WIND STRESSES

[FLEMING]

Six Monographs on Wind Stresses

WIND PRESSURE FACTORS
SPECIFICATION REQUIREMENTS
MILL-BUILDING STRESSES
RIGID JOINT · WIND BRACING
FOR OFFICE BUILDINGS

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PREFACE

Wind is everywhere. It affects all structures. Every engineer, or even every person who ever sees an engineer, has a personal interest in the effect of wind on structures. That is the subject of the present book.

Ignorance is not always bliss. If it be true—as seems very likely—that wind is preëminently a matter concerning which no one knows he knows not, then this book deserves to have many readers.

There is a fresh touch to what the author says. His main argument throughout is common sense. In this respect his frame of mind is catching; the reader will find himself saner and sounder for having read the book. When the author refers to intricate studies of wind action, claimed to show the need for radical changes in building practice, he leads us to notice the practical fact that these intricate studies are based on tests made in mild breezes, and can hardly be safe guides as to what happens in storms.

Six articles which appeared in ENGINEERING NEWS, most of them in the early months of 1915, make up this book. One of the six, however—No. 6—is so changed from the form in which it was printed March 13, 1913, that it is new. And this subject, the stress calculation for tier-building frames without diagonals, is so far without any literature.

The many slow hours of work which the author consumed in searching out and studying the material required for writing this book merit the reader's appreciation.

EDITOR *Engineering News.*

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Wind Pressure Formulas and Their Experimental Basis

SYNOPSIS—A discussion of the current formulas for relation between wind pressure and velocity, relation between pressure on normal planes and planes inclined to the wind, and several other phases of wind pressure. The author brings out strikingly how inadequate is the experimental basis for the formulas and figures commonly employed.

The purpose of this article is to give the basis from which some of the commonly used formulas for wind pressure are derived. Even the engineer who wishes to know only the wind pressure in pounds per square foot for which he shall make provision in his structure will be better equipped for designing if he is acquainted with the foundations on which ordinary practice rests.

Relation between Wind Pressure and Velocity

In view of the extent of the literature on the subject it might reasonably be supposed that the elementary principles of wind pressure are determined, at least theoretically. How near this is to being the case may be inferred from the following extracts taken from two modern American textbooks, each of which is regarded as an authority. Marburg, in his *Framed Structures and Girders,* under "Wind Pressure," writes:

Theoretically the pressure p, in lb. per sq.ft., on a plane surface normal to the direction of flow of a fluid having a relative velocity v, in ft. per sec., is equal to the weight of a vertical column of the fluid having a cross-section of 1 sq.ft. and a height h, in ft. equal to that through which a freely moving body must fall to acquire the velocity v. If w denotes the weight of the fluid, in lb. per cu.ft.,

$$p = wh = \frac{wv^2}{2g} \qquad (1)$$

For air at a temperature of 32° F. and at a barometric pressure of 760 mm., $w = 0.081$. Letting $g = 32.2$,

$$p = 0.00126\ v^2 \qquad (2)$$

If V denotes the velocity of the wind in miles per hour, $v = 1.47\ V$, whence equation (2) becomes

$$p = 0.0027\ V^2 \qquad (3)$$

Burr and Falk, in *The Design and Construction of Metallic Bridges,* under "Stresses due to Wind" write:

If the wind were directed as a finite stream against an infinitely large surface, so that the direction of the air is completely changed, an equation expressing the force against that surface may be obtained from the laws of mechanics. Let

W = the weight of air directed against any normal surface in a given time;

w = the weight in pounds of one cubic foot of air;

v = the velocity of wind in feet per second;

a = the area of cross-section of the wind stream,
Then $W = wav$.

Let

M = the mass of air of the weight W;

g = the acceleration due to gravity = 32.2 feet per second;

F = the force acting on the area a,

$$\text{Then } F = Mv = \frac{Wv}{g} = \frac{wav^2}{g} \qquad (1)$$

If a be taken at 1 sq.ft., and w at 0.0807 lb. per cu.ft. for a temperature of 32° F. and a barometric pressure of 760 mm., and if v be replaced by V, the velocity in miles per hour, then

$$F = 0.0054\ V^2 \qquad (2)$$

The reader will observe that starting with the same assumptions one author finds the resultant pressure to be twice that of the other. Both authors make haste to write that the theoretical conditions upon which their formula is based do not exist. A cushion of air is formed in front of the plate and a partial vacuum at the back; there is a certain amount of air friction and the change of direction is not complete. The student facing such conflicting theories on the very fundamentals of wind pressure may well raise the question of authority.

It is almost impossible to give undue credit to Sir Isaac Newton for his work in the realms of science and mathematics. His great book was the *Philosophia Naturalis Principia Mathematica,* or "The Mathematical Principles of Natural Philosophy," commonly called the *Principia.* Originally published in 1686, revised editions were issued in 1713 and 1726. Modern hydrodynamics had its origin in the second book, treating of Motion of Bodies in Resisting Mediums. Section VIII of this book is entitled "Of Motion Propagated through Fluids." A translation of Prop. XLVIII (Newton wrote in Latin) reads:

> **The velocities of pulses propagated in an elastic fluid are in a ratio compounded of the subduplicate ratio of the elastic force directly, and the subduplicate ratio of the density inversely; supposing the elastic force of the fluid to be proportional to its condensation.**

This means that the velocity v varies as $\frac{\sqrt{p}}{\sqrt{d}}$, or p varies as dv^2. For wind pressure, the density of the air being constant, we have the law that the pressure varies directly as the square of the velocity, which has remained almost undisputed since Newton's day.

Furthermore, according to Newton, for an area of unity, $p = dh$, in which $h = \frac{v^2}{2g}$ is the distance through which a heavy body must fall to acquire the velocity v, g being the coefficient of gravity or 32.2. This may be called the Newtonian theory, and has been followed by a host of writers, including Marburg (quoted above).

W. J. M. Rankine was one of the master mathematicians of the nineteenth century. In his fifteenth year his uncle presented him with a copy of Newton's *Principia,* which he read carefully. He remarks, "This was the foundation of my knowledge of the higher mathematics, dynamics and physics." But the pupil did not blindly follow the master. In his *Applied Mechanics,* he has a section devoted to "Mutual Impulse of Fluids and Solids." A jet of fluid A, striking a smooth surface, is deflected so as to glide along the surface in that path which makes the smallest angle with its original direction of motion. Let v be the velocity of the particle of fluid, q the volume discharged per second equal to Av, d the density, and θ the angle by which the direction of motion is deflected; then $\frac{dqv}{g}$ is the momentum of the quantity of fluid whose motion is deflected per second. With these notations the general equation for the force

Fx perpendicular to the plane in question is found to be

$$Fx = \frac{d\,q\,v}{g}(1 - \cos\theta)$$

For the particular case of the plane at right angles to the jet or $\theta = 90°$,

$$Fx = \frac{d\,q\,v}{g} = \frac{d\,A\,v^2}{g}$$

This may be called the impact theory, and is followed in some textbooks, including that of Burr and Falk.

From the time of Newton until this day a long line of investigators have sought by experiment to obtain the value of k in the formula $P = kV^2$, in which P = pressure in lb. per sq.ft. and V = velocity in miles per hour. As before noted, according to the Newton formula k is 0.0027 and with the same assumptions according to Rankine k is 0.0054. What is known as the Smeaton formula held almost universal sway for 150 years and is still in use. It is very simple, $P = {}^1/_{200}\, V^2$. In the Philosophical *Transactions* of the Royal Society, England, for the year 1759 is a lengthy paper entitled, *An Experimental Enquiry Concerning the Natural Power of Water and Wind to Turn Mills, and Other Machines, Depending on a Circular Motion. By Mr. J. Smeaton, F. R. S.* Part III is "On the Construction and Effects of Windmill-Sails." For his experiments Smeaton constructed an elaborate machine or whirling-table in which fixed sails revolved through the air about a given axis and their velocities were measured by the weights lifted. A footnote reads:

Some years ago Mr. Rouse, an ingenious gentleman of Hasborough in Leicestershire, set about trying experiments on the velocities of the wind, and force thereof upon plain surfaces and windmill sails.

It is presumed, though not so stated, that Mr. Rouse used a whirling-table similar to that described by Smeaton. Further on in the paper a table "containing the velocity and force of wind, according to their common appellations," is found introduced with:

The following table which was communicated to me by my friend Mr. Rouse, and which appears to have been constructed with great care, from a considerable number of

facts and experiments, and which having relation to the subject of this article; I here insert as he sent it to me; but at the same time must observe that the evidence for those numbers where the velocity of the wind exceeds 50 miles an hour, do not seem of equal authority with those of 50 miles an hour and under. It is also to be observed, that the numbers in column 3 are calculated according to the velocity of the wind, which in moderate velocities, from what has been before observed, will hold very nearly.

From this introduction it is impossible to tell where experiment ended and theory began. The coefficient of V^2 according to the figures given in the third column of the table is found to be 0.00492, or $^1/_{200}$ nearly. It is hard to understand how a formula resting upon such a slender foundation should have had such wide vogue.

The most careful experiments of recent years for the pressure on flat plates of moderate size normal to the direction of a uniform wind give a value of k from 0.0032 tc 0.004. Hence the formula $P = 0.004\,V^2$ may be safely used. It is interesting to note that Weisbach, in his monumental work, the *Mechanics of Engineering,* followed Newton's method but multiplied the value of k as found by this method by a coefficient 1.86, stating that about two-thirds of the action is upon the front and about one-third upon the rear surface. He based his coefficient upon the experiments of Dubuat (about 1780) and Thibault (1826).

The U. S. Weather Bureau uses the formula

$$P = 0.004 \frac{B}{30} V^2$$

in which $B =$ height of barometer in inches. This for the engineer is an unnecessary refinement as $\frac{B}{30}$ varies but little from unity. Wolff in his book *The Windmill as a Prime Mover* takes into account also the effect of temperature in determining wind pressure. At sea level for a wind velocity of 40 miles per hour he finds pressures of 8.6 lb. per sq.ft. for 0° F. to 7.08 lb. for 100° F. For a velocity of 80 miles per hour he finds pressures of 34.98 lb. per sq.ft. at 0° F. to 28.86 lb. at 100° F.

Wind-Pressure Coefficient for Inclined Surfaces

For the intensity of wind pressure on inclined surfaces we have a wide range of values from which to choose.

Tiberius Cavallo, F. R. S., etc., in 1803, published a four-volume treatise on *The Elements of Natural or Experimental Philosophy.* The writer has never seen the treatise quoted, but Chapter IV of Book II, "Of the Action of Nonelastic Fluids in Motion," and Chapter X of the same book, "Of Air in Motion, or of the Wind," are written in a truly scientific spirit and are readable today. A proposition of Cavallo's reads, "The forces of a fluid medium on a plane cutting the direction of its motion with different inclinations successively, are as the squares of the sines of these inclinations." This, however, is implied by the great Newton in the *Principia,* Book II, Prop. XXXIV. Among recent writers Spofford in "The Theory of Structures" deduces the same theoretical results.

As these results differ widely from those obtained by experiment, recourse must be had to empirical formulas. Among such, Hutton's formula has been used in England and the United States perhaps more than all others combined. It is still found in the latest editions of many technical books. The experiments upon which it is based were decidedly crude. Tract XXXVI of *Tracts on Mathematical and Philosophical Subjects* by Charles Hutton, LL.D., F.R.S., Professor of Mathematics in the Royal Military Academy of Woodwich, England, entitled, "Resistance of the Air Determined by the Whirling-Machine," records his experiments. Hutton secured a whirling-machine and during 1786 and 1787 experimented with hemispheres and cones. Under date of July 23, 1788, he records:

> Prepared the machine to make experiments with figures of shapes different from the foregoing ones. Procuring a thin rectangular plate of brass to fix on the arm of the machine; its weight $11\frac{1}{32}$ oz. and its dimensions 8 in. by 4 in., consequently its area was 32 sq.in. . . . It was adapted for fitting on the end of the arm in both directions, . . . It was also contrived to incline the surface in any degree to the direction of motion, to try the resistance at all angles of inclination. When fitted on with its length in the direction of its arm, the distance of its center from the axis of motion was 53¾ in.; and the same distance also when fitted on the other way.

Experiments were carried on at different inclinations of plate with a velocity of 12 ft. per sec. or 8.2 miles per hour. When attempting to bring the velocity up to 20 ft. per sec. or 13.6 miles per hour, the thread carrying the weight broke. These experiments are recorded under dates of July 24, 25, 31 and Aug. 11. The results obtained were tabulated and the well known formula

$$P_n = P\ (\sin x)^{1.84 \cos x - 1}$$

was deduced. This is sometimes called Unwin's formula, though for what reason is not clear, as Prof. Unwin simply quotes Prof. Hutton's formula approvingly.

The Duchemin formula

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A}$$

for inclined surfaces may be said to represent the best knowledge on the subject and is considered the most reliable formula in use. The pressures obtained are greater than those from the Hutton formula. Col. Duchemin, a French army officer, made his investigations in 1829 and the results were published in 1842 (Bixby).* Considerable weight has been attached to the work of Col. Duchemin. Weisbach quotes it, as well as most writers since his time. The Duchemin formula was verified by S. P. Langley in 1888. He had erected at the Allegheny (Penn.) Observatory a whirling-table consisting of two symmetrical wooden arms, each 30 ft. long, revolving in a plane 8 ft. above the ground. The motion thus obtained was nearly rectilinear, quite in contrast with that from Hutton's machine of less than 5-ft. radius. He also used velocities up to 100 ft. per sec., or nearly 70 miles per hour. He writes:

> At the inception of the experiments with this apparatus it was recognized that the Newtonian law, which made the pressure on an inclined surface proportional to the square of the sine of the angle, was widely erroneous. Occasional experiments have been made since the time of Newton to ascertain the ratio of the pressure upon a plane inclined at various angles to that upon a normal plane, but the published

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*The writer, while obtaining his information first hand from the sources quoted, acknowledges an obligation to a valuable report: Appendix C of the Report of Sept. 29, 1894, of the Special Army Engineer Board as to the Maximum Span Practicable for Suspension Bridges. By W. H. Bixby, Captain (now General) of Engineers, U. S. A. It is really a treatise on wind pressure in engineering construction. It is said only 500 copies were issued. This valuable paper may be found reprinted entire in "Engineering News," Mar. 14, 1895.

experiments exhibit extremely wide discordance, and a series of experiments upon this problem seemed therefore, to be necessary before taking up some newer lines of inquiry.

It is remarkable that Langley obtained results varying less than 3% from those derived from the Duchemin formula. Regarding this he writes:

Only since making these experiments my attention has been called to a close agreement of my curve with the formula of Duchemin, whose valuable memoir published by the French War Department, "Memorial de l'Artillerie" No. V, I regret not knowing earlier.

Attention is called to the monographs by Langley, *Experiments in Aërodynamics* and *The Internal Work of the Wind,* being Numbers 801 and 884 of the "Smithsonian Contributions to Knowledge."

Wind Pressure on Nonplanar Surfaces

When the wind blows on nonplanar surfaces the pressure on the projected area depends upon the form of the surface. This is important in the case of the cylinder (standpipes, chimneys and similar objects). Rankine states in his *Applied Mechanics,* "The total pressure of the wind against the side of a cylinder is about one-half of the total pressure against a diametral plane of that cylinder." A theoretical value of two-thirds is found in some treatises, but in engineering practice one-half is generally used.

Goodman in his *Mechanics Applied to Engineering,* London, 1904, gives the following ratios of pressure:

Flat plate	1.0
Sphere	0.36 to 0.41
Elongated projectile	0.5
Cylinder	0.54 to 0.57
Wedge (base to wind)	0.8 to 0.97
Wedge (edge to wind) Vertex angle 90°	0.6 to 0.7
Cone (base to wind)	0.95
Cone (apex to wind)	
Vertex angle 90°	0.69 to 0.72
Vertex angle 60°	0.54
Lattice girders about	0.8

Wind Pressure on Parallel Plates

The pressures upon parallel plates or bars with an open space between them are important in application to plate-girder bridges, the trusses in a truss bridge, or parallel bars in the same truss when one bar is behind another.

The Committee of the National Physical Laboratory, England, having decided that one of the first researches to be undertaken in the Engineering Laboratory should be the investigation of the distribution and intensity of the pressure of wind on structures, an elaborate series of experiments was conducted by Thomas Edward Stanton and the results embodied in two papers contributed by him to the Institution of Civil Engineers: "On the Resistance of Plane Surfaces in a Uniform Current of Air" and "Experiments on Wind Pressure." For circular plates 2 in. in diameter at 1½ diameters apart, he found the value of the total pressure was less than 75% of the resistance on a single plate; at 2.15 diameters apart the total pressure was equal to that on a single plate; while at a distance of 5 diameters apart the total pressure was 1.78 times that on a single plate. Stanton's first experiments were criticized because they were conducted with such small models. For his second series he built a tower and used larger surfaces, but found little to change his previous conclusions.

Baker's experiments at the Forth Bridge led him to the conclusion that in no case was the area affected by the wind in any girder which had two or more surfaces exposed more than 1.8 times the area of the surface directly fronting the wind. The Board of Trade regulations under which the Forth Bridge was built required that a wind pressure of 56 lb. per sq.ft. should be used in calculations, and this twice over the area of the girder surface exposed.*

Measuring Wind Pressure and Velocity

It has been assumed by experimenters that the pressure of the wind on a given shape with a certain velocity is the same as that of the shape moving through the air with an equal velocity. This seems to follow from Newton's Corollary V to his Laws of Motion, "The motions of bodies included in a given space are the same among themselves, whether that space is at rest or moves uniformly forward in a right line without any circular motion."

*Engineers regard the requirement of 56 lb. as needless and excessive.

Perhaps the only dissonant voice is that of T. Claxton Fidler, who in his *Bridge Construction* writes: "But it has not yet been ascertained that the pressure of the wind is the same thing as the resistance offered by the air to a moving body."

The pressure of the wind has been measured direct and independently of the velocity. The methods of doing this are so limited in their application that the pressure is almost universally determined in terms of the velocity. Hence, the prime importance of measuring the velocity of the wind correctly. Attempts to do this have been made by all manner of means for the past two centuries. The science of Anemometry has a literature of its own. The velocities obtained by all methods are more or less in error—some of them very much so. At present the Robinson Cup Anemometer or some modification of it is used pretty generally throughout the meteorological world for measuring wind velocities.

