(*Paper No. 1804.*)

"The Resistance of Viaducts to Sudden Gusts of Wind."

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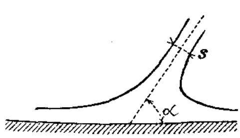
(Translated from the French by L. F. VERNON-HARCOURT, M.A., M. Inst. C.E.)

IN order to ascertain the conditions of stability of a structure exposed to wind, it is necessary, in the first place, to know the pressures which atmospheric disturbances can produce, and then to study the effects of these forces, and the additional strength necessary to resist them.

With regard to the first part of this programme, it is essentially necessary to have recourse to experience. In fact, its only theoretical basis is a doubtful similarity between a gaseous jet and a stream of liquid, which latter, though a more simple phenomenon, admits only of approximate investigations.

When a fluid stream, whose cross-section is s and velocity v, strikes against a plane surface, to which its axis is inclined at an angle α, it spreads out in a layer against the obstacle, as shown in Fig. 4; and the formula which expresses the total normal pressure on the surface is $\frac{\Pi v^2}{g} s \sin \alpha$, in which Π denotes the specific weight of the liquid, and g the acceleration due to gravity. As $\frac{v^2}{g}$ is double the height which the column of water would require to fall to attain a velocity v at the bottom of the fall, it follows that the dynamical pressure, in the case of vertical incidence, may amount to double the weight of the same column in a state of rest. The pressure, moreover, is reduced in striking against a convex surface, and increased against a concave surface.

FIG. 4.



This phenomenon was said to be comparatively simple, because the liquid, owing to its high density, is little affected by the surrounding medium of air which it displaces, or against which it rubs. Moreover, for a stream of small section, the surface is

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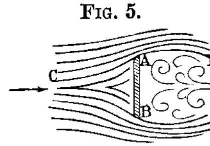
assumed to be much larger than the section, in order that the spreading out may be complete.

If now a plate having an area S, is struck by the air, the gaseous stream will have a cross-section, S sin α, limited merely by the circumference of the plate; the central filaments will always find an ample surface over which to spread, but in doing this they will push out and turn aside the other filaments; and as regards the outside filaments, their position will be so far different that, with only a very slight deflection, they will escape before having exerted all their dynamical force. On the other hand, the column of air arrested by the obstacle will be hemmed in by other layers of air in motion, which it will whirl about in forcing a sideways outlet for itself. Lastly, the partial vacuum produced on the sheltered face will enter as a cumulative force into the problem of stability. If these disturbing conditions could be neglected, taking as an average $\Pi = 1k \cdot 225$, and $g = 9 \cdot 81$, the formula of the fluid stream would give, per square metre of surface impinged on, a pressure of $0 \cdot 125\, v^2 \sin^2 \alpha$, produced by a wind having a velocity of v metres per second, and with an angle of incidence α.

In reality the numerical factor may differ more or less from this theoretical result; but, as regards the degree of influence of the velocity and the mass of the fluid, it appears to be confirmed by the following considerations. An obstacle A B (Fig. 5), being placed in the course of a fluid, the filaments C A, C B, diverge in curved lines, turning their convex side towards the obstacle. This curvature produces centrifugal reactions proportional to the mass of the molecules and to the square of their velocity; and it is the sum of these reactions which develop the "live pressure" against the front face of the body A B. At the opposite side of A B, on the contrary, the filaments A D, in tending to return to the line of their former direction, assume curves with the concave side turned towards the obstacle. Accordingly a partial vacuum, or "non-pressure," as Dubuat terms it, is produced, which has an effect similar to the "live pressure," and is additional to it in the final result. The specific weight being $\frac{\Pi}{g}$, the total resistance may be expressed by $K\, \Pi \frac{v^2}{2\, g}$, or $0 \cdot 0625\, K\, v^2$ per unit of surface, in which the value of the coefficient K must be determined by experiment. If the plate A B is replaced by a prism more or less elongated, the "live pressure" remains the same, but the "non-pressure" is reduced, and

Fig. 5.



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consequently the value also of K, which represents the resultant of both forces. Thus Dubuat, who had obtained $K = 1 \cdot 43$ for a plate moving in a liquid, obtained similarly $K = 1 \cdot 17$ for a cube, and $K = 1 \cdot 10$ for a prism whose length was thrice one of the sides of its base.

Experiments made in air appear to have given results varying between $K = 1 \cdot 3$ and $K = 2 \cdot 2$ in the case of thin plates; variations due perhaps, partly, either to the inexactness of the law of the square of the velocity, or to the influence of the size of surfaces employed, or to the rotatory motion of these surfaces in the experiments when they are paddles of wheels.

General Morin has introduced a constant into the formulæ. From experiments made at Brest, in 1823, by Thibault, by means of a fly-wheel with little sails on a horizontal axis, he deduced the formula $0 \cdot 0044 + 0 \cdot 108\, v^2$ as expressing the resistance per square metre. The coefficient $0 \cdot 108$ remains practically constant for inclinations between 90° and 50°, provided it is referred to a square metre of surface projected on a plane perpendicular to the direction of motion.

