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THE

THEORY OF STRAINS

IN

GIRDERS AND SIMILAR STRUCTURES

WITH

OBSERVATIONS ON THE APPLICATION OF THEORY TO PRACTICE

AND

TABLES OF THE STRENGTH AND OTHER PROPERTIES OF MATERIALS.

BY

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Prius quàm incipias, consulto; et ubi consulueris, mature facto opus est.

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CHAPTER XIII.

COMPRESSION AND PILLARS.

279. Compressive strain.—In most of the foregoing theoretic investigations it has been tacitly assumed that the tensile or compressive strength of any material is proportional to its sectional area, whatever that may be. This, however, is not always true of compressive strains, and one of the first difficulties which the student encounters, when seeking to reduce theory to practice, is the necessity of providing in struts or pillars not only against absolute crushing of the material, which in reality rarely occurs, but more especially against flexure and buckling, to resist which a greater amount of material is generally required than theory alone might seem to indicate. To understand the matter clearly we must recollect that the mode in which a pillar fails varies greatly, according as it is long or short in proportion to its diameter. A very short pillar—a cube, for instance, of wrought-iron, timber, or stone—will bear a weight nearly sufficient to upset, to splinter, or to crush it into powder; while a still shorter pillar—such as a penny, or other thin plate of ductile metal—will often bear an enormous weight, far exceeding that which the cube will sustain, the interior of the thin plate being prevented from escaping from beneath the pressure by the surrounding particles. Alluding to his experiments on copper, brass, tin, and lead, Mr. Rennie observes:—"When compressed beyond a certain thickness, the resistance becomes enormous." * We can thus conceive how stone or other materials in the interior of the globe withstand pressures that would crush them into powder at the surface, merely because there is no room for the particles to escape from the surrounding pressure. A long thin pillar on the contrary, such as a walking cane, will yield by flexure long before it is crushed; and if the bending be carried so far as to break the pillar, the fracture will resemble that due to transverse strain. Hence it is convenient to subdivide the results of compressive strain into flexure and crushing.

280. Flexure—Crushing—Buckling—Bulging—Splintering. —*Flexure* is the bending or deflection of a pillar whose length is very considerable in proportion to its thickness or diameter.

Crushing may be subdivided into buckling, bulging, and splintering.

(*a.*) *Buckling* is the undulation, wrinkling, or crumpling up, usually of a thin plate of a malleable material. Buckling is frequently preceded by flexure; when, for instance, long tubes of plate-iron are compressed longitudinally, they first deflect, and finally fail by the buckling or puckering of a short piece on the concave side.

(*b.*) *Bulging* is the upsetting or spreading out under pressure of ductile or fibrous materials, like lead, wrought-iron and timber.

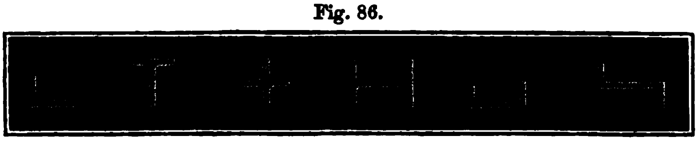
(*c.*) *Splintering* is the splitting off in fragments of crystalline, fibrous, or granular materials, such as cast-iron, timber, stone and brick; the splintering of granular and vitreous materials is generally abrupt and terminates in their being crushed to powder, while most crystalline metals are semi-ductile and therefore bulge slightly before they splinter, and flatten out when reduced to small fragments.

281. Crushing strength.—It has been found by experiment that the strength of short pillars of any given material, all having the same diameter, does not vary much, provided the length of the pillar is not less than one and a half, and does not exceed four or five diameters; and the weight which will just crush a short prism whose base equals one square unit (generally a square inch), and

* *Phil. Trans.*, 1818, p. 126.

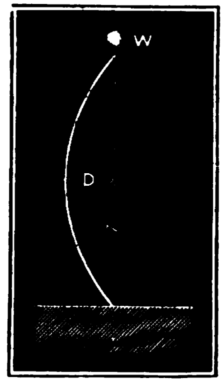
whose height is not less than one and a half, and does not exceed four or five diameters, is called the *crushing strength* of the material experimented upon. If the length of pillars never exceeded four or five diameters, all we need do to arrive at the strength of any given pillar would be to multiply its transverse area in square units by the tabulated crushing strength of that particular material. It rarely happens, however, that pillars are so short in proportion to their length, and hence we must seek some other rule for calculating their strength when they fail, not by actual crushing, but by flexure. If we could insure the line of thrust always coinciding with the axis of the pillar, then the amount of material required to resist crushing merely would suffice, whatever might be the ratio of length to diameter. But practically it is impossible to command this, and a slight error in the line of thrust produces a corresponding tendency in the pillar to bend. With tension-rods, on the contrary, the greater the strain the more closely will the rod assume a straight line, and, in designing their cross section, it is only necessary to allow so much material as will resist the tensile strain. This tendency to bend renders it necessary to construct long pillars, not merely with sufficient material to resist crushing, supposing them to fail from that alone, but also with such additional material or bracing as may effectually preserve them from yielding by flexure. In masonry, heavy timber framing, or similar massive structures, the desired effect is produced by mere bulk of material, which insures the line of thrust always lying at a safe distance within the limits of the structure. In hollow pillars the same result is attained by removing the material to a considerable distance from the line of thrust, which, though it may deviate slightly from the axis of the pillar, yet will not pass beyond its circumference. When the pillar is neither tubular nor solid, one of the forms of section represented in Fig. 86 is generally adopted.