In the *Transactions* of the Royal Irish Academy, Vol. XXII, part III (1852), is a paper: "Description of an Improved Anemometer for Registering the Direction of the Wind, and the Space Which it Traverses in Given Intervals of Time. By the Rev. T[homas] R[odney] Robinson, D.D., Member of the Royal Irish Academy, and of other Scientific Societies. Read June 10, 1850." Dr. Robinson, who was connected with the observatory at Armagh, Ireland, writes:

> After some preliminary experiments I constructed in 1843 the essential parts of the machine, a description of which I now submit to the Academy, and I added in subsequent years such improvements as were indicated by experience. It was complete in 1846, when I described it to the British Association at Southampton.

He found "from sixteen experiments made in four days with winds from a moderate breeze to a hard gale,

$$\frac{a}{a'} = 4.011$$

or, in round numbers, the action on the concave is four times that on the convex." From this he found the theoretic value m of the ratio of the velocity of the wind to that of the cup center to be $m = 3.00$. Dr. Robinson concluded that no matter what the size of the cups or the lengths of the arms, "the centers of the hemisphere move with one-third of the wind's velocity, except so far as they are retarded by friction." This has been disproved. As a necessary result, many published velocities are in error.

The U. S. Weather Bureau prescribes that each pattern of anemometer should have its particular law of rotation determined by special experiment. Its standard instruments in use throughout the United States have hemispherical cups 4 in. in diameter on arms 6.72 in. long from the axis to the center of the cups. To the observed velocity the correction Log. $V = 0.509 + 0.912$ Log. v is applied in which V is the actual velocity of the wind and v is the linear velocity of the cup centers, both expressed in miles per hour.

Effect of Variations within the Wind

Measurements of either wind velocity or wind pressure are complicated enormously by the variations in the wind. This is illustrated by two observed facts, both of which are vitally important to the structural engineer:

1. Wind pressures are less per unit of area for large surfaces than for small ones. On the Forth Bridge two pressure boards were set up, one 20 ft. long by 15 ft. high, and 8 ft. from it a circular plate of 1½ sq.ft. area. The maximum pressure registered on the small plate during the years 1884 to 1890 was 41 lb. per sq.ft. The large board showed at the same time a pressure of 27 lb. per sq.ft. The readings for the large board never exceeded 80% of those recorded for the small plate at the same time, and generally were 50 to 70%. A technical journal of the time hastily drew the inference from these experiments that pressure per square foot varies inversely as area, the velocity remaining the same—another illustration of generalizing from insufficient data!

2. Wind velocity increases with the distance from the ground. Thomas Stephenson from his experiments writes the equation

$$V = V_1 \frac{\sqrt{H}}{\sqrt{H_1}}$$

or

$$V^2 : V_1^2 :: H : H_1$$

A limiting unit of height must be established for this equation to be of any use. An anemometer placed at the top of the Eiffel Tower, an elevation of 994 ft., and another in the meteorological office at an elevation of 69 ft., showed for light winds velocities nearly four times as great at the top of the tower as at the office. For higher winds the velocities came nearer together.

Conclusion

Cavallo, previously quoted, wrote, "a great many more experiments must be instituted by scientific persons before the subject can be sufficiently elucidated." More than a hundred years after Cavallo's writing, the U. S. Weather Bureau in its monograph on *Anemometry,* after giving values for pressures and velocities with all the refinements at its command, says:

> Great dependence cannot be placed in these values for indicated velocities beyond 50 or 60 miles per hour, as thus far direct experiments have not been made at the higher velocities, though it is probable the corrected values are throughout much more accurate than values computed from older formulas and uncorrected wind velocities.

Structures have long been designed with satisfactory results to withstand wind pressure. The bracing at times may have been excessive, but in the absence of better knowledge on the subject, engineers cannot radically depart from present practice.

II

Wind Stresses in Steel Mill-Buildings

SYNOPSIS—Discusses the distribution of wind pressure on a sloping roof, referring to the experiments of Irminger, Kernot, Stanton, Smith and others. Analyses of stresses in Fink roof trusses show that a uniform vertical excess load is sufficient to take care of wind stresses if rigid members are used. In kneebraced mill-building bents, wind corrections are necessary. Suction effects are to be neglected except as regards anchorage. Recommends wind pressures and unit stresses, and discusses special bracing.

In designing ordinary mill-buildings it is common practice either (1) to neglect the wind stresses or (2) to calculate them in accordance with some textbook method and then tone down the results. In doing the latter, the general practice of designing buildings is followed, in conformity to which structures have been built that have rendered excellent service for many years. To bridge the gap between theory and practice, recourse is being had by some to what might be called a new school, which has advanced new methods and new experimental results. In the present article this school will be briefly reviewed, its conclusions negatived, and textbook assumptions made to agree as near as possible with actual conditions—the object being to present a safe, sane, workable method of determining and making provision for the wind stresses in steel mill-buildings.

Amount and Distribution of Wind Stresses

A recent writer[1] of the new school states the case thus:

In a high wind the maximum pressure against the roof is at the windward eaves. The pressure decreases upward on the windward slope, and is zero, it is claimed, at a point

[1]"Insurance Engineering," August, 1912.

three-fourths the distance to the ridge. Beyond the zero point, up to the ridge and down the leeward slope, the pressure is negative. The wind deflected upward by the windward surface of the roof rarefies the air over the leeward surface, which allows the air inside the building to exert an upward pressure in excess of the downward pressure on the roof. In other words, there is direct or inward pressure on the windward slope of the roof, center of pressure below middle of slope, and at ridge and on all of leeward slope, there is outward pressure or suction.

Suction on Roof

In 1894, J. O. V. Irminger, manager of the Copenhagen Gas Works, made a number of experiments on wind pressure, the description and results of which he embodied in a paper[2] to which reference is often made. A rectangular opening about 6½x11 in. was made in a chimney 5 ft. in diameter and 100 ft. high. Into this opening was inserted a conduit 4½x9 in., polished on the inside to reduce friction. Currents of air were made to strike plates and models placed in this conduit and the resultant pressure registered. A model of a pitched roof with 45° slopes showed a normal uplift on the leeward side due to suction three times as great as the normal pressure on the windward side. The conclusion drawn was "if the author's experiments on models represent the facts with regard to buildings, the methods with which roof principals are commonly calculated for wind-pressure need revision." An enthusiastic admirer of Irminger writes,[3] "It will be due to him that we surely in the future shall save tons of material in our roofs."

In 1891-94, Prof. W. C. Kernot, of the University of Melbourne, made the experiments connected with his name.[4] By means of a gas engine and propeller, he discharged a jet of air 12 in. by 10 in., placing into this jet the plates and models he wished to test. He concluded that the usual method of calculating wind stresses in roofs applied only to roofs supported by columns under which the air could blow freely. With roofs of a low

[2]"Engineering News," Feb. 14, 1895; "Engineering," Dec. 7, 1895; Proc. Inst. Civ. Engrs., Vol. CXVIII, p. 468.

[3]Theodore Nielsen, "Engineering," Oct. 9, 1903.

[4]"Engineering Record," Feb. 10, 1894; Proc. Inst. Civ. Engrs., Vol. CLXXI, p. 218; Australian Association for the Advancement of Science, Vol. V (1893), p. 573, Vol. VI (1895), p. 741.

pitch resting on walls having parapets, he found a tendency to an uplift.

In 1893 and later, T. E. Stanton, of the National Physical Laboratory, England, made the experiments which have become widely known from the papers he contributed to the Institution of Civil Engineers.[5] From observations on models of roofs the sides of which were 3 in. by 1 in. and sloped at 30°, 45° and 60°, placed in a current of air having velocities of 10.0, 13.6 and 16.8 miles per hour, he writes, "The experiments appear to indicate beyond question the importance of a consideration of a negative pressure on the leeward side of roofs." From later experiments on pressure boards 5x5 ft. to 10x10 ft., he found the coefficients of wind pressure to be as follows:

STANTON'S COEFFICIENTS k IN FORMULA $P_n = kV^2$

(a) Roof mounted on columns through which air can pass

	60°	45°	30°
Windward side	+0.0034	+0.0028	+0.0015
Leeward side		negligible	

(b) Roofs of buildings in which the pressure on the interior may be affected by the wind.

	60°	45°	30°
Windward side	+0.0034	+0.0028	+0.0015
Leeward side	—0.0032	—......	—0.0022

This coefficient gives the normal pressure on roof surface in lb. per sq.ft., if V is the wind velocity in miles per hour, the wind blowing horizontal.

Prof. Albert Smith in a paper read before the Western Society of Engineers, November, 1910, entitled "Wind Loads on Mill Building Bents,"[6] among his conclusions advocates "placing the wind loads equally on the two walls, and inward and outward on the windward and leeward roofs respectively, as giving important changes of stress in members of the roof truss, as giving less stress in the kneebraces and columns, and as permitting the rational design of the girts." In 1912, he made a number of observations on a model building 6 ft. wide by 15 ft. long, with wall heights of 4, 5 and 6 ft. In a paper "Wind Pressure on Buildings,"[7] he writes:

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[5]Proc. Inst. Civ. Engrs., Vol. CLVI, p. 78, Vol. CLXXI, p. 175.
[6]Journal Western Soc. Engrs., February, 1911.
[7]Journal Western Soc. Engrs., December, 1912.

> The ordinary methods of assuming wind loads on mill buildings ought to be somewhat revised. For the case of roof trusses on masonry walls, or on steel bents with long diagonals, a suction effect in the neighborhood of 0.4 of the unit wind pressure should be placed on the leeward roof of all closed buildings, and a pressure or suction derived from the curves drawn from the observations placed on the windward roof. The resulting stresses will not only be different in amount from those computed on the present basis, but will in many members differ as to sign. Wind loads on purlins might in most cases be entirely omitted. * * * * In buildings with kneebraced bents, in addition to the preceding points, the suctions on the leeward wall should be considered.

Prof. Boardman, University of Nevada, in 1911 made experiments on a model roof 10 ft. long, each slope 6 ft. wide, resting on walls 4 ft. high. His conclusions are similar to those of Prof. Smith.[8]

An English textbook, Brightmore's *Structural Engineering,* first issued in 1908, quotes the Stanton experiments as authority and the stress diagrams for the roof truss given are made with the wind forces so acting. The heading of the section is significant: "Stresses Due to Wind Pressure and Wind Suction."

Another English textbook, Andrews' *The Theory and Design of Structures,* in an appendix to the last edition, 1913, calls attention to Stanton's conclusions and gives a stress diagram for a truss with the wind loading in accordance with these conclusions. In mentioning the stresses due to suction on the leeward side the author writes, "Few designers appear to have allowed for this in their calculations for roofs, but the question is of considerable importance and the results of these experiments should either be disproved, or allowance should be made for them in design."

Marburg in *Framed Structures and Girders* alludes to the experiments of Kernot, Irminger and Stanton and reproduces one of the Irminger sketches. His practical conclusion is:

> The experiments of Kernot, Irminger and Stanton were made on much too small a scale to admit of quantitative deductions applicable to conditions in practice. They are valuably suggestive, however, in calling attention to conditions which were previously not generally or adequately recognized.

With this conclusion the writer is in thorough accord.

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[8]Journal Western Soc. Engrs., April, 1912.

Selecting a Wind Pressure for Design

Our knowledge of wind pressures is very imperfect. It is generally agreed that the fundamental equation $P = kV^2$ is correct for horizontal wind. There is little dispute that for wind with a uniform velocity and normal to plates of moderate size, the value of k is from 0.0032 to 0.004. Of the formulas for wind pressure on inclined surfaces our best knowledge indicates that of Duchemin as the most accurate. It is

$$P_n = P\frac{2 \sin A}{1 + \sin^2 A}$$

There remains to be assigned a value to V. Average wind velocities for a day or a month or a year are useless. Shall the highest wind velocity on record be taken? Is this likely to occur again?

It is useless to attempt to make provision for tornadoes or violent hurricanes "against which neither care, nor strength, nor wisdom, can avail."* Such storms are limited in area and come but seldom, perhaps once in a century. The endeavor to make a mill-building strong enough to resist them would not only add greatly to the cost but would be ineffective.†

The highest wind velocity recorded in New York City since 1871 by the U. S. Weather Bureau was 96 miles per hour sustained for a period of five minutes. During one minute of that time the velocity was 120 miles per hour. This was in Feb. 22, 1912, a Robinson anemometer being used in the same location as at present, about 20 ft. above the roof of the 33-story building at 17 Battery Place. A recorded velocity of 80 to 90 miles is not uncommon. A recorded velocity of 90 miles per hour corrected by the Weather Bureau formula gives an actual velocity of 69.2 miles per hour. With this value in the formula $P = kV^2$, $k = 0.004$, the normal pressure is 19.2 lb. per square foot on a vertical surface. Wind pres-

*This is the way it is stated in the Foreword of a little volume issued by the Home Insurance Co., of New York, advocating windstorm and tornado insurance. More than a hundred photographs in this volume of wrecks caused by windstorms illustrate the truth of the Foreword.

†Editor's Note—This argument in its terms applies just as much to office buildings and all other structures as it does to mill buildings. The author probably means to emphasize the cost limitation, for mill-buildings alone.

sure increases with the distance from the ground and decreases per square foot as the area becomes larger. When it is remembered that the instrument above mentioned is about 400 ft. from the ground and the cups are only 4 in. in diameter, the assumptions that will be made of a horizontal wind force of 20 lb. per sq.ft. in designing the trusses of mill-buildings, and 15 lb. in designing columns and kneebraces, seem to be ample and fully warranted.

Wind Stresses in Roof Trusses

Roof trusses resting on brick walls will first be considered. The example taken will be a roof truss as in Fig. 1, with pitch of 6 in. to 1 ft., and span of 60 ft. c. to c. of bearing plates. Trusses are 16 ft. apart on centers. For a horizontal wind from the left, with pressure of 20 lb. per sq.ft. on a vertical surface, the normal pressure on a surface inclined 6 in. to 1 ft. will be (by Duchemin's formula) 14.9 lb. per sq.ft.

The following cases will be considered:

(1) Wind load of 15 lb. per sq.ft. or 2012 lb. per panel, normal to one slope of roof, both ends of truss fixed.

(2) Loads as in (1), left end fixed, right end on rollers.

(3) Loads as in (1), left end rollers, right end fixed.

(4) Load of 15 lb. per sq.ft. exposed surface or 2012 lb. per panel on both sides of the roof, the loads applied vertically.

The stresses for these four cases are tabulated below:

It is seen at a glance that Case 4 is sufficient to cover wind stresses. The slight excess in a few members found in the other cases is negligible, especially when they are considered with the combined stresses due to all loads.

With the same wind velocity as before, according to Stanton, the pressure on the windward slope is about $7\frac{1}{2}$ lb. per sq.ft. and $^{22}/_{15}$ times $7\frac{1}{2}$ or 11 lb. per square foot negative pressure or suction on the leeward side. The forces acting upon the truss are as in Fig. 2 (reactions are for both ends fixed, wind shear equally divided), while Fig. 3 is the stress diagram for both ends fixed.

The tabulation of stresses given below is for

(5) Both ends of truss fixed.

(6) Left end fixed, right end on rollers.

(7) Left end rollers, right end fixed.

It might be stated here that all the above cases with one end on rollers are hypothetical, as roof trusses under 100-ft. span are seldom built with other than fixed ends.*

RECOMMENDED DESIGN LOAD—Maximum wind loading comes seldom and lasts but a short time. The working stresses used for this loading may therefore be increased 50% above those used for ordinary live- and dead-loads. A wind load of 15 lb. per sq.ft. is thus equivalent to a load of 10 lb. using the working stresses for other loads.

The snow load varies from 20 lb. per sq.ft. horizontal projection in the latitude of New York City to 30 lb. in parts of New England. This is equivalent to 16.6 up to 25 lb. vertical load per sq.ft. surface of a 6-in. pitch roof.

For combined snow and wind a load of 25 lb. per sq.ft. over entire surface, acting vertically, is ample for roofs in the latitude of New York City. If to this is added the weight of trusses, purlins, and roof covering, reduced to square foot of exposed surface, we have the total load for which the ordinary roof truss should be designed. However, not less than 40 lb. should be used except in tropical climates with no snow, where the minimum loading should be 30 lb. Where snows are severe 5 to 10 lb. should be added to the 40 lb.

NO ALLOWANCE FOR SUCTION—Turning to the tabulation of stresses found by the Stanton assumptions, and taking into account the total stresses from all loads, the saving due to reduced wind stresses is small. A serious objection to taking advantage of even this saving is that with a monitor along the ridge, or openings in the building and roof, the closed roof may become a partly open roof, thus changing the conditions for which the assump-

*Editor's Note—The wind shear may, however, come wholly on one or the other wall, due to unequal bedding of the anchor bolts or to temperature movement. The condition then, as regards the present calculation, is identical with one end on rollers.

tions were made. For a truss resting on brick walls the tendency to an uplift can be met by firmly anchoring it at the ends. The tendency to reversals of stress can be sufficiently met by using stiff shapes for all members; flats and rounds have no place in an ordinary roof truss.

The writer believes that the assumption of a total uniform load per square foot of exposed surface applied vertically at the panel points, with the same working stresses used throughout, is specially well adapted to the design of roof trusses.

Wind Stresses in Kneebraced Bents

Kneebraced Bents—The case of an intermediate transverse bent of a kneebraced mill-building will now be considered. The example taken will be that shown in Fig. 4; span 60 ft., roof pitch 6 in. to 1 ft., height 14 ft. to foot of kneebrace and 20 ft. to bottom chord. Trusses are 16 ft. apart c. to c.

The wind pressure will be taken at 15 lb. per sq.ft. perpendicular to the sides of the building and the corresponding normal component on the roof at 11.2 lb. (For buildings over 25 ft. to the eave line the normal component of a wind load of 20 lb., or 14.9 lb., would be used for the roof.) The columns are assumed partially fixed at the lower end, with the point of contraflexure at one-third the distance between the lower end and the foot of the kneebrace; the upper ends are considered supported. The wind shear is divided equally between the two columns. Fig. 5 is the stress diagram.