In 1835–37 Piobert, Morin, and Didion made observations on the fall of a plate suspended to a cord; the laws of the motion being indicated, with respect to the guide pulley, by a clockwork apparatus. The resulting formula, namely, $0 \cdot 036 + 0 \cdot 084\, v^2 + 0 \cdot 164\, j$, contains a term proportionate to the acceleration j in the case of variable motion, which vanishes for uniform motion. Analogous formulæ have been obtained for parachutes. The velocities observed did not exceed 10 metres (33 feet) per second.

Whereas for slow motion the law of pressure appears to be best expressed by formulæ having two terms, of which one is proportional to the square of the velocity, and the other is taken as a constant by some, or proportional to the simple velocity by others; it is found, on the contrary, that the intensified phenomena of ballistics indicate a greater variation than the square of the velocity. Piobert estimates the resistance to motion of a projectile whose section is s as $0 \cdot 023\, s\, v^2\, (1 + 0 \cdot 0023\, v)$; but sometimes it is expressed by a single term proportionate to v^3. As regards the reduction of pressure due to the obliquity of the current, experiments indicate a less rapidly diminishing factor than the square of the sine. Didion found that in bending the opposing surface so as to form a convex two-sided angle, and inclining each of the two faces thus formed at the same angle a to the direction of motion, the formula has simply to be multiplied by $\frac{a}{90°}$, so long as

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α is between 90° and 65°. Hutton had arrived at the complicated formula $0 \cdot 135\ s^{1 \cdot 1}\ v^2\ (\sin \alpha)^{1 \cdot 84 \cos \alpha}$ for the total pressure upon the surface s of a plate in the case of velocities below 10 metres (33 feet). It will be noticed in this formula that the pressure per unit of surface is considered to be proportional to $\frac{s^{1 \cdot 1}}{s}$ or to $\sqrt[10]{s}$, which agrees with Borda's experiments, which indicated a pressure of $0 \cdot 09\ v^2$ per unit of surface, on a square whose side was 0·11 metre ($4\frac{3}{8}$ inches), and $0 \cdot 105\ v^2$ when the side amounted to 0·25 metre ($9\frac{7}{8}$ inches). The influence of the size of the area on the result is explained by the fact that the filaments of the current near the sides only produce a partial effect, and the larger the surface, the smaller is the proportion of the perimeter to the area. However Didion, Thibault, and other observers, have, on the contrary, arrived at the conclusion that the total pressure is proportional to the surface, and independent of its form. Morin gave as an objection to Borda's experiments, made with a fly-wheel having small sails turning a vertical axis, that the effect of the friction of the apparatus had not been calculated.

The resistances offered by the air to railway carriages in motion have been variously estimated: thus Harding gives $0 \cdot 0627\ v^2$, and Ruhlmann $0 \cdot 117\ v^2$ per square metre of front section. The circumstances, however, are complex, and when it is desired to estimate the resistances as closely as possible, it is necessary to go into the details of the carriages in order to ascertain the effect of the air in the spaces between them.

It is generally accepted as an axiom that the resistance offered by air at rest to a moving body is equal to the pressure which wind moving with the same velocity would exert on the body at rest. Smeaton, adopting a Table drawn up by Rouse for winds having velocities not exceeding 72 feet per second, appears to have accepted pressures denoted by the formula $0 \cdot 0023\ v^2$, which are given in a tabular form in the Minutes of Proceedings, vol. v., p. 292. In the same volume (p. 296) will be found the results of the careful experiments made by Colonel Beaufoy in 1815, with plates however only 1 foot square, which may account for these pressures being less than those adopted by Smeaton. General Morin deduced a formula from some experiments by Thibault in 1826, which gives results approximate to those of Smeaton, but decidedly greater than the resistances experienced in moving flat disks in still air, which would support Dubuat's opinion as to the incorrectness of the axiom mentioned above.

It would appear from calculation[1] that the pressure on a

[1] "Mécanique Appliquée." Bresse (Hydraulique, § 109).

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cylinder is two-thirds, and that on a sphere half of the pressure on their diametral sections. Borda, however, obtained by experiment the smaller values 0·57 and 0·41 as the relations of these pressures. For a prism presenting a right-angled isosceles triangle to the air, he obtained the proportion 0·73, and for a cone the values 0·69 or 0·54, according as the angle at the apex was 90° or 60°.

The velocity of the wind is recorded by anemometers. Thibault obtained the pressures by plates attached to springs for measuring the resistance.[1] In a similar manner Mr. Pâris took measurements of the wind at sea by fastening small boards to a deal rod which served as the spring, and he obtained the following results:—

	Feet per Second.										
Velocity of the wind	2·6	5·9	11·2	19·7	30·2	43·0	59·7	77·4	98·8	125·3	150·9
	Lbs. per Square Foot.										
Pressure exerted . .	0·02	0·09	0·30	0·87	2·13	4·25	8·03	12·56	22·12	37·90	51·20

These figures approximate to those given by Smeaton's formula, and are smaller than those derived from Hutton's formula, which formulæ would give for a great storm of 151 feet velocity per second about 51·8 lbs. and 57·3 lbs. per square foot respectively. In the higher regions of the atmosphere the velocities may be very great, as it is stated that, in 1823, Green travelled in a balloon at the rate of 210 feet per second.

