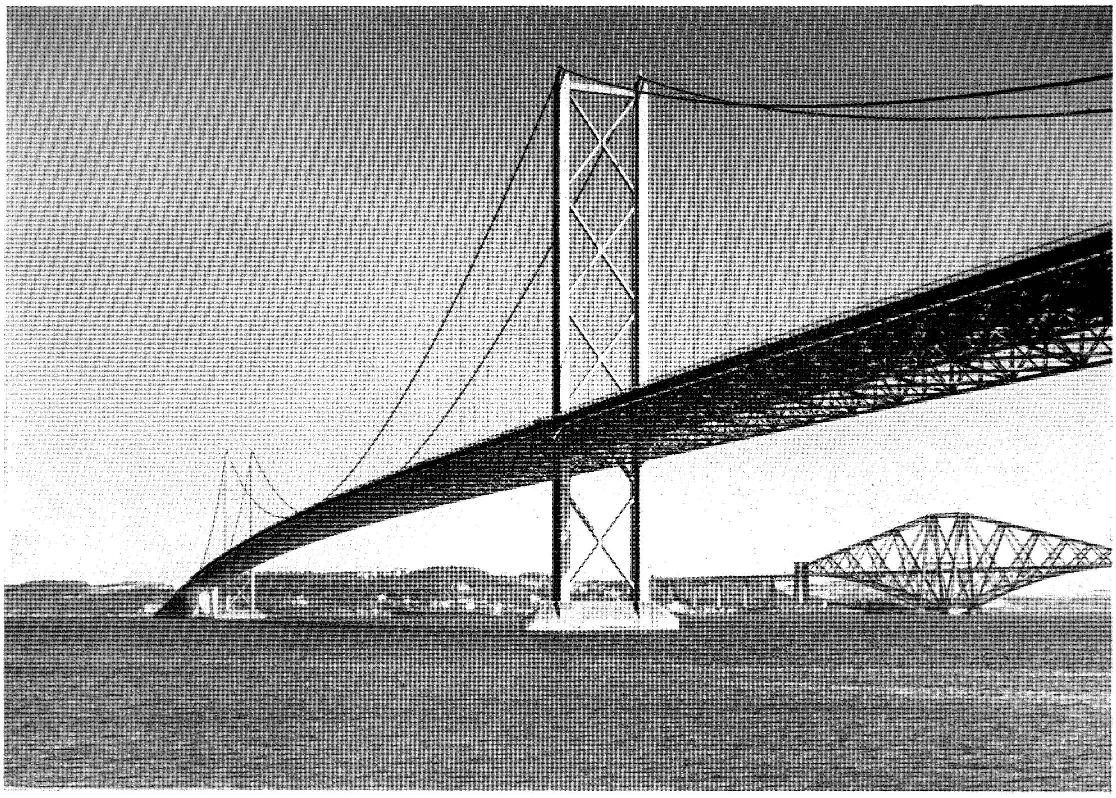

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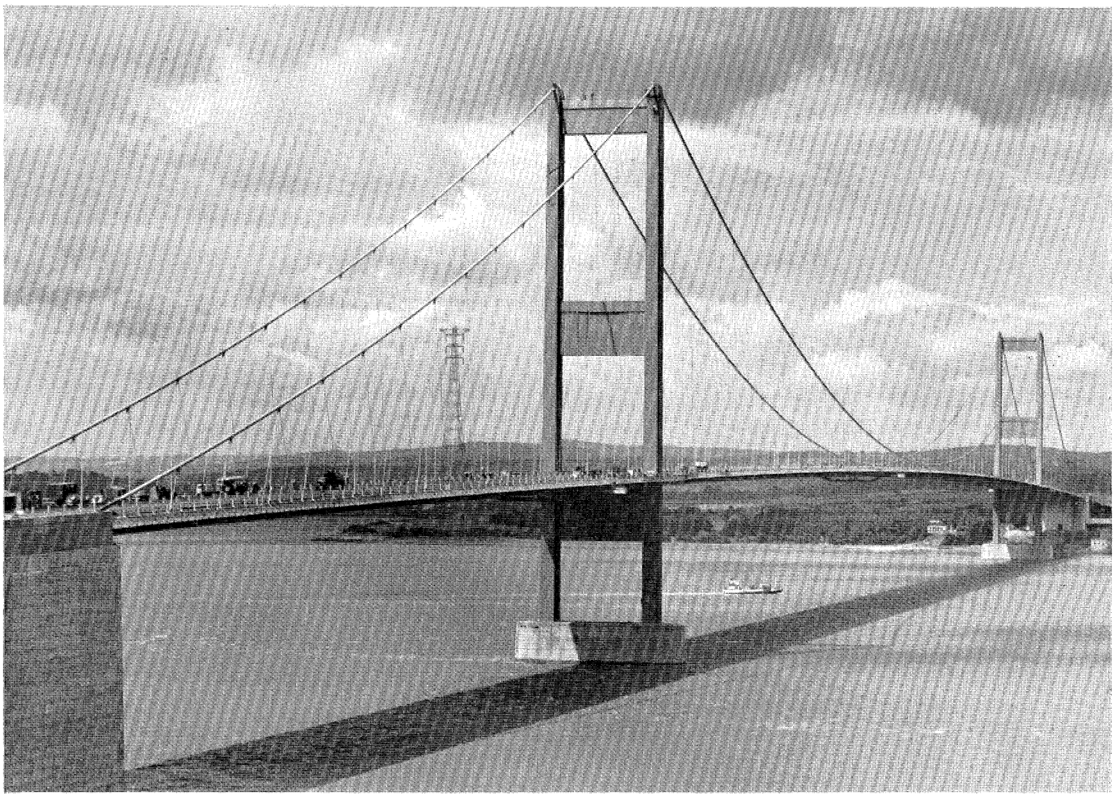

FIGURE 2. Forth Road Bridge.
FIGURE 4*b*. Severn Bridge.

wind speed. The 'flutter' speed is determined by a combination of the following factors:

(*a*) Natural frequencies of the structure in bending and torsional oscillations.
(*b*) Natural mode shapes in bending and torsion.
(*c*) Weight and rotational mass inertia of the structure.
(*d*) Width of the deck platform.
(*e*) The aerodynamic behaviour of the deck platform (approaching 'flat plate').

In order to achieve as high a flutter speed as possible it is desirable to have high natural frequencies and to get the torsional frequencies well above the bending frequencies. In practice it is usually sufficient to ensure that the deck structure is a torsionally stiff box, either latticed or plated. For very long spans, however, it may be too expensive to raise the torsional frequencies sufficiently and in that case it may be necessary to control flutter by some other means, which are yet to be developed.

Apart from the aerodynamic stability of the structures as a whole, consideration has to be given to the stability of individual elements. The hanger ropes of suspension bridges are particularly vulnerable. In many cases the problem can be overcome by connecting ropes to adjacent ones in the same group at sufficiently close spacing but in the case of the Severn Bridge where the inclined hangers occur singly the oscillation was overcome by means of attaching Stockbridge dampers similar to those used on overhead power transmission lines. Hangers of structural shapes are also subject to oscillations and in the case of Storstrom Arch Bridge some vibrated violently until replaced by sections of different inertia.

Transmission towers

In the design of transmission towers the problem is essentially one of dynamics and not statics. It is necessary to know the duration of the gusts required for critical response of the structure and the gust size. This knowledge is almost lacking.

A large tower is at present equipped with strain gauges and wind recording instruments to correlate actual structural response with wind loadings. In the meantime, engineers have to use approximations.

Another big unknown is the extent of co-existence of wind and ice. Very crudely, maximum ice and maximum wind do not co-exist, but how much can actually co-exist is unknown and hardly any serious work has been done in this field.

C.E.G.B. design their conductors for half ice and about 80 km/h. Both the conductors and structures have been known to collapse, but the reasons are usually obscure.

Cylindrical structures

Examples of cylindrical structures are cantilever steel chimneys, guyed television masts, pipe line suspension bridges and tubular truss bridges. Critical wind velocities are usually low.

If aerodynamic instability is expected, spoiling devices or dampers can be incorporated in the design. For example, helical strakes are a well-known device for stabilizing cylinders. However, a smooth cylinder with large aspect ratio at high Reynolds number has a drag coefficient of 0.6, but when fitted with strakes the drag coefficient becomes 1.2 or so, so that the high-speed wind effects are about doubled.