Bending moments in the columns are as follows:

At the foot of windward column	12,320 ft.-lb.
At foot of leeward column	14,940 ft.-lb.
At foot of windward kneebrace	19,410 ft.-lb.
At foot of leeward kneebrace	29,870 ft.-lb.

It is seen that the maximum bending moment is at the foot of the leeward kneebrace.

Recommended Method—For mill-building bents the writer first determines the stresses in the truss due to a total uniform load and then proportions it for the same. The ordinary working values for medium steel are gen-

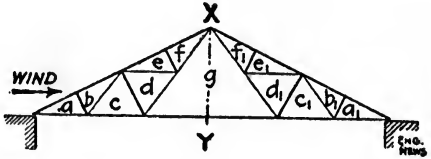


FIG. 1

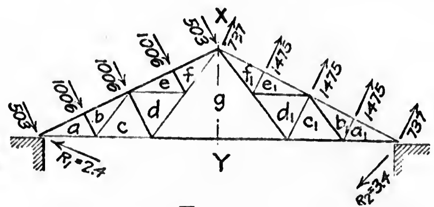


FIG. 2

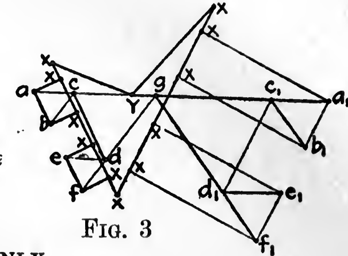


FIG. 3

STRESSES IN ROOF TRUSS FIG. 1, PRESSURE ONLY

Member Case	Xa	Xb	Xe	Xf	Ya	Yc	Yg	ab	bc	cd	de	ef	dg	fg
1	—9.1	—9.1	—9.1	—9.1	+10.2	+8.0	+3.5	—2.0	+2.3	—4.1	+2.3	—2.0	+4.5	+6.8
2	—9.1	—9.1	—9.1	—9.1	+11.4	+9.1	+4.6	—2.0	+2.3	—4.1	+2.3	—2.0	+4.5	+6.8
3	—9.1	—9.1	—9.1	—9.1	+7.7	+5.5	+1.0	—2.0	+2.3	—4.1	+2.3	—2.0	+4.5	+6.8
4	—15.8	—14.9	—14.0	—13.1	+14.1	+12.1	+8.1	—1.8	+2.0	—3.6	+2.0	—1.8	+4.1	+6.1

Member Case	Xa_1	Xb_1	Xe_1	Xf_1	Ya_1	Yc_1	a_1b_1	b_1c_1	c_1d_1	d_1e_1	e_1f_1	d_1g	f_1g
1	—5.1	—5.1	—5.1	—5.1	+3.5	+3.5	0	0	0	0	0	0	0
2	—5.1	—5.1	—5.1	—5.1	+4.6	+4.6	0	0	0	0	0	0	0
3	—5.1	—5.1	—5.1	—5.1	+1.0	+1.0	0	0	0	0	0	0	0
4	—15.8	—14.9	—14.0	—13.1	+14.1	+12.1	—1.8	+2.0	—3.6	+2.0	—1.8	+4.1	+6.1

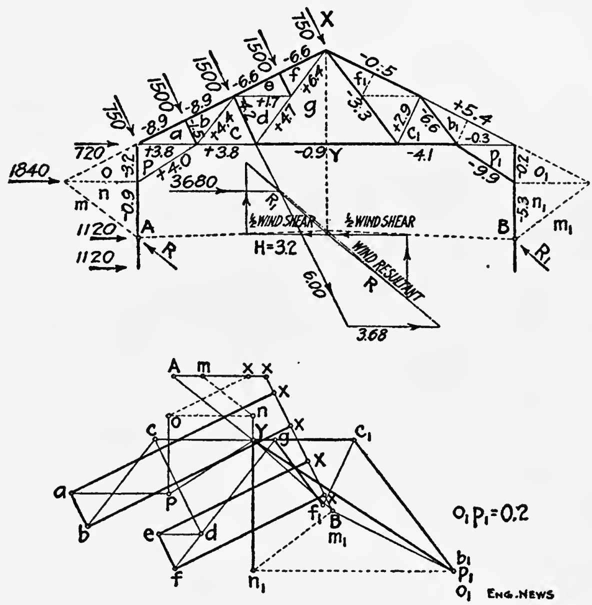
Stresses are given in thousands of pounds. — Denotes compression. + Denotes tension

STRESSES IN ROOF TRUSS FIG. 2, PRESSURE AND SUCTION

Member Case	Xa	Xb	Xe	Xf	Ya	Yc	Yg	ab	bc	cd	de	ef	dg	fg
5	—0.8	—0.8	—0.8	—0.8	+2.7	+1.6	—0.7	—1.0	+1.1	—2.0	+1.1	—1.0	+2.2	+3.4
6	—0.8	—0.8	—0.8	—0.8	+4.9	+3.8	+1.6	—1.0	+1.1	—2.0	+1.1	—1.0	+2.2	+3.4
7	—0.8	—0.8	—0.8	—0.8	+0.5	—0.6	—2.9	—1.0	+1.1	—2.0	+1.1	—1.0	+2.2	+3.4

Member Case	Xa_1	Xb_1	Xe_1	Xf_1	Ya_1	Yc_1	a_1b_1	b_1c_1	c_1d_1	d_1e_1	e_1f_1	d_1g	f_1g
5	+4.1	+4.1	+4.1	+4.1	—5.6	—3.9	+1.5	—1.7	+2.9	—1.7	+1.5	—3.3	—5.0
6	+4.1	+4.1	+4.1	+4.1	—3.4	—1.8	+1.5	—1.7	+2.9	—1.7	+1.5	—3.3	—5.0
7	+4.1	+4.1	+4.1	+4.1	—7.8	—6.2	+1.5	—1.7	+2.9	—1.7	+1.5	—3.3	—5.0

Note—Stresses are given in thousands of pounds. — Denotes compression. + Denotes tension erally used—16,000 lb. per sq.in. in net tension and (reduced by formula) for gross compression. If the wind stresses in any member from Figs. 4 and 5 are greater than the wind stresses from the uniform wind loading of 10 lb. per sq.ft. applied vertically, that member is proportioned for the maximum wind stress plus the stresses from the uniform loads other than wind, using working stresses 50% more than in the first calculation; but in no case is a less section used than that first obtained. The



FIGS. 4 AND 5. KNEEBRACED BENT OF A MILL-BUILDING; STRESS DIAGRAM FOR WIND LOAD

(Equal shears; contraflexure one-third height to kneebrace)

members bc and b_1c_1 will generally need be increased; often cd and c_1d_1; occasionally gd, gf, gf_1 and g_1d_1. A reversal of stresses is noted in certain members, particularly in bc and the lower chord. The diagonals bc and b_1c_1 can be made of two angles instead of one as when designed for tension alone. The compressive stresses in the lower chord are overcome by the tensile stresses due to the dead-load. The kneebraces have wind stresses only, and are proportioned for the larger working stresses.

The column is first proportioned for carrying the direct stress due to the total uniform load from the truss, noting the flange area required. From the maximum bending moment due to the wind, as in Fig. 4 and 5, the sectional area required for the flanges is found using the larger working stresses and considering the column as a beam. If this flange area is not more than one-third of that first found no change is made; if more than one-third, the excess is added. The compressive stress due to overturning need not be considered unless it exceeds the stress from the wind portion of the uniform roof load.

Girts—The side and end girts are proportioned as beams supported at each end with a uniform horizontal load of 15 lb. per sq.ft.; an extreme fiber stress of 24,000 lb. per sq.in. is used.

The girts are apparently the simplest portion of the building to design, but if observation and experience count they are the most difficult. Side and end girts of $3\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$-in. or $\frac{5}{16}$-in. angles, 16 to 20 ft. long and 5 ft. apart, are still in use after 20 years' service, in defiance of all figures. Notwithstanding this, such designing practice is reprehensible.

Special Cases of Mill-Building Bracing

Traveling cranes running through a building often bar the use of kneebraces. The gussets connecting the trusses to the columns should then be as large as possible and calculations made accordingly. An ideal way is to transfer the transverse thrust from the wind as well as that from the cranes to the ends of the building by means of diagonal bracing in the plane of the lower chords of the trusses, and thence by diagonals in the ends of the building to the ground. Openings in the ends often interfere with this expedient. Neither is it feasible in buildings so long as to require provision for longitudinal expansion and contraction due to changes in temperature.

In all cases diagonal bracing should be introduced into the planes of both top and bottom chords for stiffness as well as to take calculated stresses. This applies also to roof trusses resting on brick walls. Adjustable rods can be used for top-chord bracing, but the bracing of the bottom chord should be entirely of angles or other rigid shapes, with bolted or riveted connections.

The ends of a building, the gables in particular, are more liable to be severely strained from wind than any other portion of the building. Generally diagonals in the planes of the chords of each end panel, and in each end side bay to the ground will be sufficient to take care of the induced stresses. If not, the shear may be divided with other braced bays.

Special types of buildings should be considered in reference to their own requirements. Open sheds, especially if the gables are closed, may have an uplift as well as a vertical load.

Comments on Prior Recommendations

The leading textbook on the subject of mill buildings is Ketchum's *Steel Mill-Buildings.* In this book a knee-braced mill bent is considered for four cases:

(1) A horizontal wind load of 20 lb. per sq.ft. on the side and vertical projection of the roof, with the columns hinged at the base.

(2) Same wind load as in Case 1, with columns fixed at the base.

(3) A horizontal wind load of 20 lb. per sq.ft. on the side, and the normal component of a horizontal wind load of 30 lb. per sq.ft. on the roof, with columns hinged at the base.

(4) Same wind load as in Case 3, with columns fixed at the base.

The writer believes that the loads of 20 and 30 lb. are larger than need be used. The columns are seldom if ever rigidly fixed at the base, neither are they hinged. That they are partially fixed and the point of contraflexure is at one-third the distance from base to foot of kneebrace is believed to be nearer correct than either assuming it one-half the distance or assuming the columns supported at the base. There seems no good reason for assuming a normal component of a horizontal wind load of 30 lb. on the roof while a horizontal wind load of 20 lb. is taken on the sides. It is not clear why the Hutton formula is used to find the normal component. Near the beginning of the book we read, "Hutton's formula is based on experiments which were very crude and probably erroneous. Duchemin's formula is based on very careful experiments and is now considered the most reliable formula in use." The specifications near the close of the book call for the Duchemin formula to be used in computing the normal wind pressure; by this formula, for a 30-lb. load, the 18-lb. normal pressure in Cases 3 and 4 would be 22.4 lb. The only increase of the usual working stresses allowed is 25% for laterals and 50% for combined direct and flexural stress due to wind. This increase does not apply to the combination of wind with other loads though with the maximum wind load a minimum snow load of 10 lb. per sq.ft. is allowed. (For the purpose of aiding those who wish to make comparisons the same roof truss and bent have been taken in this article as found in Ketchum.)

The chapter on "Iron and Steel Mill-Building Construction" by G. H. Hutchinson in Johnson, Bryan & Turneaure's *Modern Framed Structures*, considers three cases of a mill-building bent, arriving at conclusions similar to those of Ketchum. The method of obtaining reactions and moments is quite abstruse and difficult to follow. No mention is made in the chapter of working stresses to be used.

Smith, in "Wind Loads on Mill-Building Bents,"[6] assumes a total horizontal wind force of 30 lb. per sq.ft. on the bent considered in this article. The pressure on the sides is divided equally between the two columns. One-third of the normal component of the total pressure on the roof, found by Duchemin's formula, is taken by the windward slope and two-thirds as suction or an outward pressure on the leeward slope. The bases of the columns are considered hinged. Comparing his results with those obtained by Ketchum, he shows reduced stresses but is unfortunate in the example selected for comparison: Ketchum's stresses are taken and 50% is added to them for a 30-lb. load; but Ketchum calculates his roof for the normal component of a horizontal wind of 30 lb. and the side for 20 lb., so that Smith is actually comparing his results with those obtained from a bent having a pressure on the sides of 30 lb. per sq.ft. and on the roof the normal component (by the Hutton formula) of a horizontal force of 45 lb. per sq.ft.

However, with the same wind force, the Smith method does give reduced stresses, especially in the columns, kneebraces and girts. The important point is whether these reductions are permissible. In a modern mill-building the sides are from one-fourth to one-half or more of glass, a large proportion of which can be opened to permit of ventilation. If opened on one side only, Smith's assumption that the pressure on the inside of a mill-building is a mean between the windward pressure and the leeward suction disappears. In a high wind the windows on the leeward side are liable to be open and those on the windward side closed; there is then little suction. In a building the sides of which are covered with sheet metal there is always a probability of the covering on one side or end being removed for 8 ft. or more from the ground, thus completely doing away with the suction theory. For these reasons it is unwise to take advantage of a theory based upon assumptions which are destroyed by a probable change of conditions.

In conclusion, while the methods advocated for treating wind stresses may not be thoroughly scientific, *they are easily workable, and experience proves that they are safe and sane.* The load of 15 lb., the working stress of 24,000 lb., and the assumed point of contraflexure, may all be criticized, but for the ordinary mill-building it is more rational to use these assumptions and make strict provision for them than to follow the present method of giving an intellectual assent to the theories of the textbook and ignoring them in actual practice.

III

Wind Stresses in Railroad Bridges

SYNOPSIS—The Tay Bridge failure reviewed. English practice in the 70's ignored wind stresses, while American engineers used methods nearly equal to those of today. Empirical development is the basis of practice. Modern specifications show a great number of variations in detail, but may be brought nearer uniformity in the future. Lateral-oscillation forces should be specified separate from wind pressure.

The purpose of this article is to review past and present practice of the treatment of the wind forces acting on railroad bridges.

THE TAY BRIDGE

On the evening of Dec. 28, 1879, occurred the "Tay Bridge Disaster." During a violent gale, 11 spans of 245 ft. and two of 227 ft., with the train passing over them at the time, fell into the river. This failure of what was at the time the largest bridge in the world after a service of less than two years marked an epoch in bridge building in Great Britain. Wind stresses in railroad bridges had previously been almost neglected; from that time they have been fully considered, if not magnified. A Court of Inquiry was appointed by the English Board of Trade to report on the causes of the disaster. Today the testimony taken regarding the provision made in the design of the bridge for wind stresses is interesting reading.

Sir George Airy, Astronomer Royal, testified that about seven years previously he had been consulted on the subject of the provision which should be made for wind pressure on the plans prepared by Sir Thomas Bouch of a bridge of two spans of 1600 ft. over the Forth. He gave as his opinion that the greatest wind pressure that might be expected over the whole extent of such a surface was 10 lb. per sq.ft.

Sir Thomas Bouch, the designer of the Tay bridge, testified that he did not specially make any allowance for wind pressure, but he had seen the report on the Forth Bridge; he thought the greatest pressure would be about 10 lb.

The majority report of the Court of Inquiry ends:

In conclusion, we have to state that there is no requirement issued by the Board of Trade respecting wind pressure, and there does not appear to be any understood rule in the engineering profession regarding wind pressure in railway structures; and we therefore recommend that the Board of Trade should take such steps as may be necessary for the establishment of rules for that purpose.

A minority report was submitted by the third member of the Court.† An extract is:

It is said that Sir Thomas Bouch must be judged by the state of our knowledge of wind pressure when he designed and built the bridge. Be it so; yet he knew or might have known that at that time the engineers in France made an allowance of 55 lb. per sq.ft. for wind pressure, and in the United States an allowance of 50 lb.

In the engineering literature of 1880 and 1881, a paper often referred to is "The Tay Bridge," by Edgar Gilkes, a member of the firm building the bridge. It was read before the Cleveland Institution of Engineers, Nov. 6, 1876. The special paragraph that must have plagued the author reads:

A consideration of the action of the wind on this bridge will dissipate the often-advanced theory that at some period it will be blown over. The exposed surface of one large pier is about 800 sq.ft., and of the superstructure which depends upon it, about 800 more, and so, giving 800 ft. for a train

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†The majority of the Court of Inquiry reported that in their opinion the cross bracing at the pier and its fastening by lugs was the first part to yield. The evidence, however, was ample to justify the language of the minority report, "that this bridge was badly designed, badly constructed, and badly maintained, and that its downfall was due to inherent defects in the structure which must sooner or later have brought it down."

The piers which failed carried two adjoining trusses and consisted of a hexagonal group of six cast-iron columns filled with portland cement, two 18 in. and four 15 in. in diameter. Each column in case of the higher columns was made of seven lengths 10 ft. 10 in. long, united by flanges. These columns were braced vertically on the exterior sides by struts of two 6-in. channels and diagonals of 4½x½-in. flats fastened to cast-iron lugs with 1⅛-in. bolts. Horizontal bracing connected the four interior columns. It will readily be seen that such a system is weak throughout.

As a matter of interest it may be noted that during the progress of construction on the night of Feb. 2, 1877, two of the 245-ft. spans were blown into the river. To this accident "Engineering" of Feb. 9, 1877, devoted just 14 lines.

above, we have 2400 ft. Twenty-one pounds per sq.ft. is the force of a very strong gale, but it would take no less than 96 lb. per sq.ft. on the surface given to overturn the pier. Even the most severe hurricane on record would equal only one-half this resultant power.

C. Shaler Smith, after a careful calculation in accordance with American rules, found the exposed surface of the superstructure to be 2576 sq.ft. instead of 800, while the London *Engineering* shows the exposed train surface was 1630 sq.ft. instead of 800.

Early English Practice

It is surprising how little is found in the English technical books and papers of those days regarding the force of the wind on structures. Humber, in his voluminous "Complete Treatise on Cast- and Wrought-Iron Bridge Construction," published in 1861, does not mention it. Unwin's "Wrought-Iron Bridges and Roofs," 1869, was a far better textbook than any that had preceded it. What he writes regarding wind pressure on roofs is still quoted; yet he says nothing of wind stresses in bridges. The article on Bridges, by Prof. Jenkin, in the ninth edition of the "Encyclopedia Britannica" (1876), afterward issued as a separate treatise, fills 58 closely written pages, but not a line is found concerning wind bracing.