The absolute relation between the pressure and the velocity is by no means indispensable for ascertaining the stability of structures exposed to the wind. It is sufficient for this purpose to find the greatest pressure that may occur in a given locality during a sudden squall.

Rankine states about 55 lbs. on the square foot as the greatest wind-pressure observed in England by anemometers or dynamometers, which is confirmed by the fall of chimneys and other buildings. However, a pressure of 61 lbs. on the square foot was recorded at Liverpool during the storm of the 7th of February, 1868, and of 71 lbs. on the 27th of September, 1875.

The violent storm of 1876, which overturned several chimneys

[1] Dr. Lind, in 1775, employed a reversed siphon containing water; and the wind entering one branch made the water rise in the other branch, thus affording a measure of the pressure exerted. (*Vide* Minutes of Proceedings Inst. C.E., vol. v., p. 290, and Philosophical Transactions, 1775, p. 353.—L. F. V.-H.)

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in Germany, was reckoned to have a velocity of 102 feet, and a direct pressure of 29·5 lbs.; but, taking into account the "non-pressure," due to suction at the back face, it is estimated that the total resultant pressure on these structures must have been a third more, and consequently equal to 39·3 lbs. per square foot.

The upsetting of a train between Narbonne and Perpignan, in December 1867, indicated a pressure of between 30 lbs. and 50 lbs.; and other similar accidents with empty wagons on the same railway in February 1860, and January 1863, indicated a pressure of from 25 lbs. to 33 lbs. No other part of France is exposed to such violent storms; nevertheless, in considering the stability of lighthouses, Fresnel allowed for the possibility of wind-pressures up to 56 lbs.

It would appear that American engineers, for the resistance of bridges, assume wind-pressures of 30 lbs. per square foot upon the loaded and 50 lbs. upon the unloaded structure, although certain local tornadoes in that country might have exerted forces amounting to as much as 84 and even 93 lbs.[1]

Instead of waiting for chance accidents, which have to be investigated after the event with inadequate data, it would be advisable to set up apparatus at once in certain meteorological observatories for registering the pressure of great gales. For example, a kind of case of pigeon-holes might be placed in windows facing in a suitable direction, these holes being closed by a series of little shutters one above the other, capable of moving inwards under certain pressures of wind, being guided by little rollers, and made to close again against the external rabbets of their respective frames by springs or counterpoises with suitably graduated power. Lastly, each of these movable panels might be so arranged that the moment it began to open it should unhook a signal which would bear evidence to the movement even after it had closed again. It would suffice after each storm to ascertain, by a rapid inspection, which of the panels had yielded to the wind, and then whichever of these panels offered the greatest resistance would measure the pressure experienced.

Of all engineering structures, suspension-bridges are the most easily acted upon by wind. Their primitive methods of construction were defective through excessive flexibility. The accident which happened to the Roche-Bernard bridge on the Vilaine, on the 26th of October, 1852, and the successive injuries to the Menai bridge in 1826, 1836, and 1839, may be cited as examples. The

[1] Minutes of Proceedings Inst. C.E., vol. lxiv., p. 352, and vol. lxvi., p. 388.

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chains of the latter bridge, though clashing together violently, bore the strain; but a number of transverse pieces and suspension rods broke, and 160 feet of flooring hung in the air in 1839. According to the bridge-keeper, the undulations of the roadway attained an amplitude of 13 or 16 feet, and the greatest deflections were observed at the distance of a quarter of the span from the piers. It is evident that everything gives way in these irregular undulations, which are different for the chains and the roadway. The Menai bridge was strengthened by various means. The Roche-Bernard bridge was provided with a counter-cable, curving upwards and placed under the roadway; and notable progress has been achieved in the design of more recent works. The Americans, in developing the principle of the stiffening girder, have also added a series of straight and sloping cables coming from the top of the piers and supporting various parts of the roadway. They have, moreover, in some large bridges, anchored the roadway to the rocks by stays underneath, a method which is not free from objections any more than the parabolic counter-cable of the Roche-Bernard bridge, for the variations in temperature may at one time loosen and at another time stretch these understays.

In the Ordish system, as applied to the Albert bridge, Chelsea, the upper stays, starting from the tops of the piers and ending at various parts of the roadway, are connected with the vertical suspension rods at divers points of crossing, which increases the total rigidity. Sometimes, as at the Lambeth bridge, rigidity is obtained by the introduction of cross bracing or diagonal bars between the suspension rods; or, as at Pittsburg, the chain itself is made rigid, assuming the appearance of two sloping lattice girders of variable height, and attached by their narrowed extremities, at one end to each other in the centre of the span, and at the other end to the tops of the piers.

The great transversal inclination in certain bridges to the two funicular planes, by which the cables, spreading out at the top of the piers, come together in the centre of the span, affords a powerful resistance to lateral oscillations.