Other devices available are 'sharks fins' and perforated cylindrical sheaths. Suspension pipe bridges up to 180 m (600 ft) span have shown no tendency to oscillate. A 1830 m (6000 ft) long triangular section pipe bridge of all welded h.t. tubular construction was strongly excitable in the

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tunnel tests and had very poor damping characteristics (figure 5, plate 8). Triangular 'splitters' in the plane of the bottom laterals were provided with satisfactory results with little or no effect on the drag.

Spiral strakes as a cure for existing oscillating masts are not practicable because of the increase in drag. Aerodynamic instability should be dealt with in the design stage as remedial measures are difficult and expensive.

Wind tunnel as a tool in structural design

The aeronautical engineers have used and developed the wind tunnel for over 50 years and can now actually predict pressures and forces on an aircraft. However, the conditions required by the aeronautical engineer materially differ from those required in the structural field and much work is yet to be done in that field.

The aeronautical wind tunnel requires constant velocity and very low turbulence over the whole of its area, while the building wind tunnel should represent the atmospheric boundary layer, i.e. requires an air stream with mean velocity varying with distance from the surface and having very high turbulence, catering for winds in all directions. Furthermore, unless the body is sharp-edged the models should be tested at correct Reynolds numbers to obtain correct conditions at separation. To make matters worse, the conditions which the wind-tunnel worker has to imitate are not as yet completely defined by the meteorologist.

It would appear that the structural wind tunnel must yet be fully proven as the experimental tool for structural design and more full-scale measurements with which to compare the tunnel results appear to be very necessary.

Several experiments are in hand (Royex House, Post Office Towers, Aylesbury), but the work is slow and laborious. Yet, until the tunnel tests are fully verified against the actual buildings the experimental results will remain suspect.

The necessity to resort to wind tunnel testing is a serious handicap in design because of the time and costs involved, hence numerous attempts in all industrialized countries to codify the data.

The wind tunnel worker and the designer must get together on the type and form of the results required. The tunnel worker must understand the flow patterns and present them to the designer in a form in which the designer can use them. The designer has to learn suitable techniques to enable him to use the available information. Only in such partnerships can satisfactory results be achieved.

Effects on environment

The structural engineer is normally concerned with effect of environment on wind in so far as this affects the action of wind on the structure in question. Full scale and model tests are the normal method of ascertaining these effects. Relief from shielding by other structures should only be allowed if permanancy of the set-up is assured. Changes in land use can drastically alter the wind effects on the neighbourhood and on the structure itself.

Lately some concern has been experienced about the effects of structures on wind which could be detrimental to the existing environment. But practically no work on acceptable limits has been published. The tunnelling effects of narrow streets are well known. Drivers of cars on modern motorways steel themselves for the sudden changes of pressure on the vehicle when passing under massive overbridges (M1). On the other hand the tunnelling effect of viaducts is very small and normally can be ignored. However, the wind passing over a bridge or viaduct can be very disturbing to the traffic. Traffic on both Forth and Severn bridges has to be often warned and on rare occasions some high empty vehicles have been diverted. So far practically no work has been done on protection of traffic from wind on long span bridges. Some kind of deflectors probably could be devised, but the cost of fully screening such bridges against the wind would be almost prohibitive. This is probably one of the strongest arguments against the bridge across the Channel.

Future research

A list of items on which more information or guidance is required may, perhaps, be helpful:

(1) The physical nature of wind.

(2) The statistical prediction of wind speeds in a given locality.

(3) The minimum duration of gusts to produce maximum response of structures of different type and weight.

(4) The confirmation of tunnel tests by full-scale tests.

(5) Effects of wind on environment and of environment on wind.

(6) Vortex shedding.

(7) Local wind effects in urban environments.

(8) The effect of roughness of building surfaces.

(9) Wind loadings on projections such as balconies.

(10) Pressure distribution on unusual shapes.

(11) Dynamic behaviour of canopies (grandstands, etc.).

(12) Wind effects on porous structures (open multi-storey car parks).

Economics

It is difficult, if not impossible, to assess the economic return from better knowledge and application of wind effects. Cost of a major disaster, be it the collapse of a bridge or of cooling towers or of a tall building, cannot be foreseen and can only be viewed on a statistical basis. Political repercussions from loss of life are unpredictable. Breakdown in electrical network, not costly by itself, may involve millions of pounds in consequential damages. Avoidance of failure, therefore, must be our constant endeavour.