Thomas Cargill, in his "The Strains upon Bridge Girders and Roof Trusses," 1873, writes:

> No allowance is made in the theoretical calculation for the violent shock, concussion and consequent vibration that attend the passage of a heavy train over a bridge. This must be allowed for by experience, by the introduction of such additional bracing as the skill of the engineer suggests. These are points which cannot be learned from books.

To the effect of the wind on a bridge he makes no allusion, though in the chapter "Curved Roof Trusses," he says:

> Some writers lay great stress upon providing a large margin of strength for wind pressure, but there is more theoretical than practical knowledge displayed in such statements.

Rankine, in his "Manual of Civil Engineering," in the early 60's, gives a formula for the effect of wind on tubular girders, and in a footnote states that the greatest pressure of wind ever observed in Britain was 55 lb. per sq.ft., but we have no record that this observation ever entered into the consideration of bridge engineers.

The first German edition of Ritter's "Elementary Theory and Calculation of Iron Bridges and Roofs," was issued in 1862. An English translation was published in 1879. In this book the lateral force is taken as a percentage of the combined vertical live- and dead-loads. In one particular bridge under consideration, it is assumed to be one-seventh.

The outcome of the agitation following the fall of the Tay Bridge was a commission appointed by the Board of Trade to consider the question of wind pressure on railway structures. This committee made its report in 1881. The substance of its five recommendations was: (1) That for railway bridges and viaducts a maximum wind pressure of 56 lb. per sq.ft. should be assumed for the purpose of calculation. (2) That the area of exposed surface should be taken at once to twice the front surface, according to the extent of the openings in the trusses or lattice-girders. (3) That a factor of safety of 4 should be used for strains caused by wind pressure, and for the whole structure overturning as a mass a factor of safety of 2 should be used. These recommendations became law in Great Britain.

Early American Practice

Not less interesting is the historical development of wind bracing in the United States. In 1851 was published, "General Theory of Bridge Construction," by Herman Haupt, A.M., General Superintendent of the Pennsylvania R.R., formerly professor of Mathematics in Pennsylvania College. The pioneer book in which bridge trusses are correctly analyzed is "A Work on Bridge Building," by Squire Whipple, published at Utica, N. Y., in 1847. Whipple's book was little known for a ong time after its publication, while Haupt's book, written without any knowledge of the existence of Whipple's, soon became widely circulated and for years was regarded as an authority. In Part 1 of Haupt's book we read:

The use of lateral bracing is principally to guard against the effects of wind, and other disturbing causes, tending to produce lateral flexure in the roadway * * * The greatest

lateral strain is that produced by the action of a high wind; assuming the force of wind at 15 lb. per sq.ft., as a maximum, * * *

In Part II, written some time after Part I, we read, "The heaviest locomotives in use weigh about 23 tons, and their length is 23 ft.," and further on:

The greatest strain upon the lateral bracing of a bridge would be that caused by the action of the wind in a violent tornado. It is probable that this force is far greater than it is usually estimated. The observations of the writer at the Susquehanna Bridge, during the tornado which caused the loss of six of the unfinished spans, led him to believe that the direct effect of the storm was increased by reflection from the surface of the water. * * * If we suppose a storm could be so violent as to cause a pressure of 30 lb. per sq.ft., * * *

The tornado alluded to occurred Mar. 27, 1849. A viaduct across the Susquehanna River, near Harrisburg, was being built for the Pennsylvania R.R. It was supported on 22 piers, 160 ft. center to center. The trusses were of the Howe type, with the addition of wooden arches. After the fourteenth span had been raised, the storm came and carried off six spans. The contractor was busy at the time putting in the arches, and as the diagonal braces could not be fastened until after the arches were in place, they had been omitted except over the piers and in the middle of the spans. The wind came at right angles to the bridge and the six spans without lateral bracing gave way.

As late as the early 70's American textbooks had little or nothing to say on wind bracing. De Volson Wood devotes less than one-half a page to the subject in his "Theory on the Construction of Bridges," while Greene in "Bridge Trusses" spares only a page. Col. Merrill, in his "Iron-Truss Bridges for Railroads," makes no mention of wind bracing. Shreve, whose "Treatise on the Strength of Bridges and Roofs" was translated into French, finds vertical strains, horizontal strains, chord strains, brace strains, but the word wind does not occur in the book, nor is any mention made of bracing in a horizontal plane. Nearly the same might be written of Roebling's "Long and Short Span Railway Bridges," 1869.

American practice, however, was ahead of the teaching profession. C. Shaler Smith, on Dec. 15, 1880, presented a masterly paper to the American Society of Civil Engineers, entitled, "Wind Pressure upon Bridges." He gives specifications which he had used in constructing a number of bridges, some of them high and in exposed localities. He specifies:

> The portal, vertical and horizontal bracing shall be proportioned for a wind pressure of 30 lb. per sq.ft. on the surface of a train averaging 10 sq.ft. per lin.ft., and on twice the vertical surface of one truss. The 300 lb. pressure per lin.ft. due to the train surface shall be treated as a moving load, and the pressure on the trusses as a fixed load. Trusses of less than 200 ft. span shall also be proportioned for a pressure of 50 lb. per sq.ft. where unloaded, and the greatest strain by either method of computation shall in each case be used in determining the sectional area of the bracing.

Several leading railway companies at that time were using practically these specifications. From this same paper the following is significant:

> Many engineers prefer to express wind force in pounds per lineal foot of bridge instead of per square foot of exposed surface. Using a 200-ft. span as an example, the specifications in question can be condensed as follows:
>
> Fixed load in plane of roadway, 210 lb. per lin.ft.
> Fixed load in plane of other chord, 130 lb. per lin.ft.
> Moving load in plane of roadway, 300 lb. per lin.ft.

It is refreshing to see C. Shaler Smith quoted as the exponent of American practice for wind bracing in the article on Bridges, by Unwin, in the eleventh edition of the "Encyclopedia Britannica."

The rules in the Erie Specifications, formulated in 1878 by Theodore Cooper, were:

> To provide for wind strains and vibrations, the top lateral bracing in deck bridges and the bottom lateral bracing in through bridges shall be proportioned to resist a lateral force of 450 lb. for each foot of the span, 300 lb. of this to be treated as a moving load.
>
> The bottom lateral bracing in deck bridges and the top lateral bracing in through bridges shall be proportioned to resist a lateral force of 150 lb. for each foot of the span.

It is thus seen that in the early days of iron railway bridges, the American engineers were far in advance of their English brethren in the recognition of wind forces.

Factors in the Problem

"A Practical Treatise on Bridge Construction," by T. Claxton Fidler, was published in London in 1887. Chapter XXIV is "On Wind-Pressure and Wind-Bracing." In 1894, Captain (now General) Bixby, U. S. A., in a monograph reviewing the literature on wind pressure, writes: "The chapter is perhaps the best single, short, concise, comprehensive and practical review of the whole subject yet in print." This characterization in a large measure still holds true. Some concluding sentences from the chapter are:

> **We have seen, for example, how large a proportion of the metal in a long-span bridge is required for the purpose of resisting wind-pressure and for the purpose of carrying the metal that resists wind-pressure. But we have also seen that it is really impossible to estimate the wind stresses within 100% of their real value. * * * In this state of uncertainty, the responsible engineer will generally be disposed to err on the safe side; but it must be remarked that this will be a very expensive proceeding. * * * On the other hand, he knows that an error in the opposite direction might be attended with still more disastrous results.**

The sting of these sentences is in their truth. Our knowledge of the wind is uncertain, especially regarding the higher velocities. Although there are many unknown quantities in the problem of wind stresses in a bridge, the main questions to be considered are two:

(1) What is the pressure to be assumed per unit of area?

(2) What shall be taken as the area exposed to the action of the wind?

Wind pressure is generally measured in terms of the velocity. According to the best information we have, an indicated velocity by the (Weather-Bureau standard) anemometer of 100 miles per hour denotes an actual velocity of 76 miles, which is equivalent to a pressure of 23 lb. per sq.ft. on a surface at right angles to its direction. A pressure of 30 lb. per sq.ft., which corresponds to an indicated velocity of about 120 miles per hour, will overturn empty freight cars, the ordinary passenger car, and acting over an extended area of land would sweep from it all trees. No engine driver could take his train upon a bridge with such a pressure, though it is possible that the train during a sudden gust might be caught there. A man could not keep his feet with such a pressure, no matter at what angle his legs were inclined to the ground.

It would seem that 30 lb. per sq.ft. is ample for assumed wind pressure.

The second question to be considered is even more difficult to answer than the first. In a bridge composed of two or more trusses several feet apart, and each truss made up of members which may shelter other members, the case is far different from that of wind on a plate or on a solid body. Our actual knowledge of the subject is slight. Baker's experiments at the Forth Bridge* and Stanton's experiments at the National Physical Laboratory† are generally quoted. Bridge engineers and writers on the subject vary in their methods. C. Shaler Smith, as previously noted, uses an exposed area of "twice the vertical surface of one truss." In estimating the vertical surface of *one* truss he adds to the elevation of the upper chord and posts, as seen on the drawing, 1½ times the surface of the ties, and twice the surface of the lower chord.‡ Du Bois, in his "The Stresses in Framed Structures," gives as a rule this method for finding the area of surface of a single truss. "In preliminary estimates," he writes, "we may take the exposed surface for *both trusses* at 10 sq.ft. per lin.ft."

Johnson, Bryan and Turneaure, in "The Theory and Practice of Modern Framed Structures," use "the exposed surface of all trusses and the floor as seen in elevation." Merriman and Jacoby, in "A Text Book on Roofs and Bridges," Heller in "Stresses in Structures," and a number of other writers do the same. "Structural Engineering," an English textbook by Husband and Harby, says "The area of the bridge exposed to the higher pressure will be from once to about three times the area as seen in elevation, depending on the type of construction." Another English textbook, Anglin, "The Design of Structures," says, "In double-webbed lattice girders, the area of both webs should be taken, or double the web area as seen in elevation, ... If a bridge consists of two such main girders, the wind pressure must be taken as acting on an area equal to *four times* that as seen in

*"Engineering," Sept. 5, 1884.

†"Proc. of Inst. of C. E.," Vol. CLVI and Vol. CLXXI.

‡"Trans. Am. Soc. C. E.," Vol X, pp. 170. (Private letter to O. Chanute.)

elevation." This same textbook adds, "American engineers assume a wind pressure of 30 lb. per sq.ft. upon the loaded, and 50 lb. upon the unloaded structure."

From the foregoing it is readily seen that specifications that simply give the load per square foot of exposed surface to be used do not fully specify. Descriptions of bridges which state the lateral pressure per square foot used in the calculations without defining the extent of exposed surface intended by the designer are incomplete in their description.

It is to be remembered that there are stresses due to lateral forces other than the wind. A considerable lateral force is developed by a rapidly moving train, or the lurching of a locomotive when it first strikes a bridge. This lateral vibration appears to be much more accidental in its character than the vertical vibration.* Even were there no wind, rigidity would have to be maintained against this lateral vibration, which in short spans is probably a greater factor than the wind pressure itself. We have nothing to determine a relation between lateral vibration and wind strain.

Further, the compression chords of bridges must be held in alignment by the lateral bracing. The amount of material required to do this is not, with our present knowledge, a matter of exact calculation.†

In some specifications provision for all lateral forces, except the centrifugal force when the track is on a curve, is included in that for wind pressure without being so stated. In others, "for lateral forces" or "for wind loads and lateral vibration" are the words used and more clearly express what is intended.

Giving the lateral force in terms of pounds per lineal foot of bridge (rather than in pounds per square foot) has a decided advantage in the preparation of designs for competition, as all bidders are working upon the same basis. Theoretically, for the wind itself the pressure per square foot is to be preferred and the force that produces lateral vibration is best represented by a percentage

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*Robinson, "Vibration of Bridges." Trans. Amer. Soc. C. E., Vol. XVI, p. 42.

†Reichman, "Journal of the Western Soc. of Eng's," Vol. 29, p. 93.

of the moving load. As engine weights and car loads are increased, provision is thus made for the increased tendency to vibration. Again, a different lateral force for spans under 200 ft. from that for spans over 200 ft. is assumed by some engineers.

While the wind pressure on a moving train should be treated as a moving load, engineers are divided in their opinions as to the wind load on the structure itself; some considering it uniform and some moving.

In regard to end anchorage, the following will be quoted from Waddell's "De Pontibus": "No matter how great its weight may be, every ordinary fixed span should be anchored effectively to its support at each bearing on same." (Principle XXVI, in chapter "First Principles of Designing.")

In passing, a criticism will be launched at the English bridges of "an early Victorian type,"* having an arched portal strut at every post and no top laterals. Some of these are of late date. All are wasteful in material, and there is great ambiguity in regard to the lateral stresses.

Present Specifications

The specifications of the American Railway Engineering Association, 1910, read:

> All spans shall be designed for a lateral force on the loaded chord of 200 lb. per lin.ft. plus 10% of the specified train load on one track, and 200 lb. per lin.ft. on the unloaded chord; these forces being considered as moving.

The American Bridge Co. or Schneider specifications assume the wind pressure:

> First, at 30 lb. per sq.ft. on the exposed surface of all trusses and the floor as seen in elevation, in addition to a train of 10 ft. average height, beginning 2 ft. 6 in. above base of rail, moving across the bridge. Second, at 50 lb. per sq. ft. on the exposed surface of all trusses and the floor system. The greatest result shall be used in proportioning the parts.

The Cooper specifications call for provision to be made to resist a lateral force of 600 lb. per lin.ft. on the loaded chord, of which 450 lb. is to be treated as a moving load acting on a train of cars at a line 6 ft. above base of rail.

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*The Tugela Bridge, "Engineering," Jan. 26, 1900.

The unloaded chord is to resist a lateral force of 200 lb. per lin.ft. for spans up to 200 ft., and 25 lb. for each additional 50 ft.

The specifications of the railroads mentioned below are selected from a larger number to show the varying assumptions made of the amount of wind and lateral forces to be used in the design of railroad bridges. The wind is assumed to act horizontally at right angles to the bridge. Pounds per lineal foot means lineal foot of bridge.

Baltimore & Ohio R.R. Co.—A moving lateral force of 600 lb. per lin.ft. against the loaded chord, and 200 lb. per lin.ft. against unloaded chord.

Buffalo, Rochester & Pittsburgh Ry. Co.—(a) On the loaded structure, 30 lb. per sq.ft. on the exposed surface of all trusses and the floor system as seen in elevation, and on a moving train surface of 10 ft. average height beginning 2 ft. 6 in. above base of rail. (b) On the unloaded structure, 50 lb. per sq.ft. (instead of 30). In no case shall a lateral force of less than 200 lb. fixed and 300 lb. moving per lin.ft. be used for the loaded chord and less than 150 lb. per lin.ft. fixed for the unloaded chord.

Canadian Pacific Ry. Co.—Same as the Schneider specifications.

Chesapeake & Ohio Ry. Co.—Against the unloaded chord a fixed force of 200 lb. per lin.ft. for all spans of 200 ft. and under, and an additional force of 10 lb. per lin.ft. for every 25 ft. increase in span over 200 ft. Against the loaded chord same as above with an additional force of 500 lb. per lin.ft. acting 8 ft. above base of rail and treated as a moving load.

Chicago, Milwaukee & St. Paul Ry. Co.—A lateral force of 750 lb. per lin.ft. against the loaded chord, and 200 lb. per lin.ft. against the unloaded chord, these forces being considered as moving.

Delaware & Hudson Co.—For the loaded chord 300 lb. per lin.ft. moving load and 200 lb. per lin.ft. dead-load. For the unloaded chord, 200 lb. per lin.ft. dead-load. For double-track bridges these loads shall be increased one-half.

Delaware, Lackawanna & Western R.R.—A moving load of 300 lb. per lin.ft. against the loaded chord, and a uniform load of 300 lb. per lin.ft. divided equally between loaded and unloaded chords.

Grand Trunk Railway System has "Private" printed on the title page of its specifications and hence they can not be quoted.

Harriman Lines—Same wording as Buffalo, Rochester & Pittsburgh Ry. Co. above.

Lehigh Valley R.R. Co.—A moving load of 700 lb. per lin.ft. against the loaded chord and a moving load of 300 lb. per lin.ft. against the unloaded chord.

Long Island R.R. Co.—(1st) A load of 30 lb. per sq.ft. "on the exposed surface of entire surface as seen in elevation" (but never less than 200 lb. per lin.ft. at the unloaded chord), and on a moving train 10 ft. high beginning 2 ft. 5 in. above base of rail; (2d) 50 lb. per sq.ft. on "the exposed surface of the entire structure as seen in elevation."

Mexican International R.R. Co.—Six hundred pounds per lineal foot against the loaded chord and 200 lb. per lin.ft. against the unloaded chord, both forces considered as moving.

National Lines of Mexico—On the unloaded structure 50 lb. per sq.ft. "on the geometrical elevation of the completed structure and track." On the loaded structure, "30 lb. per sq.ft. of said elevation," plus the moving surface of train 10 ft. high, beginning 2½ ft. above the base of rail. In no case shall the fixed wind pressure be less than 150 lb. per lin.ft. for each chord of any bridge.

New York Central Lines—(1st) A moving load of 30 lb. per sq.ft. on 1½ times the vertical projection of the structure on a plane parallel with its axis (but never less than 200 lb. per lin.ft. at the unloaded chord), and a moving load of 360 lb. per lin.ft. applied 8 ft. above the base of the rail. (2d) A moving load of 50 ft. per sq.ft. on 1½ times the vertical projection of the unloaded structure on a plane parallel with its axis.

New York, New Haven & Hartford R.R. Co.—Same as the A. R. E. Assn.

New York, Ontario & Western Ry. Co.—Same as the A. R. E. Assn. For double-track bridges the constants are increased 50% but the percentage of live-load remains the same.

Norfolk & Western Ry. Co.—Same as the Schneider specifications, but omitting "beginning 2 ft. 6 in. above base of rail."

Pennsylvania R.R. Co.—Same as the Schneider specifications.

Pennsylvania Lines West of Pittsburgh—A uniform load of 150 lb. per lin.ft. against the unloaded chord and 200 lb. per lin.ft. against the loaded chord. A moving load of 300 lb. per lin.ft. against the loaded chord acting at a line 6 ft. above the base of rail.