With these improvements the suspension system, without losing its inherent lightness, is protected from irregular undulations when exposed to wind; so that the wind-pressure merely acts on it, like on any other structure, in producing an increased molecular strain which has to be provided for by strengthening the parts liable to be affected.

It is true that a great number of suspension-bridges exist which were constructed on the old flexible principle, and have stood for

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many years; but their preservation is doubtless due, in most cases, to their not having experienced the full force of the wind whirling under their roadways, owing to their small height above the water, or other circumstances. The most exposed bridges are those which traverse deep and shut-in gorges at a great height.

Wind has no effect on massive stone bridges; but every light bridge, whether of iron or wood, although rendered rigid, is liable to side strains, or small elastic vibrations producing molecular deformations, upon which the conditions of resistance of the material depend.

Though the motion of wind is generally parallel to the ground, its action on the underside of the roadway may become considerable, owing to the rebound of the wind from the bottom of ravines, which occasions the great danger to light flexible suspension bridges of being raised and falling again violently. When the wind, blowing in sudden gusts, lifts the platform slightly, the platform falls again for a moment below its normal level to a similar extent, so that the pressure of the wind from below produces eventually the same strain as if its action was added to the load. Accordingly in special cases, where it might be possible to estimate at an appreciable amount the vertical resultant of a storm beating against the roadway of a bridge, it would be correct to treat it as an extra load on the bridge.

The effect might be still more serious in a bridge with several continuous spans, for, as nothing could ensure the concordance of the oscillations of the various spans, it would be necessary to provide against the worst case of a pressure from above on certain spans aggravated by a pressure from below on certain other spans.

Putting aside, however, these accessory or derived effects, let the wind be considered solely in its horizontal direction, in which it displays its greatest power, and, knowing its force on a single solid surface, let an endeavour be made to calculate the force exerted on several open, or partly open, surfaces.

Taking the case of a bridge consisting of two solid girders, though these girders cover each other completely in a geometrical sense, yet the first, whilst exposed to the full force of the wind, does not completely shelter the other. Thibault experimented on two square screens covering each other, and placed at a distance apart equal to the length of one of their sides, and found that the wind-pressure on the one screen being 1, a total wind-pressure was experienced on the two of 1·7. In the case of a bridge, the wind-pressure cannot be so high, as instead of four edges there are only two at the most (when the platform is halfway up the girders),

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round which the wind can whirl and beat against the second surface; the coefficient of increase in such a case, deduced from the preceding instance, will perhaps amount at most to 1·4. It would be reduced to 1, and even less, if the girders were connected by solid platforms at their upper and lower edges. Lastly, in the case of a single platform, placed at the top or the bottom, it would be perhaps necessary to estimate the total lateral pressure as equal to 1·2 time that which the side directly exposed would experience. It is evident that if a train is on the bridge at the time when the storm is raging, the resistance that it offers to the wind aggravates the strains on the structure.

Considering, now, the case of trellis girders, each opening may be regarded as an orifice, with thin sides, through which a jet of air rushes; there will be some contraction of the fluid vein, and the side will experience a little greater resistance than the ratio between solid and void would indicate. If p denotes the wind-pressure, s the whole surface of the side of the girder, σ the open portion of this surface, and k the coefficient of contraction, the pressure on the girder will be $p\,(s - k\,\sigma)$. The value of k, according to D'Aubuisson, would equal 0·65 for small orifices, but as it doubtless varies inversely as the ratio of the perimeter to the surface, which diminishes as the dimensions increase, it may be assumed that k approaches unity in the case of large openings. However, as its real value is not known, it will be better to risk exaggerating it in the case under consideration.

Suppose, now, that a second side exactly similar is placed behind the first, it receives the shock of the portion of wind which has passed through. This wind may be considered to have been made homogeneous by the whirling which occurs in the interval between the two girders, and to have a reduced force $p\,\frac{k\,\sigma}{s}$, according to the relation between the amount of air which has traversed the first girder and the total original mass. Consequently the second trellis will experience a pressure $\frac{p\,k\,\sigma}{s}\,(s - k\,\sigma)$; and similarly the wind which passes through it will have its force reduced to $p\left(\frac{k\,\sigma}{s}\right)^2$. If there are n successive girders, the sum of the pressures experienced will be

$$p\left(s - k\,\sigma\right)\left(1 + \frac{k\,\sigma}{s} + \frac{k^2\,\sigma^2}{s^2} + \ldots + \frac{k^{n-1}\,\sigma^{n-1}}{s^{n-1}}\right) = p\,\frac{s^n - k^n\,\sigma^n}{s^{n-1}}.$$

As the above calculation does not take into account the wind which may come round the sides of the front girder, a certain

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coefficient must be introduced, smaller than in the case of solid girders, as some opposition is offered to the inflowing wind by the wind passing through the girder. Perhaps the coefficient 1·10 would apply suffice in the majority of cases.

Another process of approximate calculation of the pressure of wind on a trellis girder has been employed by Mr. Nordling. He assumes that the filaments of air slant a little, so that those which pass through the openings of the first girder strike against the solid portions of the second. In this way a succession of trellises would finally act as a solid girder, when no openings are visible in a direction only slightly deviating from the normal.