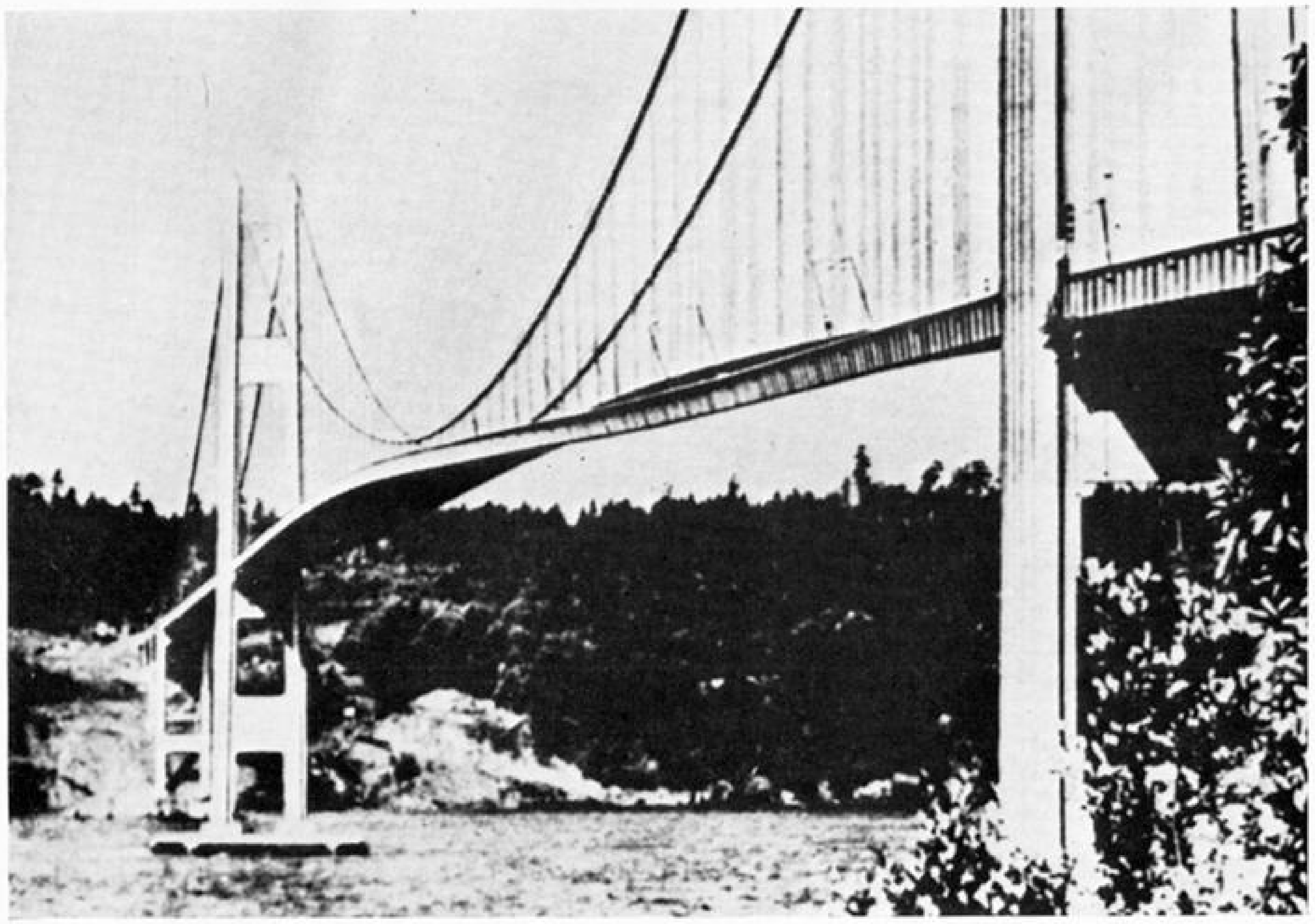
In cases where wind has a major effect on the design of a structure, e.g. towers, chimneys, long-span bridges, very tall buildings and structures of all kinds subjected to hurricanes, proper understanding of wind effects is essential, and must lead to safer and more economic structures.