Philadelphia & Reading Ry. Co.—A uniform load of 200 lb. per lin.ft. against each chord and a moving load of 400 lb. per lin.ft. against the loaded chord with its point of application 7½ ft. above the rail.

Piedmont & Northern Lines—Same as A. R. E. Assn.

Seaboard Air Line Ry.—Same as A. R. E. Assn.

Southern Ry. Co.—Same as A. R. E. Assn.

Western Maryland Ry. Co.—Dead-load, 150 lb. per lin.ft. against the unloaded chord, and 200 lb. per lin.ft. against the loaded chord. Moving load, 400 lb. per lin.ft. against the loaded chord, applied at a distance of 6 ft. above the base of rail.

Western Pacific Ry. Co.—Same as Western Maryland.

The wide range of requirements demanded in existing specifications shows the difficulty of uniting on a common standard. At present an increasing number of railroad engineers is following the specifications of the American Railway Engineering Association. In Europe, bridges are built in accordance with rules and regulations prepared by the respective governments. This at times is an advantage, at other times it is not. Unwin writes: 'English bridge builders are somewhat hampered in adopting rational limits of working stresses by the rules of the Board of Trade."

Working Stresses

The required material in a bridge depends upon assumed unit stresses as well as upon assumed loadings. Some ten years ago, the late Professor Heller* found that for the same live- and dead-load stresses in a bottom-

*"Engineering News," Nov. 19, 1903.

chord member of a 134-ft. span, there was a variation from 11.4% below to 18.6% above the average section of 25.4 sq.in. required by the 28 specifications he examined. In the specifications of the 25 railroads mentioned above. with the total stresses assumed by Heller, the variation is from 11.65% below to 9.33% above the average of 24.97 sq.in. required. There is nearly a unanimity in using for the combined stress due to lateral forces, plus live- and dead-loads, a unit-stress 25 or 30% greater than that due to the live- and dead-loads alone.

In this connection attention is called to the bending stresses in the end posts due to portal bracing; and the stresses induced in different members when the bridge is figured for overturning moments. These are not to be neglected, nor are centrifugal stresses when track is on curve.

Long Spans

What has been written and the specifications quoted apply primarily to railroad bridges of noncontinuous truss spans. When the Ohio River bridge between Cincinnati and Covington was finished in 1889 with a center span of 545 ft. and two spans of 486 ft., it had the distinction of having the longest and heaviest simple truss that had been built either in the United States or in Europe. The specifications called for a wind pressure of 30 lb. per sq.ft. on the exposed surface of both trusses and the vertical projection of the floor system, and on a moving-train surface averaging 10 sq.ft. per lin.ft.

The St. Louis Municipal Bridge has three spans, each of 668 ft. center to center of end pins, at present the longest simple truss spans in the world.* The permissible lengths of the spans are explained by 58% of the metal being nickel-steel. The wind loads assumed were 300 lb. per lin.ft. for the upper lateral system, and 600 lb. per lin.ft., one-half moving and one-half fixed, for the lower lateral system.

*Merriman and Jacoby in the last edition of their "Roofs and Bridges" enumerate 31 railway bridges and 6 combined railway and highway bridges which have simple truss spans of 400 ft. and over in length. Of these 15 are over 500 ft. and two, including the St. Louis bridge, are over 600 ft. The new Ohio River Bridge at Metropolis, recently contracted for, has a noncontinuous channel span of 700 ft. clear distance between piers.

The Hell Gate Bridge, now building, will have the longest arch in the world—a span of 977½ ft. The paragraph in the specifications relating to wind pressure reads:

Wind pressure shall be assumed as a moving load of 500 lb. per lin.ft. in the plane of the tracks, plus 30 lb. per sq.ft. on such vertical surface of the unloaded bridge as shall be exposed at any angle between 20° above or 20° below the horizontal or at an angle of 45° from the axis of the bridge, but not less than 200 lb. per lin.ft. on any chord.

For 25 years the Forth Bridge of cantilever design, in Scotland, has remained the greatest bridge in the world. Its spans of 1700 ft. exceed in length and magnitude any other now standing. The wind loads and unit stresses used in the design were those of the English Board of Trade Regulations, which most engineers regard as excessive and needlessly severe. The table below, made by Sir Benjamin Baker from his calculations of stresses due to the separate loadings, will show the important part wind forces played in the design.

	Dead Load Stresses	Live Load Stresses	Wind Stresses	Total Stresses
Bottom member	2282	1022	2920	6224
Top member	2253	997	544	3794
Vertical member	1550	705	1024	3279
Diagonal struts	802	167	414	1383
Diagonal ties	754	186	194	1134
Horizontal wind bracing..	80	5	265	350
Vertical wind bracing.....	42	169	108	319
Central girder—top	337	303	182	822
Central girder—bottom ...	330	301	247	878

Stresses are given in tons of 2240 lbs.

The Quebec Bridge, also of cantilever design, now building, will, when finished, eclipse the Forth Bridge, its enormous channel span being 1800 ft. long. The assumed wind loads are:

A wind load normal to the bridge of 30 lb. per sq.ft. of the exposed surface of two trusses and 1½ times the elevation of the floor (fixed load), and also 30 lb. per sq.ft. on travelers and falsework, etc. during erection.

A wind load on the exposed surface of the train of 300 lb. per lin.ft. applied 9 ft. above base of rail (moving load).

A wind load parallel with the bridge of 30 lb. per sq.ft. acting on one-half the area assumed for normal wind pressure.

Conclusion

The writer has no intention of passing judgment upon the specifications of engineers who have carried American bridge building to such a marked success. Standard specifications, as far as wind stresses are concerned, may not be practicable, but it should be possible to come nearer to points of agreement than at present. As an instance, the Lehigh Valley, the Philadelphia & Reading, and the Lackawanna railroads, all in the same territory, must have practically the same lateral forces; but the assumed forces vary nearly 75%. A discussion of the reasons for these variations would be interesting.

In no specifications that the writer has ever read is there an attempt to separate the stresses due to wind from those due to other lateral forces. The 10% of the weight of the train often specified for the lateral force on the loaded chord includes the wind pressure on the train. The wind pressure of 30 lb. per sq.ft. on a moving train, sometimes called for, includes the lateral force due to vibration. When the assumed loading is given in lb. per lin.ft., the total pressure due to all lateral forces is intended.

Perhaps in the future some engineer may be able to assign definite amounts to the different items that make up the total lateral force on a bridge. This would be one of the first steps to be taken to secure any degree of uniformity in proposed requirements for lateral stresses. As a beginning in this direction, the writer suggests that the lateral force on the loaded chord due to oscillation of the train be taken at 4% of the train load.

IV

Wind Stresses in Highway Bridges

SYNOPSIS—A review of the varying assumptions that have been made regarding the wind stresses in highway bridges. Problem complicated by lateral forces due to traffic. Present-day specifications, and variation of practice.

Early American Writers

Whipple—In the early part of 1847 there appeared a pamphlet of 48 pages with the title, "An Essay on Bridge Building, containing analyses and comparisons of the principal plans in use, with investigations as to the best plans and proportions, and relative merits of wood and iron, for bridges. By S. Whipple, C. E., Mathematical and philosophical instrument maker. Utica, N. Y. H. H. Curtiss, printer, Devereux Block, 1847." After distributing 50 or 60 copies among friends, the author bound the remainder of the edition with "Essay No. II on Bridge-Building Giving Practical Details and Plans for Iron and Wooden Bridges," which he had written and printed later in the year. This little book of 120 pages and 10 plates was the pioneer in the mathematics of bridge construction. To Squire Whipple, its author, the inventor of the Whipple bridge, belongs the honor of being the first to publish a correct analysis of the stresses in a simple truss. His work did not become widely known. In 1869 he took the copies remaining of his original edition and bound them "with an appendix, containing Corrections, Additions & Explanations, Suggested by Subsequent Experience: to which is annexed an Original Article on the doctrine of Central Forces." This addition of about 150 pages the author prepared and printed with his own hands.

In 1872 an enlarged and rewritten edition was published by the D. Van Nostrand Co. In 1873 a chapter of 35 pages on Drawbridges was added. From a copy of this 1873 edition, the following quotations regarding swaybracing are taken.

> The primary and essential purpose of a bridge is to withstand vertical forces which are certain and, to a large extent, determinate in amount . . . . . But the lateral or transverse forces to which a bridge superstructure is liable, are of a casual nature, depending upon conditions of which we have only a vague and general knowledge; . . . . But in arranging his system for securing lateral stability and steadiness, science can lend him but little assistance. . . . . He knows the wind will blow against the side of his structure, but whether with a maximum force of one hundred pounds, or as many thousands, he has no means of knowing with any considerable degree of certainty or probability . . . . No attempt will be made here to assign specific stresses as liable to occur in sway rods or braces, based upon calculations from the uncertain and indeterminate elements upon which the lateral action upon bridges depends. But judging from experience and observation, it may be recommended that iron sway rods be made of iron not less than ⅝ inch in diameter, for bridges of five panels or under, ¾-in. from six to ten panels inclusive. For twelve and fourteen panels, ¾-in. for ten middle panels and ⅞-in. for the rest; and for sixteen the same as last above, with the addition of a pair of 1-in. rods in the end panels.

These are opinions from the father of modern bridge building, written only 42 years ago.

Boller—Boller's "Iron Highway Bridges," first published in 1876, which has passed through several editions, says,

> The horizontal or sway bracing may consist of very light rods, if the floor is well laid, forming as it does a very effective system of bracing against lateral movement. Rods from ¾- to 1-in. round will cover all but extreme requirements, and they are attached by any convenient means to the floorbeams near their point of support.

In modern textbooks calculation has taken the place of speculation. Most of what has been written about the wind stresses in railroad bridges* applies to highway bridges. The same questions of the intensity of wind pressure per unit of area exposed, and the amount of area to be considered as exposed surface, are to be met. Induced stresses, load uniform or moving, and lateral forces other than wind are also to be considered.

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*In the preceding article.

Changed Traffic Requirements

The whole subject of loadings on highway bridges is being revised. This is the day of heavy concentrated loads. Many present bridges are seriously overloaded by the traffic coming upon them, especially in the floor-beams and joists. They were often built to carry uniform live-loads of, say 125 lb. per sq.ft. for the floor-beams and 80 or 100 lb. for the trusses. Sometimes a road roller was mentioned in the specifications. Manufacturers are constantly increasing the weight of road rollers and traction engines, and with the good-roads movement many bridges are called upon to carry rollers and engines for which they were not designed. Then there is the automobile, often run at a speed of 30 mi. per hr.

But the severest tests to which some of our highway bridges are being put are those from the auto trucks.** The traffic of towns and cities now reaches far out into the country. The road roller runs slowly, while the auto truck may be driven at a speed of 12 mi. per hr. and two trucks may meet or pass each other on the same bridge. A load of 10 tons is often carried and the weight of the truck adds another 6 tons. (In New York City a load of 75 tons has been carried on a truck weighing 10 tons, most of the load being on the two rear wheels.) † Trucks are being made heavier and with increased capacity. Greater impact stresses are induced and the tendency to both vertical and lateral vibration becomes greater. Sometimes centrifugal forces are introduced. All this is particularly true of the auto truck when fully loaded and with a driver ignorant or indifferent to loadings, speed, and the strength of bridges. One writer,‡ however, does not think the vibration effects greater than those produced by a horse and wagon. Anyone who has stood on a country bridge of 150-ft. span while a horse drawing a light buggy was crossing at a trot may have felt a decided jolting of the whole structure, a condition largely remedied by rigid connections and stiff members.

**Motor Truck Loading on Highway Bridges, "Eng. News," Sept. 3, 1914.

†Seaman, Proceedings, Am. Soc. C. E., December, 1911.

‡Neff (Am. Assoc. for Advancement of Science), "Engineering and Contracting," Jan. 22, 1913.

Electric-car lines are being extended, and cars are being increased in weight and run at greater speed. When a highway bridge carries electric cars it becomes in reality a miniature railroad bridge. City or county officials who, without examination by a competent engineer, will sanction the use of existing bridges to carry electric cars, belong to the class of undesirable citizens.

Lateral Forces Other Than Wind Pressure

The assumed wind load in bridge specifications includes all the lateral forces whether so stated or not. The writer believes that this is sufficient (in nearly all present specifications) to take care of the increased lateral forces due to changed traffic requirements. With the exception of the electric car, it is improbable that the full wind load will be acting at the same time that the lateral vibration occurs, due to the moving load. It should be remembered that, if by any means a highway bridge is blown over, there is not likely to be any loss of life, neither will traffic be seriously interrupted. The actual loss to the authorities is little more than the cost of the structure itself. Hence, excessive bracing in all bridges to guard against a remote possibility in a single one is unnecessarily expensive. With a railroad bridge it is different; provision must there be made for remote possibilities.

The weakness of lateral systems of highway bridges in the past has not been so much in the assuming of loads as in the abominable details used. Witness the common practice of 25 years ago and still prevalent in some quarters of fastening the lower laterals in a nondescript way to the floor-beams, which, in turn, are suspended from the pins by U-bolt hangers; or, the top laterals having bent eyes taking the top-chord pins and pulling against struts attached to the same pins by bent plates. It is better to design for a safe and sane wind loading, taking care of induced stresses, and with all details fully up, than to proportion the body of the lateral members for larger stresses and use inefficient details.

In high-truss bridges the compression chord is kept in alignment by the top lateral system. In the pony truss, recourse is often had to doubtful expedients. "One has only to shake the top chord of a pony truss to see how loosely it is secured laterally and to demonstrate its lack of fixity at intermediate points."* With the moving loads now coming into use, the pony truss is doomed. In some specifications it is prohibited altogether.

The wind is generally assumed to blow horizontally, but it may vary greatly from the horizontal. For high bridges in exposed localities, the upward pressure should be taken into account; the end anchorage should provide for any possible uplift and against the structure being moved off its seats either by wind pressure or by a blow from a passing object. A study of the wreck of the High Bridge over the Mississippi River at St. Paul† is interesting.

PRESENT SPECIFICATIONS

SCHNEIDER—Passing to well known specifications, the American Bridge Co. or Schneider specifications for steel highway bridges read:

> The wind pressure shall be assumed acting in either direction horizontally:
>
> First. At 30 lb. per sq.ft. on the exposed surface of all trusses and the floor as seen in elevation, in addition to a horizontal live-load of 150 lb. per lin.ft. of the span moving across the bridge, but not less than 300 lb. per lin.ft. shall be used for bracing of loaded chord nor less than 150 lb. per lin.ft. of unloaded chord.
>
> Second. At 50 lb. per sq.ft. on the exposed surface of all trusses and the floor system.
>
> The greatest result shall be assumed in proportioning the parts.

COOPER—Probably more highway bridges have been built in accordance with the specifications of Theodore Cooper than any other. The paragraphs stating amount of lateral forces are:

> To provide for wind and vibrations, the top lateral bracing in deck bridges and the bottom lateral bracing in through bridges shall be proportioned to resist a lateral force of 300 lb. for each foot of the span; 150 lb. of this to be treated as a moving load.
>
> The bottom lateral bracing in deck bridges and the top lateral bracing in through bridges shall be proportioned to resist a lateral force of 150 lb. for each foot of the span.
>
> For spans exceeding 300 ft., add in each of the above cases 10 lb. additional for each additional 30 ft.

Johnson, Bryan & Turneaure; Merriman & Jacoby; Ketchum; and others in their textbooks follow Cooper's specifications regarding wind pressure. Others, as Mar-

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*Smith, Proceedings, Indiana Eng. Soc., 1911, p. 209; "Engineering Record," Jan. 21, 1911.

†Turner, Trans. Am. Soc. C. E., June, 1905, Vol. LIV, p. 31.

burg, and Burr & Falk, quote both the Schneider and the Cooper specifications.

WADDELL—Waddell in his "Ordinary Highway Bridges" assumes a wind pressure of 40 lb. per sq.ft. for spans 100 ft. and under, 35 lb. for spans 100 to 150 ft., and 30 lb. for spans greater than 150 ft.; these pressures to be increased 10 lb. for bridges in unusually exposed locations. The loads are considered moving.

The total area opposed to the wind is to be determined by adding together the area of the vertical projection of the floor and joists, and twice the area of the vertical projection of the windward truss, hub plank, guard rail, and ends of floor-beams.

In his "Specifications for Steel Highway Bridges," 1906, the wind loads per lineal foot of span for both the loaded and unloaded chords are taken from curves shown on a diagram. The diagram was figured (for a clear roadway of 20 ft.) with intensities varying from 40 lb. for very short spans to 25 lb. for very long ones. For spans up to 600 ft., the curves show loads from 200 to 355 lb. per lin.ft. of bridge on the loaded chord and 100 to 265 lb. on the unloaded chord, according to the length of span and the class of the bridge. For wider structures, the wind loads are to be increased 2% for each foot of width in excess of 20 ft.

GREINER—The "Specifications for Steel Stationary Bridges," by Greiner, require that

for city, interurban and country bridges the lateral force against unloaded chords shall be assumed not less than 150 lb. per lin.ft. plus 10% of the uniform load on one car track or on a width of 12 ft., and for the unloaded chords 150 lb. per lin.ft. In cases where a lateral force of 30 lb. per sq.ft. on 1½ times the vertical projection of the structure produces greater stresses than the above loads, it shall be considered. All lateral loads shall be treated as moving.

OSTRUP—Ostrup, in his "Standard Specifications for Highway Bridges," calls for wind bracing to be designed to resist one of the following lateral loadings, whichever produces the greater stress: (a) *Structure unloaded,* 50 lb. per sq.ft. on the exposed surface of all trusses and the floor as seen in elevation; or (b) *Structure loaded,* bridges (of all classes) carrying highway traffic only, 30 lb. per sq.ft. on the exposed surface of all trusses and the floor as seen in elevation in addition to a uniform load of 150 lb. per lin.ft. of structure applied on the loaded chord; or (c) *Structure loaded,* bridges of all classes carrying electric-railway traffic, the same loading as under (b) except that the additional uniform load is 300 lb. per lin.ft. of structure and is applied 7 ft. above the base of rail. The minimum value of the pressure is to be 250 lb. per lin.ft. for the loaded and 150 lb. for the unloaded chord of the structure.