Having ascertained the lateral force exerted by the wind against the roadway of a bridge, it is necessary to calculate the special molecular strain which it tends to set up, in order to add it to that produced by the permanent and moving loads. In resisting the wind, the roadway acts as an imaginary girder whose flanges are the actual girders of the bridge, and whose lattices are the horizontal braces and wind ties. The resistance, moreover, offered by the irregular interlacing motion of the trains must be taken into consideration. Owing also to the wind coming in gusts, thus causing a reaction, its effect on each girder, whether tensive or compressive, must be considered as added to the strain due to the load, and in the case of several spans the most unfavourable condition must be allowed for.

An arch has the advantage over a straight girder of opposing less surface to the wind in the central portion, whilst the opposite is the case with a bow-string.

Two examples of iron arches, with narrow roadways, spanning very large openings, are those of Oporto, over the Douro, which has a width of 14 feet 9 inches between the parapets and a span of 525 feet, and that of the Montereale, over the Cellina torrent,[1] which has a width of 9 feet 10 inches and a span of 272 feet. But these bridges are secured against the wind by special contrivances; the first, by giving a batter of 0·1164 to each face of the bridge, so that the distance from centre to centre of the arched ribs, which is only 12 feet 10 inches at the crown, is increased to 49 feet 2½ inches at the springings; the second by an external wind bracing, namely, by side buttresses coming from the haunches of the arch, and butting against the masonry at two points 27 feet 7 inches apart, whereas the distance between the arched ribs is only 9 feet 10 inches.

[1] Minutes of Proceedings Inst. C.E., vol. lxiv., p. 354.

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Certain structures may be liable to be wholly overturned by a gust of wind. Iron superstructures are generally free from this danger in consequence of their weight, except perhaps during a dangerous stage in some methods of putting them in place, especially if detached girders are being moved. On the contrary, the iron piers of very high viaducts need to be very firmly anchored in their masonry pedestals, as Mr. Nordling has pointed out in his memoir about various works on the branch lines of the Orleans Company.[1] These kinds of piers are eventually strained as elastic braced structures fastened at their base and subjected at their summit to violent horizontal thrusts. On this account, instead of distributing their mass in a number of external and internal uprights, it is better to concentrate it at the angles in only four ribs connected together by cross-bracings. The anchorage at the base is rendered more economical, or more powerful, by fastening buttresses to the piers near their foot so as to enlarge their base. If the height does not exceed 130 feet, as for instance at the Bellon viaduct, the uprights may be curved outwards towards their base, so as to spread out without the aid of special stays. It would be equally feasible to secure the tops of high piers by stays fastened near the top of the piers and firmly anchored to the ground; but the system of buttresses is more æsthetic, and is not liable to get loose. One of the high piers of the Bouble viaduct, Fig. 6, will serve as an example to illustrate, by an approximate process, to what severe strains such a structure might occasionally be exposed. Mr. Nordling has assumed the wind-pressure at 55·3 lbs. per square foot, without allowing for a train on the bridge, as, in his opinion, if such a storm ever burst upon these structures the traffic would be suspended for a time; and, moreover, the above pressure appears to him excessive for the locality. Let, however, the worst possible case be considered by imagining a concurrence of adverse circumstances, the structure being in a very exposed situation, and the full fury of the gale suddenly occurring whilst a train is passing over.

Taking only a half pier containing two uprights and the intermediate bracing, the span being 164 feet, crossed by two lattice girders 14 feet 9 inches high, it appears that, allowing for the spaces, the wind, having a pressure of 55·3 lbs., would exert a total stress of about 20 tons at a height of 196·2 feet above the footings, which gives a moment of 3,924. The pressure on the train is 16·2 tons, with a leverage of 210·3 feet, giving a moment of 3,407. Lastly, the moment of the pressure of the wind on the half pier amounts to 20 tons $\times$ 92·85 feet = 1,857. Thus the total moment of

[1] Annales des Ponts et Chaussées, 1864 and 1870.

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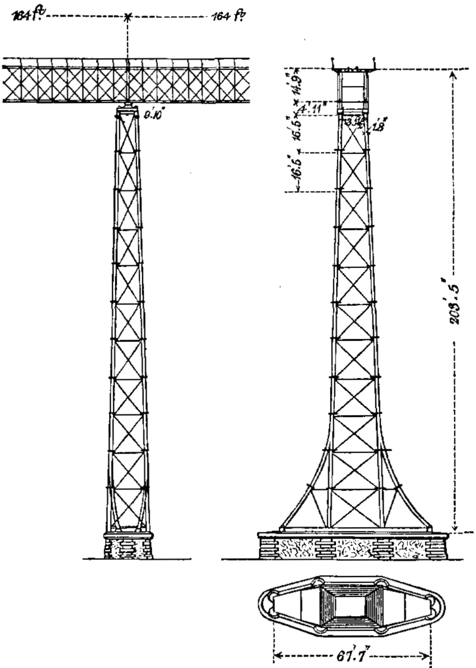
overturning on the edge of the base is 9,188. The moment of stability due to the loads is obtained as follows: taking 60 tons as the weight of the half span, and 120 tons as the weight of the half pier (the cast-iron cylinders being ballasted with concrete), and allowing 42·5 tons as the weight of the train which suffices to prevent its being overturned by the gale, the total weight amounts to 222½ tons, and the half width of the base being 33·8 feet, the

FIG. 6.