The advance from the Forth to the Severn Bridge is a good illustration of this. Something like one-third of the cost was saved through a better control of the wind effects. On the other hand, it has been suggested that application of the new CP. 3 values to old and new structures in accordance with the present 'simple' methods of design would cost the nation many millions of pounds. More sophisticated understanding of the behaviour of complete buildings and of structural details must be developed if waste of materials is to be avoided.

Acceptance of the probability of failure is not for the structural engineer—the general public and responsible authorities would not tolerate it as a deliberate proposition—fools paradise is preferred, but if aeroplanes were designed like structures they would never fly.

The author gratefully acknowledges the help of his colleagues and friends, Dr W. C. Brown O.B.E., K. A. Goodearl, M. F. Parsons, B. P. Wex and Dr T. A. Wyatt.

Some of the data was drawn from the work in the Institution of Structural Engineers' Committee on Environmental Wind of which the Author is a member.



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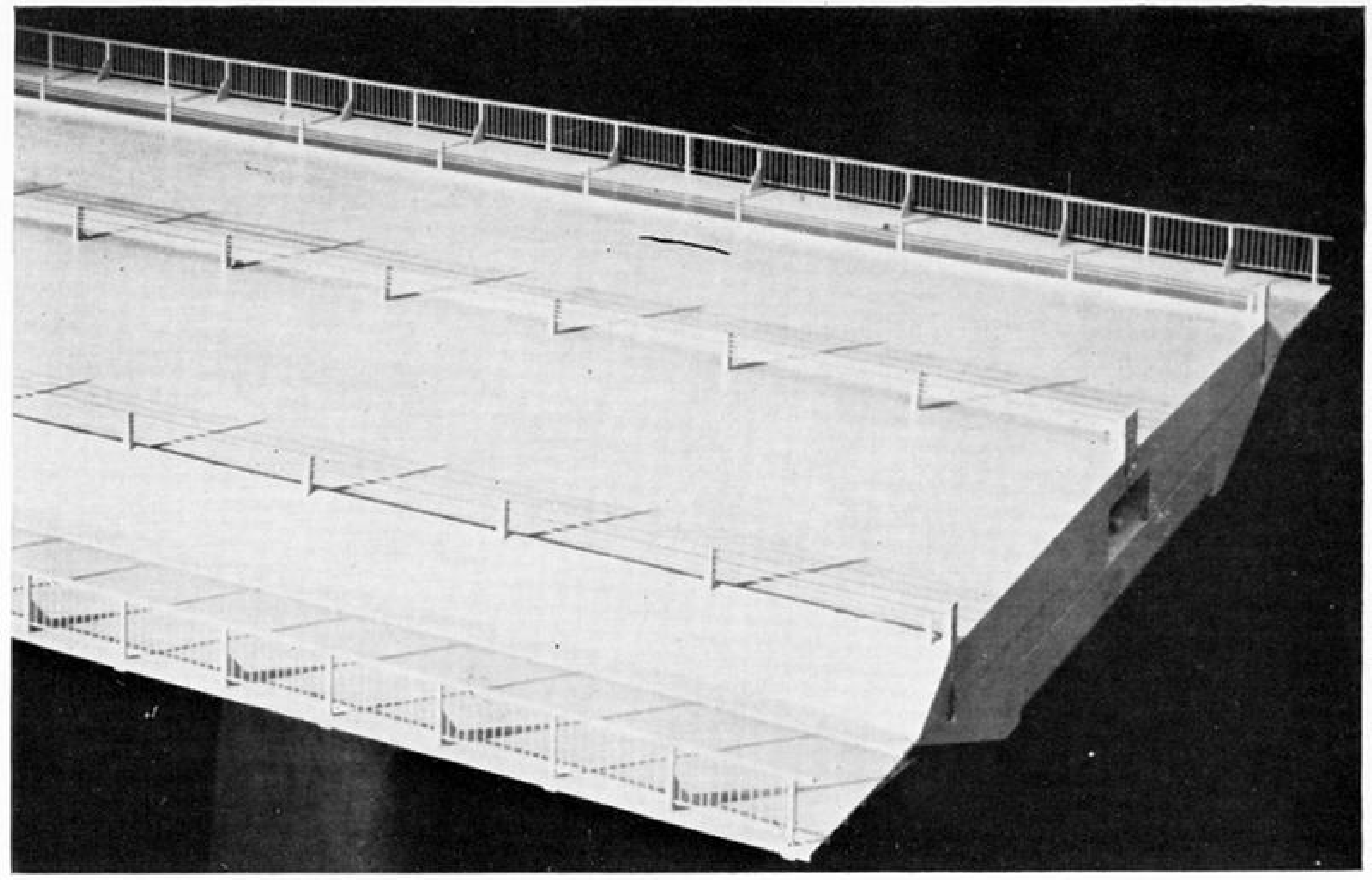