BOWSER—Bowser, in a "Treatise on Roofs and Bridges," gives 30 lb. per sq.ft. of exposed surface of both trusses as the maximum wind load upon a highway bridge. To estimate the 30-lb. pressure when the surface is not known, he writes, "it is customary to use the following rule: Take 150 lb. per lin.ft. per truss, or 75 lb. per lin. ft. for each chord."

MERRIMAN—In the earlier editions of Merriman's work, "A Textbook on Roofs and Bridges," are found the sentences:

> For a highway bridge the surface exposed to wind action is usually taken as double the side elevation of one truss. If the area of this be not known, an approximation to its value may be found by taking it as many square feet as there are linear feet in the skeleton outline of the truss.

A number of the states, through highway commissioners or otherwise, have issued specifications for steel highway bridges. Some of them are incomplete and written by men without a clear knowledge of the subject. Below are given quotations from these and some other sources, as to horizontal wind pressure.

COLORADO—300 lb. per lin.ft. on the loaded chord and 150 lb. per lin.ft. on the unloaded chord.

ILLINOIS — Cooper's Specifications for Highway Bridges, ed. of 1909, "except as hereinafter specified or as may be specially indicated on the drawings."

MICHIGAN—No mention made of wind loads but they may be covered by the paragraph, "Any questions that may arise as to the quality of material and labor shall be settled in accordance with the provisions of Theodore Cooper's Specifications for Steel Highway Bridges, under Class B-1.

NEBRASKA—300 lb. per lin.ft. on the loaded chord and 150 lb. on the unloaded chord.

OHIO—Same as the Cooper Specifications, ed. of 1909.

VIRGINIA—300 lb. per lin.ft. on the loaded chord, 150 lb. of which is to be treated as a moving load, and 150 lb. per lin.ft. on the unloaded chord.

MASSACHUSETTS—The Massachusetts Railroad Commission, George F. Swain, Consulting Engineer, specifies that, for bridges carrying electric railways "a lateral force of 50 lb. per sq.ft. on the unloaded structure, or of 30 lb. per sq.ft. on the loaded structure, shall be provided for. The surface of the unloaded structure shall, in the case of a truss, be taken as twice the area of the vertical elevation of one truss, plus that of the floor; and in the case of a girder, as 1½ times the vertical surface. The surface of the loaded structure shall be that of the unloaded structure plus a vertical surface 10 ft. in height and 50 ft. long, the pressure on which is to be considered a moving load upon a car."

NEW YORK—The Department of the State Engineer and Surveyor of New York specifies: "The intensity of the wind pressure shall be assumed at: First, 30 lb. per sq.ft. on the exposed surface of all railings, trusses, trestle posts, bracing and the floor in addition to a load of 150 lb. per lin.ft. applied at 4 ft. above the floor line for all bridges which do not carry electric cars, and 300 lb. per lin.ft. applied 8 ft. above the floor line for all bridges which do carry electric cars. Second, 50 lb. per sq.ft. on all exposed surface of the unloaded structure. All parts shall be proportioned for that one of these loads which gives the greater results, but in no case shall the wind pressure be assumed at less than 100 lb. per lin.ft. at the unloaded chord, or less than 250 lb. per lin.ft. at the loaded chord. All wind loads shall be considered as moving loads."

PHILADELPHIA—The Department of Public Works of the City of Philadelphia specifies for its bridges a wind pressure of 30 lb. per sq.ft. against the side area of all trusses, railings, and the end area of the floor construction. In no case is less than 150 lb. per lin.ft. to be used. In addition the system attached to the floor is to carry a moving load of 150 lb. per lin.ft. of bridge.

HARRIMAN LINES—A number of railway companies have specifications for highway bridges attached to or a part of their specifications for railroad bridges. The Harriman Lines issue separate specifications for highway bridges in which the wind pressure is taken: (a) On the loaded structure at 30 lb. per sq.ft. on the exposed surface of all trusses and the floor system as seen in elevation, together with a moving load of 150 lb. per lin.ft. of bridge. (b) On the unloaded structure at 50 lb. per sq.ft. on the exposed surface taken as in (a).

U. S. ROADS—The Office of Public Roads, U. S. Department of Agriculture, issues "Typical Specifications for the Fabrication and Erection of Steel Highway Bridges." The Director of the Office states that they are prepared "with the view of furnishing a suitable guide for local highway officials in fixing requirements to which bridge structures must conform." He further writes, "In the past many steel bridges have been very poorly constructed, and it is believed that lack of information on the part of highway officials concerning proper specifications for this class of work has been in a large measure responsible for the unsatisfactory results." It may be remarked that unless the highway officials are reinforced by competent engineers, the use of these specifications will not prevent "unsatisfactory results." The wind loads assumed are a load of 300 lb. per lin.ft. on the loaded chord, one-half of this to be treated as moving, and 150 lb. per lin.ft. on the unloaded chord.

ONTARIO—The "General Specifications for Steel Highway Bridges," of the Canadian province of Ontario are quite detailed in their provisions for wind and lateral stresses. For Class A (bridges suitable for main county roads) in spans of 200 ft. or less, a uniform load of 150 lb. per lin.ft. per span is used on the unloaded chord, and the same with the addition of 150 lb. per lin.ft. moving load on the loaded chord; for spans exceeding 200 ft. the uniform load in each system is to be increased 10 lb. for each 30 ft. of span. For Class B (bridges carrying light rural traffic), same as Class A. For Class C (bridges for heavy traffic in towns and cities), for spans of 200 ft. and less, a uniform load of 200 lb. per lin.ft. of span on the unloaded chord and a uniform load of 250 lb. per ft. in addition to a moving load of 250 lb. per ft. on the unloaded chord; for spans over 200 ft. the uniform load in each system is to be increased 10 lb. for every 30 ft. increase of span.

TYRRELL—Merriman & Jacoby in their enumeration of noncontinuous bridges of 400-ft. span and over include 21 which are exclusively highway. Of these 21 the longest span is that over the Miami River at Elizabethtown, Ohio (destroyed by flood in March, 1913). This structure was proportioned "for a wind load of 30 lb. per sq.ft. acting on the exposed surface of both trusses, and all bracing that is likewise exposed to wind pressure."*

Unit-Stresses

In the specifications mentioned, the values allowed for stresses due to combined dead- and live-load and wind are 20 to 30% greater than that allowed for combined live- and dead-loads. The proviso is attached that the section used must not be less than that required for the combined dead- and live-loads. One exception is that of the U. S. Department of Agriculture: these specifications require that the wind stresses be proportioned with the same values as other stresses without allowance for any combination with other loads. This may not be intended, but there is no doubt of the literal interpretation. Another exception is the Waddell specifications, where in highway bridges no reduction of working stress is allowed for any combination of loads. Unless the structure carries an electric railway, it is assumed that the live-load and wind-load cannot act together, "for the reason that no person would venture on the bridge when even one-half of the assumed wind-pressure is acting."

*Tyrrell, The Elizabethtown Bridge.

Conclusion

It will be seen from the above that the current requirements regarding lateral bracing vary greatly. A number of states have already legislated upon the subject of highway bridges and others will soon follow. As far as lateral bracing is concerned it might be well to divide highway bridges into three classes, those which carry electric cars, those which carry heavy loads other than cars, and ordinary country bridges. A different lateral loading should be assigned to spans over 150 ft. than to those under. It is better to state the wind pressure in pounds per lineal foot of bridge rather than in pounds per square foot of exposed surface, because contracts for highway bridges are almost invariably let by competition, and if wind loadings are given in pounds per lineal foot, the designs of different bidders are on the same basis, which may not be the case when given in pounds per square foot.

After a study of many specifications, the writer suggests the following:

RECOMMENDED SPECIFICATIONS

For bridges carrying electric-railway traffic the lateral system shall be designed to resist a lateral force of 300 lb. per lin.ft. on the loaded chord and 150 lb. per lin.ft. on the unloaded chord, for spans of 150 ft. and under. An additional allowance of 10 lb. for every 30 ft. of span shall be made to the loaded chord and 5 lb. to the unloaded chord for spans of more than 150 ft.

For bridges not carrying electric cars, but subject to heavy loads such as auto-trucks, road rollers, and traction engines, the lateral force shall be assumed at 250 lb. per lin. ft. on the loaded chord and 150 lb. per lin.ft. on the unloaded chord, for spans of 150 ft. and under. An additional allowance of 5 lb. for every 30 ft. of span shall be made to each chord for spans of more than 150 ft.

Ordinary country bridges shall be designed for a lateral force of 225 lb. per lin.ft. on the loaded chord and 150 lb. on the unloaded chord with an additional force on the loaded chord of 5 lb. for every 30 ft. of span exceeding 150 ft.

All lateral loads are to be considered as moving loads.

In members subject to stresses from lateral forces alone the unit-stresses may be increased 25% over those assumed for live- and dead-loads. In bridges carying electric cars the unit-stresses in chords and floor-beams for the stresses due to lateral forces combined with those from the vertical forces may be increased 25% over those assumed for dead- and live-loads. If the track is on curve the centrifugal force shall be added to the lateral live-load. For bridges not carrying electric traffic, unit-stresses of 50% increase may be used instead of 25%. In no case shall the section be less than that required for the live- and dead-loads.

Provision shall be made for reversal of stress in any member due to any combination of wind with other loads. The end seats shall in all cases be firmly anchored against lateral movement and uplift. In bridges in unusually exposed situations or at a great height above the water the amount of anchorage shall be determined by calculation.

All details shall be designed to carry the stresses in the main members.

V

Wind-Bracing Requirements in Municipal Building Codes

SYNOPSIS—How 120 American cities specify wind pressure for the design of buildings. Great range of pressures and working stresses. Recommendation that 20 lb. per sq.ft., and for combined stress 50% increase in working stress, be adopted as standard.

The assumptions that are made for wind pressure and working stresses due to wind loads play an important part in the design of a many-storied hotel or office building. These are seldom left to the judgment of the designer, but are determined by the building code of the city in which the building is located.

According to the census of 1910 there were in the United States 50 cities each having a population over 100,000. The building codes of 45 of these cities, together with those of about 75 cities below 100,000, have been examined with respect to their requirements for wind bracing. The purpose of this article is to show the wide variation in requirements in these codes, and to make a plea that assumptions be made more nearly uniform.

The present Building Code of the City of New York, affecting more building operations than that of any other city on the continent, was adopted in 1899. The Board of Aldermen passed a new code in 1909, after extended discussion and bitter controversy, but the Mayor vetoed it. The present code is archaic in some of its provisions and is inadequate for present needs. It has been used as the basis for the codes of a host of other cities. Sometimes it has been copied with but little change, and in other cases some sections have been modified or rejected.

Regarding wind pressure the New York code requires that all structures exposed to the wind (except those under 100 ft. in height in which the height does not exceed four times the average width of the base) be designed to resist a horizontal wind pressure of 30 lb. in any direction for every square foot of surface exposed from the ground to the top, including the roof. Regarding unit-stresses the code reads:

In calculations for wind bracing, the working stresses set forth in this code may be increased by 50%.

This sentence is ambiguous as it does not state whether the high unit-stress is applicable to the combined stresses due to wind and other loads or whether it is to be used for the wind stress only. There is considerable difference between the two interpretations as to the amount of material required in bracing a high and narrow building. The Chicago code removes all doubt by specifying:

For stresses produced by wind forces combined with those from live- and dead-load, the unit-stress may be increased 50% over those given above; but the section shall not be less than that required if wind forces be neglected.

It may be said that the practice in New York and elsewhere is to interpret the 50% as applying to combined stresses.

Another sentence in the New York code reads:

In all structures exposed to wind, if the resisting moments of the ordinary materials of construction, such as masonry, partitions, floors and connections, are not sufficient to resist the moment of distortion due to wind pressure, taken in any direction on any part of the structure, additional bracing shall be introduced sufficient to make up the difference in the moments.

Good practice does not permit and it is not common to carry the wind stresses in steel buildings either in whole or in part to the ground by walls or partitions. The Bridgeport code has a clause which should be followed, reading:

In buildings of skeleton construction the frame must be designed to resist this wind pressure.

Manchester, Albany, Utica, Jersey City, Paterson, Terre Haute, Kalamazoo, Milwaukee, St. Paul, Minneapolis, Louisville, Tampa, Atlanta, Dallas and Tacoma all follow the New York code regarding wind bracing except for an occasional variation for buildings under 100 ft. in height.

In Philadelphia a pressure of not less than 30 lb. per sq.ft. is called for on all buildings erected in open spaces or on wharves. On tall buildings erected in built-up districts the wind pressure is not to be figured for less than 25 lb. at tenth story, 2½ lb. less on each succeeding lower story, and 2½ lb. additional on each succeeding upper story to a maximum of 35 lb. at the fourteenth story and above. In proportioning members subject to stresses due to wind loads the working stresses may be increased 30%. In Washington buildings are practically limited to twelve stories in height. The prescribed wind pressure is the same as in Philadelphia, but no mention is made of any increase of working stresses. Lowell, Bridgeport, Baltimore, Buffalo and Sioux City assume wind pressure at 30 lb. per sq.ft. and are also silent on the subject of working stresses being increased.

Pittsburgh calls for 25 lb., Detroit and Jacksonville 30 lb. per sq.ft. wind pressure, and each allows the working stresses to be increased 25%.

Cincinnati requires provision to be made for a pressure of 20 lb. per sq.ft. for the surface exposed above surrounding buildings; working stresses may be increased 25%. St. Louis assumes a pressure of 30 lb. per sq.ft. and allows an increase of 20% to working stresses. The St. Louis code has this provision:

> Where there are buildings immediately adjoining, the wall surface covered by such buildings will be considered as not exposed to wind pressure.

The question might be asked concerning buildings in Cincinnati and St. Louis as to what would take the wind pressure if the surrounding buildings were removed.

Chicago, San Francisco, Covington and Akron call for 20 lb. per sq.ft. wind pressure. An increase of 50% to the working stresses is allowed in Chicago and San Francisco, 25% in Covington and none in Akron.

Poughkeepsie, Evansville and Chattanooga call for 30 lb. per sq.ft. horizontal wind pressure, and state as follows:

> The additional loads caused by the wind pressure upon beams, girders, walls and columns must be determined by calculation and added to other loads for such members. Special bracing shall be employed wherever necessary to resist the distorting effect of the wind pressure.

No mention is made of higher unit-stresses for wind loads.

Syracuse, Erie, Cleveland, Duluth, Denver, Macon, Birmingham and Portland (Oregon) for all buildings whose heights exceed 1½ times the width of the base follow the wind pressures given in the Philadelphia code. The Syracuse code alone allows an increase of working stresses—25%. Each code has this provision:

Every panel in a curtain wall shall be proportioned to resist a wind pressure of 30 lb. per sq.ft.

The code of Grand Rapids copies the Schneider "Specifications for Structural Work of Buildings." The wind pressure is assumed as acting horizontally in any direction, as follows:

First—At 20 lb. per sq.ft. on the sides and ends of buildings and on the actual exposed surface, or the vertical projection of roofs.

Second—At 30 lb. per sq.ft. on the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors.

For bracing and the combined stresses due to wind and other loading, the permissible working stresses may be increased 25%, or to 20,000 lb. for direct compression or tension.

The code of Memphis has the same provisions though differently worded.

The code of Oakland is unusually explicit in the treatment of wind bracing. For buildings of Class A over 100 ft. high, or where the height exceeds three times the least horizontal dimension, or for buildings of Class B over 80 ft. high where the height exceeds two times the least horizontal dimension, it provides:

The steel frame shall be designed to resist a wind force of 30 lb. per sq.ft. acting in any direction upon the entire exposed surface. All exterior wall girders shall have kneebrace connections to columns. Provision shall be made for diagonal, portal or kneebracing to resist wind stresses, and such bracing shall be continuous from top story to and including basement.

An increase of 50% above the allowed dead- and live-load stress shall be used for wind stress. Columns subjected to cross-bending by wind or eccentric loading shall have additional area to provide for the stresses, the eccentric loading being calculated as dead-load and the wind provided above. The area of metal thus obtained for wind, cross-bending and eccentric loading shall be added to the area provided for dead- and live-load to obtain the total metal in column.

In the case of reinforced-concrete buildings where provision must be made for wind pressure, there is this provision:

The reinforcing rods of columns shall be connected by threading the rods and by threaded sleeve nuts or threaded turnbuckles, or methods equally effective and satisfactory to the Bridge Inspector.

In Salt Lake City for buildings over 102 ft. high, or where the height exceeds three times the least horizontal dimension, "the steel frame shall be designed to resist a wind force of 20 lb. per sq.ft. in any direction upon the entire exposed surface." As in Oakland, it is required that the exterior wall girders shall have kneebrace connections to the columns and that diagonal, portal or kneebracing to resist wind pressure shall be used from the top story to and including basement. Unlike Oakland no increase of working stresses for wind loads is mentioned.

The code of Waltham, Mass., has the provision:

All buildings exposed to the wind shall be calculated to resist a pressure on either side so exposed, and upon the roof, if pitched, amounting to 10 lb. per sq.ft. of vertical projection of roof between the ground and a height of 40 ft. above the ground, a pressure of 15 lb. per sq.ft. on parts between 40 and 60 ft. above the ground, and 20 lb. per sq.ft. on parts 60 ft. above the ground.

No increased working stresses for wind are mentioned.

The code of Columbus, Ohio, adopted ten years ago, reads the same on wind pressure as the New York code except that working stresses may be increased 25% instead of 50%. There is added the sentence:

In buildings constructed of structural steel the wind pressure shall be allowed for as follows: Ten lb. per sq.ft. of exposed surface for buildings 20 ft. or less to the eaves; 20 lb. per sq.ft. of exposed surface for buildings 60 ft. to the eaves; 30 lb. per sq.ft. of exposed surface for buildings over 60 ft. to the eaves.

The codes of Boston, Cambridge, Haverhill and New Orleans have the sentence: "Provision for wind bracing shall be made wherever it is necessary." This is indefinite and tends to put a premium on ignorance. If all designers were experts there would still be enough difference of opinion as to the amount of wind bracing necessary. But a design with little or no wind bracing is also entitled to consideration if the maker gives assurance that he is furnishing bracing "wherever it is necessary." The same might be said concerning the codes of New Haven, Providence, Worcester, Springfield, Wheeling, Youngstown, Toledo, Omaha, Lincoln, Montgomery, Fort Worth, Los Angeles and others, which while giving loads and stresses for structural steel generally say nothing on the subject of wind pressure. Indianapolis and Seattle allow an increase of 50% to the working stresses but do not state the amount of pressure.