PIER OF THE BOUBLE VIADUCT.

moment is 7,520, leaving a deficiency of 1,668. To provide for this the anchorage tie must exert a tension of $\frac{1,668}{67 \cdot 6} = 24 \cdot 69$ tons. Without the help of the buttresses the width of the base of the pier would be only 24 feet 3 inches, instead of 67 feet 7 inches, and the anchorage would be subjected to the great strain of 267 tons.

In order to form a notion, not merely of the strain on the anchorage, but of the strain on the whole structure of the half pier, a graphic illustration is given of the polygon of forces, considering, for the sake of simplicity, the imaginary case of an

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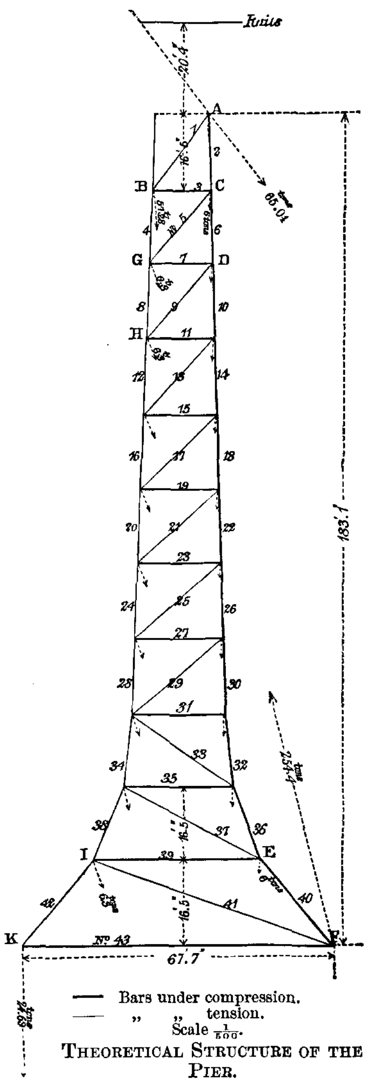
articulated structure. The lattice, moreover, is hypothetically reduced to the lines of Fig. 7, by omitting as well the foot of the straight uprights, replaced by the corresponding curved or polygonal stay, as in each row of bracing, that of the two diagonals which, exposed to a wind from the left, would be strained in compression, and are considered to be too flexible to offer an effectual resistance in this way.

Fig. 7.



Theoretical Structure of the Pier.

The external forces applied to the various summits produce the following horizontal components. At the summit A the whole force of the wind against the beams and the train is brought to bear, namely, a force of 40·04 tons obtained by dividing the moment, 7,331, by the height, 183 feet, of the point A above the base. The pressure against the half pier amounts to about 2 tons acting at each of the points B, G, H, . . . I, situated on the side which the wind strikes. The weights or vertical components are:—51·25 tons at A, due to the loaded roadway; the same weight at B increased by a portion of the pier, amounting altogether to 57·25 tons; lastly, in each of the points G, H, . . . I and C, D, . . . E, a vertical force of 6 tons. The reactions in equilibrium developed by the base of support are: at K, the tension of anchorage, amounting to 24·69 tons as calculated above, acting from the top to the bottom; in F, a vertical upward reaction equal to the total weight increased by the strain of anchorage, namely,

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to 247·2 tons; and a horizontal force acting from right to left, which, counteracting in projection all the wind-pressures, is equal to 60·04 tons.

The resultants at the different points consequently assume oblique or vertical directions. The oblique resultants are: 65·04 tons at A; 6·3 tons at each of the points G, H, . . . I of the left upright; and 254·4 tons at the point F of the right upright. The state of equilibrium of the external forces is shown by a closed polygon in Fig. 3. Moreover, this figure is completed by the addition or grouping of a series of other closed polygons representing the respective states of equilibrium of the various summits of the articulated system of Fig. 7, under the influence of the internal and external forces acting on each of them. The inscription of identical numbers in Figs. 7 and 8, serves to indicate their connection; thus, for example, the closed polygon 8, 9, 11, 12, 6·3 tons in Fig. 8 proves that the point H of Fig. 7 is in equilibrium under the external force 6·3 tons, the tensional strains of the bars Nos. 8, 9, 12, and the compression of the bar No. 11, the intensities of the forces being measured by the size of the lines on the diagram, Fig. 8. It will be observed that the left side is in tension from G to K, the greatest tensional strain, of about 190 tons, occurring on the portion No. 34. With a cast-iron pipe having an external diameter of 1 foot 8 inches, and an internal diameter of 1 foot 4 inches, this strain would amount to 1·9 ton per square inch; but, as previously stated, the Bouble viaduct was constructed on the supposition of the maximum pressure being less. The compressive strain reaches 422 tons at the portion No. 40, which would amount to 4·1 tons per square inch, but in reality the strain is less if the uprights are made complete, as shown in Fig. 6.