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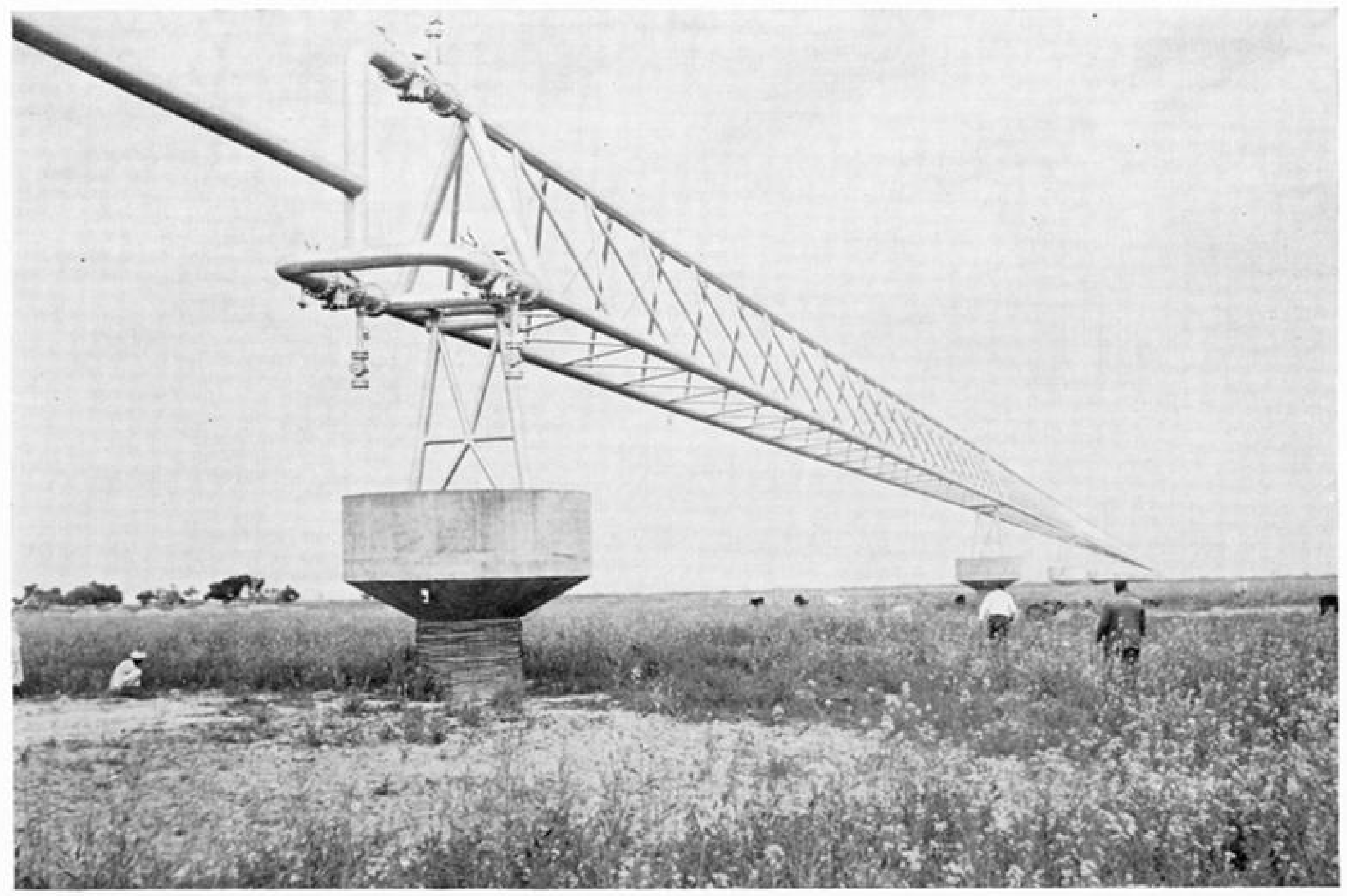