The codes of Fall River, Pawtucket, Elizabeth, Allentown, Altoona, Fort Wayne, Dubuque and Topeka are very meager or altogether silent on the whole subject of loads, stresses, and structural steel.

Codes often contain blanket clauses which might be used to cover a wide range of omissions—thus, Cleveland, Duluth, Little Rock, Fort Worth and others say:

> The allowable factor or units of safety or the dimensions of each piece or combination of materials required in a building or structure, if not given in this ordinance, shall be ascertained by computation according to the rules prescribed by the modern standard authorities on strength of material, applied mechanics and engineering practice.

Erie, Pa., has the sentence: "In general all stresses shall be figured *in accordance with the standard specifications of the American Society of Civil Engineers.*"

The New Haven code reads:

> The dimensions of each piece or combination of materials required shall be ascertained by computation according to the rules and data given in Haswell's Mechanics' and Engineers' Pocket Book, Trautwine's Engineers' Pocket Book, or Kidder's Architects' and Builders' Pocket Book, except as may be otherwise provided in this title. Stresses for materials and forms of same not herein mentioned shall be those determined by the best modern practice.

The last code from which quotations will be made is that of the largest city in the world. The London County Council (General Powers) Act, 1909, in Section 22, "Provisions with respect to Buildings of Iron and Steel Skeleton Construction," requires:

> All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than 30 lb. per sq.ft. of the upper two-thirds of the surface of such buildings exposed to wind pressure.

Working stresses exceeding those specified "by not more than 25% may be allowed in cases in which such excess is due to stresses induced by wind pressure."

Conclusion

It might seem from the foregoing that our American municipalities have exhausted the combinations of wind pressure and wind stress that can be made. The fact that one code differs from another is not in itself a cause for criticism, but a code is decidedly at fault when it contains absurd or needless requirements or when its requirements are not clearly expressed. To assume wind pressure over a large area at 30 lb. per sq.ft. and then to add the sectional area necessary to resist wind stresses to that required for live- and dead-loads is needless. Where this is specified in a code it is evaded in practice. It would be far better to make rational assumptions and insist on a rigid adherence to them, than to insert in a code improbable loadings or working stresses that will be ignored in actual construction.

That the need of revision in our building codes is being felt by the public is evidenced by the number now being revised. Although our knowledge of wind action is limited we should be able to come nearer to a common ground of requirement for wind bracing than we have at present. As a basis for uniformity the writer suggests the building ordinances of Chicago. The paragraph on Wind Resistance reads:

> All buildings shall be designed to resist a horizontal wind pressure of 20 lb. per sq.ft. for every square foot of exposed surface. In no case shall the overturning moment due to wind pressure exceed 75% of the moment of stability of the building due to the dead load only.

The paragraph relating to Wind Stress, previously quoted, reads:

> For stress produced by wind forces combined with those from live- and dead-loads, the unit-stress may be increased 50% over those given above; but the section shall not be less than required if wind forces be neglected.

VI

Windbracing Without Diagonals for Steel-Frame Office-Buildings

SYNOPSIS—Exact elastic analysis of rigid square-panel tier-building frames being impossible in practice, approximate methods based on certain arbitrary assumptions are used. The first summarized statement of these methods was given by the author in ENGINEERING NEWS, *Mar. 13, 1913. The present article—an enlarged revision of that article—gives four methods, and compares their results for a specific example. Method* II-A *of this article has been added, and the treatment of the other three revised and corrected.*

If an apology is needed for adding to the literature of the above subject, it may be found in the fact that many of the methods given in technical papers for determining stresses due to wind loadings are not *workable.* That is, the average engineer to whom falls the lot of designing the average office-building has neither the time nor the ability to handle the cumbersome equations involved. One paper published a few years ago and now before the writer has for its purpose "to develop the exact theory of framework with rectangular panels, and then to suggest such short-cuts as may be of use in actual designing." This is an elaborate paper in which the theorem of four moments is used. A bent of two unequal bays, three columns and two girders, is considered and by the "short-cuts" seven equations are found from which the values of all the moments for the floor in question may be found. Whatever may be the merit of this and similar papers, it has not been recognized sufficiently to be followed to any appreciable extent. It is to be regretted that the treatment of the subject in our textbooks is not more complete and adequate.

Buildings like the Trinity, the Fuller, the Singer, the Woolworth, or the Metropolitan Tower, in New York City, are each in a class by itself, and of necessity careful study is given to the windbracing. For another and a large class of office-buildings, little or no attention is given to the matter of bracing for wind, either in the proportioning of main members or in details.

Without further introduction the writer gives four methods in current use of calculating wind stresses and moments in office-buildings where diagonals are not permissible. Each method has its own advocates.

Considering a single bent: It will be assumed that all columns in any given story have the same sectional area and the same section modulus, that all girders of the same floor have the same section modulus, and that the joints are perfectly rigid. It is obvious that if the forces in the several members of the frame are small in relation to the stiffness of the members, the longitudinal distortions may be neglected; hence the adjacent joints occupying the corners of a rectangle will after distortion occupy the corners of an oblique parallelogram.

It is assumed that the point of contraflexure of each column is at midheight of the story. The first method described further involves the tacit assumption that the girders have their points of contraflexure at midlength. Specific assumptions as to the distribution of column shears and direct stresses are made in the several methods. In only one of the four methods are the assumptions strictly consistent. For example, in Method I the assumption as to location of points of contraflexure would make the distorted shape of panel constant in any given story, and from this would follow that the column shears must be equal; but the calculation gives column shears of different amount.

The resistance to overturning will cause a direct stress in tension on the windward side of the neutral axis, taken by all or some of the columns on that side according to the method used, and a direct stress in compression on the leeward side taken by the columns on that side.

Figs. 1, 7, 9, and 12, give results obtained from calculations according to Methods I, II, II-A and III respectively. Loads and stresses are given in thousands of pounds and bending moments in thousands of foot-pounds. Direct stresses are given in parentheses ().

Method I

This may be called the Cantilever Method and is a restatement with some modifications of an arti-

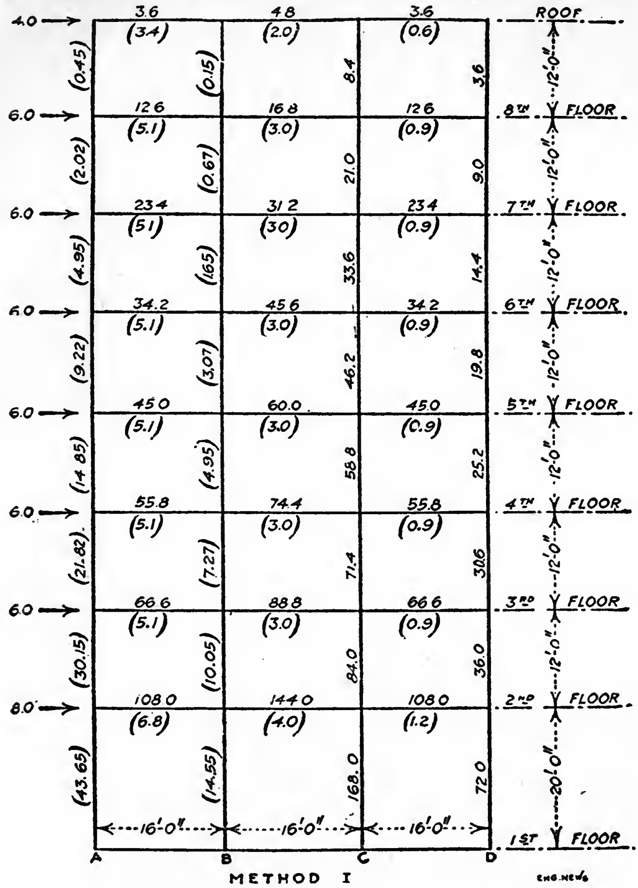


FIG. 1. RECTANGULAR BUILDING-FRAME: DIRECT AND BENDING STRESSES CALCULATED BY APPROXIMATE METHOD I

144.0 = Moment of 144,000 ft.-lb.
(4.0) = Direct stress of 4000 lb.

cle entitled "Windbracing with Kneebraces or Gusset-plates," by A. C. Wilson, in *The Engineering Record*, Sept. 5, 1908. A section or bent of the building is considered similar to a beam loaded as a cantilever.

If a beam of rectangular section be loaded as a cantilever with concentrated loads, it is possible by the theory of flexure to find the internal stresses at any point. If, however, rectangles be cut out of the beam between the loads, there will then be a different condition of stress. What was the horizontal shear of the beam will now be a shear at the point of the contraflexure of the floor girders, causing bending, and, as in the beam, the nearer the neutral axis the greater the shear. The vertical shear in the beam would be taken up by the columns as a shear at the points of contraflexure and the amount of this shear taken by each column would, as in the beam, increase toward the neutral axis. The direct stresses of tension or compression in the beam would act on the columns as a direct load of either tension or compression, and as in the beam would decrease toward the neutral axis.

Each intersection of column with floor girders would be held in equilibrium by forces acting at the points of contraflexure; and to find all the forces acting around a joint at any floor the bending moments of the building at the points of contraflexure of the columns above and below the floor in question are found as will be explained later,

It is assumed that if a beam of constant, symmetrical cross-section and homogeneous material is fixed at both ends, and that if forces tend to move those ends from a position in the same straight line to a position to one side with the ends still parallel, reversed bending will occur with the point of contraflexure in the center of the unsupported span. And since this condition exists in all columns and floor girders it will be necessary to find the shears at the points of contraflexure as well as the direct stresses in all members.

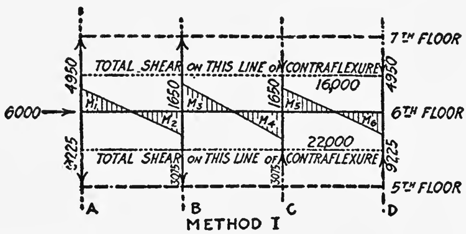


FIG. 2. COLUMN SHEARS AND GIRDER MOVEMENTS AT SIXTH FLOOR, CALCULATED BY METHOD I

Fig. 1 gives stresses and maximum moments in all members of a section of the building in accordance with the above statement.

The calculation of stresses and bending moments in members about the sixth floor will be given in detail. The direct stress in any column is assumed to be proportional to its distance from the neutral axis of the cross-section of the building. In the cross-section considered, the neutral axis coincides with the center line of the building. The total moment of the wind loads above the sixth floor about the line of inflection of the sixth-story columns must equal the moment of the direct stresses in these columns about the neutral axis. Let $8X$ be the direct stress in each of the sixth-story columns B and C, then $24X$ will be the direct stress in each of the sixth-story columns A and D. Hence we have

$$(4000 \times 30) + (6000 \times 18) + (6000 \times 6) = (24X \times 24) + (8X \times 8) + (8X \times 8) + (24X \times 24)$$

From which $8X = 1650$ and $24X = 4950$.

In the same way for the fifth-story columns we have the equation

$$(4000 \times 42) + [6000 \times (30 + 18 + 6)] = [(24X \times 24) + (8X \times 8)] \times 2$$

From which $8X = 3075$, the direct stress in the fifth-story columns B and C; and $24X = 9225$, the direct stress in the fifth-story columns A and D.

The total horizontal shear on any line across the building is the sum of the wind loads above that line. The shear taken by any column in any story is proportional to the total horizontal shear in that story.

In Fig. 2, if $X =$ shear of any fifth-story column at its point of inflection, then $\frac{16,000}{22,000} X$ or $\frac{8}{11} X =$ shear at point of inflection of the sixth-story length of the same column, and $\frac{6,000}{22,000} X$ or $\frac{3}{11} X =$ increment of shear taken by the column at the floor girder.

We are now ready to consider the forces about the first joint, or the intersection of Col. A with the sixth-floor girder, sketched separately as Fig. 3.

The difference between 9225 and 4950 = 4275 is taken up as a shear in the floor girder between Cols. A and B. The moments of the shears must hold the joint in equilibrium. Taking moments about the lower point of inflection we have

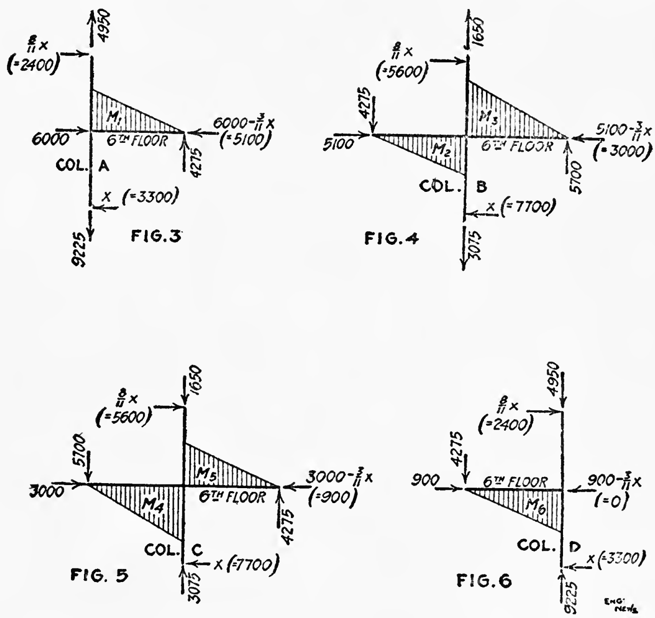
$$(\tfrac{8}{11} X \times 12) + (\tfrac{3}{11} X \times 6) = 4275 \times 8$$

from which $X = 3300$, $\tfrac{8}{11} X = 2400$ and $\tfrac{3}{11} X = 900$. The bending moment M_1 for the floor girder is $4275 \times 8 = 34{,}200$ ft.-lb. The bending moment for the fifth-story column is $3300 \times 6 = 19{,}800$ ft.-lb., and that for the sixth-story column is $2400 \times 6 = 14{,}400$ ft.-lb. The direct thrust on the floor girder is $6000 - 900 = 5100$.

Proceeding to the second joint, sketched in Fig. 4: The difference between 3075 and 1650 = 1425 acts as a shear in the girder between Cols. *B* and *C*. This added to the 4275 shear continued from the girder between *A* and *B* makes a total shear of 5700 in the girder. The equation of moments is

$$(\tfrac{8}{11} X \times 12) + (\tfrac{3}{11} X \times 6) = (4275 \times 8) + (5700 \times 8)$$

From which $X = 7700$; $\tfrac{8}{11} X = 5600$, and $\tfrac{3}{11} X =$



FIGS. 3-6. SIXTH-FLOOR JOINTS OF BUILDING-FRAME, WITH STRESSES CORRESPONDING TO METHOD I

2100, are the shears taken by Col. B to hold the joint in equilibrium.

The bending moment M_2 of the girder from A to B at Col. B is the same as at Col. A with an opposite sign; M_3, of the girder from B to C, is $5700 \times 8 = 45{,}600$ ft.-lb. The bending moment of the fifth-story column is $7700 \times 6 = 46{,}200$ ft.-lb., and that of the sixth-story column is $5600 \times 6 = 33{,}600$ ft.-lb. The direct thrust on the girder between B and C is $5100 - 2100 = 3000$.

At the third joint, Fig. 5, the shear taken by the girder between C and D is $5700 - (3075 - 1650) = 4275$. From the equation of moments

$$(\tfrac{8}{11} X \times 12) + (\tfrac{8}{11} X \times 6) = (5700 \times 8) + (4275 \times 8)$$

whence

$$X = 7700, \ \tfrac{8}{11} X = 5600, \ \tfrac{3}{11} X = 2100$$

As expected, the moments in Col. C are numerically equal to those in Col. B, and the girder moments $M_4 = M_3$, and $M_5 = M_2$. The compression in the floor girder between C and D is $3000 - 2100 = 900$.

At the fourth joint, Fig. 6, we have

$$(\tfrac{8}{11} X \times 12) + (\tfrac{8}{11} X \times 6) = (4275 \times 8)$$

the same equation as at the first joint, and hence the same numerical values for moments and shears.

The designer in following this method for the various floors will find many short-cuts. A relationship between the floors can soon be established. If the distances between columns are not even spaces, or the columns have different sectional areas, the direct stresses vary both in proportion to their distances from the neutral axis and their sectional areas. It will be necessary to first find the neutral axis of the cross-section in question and then the direct stresses. With these the shears and bending mc ments can be obtained.

Method II

This may be called the Method of Equal Shears. It is assumed that the horizontal shear on any plane is equally distributed among the columns cut by that plane. The stresses and maximum bending moments for a cross-section of the building are as given in Fig. 7.

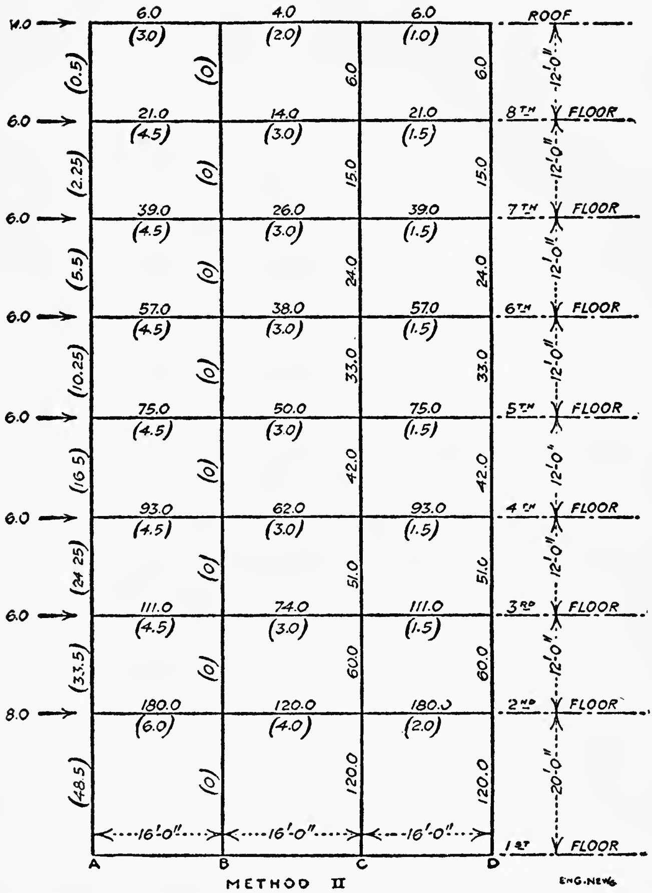


FIG. 7. RECTANGULAR BUILDING-FRAME: DIRECT AND BENDING STRESSES CALCULATED BY APPROXIMATE METHOD II

120.0 = Bending moment of 120,000 ft.-lb.
(4.0) = Direct stress of 4000 lb.