Moreover, it is certain that the rigidity of the cast-iron columns and their bolted flange-joints must considerably modify the conditions of the problem. Instead, therefore, of merely comparing the pier to an articulated system, each member of which is considered to be free to deflect in any way, as assumed above, it would be necessary, in a complete design, to study the transmission of force resulting from impeded deflections.

In certain mechanical structures, as, for instance, in swing bridges with short tail ends, the action of high winds may stop or impede their motion without actually producing any dangerous amount of damage.

High timber stagings, owing to their lightness and the broad surface presented by their planks, are exposed to considerable risks

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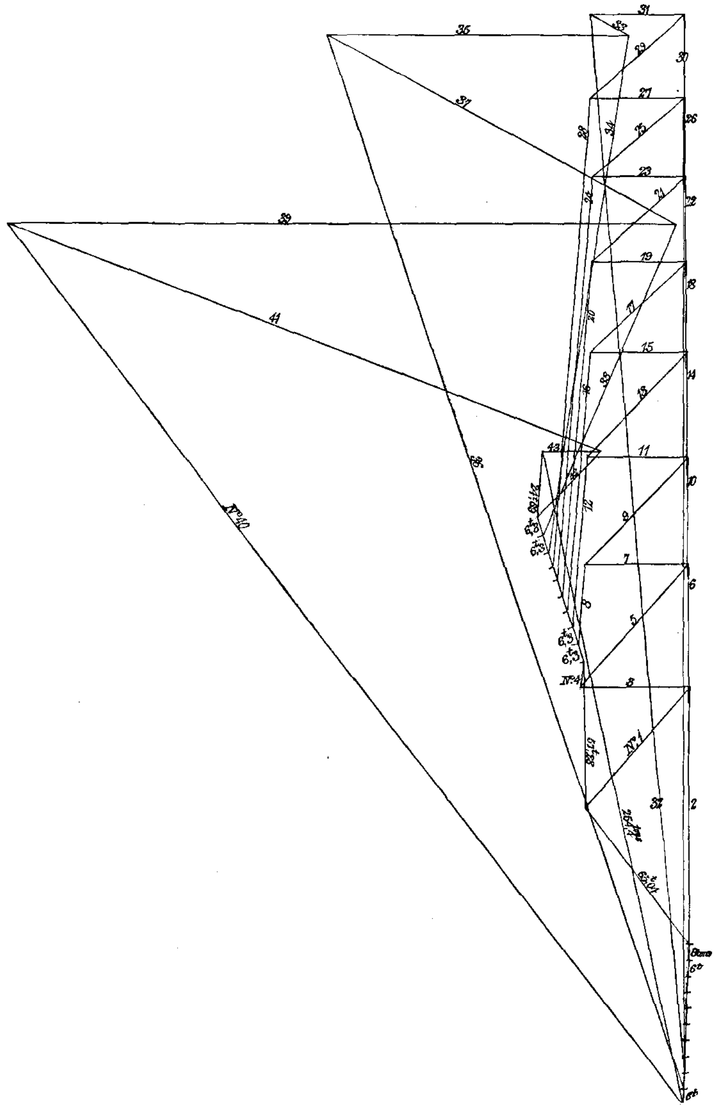
of damage by wind. An excellent method for strengthening them was adopted at the Chaumont viaduct, which is 164 feet high,

FIG. 8.



and has three tiers of arches, each of which was provided with a temporary platform for the supply of materials. The staging was

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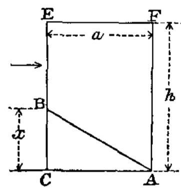
braced in various directions by iron wire cables, very tightly stretched and firmly anchored.

When a structure rests without sufficient adherence on a fixed base, a lateral thrust would turn it over by detaching it from its support; but if its fall cannot be effected without some indeterminate or chance cleavage, the rupture will take place in an oblique and downward direction B A, Fig. 9, because a certain triangular prism, B A C, possesses a stable position, on account of the leverage of the weight being great, and that of the impact of the wind small, in relation to the axis of rotation.

Fig. 9.



In reality, so long as the solid is not broken, the pivoting does not tend to take place on the extreme edge A, but upon some neutral axis of the section of rupture A B; as in every prismatic body, subjected to a bending strain, fracture results from the crushing of some portions and the tearing of others.

The direction A B being defined by the indeterminate C B = x, the external forces acting are, the weight of the prism A B E F, and the pressure of the wind on B E. In calculating the combined effects of pressure and flexure exerted on A B, the chance of fracture would be investigated from the position of the critical point A or B. The first of these points is the place of maximum compression; assuming that it reaches the limit of imminent crushing, an equation of ultimate resistance could be formed containing x and the pressure p of the wind per unit of surface as the variables. Then, by finding what value of x in this equation would make p a minimum, the direction of rupture would be obtained, provided that it is the point A where the disintegration begins. Such would be the condition of a building very much strained by its own weight before the intervention of the wind.