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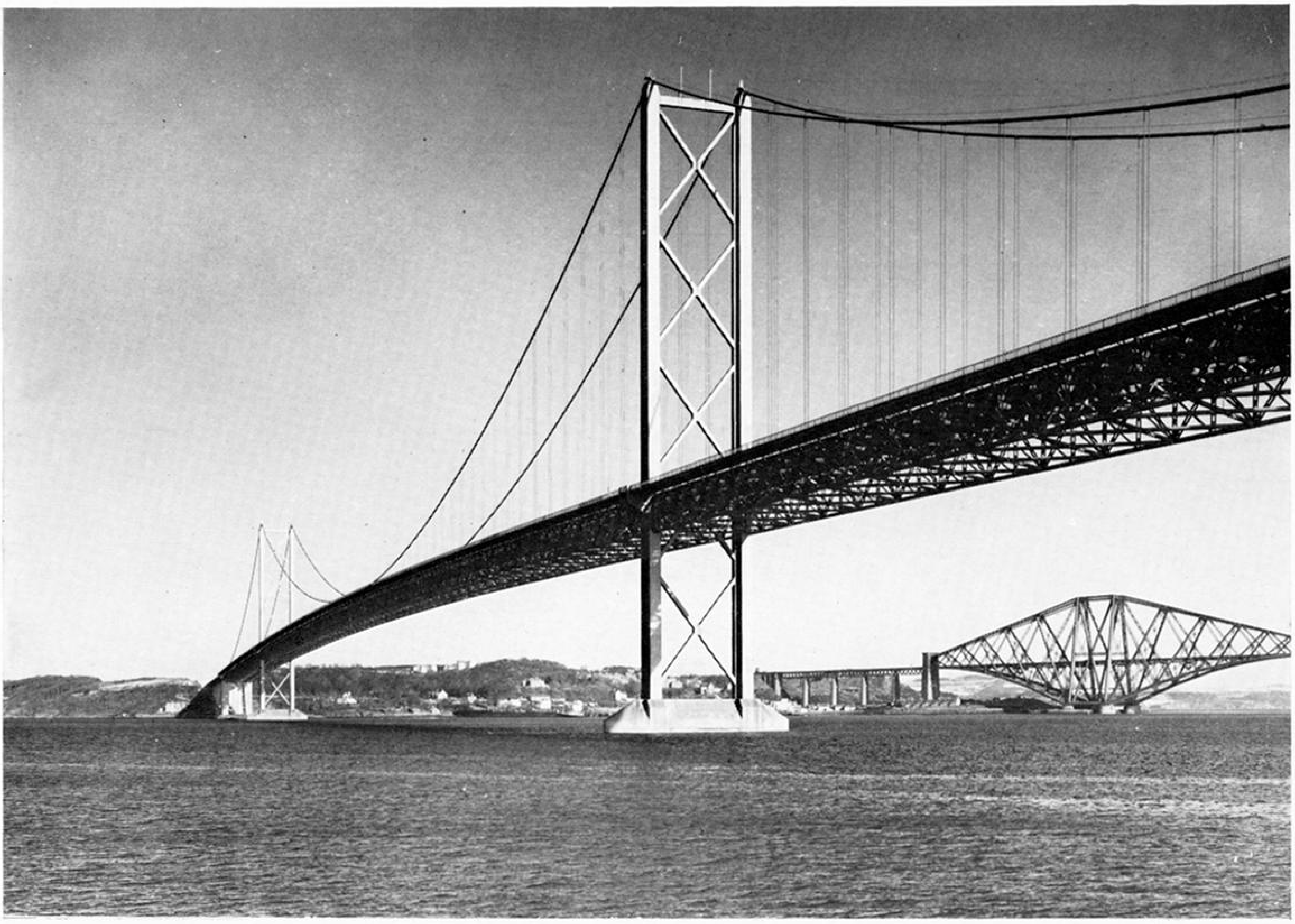

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FIGURE 2. Forth Road Bridge.

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