Taking any aisle we find the direct stress in the fifth-story columns to be $\left(\frac{4000}{3} \times 42\right) + \left(\frac{6000}{3} \times 30\right) + \left(\frac{6000}{3} \times 18\right) + \left(\frac{6000}{3} \times 6\right) \div 16 = 10{,}250$

The direct stresses coming upon any interior column from the adjacent aisles are equal in amount but opposite in direction. Hence their algebraic sum is zero

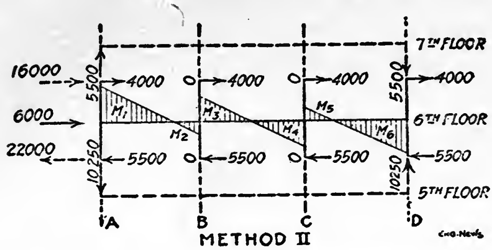


FIG. 8. COLUMN-SHEARS AND GIRDER MOMENTS AT SIXTH FLOOR, CALCULATED BY METHOD II

and only the outside columns have direct stresses. This may be found directly for any story, say the sixth,

$$(4000 \times 30) + (6000 \times 18) + (6000 \times 6)$$
$$\text{divided by } 48 = 5500$$

Considering in detail, as in Method I, the sixth floor, we have in Fig. 8 the direct stresses and shears in the columns.

The shear in each girder is 10,250 — 5500 = 4750. The equations for bending moments in the girders can be written as follows:

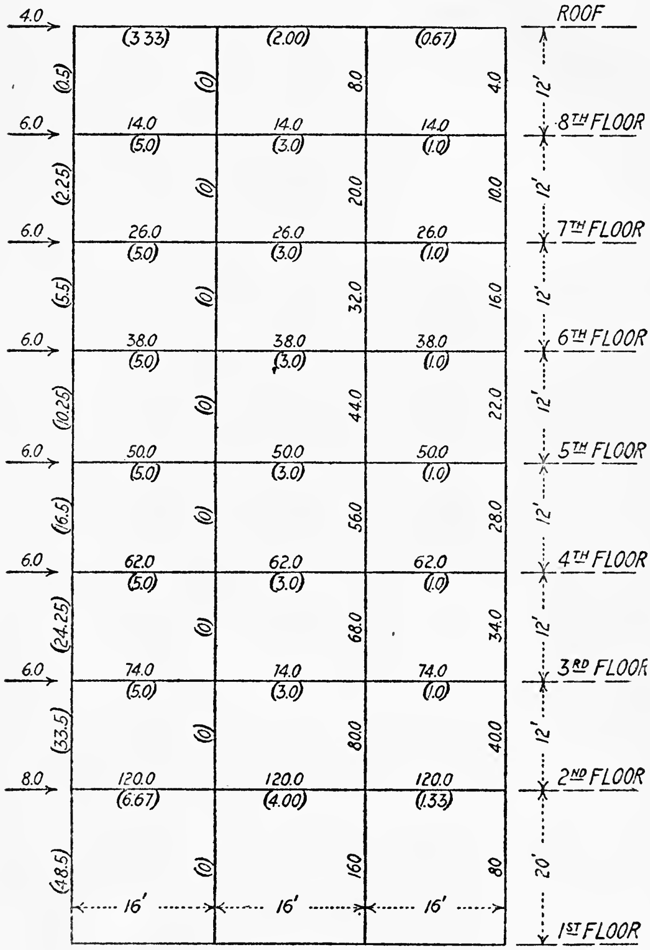
$$
\begin{aligned}
M_1 &= [(4000 \times 6) \times (5500 \times 6)] &&= +57{,}000 \text{ ft.-lb.}\\
M_2 &= [(4000 \times 6) + (5500 \times 6)] - [(10{,}250 - 5500) \times 16] &&= -19{,}000 \text{ ft.-lb.}\\
M_3 &= [2(4000 \times 6) + (5500 \times 6)] - [(10{,}250 - 5500) \times 16] &&= +38{,}000 \text{ ft.-lb.}\\
M_4 &= [2(4000 \times 6) + (5500 \times 6)] - [(10{,}250 - 5500) \times 32] &&= -38{,}000 \text{ ft.-lb.}\\
M_5 &= [3(4000 \times 6) + (5500 \times 6)] - [(10{,}250 - 5500) \times 32] &&= +19{,}000 \text{ ft.-lb.}\\
M_6 &= [3(4000 \times 6) + (5500 \times 6)] - [(10{,}250 - 5500) \times 48] &&= -57{,}000 \text{ ft.-lb.}
\end{aligned}
$$

The bending moment at the sixth-floor girder of each sixth-story column is 4000 × 6 = 24,000 ft.-lb., and of each fifth-story column is 5500 × 6 = 33,000 ft.-lb.

The compression in the floor girders is 6000 — 1500 = 4500 between Cols. *A* and *B*, 4500 — 1500 = 3000 between *B* and *C*, and 3000 — 1500 = 1500 between *C* and *D*. General equations can easily be deduced which will simplify the calculation of stresses and moments for other floors. If the spaces between columns are unequal, the direct stresses from adjacent aisles will be unequal. This difference is a direct stress in the column between the two aisles considered. If the columns have different sectional areas, the horizontal shear taken by each column will be in proportion to its moment of inertia.

Method II-A

This is a special case of Method II and may be called



METHOD II-A

Fig. 9. Rectangular Building-Frame: Direct and Bending Stresses Calculated by Approximate Method II—A

120.0 = Bending moment of 120,000 ft.-lb.
(4.0) = Direct stress of 4000 lb.

the Portal Method. The structure is regarded as equivalent to a series of independent portals. The total horizontal shear on any plane is divided by the number of aisles instead of by the number of columns as in II. An outer column thus takes but one-half the shear of an interior column. The stresses and maximum bending moments for a cross-section of the building are as given in Fig. 9.

For equal spacing the direct or vertical axial stress due to the overturning moment of the wind is all taken by the outside columns and is the same in amount as in Method II.

Considering in detail the sixth floor, we have in Fig. 10 the direct stresses and shears in the columns.

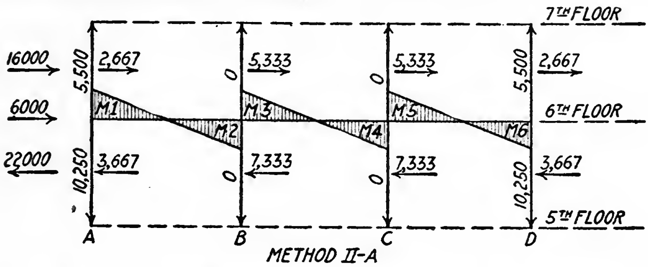


Fig. 10. Column-Shears and Girder Moments at Sixth Floor, Calculated by Method II-A

The shear in each girder is 10,250 — 5500 = 4750. The equations for bending moments in the girders are as follows:

$$
\begin{aligned}
M_1 &= [(2{,}667 \times 6) + (3{,}667 \times 6)] = && +38{,}000 \\
M_2 &= [(2{,}667 \times 6) + (3{,}667 \times 6) - (10{,}250 - 5{,}500) \times 16] = && -38{,}000 \\
M_3 &= [(2{,}667 \times 6) + (3{,}667 \times 6) + (5{,}333 \times 6) + (7{,}333 \times 6) \\
&\qquad - (10{,}250 - 5{,}500) \times 16] = && +38{,}000 \\
M_4 &= [(2{,}667 \times 6) + (3{,}667 \times 6) + (5{,}333 \times 6) + (7{,}333 \times 6) \\
&\qquad - (10{,}250 - 5{,}500) \times 32] = && -38{,}000 \\
M_5 &= [(2{,}667 \times 6) + (3{,}667 \times 6) + 2(5{,}333 \times 6) + 2(7{,}333 \times 6) \\
&\qquad - (10{,}250 - 5{,}500) \times 32] = && +38{,}000 \\
M_6 &= [(2{,}667 \times 6) + (3{,}667 \times 6) + 2(5{,}333 \times 6) + 2(7{,}333 \times 6) \\
&\qquad - (10{,}250 - 5{,}500) \times 48] = && -38{,}000
\end{aligned}
$$

The bending moment at the sixth-floor girder of each outer sixth-story column is 2667 × 6 = 16,000 ft.-lb., and of each inner sixth-story column is 5333 × 6 = 32,000 ft.-lb. At the fifth-floor girder the bending moment of each fifth-story outer column is 3667 × 6 = 22,000 ft.-lb. and of each fifth-story inner column is 7333 × 6 = 44,000 ft.-lb.

The compression in the floor girders is 6000 — 1000 = 5000 between Cols. A and B, 5000 — 2000 = 3000 between B and C, and 3000 — 2000 = 1000 between C and D.

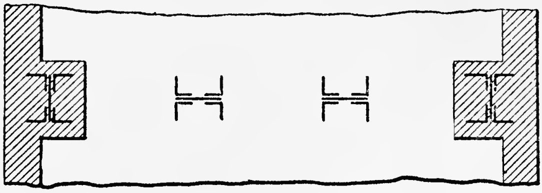


FIG. 11. CROSS-SECTION OF COLUMNS IN TRANSVERSE BENT

It is noted from the above that the bending moment in an outer column is one-half that in an interior column; that the point of contraflexure of each girder is at its center; and the bending moments due to wind for all girders of any transverse bent on the same floor are alike. This is an ideal condition for the detailer and the shop. The designer finds this method very simple and his work easily checked. The bending moment in a girder is the mean between the bending moments in the interior column above and below the girder. The width of the aisle does not affect the value of the bending moment.*

Methods I and II-A are specially adapted to transverse bents when the columns are turned as in Fig. 11; also when the outer columns carry floor loads only and the stresses are but one-half those of the inner columns.

METHOD III

This may be called the Continuous Portal Method. The direct stresses in the columns are assumed to vary as their distances from the neutral axis, and the horizontal shear on any plane is equally distributed among the columns cut by that plane. Stresses and maximum bending moments for a cross-section of the building are as given in Fig. 12.

The direct stresses in the columns are found the same way and are the same in amount as in Method I.

---

*Burt, "Steel Construction" Section, Wind Bracing.

Considering in detail the sixth floor, we have in Fig. 13 the direct stresses and shears in the columns. The shear in the girder A to B and the girder C to D is 9225 — 4950 = 4275. The shear in the girder B to C is (9225 — 4950) + (3075 — 1650) = 5700.

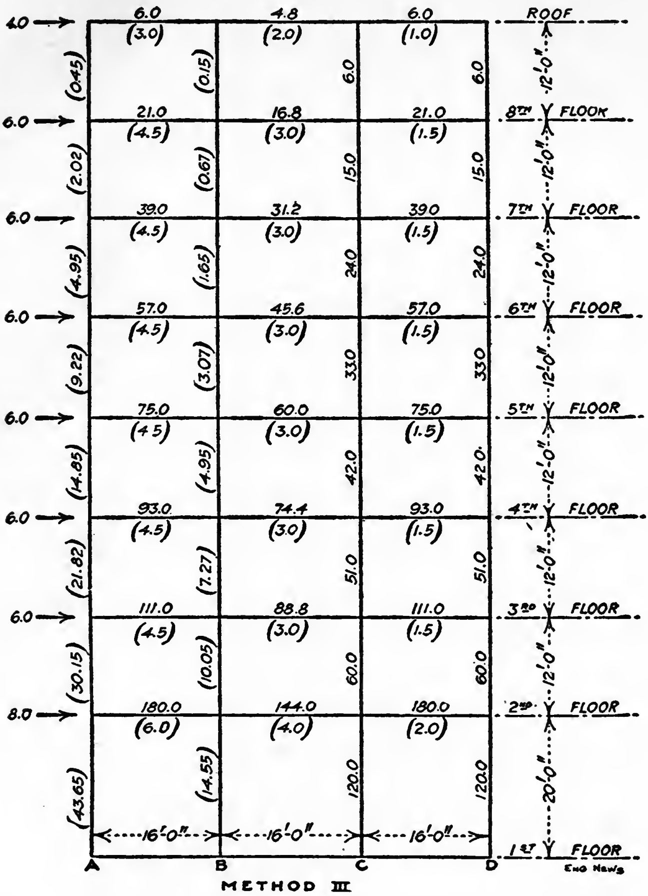


FIG. 12. RECTANGULAR BUILDING-FRAME: DIRECT AND BENDING STRESSES CALCULATED BY APPROXIMATE METHOD III

144.0 = Bending moment of 144,000 ft.-lb.
(4.0) = Direct stress of 4000 lb.

The equations for bending moments in the girders can be written as follows:

M_1 = [(4000 × 6) + (5500 × 6) = +57,000 ft.-lb.
M_2 = [(4000 × 6) + (5500 × 6)] — [(9225 — 4950) × 16] = —11,400 ft.-lb.
M_3 = 2[(4000 × 6) + (5500 × 6)] — [(9225 — 4950) × 16] = +45,600 ft.-lb.
M_4 = 2[(4000 × 6) + (5500 × 6)] — [(9225 — 4950) × 32] — [(3075 — 1650) × 16] = —45,600 ft.-lb.
M_5 = 3[(4000 × 6) + (5500 × 6)] — [(9225 — 4950) × 32] — [(3075 — 1650) × 16] = + 11,400 ft.-lb.
M_6 = 3[(4000 × 6) + (5500 × 6)] — [(9225 — 4950) × 48] — [(3075 — 1650) × 32] + [(3075 — 1650) × 16] = —57,000 ft.-lb.

The bending moment at the six-floor girder of each sixth-story column is 4000 × 6 = 24,000 ft.-lb., and of

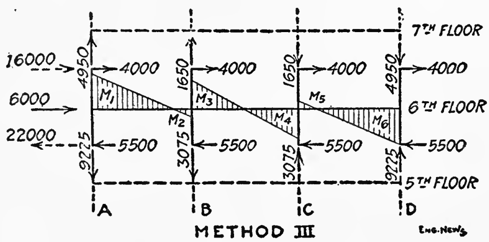


FIG. 13. COLUMN-SHEARS AND GIRDER MOMENTS AT SIXTH FLOOR, CALCULATED BY METHOD III

each fifth-story column is 5500 × 6 = 33,000 ft.-lb. The compression in the floor girders is 6000 — 1500 = 4500 lb. between Cols. *A* and *B*, 4500 — 1500 = 3000 between *B* and *C*, and 3000 — 1500 = 1500 between *C* and *D*.

If the columns are unequally spaced or their sectional

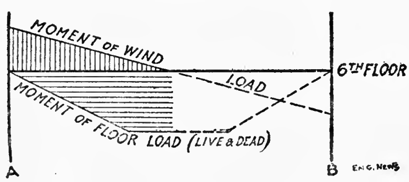


FIG. 14. GRAPHICAL COMBINATION OF MOMENTS FROM VERTICAL LOADS AND WIND LOADS IN FLOOR GIRDER

areas are different, the location of the neutral axis must first be found. The direct stresses in the columns will vary both as their distances from the neutral axis and their sectional areas. The horizontal shears taken by the columns will vary as their moments of inertia.

Conclusion

It can be said of each of the above methods of calculating wind stresses that it is easily workable; and to quote Prof. W. H. Burr: "So long as the stresses found by one legitimate method of analysis are provided for, the stability of the structure is assured." At the present time Method I is probably more used than any of the others, though Methods II and II-A have been used quite extensively. In the 36-story Equitable Building of New York City, the largest office building in the world, Method I was followed. In its near neighbor, the 32-story Adams Express Building, Method II-A was used. Method III is found in some text-books; it has been used but little about New York, and only to a limited extent elsewhere. The writer personally prefers Method I, though during the past ten years he has used I, II, and II-A. In a 20-story building in Philadelphia built in 1914-1915 he used I. In an 18-story building in Atlanta, designed in 1912, he used II-A. To Method III he objects not only because of its practical limitations but because in theory it seems farther from the truth than any of the others—especially when it comes to distributing the shear for bents in buildings more than four aisles wide.

The practice of the writer in calculating wind stresses, using Methods I or II-A (preferably I), is first to find the distance of each column from the neutral axis of the transverse bent to which it belongs, and then to assume the moments of inertia of the inner columns in that bent to be the same and of the outer columns to be one-half that of the inner. The columns are proportioned for all stresses coming upon them, including both direct and cross-bending due to wind. It is seldom that corrections are made for moments of inertia that differ from the assumptions.

It is often convenient to assume the wind loads on the basis of using the same unit-stresses as for live- and dead-loads. A number of building codes call for a horizontal wind pressure of 30 lb. per sq.ft. and allow unit-stresses to be increased 50% for wind-bracing. A wind load of 30 lb. per sq.ft. with unit-stress of 24,000 lb. per sq.in. is equivalent to a load of 20 lb. per sq.ft. with a unit-stress of 16,000 lb. per sq.in.—the working stress generally used for live- and dead-loads. The diagrams of moments for any floor girder can easily be combined in one figure (see Fig. 14), and the total moment at any point read by scaling. Fig. 14 is drawn for beam with ends supported. If the ends were considered fixed the beam would be restrained and the diagram for both wind and floor loads would show smaller bending moments. Any saving thus made is doubtful economy as in actual practice it is uncertain to what extent the beams are fixed (under vertical load).

The building should be examined for wind in a longitudinal direction as well as transversely and calculations made if necessary. This is a simple thing to do but in some marked instances it has been neglected.

Special attention should be given the column splices, and the connection of floor girders to columns. It is folly to add material to columns or floor girders to meet stresses and moments due to wind, and then neglect their connections. Care should be taken that in all cases the connections are made strong enough for the bending moments coming upon them. Many buildings have main members sufficient to meet wind stresses without efficient connections. In such cases it matters little what particular theory of wind distribution had been adopted.