Under other circumstances, however, the point B might eventually be subject to a tension liable to prove more dangerous, though smaller in amount, than the pressure at A, owing to the material being less able to bear tension than compression. It would be necessary, therefore, to examine the equation of rupture with regard to the point B, which might lead to another value of x applicable to the case where the disintegration commenced at this edge.

Nevertheless, nothing indicates that the fracture must be a plane surface. It might possibly slope somewhat in a homogeneous body; and in a masonry structure the fracture would run along the joists in some zigzag line; and these considerations limit the value of theoretical investigations.

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Another reason for avoiding putting down the equations is, that they would lead to the disputed question of ultimate resistance in the complicated case of a material opposing an unequal resistance to tension and compression. With reference to the practical and legitimate need of a method or formula of safety applicable to the case in question, it is allowable to start on the simplifying hypothesis, commonly admitted in investigations of the stability of masonry, of the absence of cohesion, or neglect of the resistance to tension. If the line A B, whatever its direction, is regarded as a pre-existing fissure, the initial effect of the gust of wind, instead of being a pivoting on some neutral axis, would be from the first a rotation on the point A itself, at least, if the slight crushing of the edge is neglected. If, for example, Fig. 1 represents a wall with a rectangular base, the equation of actual equilibrium of rotation is $p\,\frac{h^2 - x^2}{2} = \Pi\, a^2\, \frac{3\,h - 2\,x}{6}$, where Π is the weight of a cubic foot of masonry. To remain stable against a given wind-pressure p, the wall must have a thickness a sufficient for the most dangerous value of x. Now the value of x, which makes a a maximum in the above equation, is given by $x^2 - 3\,h\,x + h^2 = 0$, or $x = 0{\cdot}382\,h$; and the corresponding thickness is $a = 1{\cdot}0705\sqrt{\frac{p\,h}{\Pi}}$. If, for instance, $p = 55{\cdot}3$ lbs. per square foot, and $\Pi = 150$ lbs. per cubic foot, the proper thickness would be $a = 0{\cdot}65\,\sqrt{h}$, where a and h are in feet. With this value there would remain the cohesion, which has been neglected, as a factor of safety; and there would be no fear of the occurrence of extensions or of fissures, since, even with pre-existing fissures, the wall would not stir. If, however, a greater degree of stability was requisite, it would suffice to increase a by an optional amount.

An interesting instance of oblique rupture, caused, not by the wind, but by a stroke of the sea, occurred on the 8th of January, 1867, to the masonry tower beacon of "Petit Charpentier" at the mouth of the Loire. From an investigation of this accident, Mr. Leferme arrived at the conclusion that the pressure exerted by the blow of the wave must have amounted to about 6,140 lbs. per square foot. Mr. Thomas Stevenson, M. Inst. C.E., deduced some equally high pressures from observations at the Skerryvore rocks, which appear to be confirmed by the jets of water sometimes dashed to a height of 100 feet against lighthouse towers. Nevertheless, in most storms, and on most sea-coasts, the dynamical pressures exerted by the shock of the waves are generally estimated not to exceed from 600 to 1,000 lbs. per square foot. Even with this

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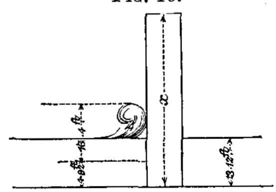
reduced value it is questionable whether, in the case of lighthouses and other structures in the sea, the wind-pressure is not less dangerous than the shock of the waves. Taking the latter at 1,024 lbs., and the wind-pressure at 55·3 lbs. per square foot, and assuming a tower to be immersed 13 feet (4 metres) in the water (Fig. 10), to what height would the tower have to be raised for it to be in as much danger of being overturned by the wind as by the waves? The sea in a storm would perhaps rise 8·2 feet (2·5 metres) above its ordinary level; and if the smaller pressure on the bottom 5 feet (1·5 metre) is neglected, the total pressure on a height of 16·4 feet (5 metres) would amount to 16,794 lbs. With a leverage of 13·12 feet (4 metres), the overturning moment with respect to the base is 220,000. Now, supposing the height of the tower to be x, the portion out of water will be exposed to a wind-pressure of 55·3 lbs. $(x-13)$, and the moment of this force, $27\cdot65\ (x^2-169)$, will only become equal to the former moment when x reaches the height of $90\frac{1}{4}$ feet.

FIG. 10.



If the same calculation is repeated on the assumption that the shock of the sea has its greatest possible degree of intensity, namely, that the wave rises 13·12 feet above its ordinary level, and exerts a pressure at the same instant of 6,144 lbs. on the whole height of $26\frac{1}{4}$ feet, the corresponding moment of 2,116,000 could not be equalled by the wind-pressure on a tower less than 277 feet high.

On the contrary, in the case of a viaduct only opposing a resistance to the water at the lower extremities of its piers, whilst the wind beats upon the lofty superstructure as well as against the piers, there is in all probability more danger to be apprehended from the wind. As to the conditions under which the Tay bridge catastrophe occurred, the Author is not in a position to discuss them.

This communication is accompanied by several small scale diagrams, from which the woodcuts have been engraved.

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