Fig. 86.



282. Very long thin pillars.—The law which determines the flexure of very long thin pillars may be investigated as follows:—Let Fig. 87 represent a pillar of uniform section throughout, not fixed at the ends, very long in proportion to its breadth, and just on the point of failing from flexure.

Fig. 87.



Let W = the deflecting weight,
b = the breadth of the pillar,
d = its diameter or least lateral dimension,
l = its length,
D = the central deflection,
M = the moment of rupture (58),
f = the longitudinal unit-strain in the extreme fibres in a horizontal section across the middle of the pillar,
λ = the difference in length between the convex and the concave edges of the pillar,
C = the resultant of all the longitudinal forces of compression in the concave side at the plane of section,
T = the resultant of all the longitudinal forces of tension in the convex side at the plane of section,
E = the coefficient of elasticity.

The upper half of the pillar is held in equilibrium by three sets of vertical forces—viz., the weight, acting in the chord-line of the curve; the longitudinal tensile strains in the convex side at the middle section; and the longitudinal compressive strains in the concave side, also at the middle section.

When the pillar is very long in proportion to its width, and the deflection therefore considerable even though the curvature be small,* we may assume D equal to the distance from the chord-line to either the centre of tensile or the centre of compressive

* Mr. Hodgkinson's experiments show that this investigation is not applicable to pillars whose length is less than thirty diameters if cast-iron, or sixty diameters if wrought-iron; even with such short pillars it requires certain modification, as will be seen hereafter.

strains (56). Taking moments round either of these points indifferently, we have

$$WD = M \text{ nearly.} \tag{a}$$

Again, assuming that the deflection curve is a circle, from which it can differ but slightly, we have from eq. (130),

$$D = \frac{\lambda l}{8d} \text{ nearly,} \tag{b}$$

whence, by substitution in eq. (*a*), we have

$$W = \frac{8dM}{\lambda l} \tag{c}$$

Further, recollecting that λ is equal to the contraction of the concave, plus the extension of the convex edge, we have from eq. (2),

$$\lambda = \frac{2fl}{E}$$

Substituting this in eq. (*c*), we have

$$W = \frac{4dEM}{fl^2} \tag{208}$$

Replacing M by its values in (70) and the following sections, and recollecting that the ratio $\frac{d}{f}$ in eq. (208) is equal to the ratio $\frac{2c}{f}$ in the 45th and following eqs., we obtain the following values for the strength of long pillars of various sections:—

288. Solid rectangular pillars—Solid round pillars—Hollow round pillars—Strength of long pillars depends on the coefficient of elasticity.—From equations (45) and (208) we have for long solid rectangular pillars,

$$W = \frac{2Ebd^3}{3l^2} \tag{209}$$

where d = the least lateral dimension.

From equations (47) and (208) we have for long solid round pillars,

$$W = \frac{\pi Ed^4}{8l^2} \tag{210}$$

where d = the diameter of the pillar.

From equations (48) and (208) we have for long hollow round pillars,

$$W = \frac{\pi E(d^4 - d_1^4)}{8l^2} \tag{211}$$

where $d =$ the external diameter,

$d_1 =$ the internal diameter

These equations prove that the strength of long square or round pillars varies as the fourth power of their diameter divided by the square of their length, and the longer the pillar is in proportion to its diameter, the closer will these equations represent the truth; in such pillars the neutral surface will not lie far from the axis, and the deflecting weight, W, will be small compared to that which would crush a very short pillar of the same diameter.

It is also to be observed that the strength of very long pillars depends, not on the strength of the material, but on E, which represents its stiffness and capability of resisting flexure. This theoretic result agrees with the fact that, although a short pillar of cast-iron will bear a much greater weight than a similar pillar of wrought-iron, as the crushing strength of cast-iron is from two to three times greater than that of wrought-iron, yet a very long wrought-iron pillar will support a greater weight than a similar one of cast-iron, as the coefficient of elasticity of wrought-iron is considerably higher than that of cast-iron.

284. Strengths of long similar pillars are as their transverse areas—Weights of long pillars of equal strength but different lengths are as the squares of their lengths.—These equations also prove that the strengths of similar long pillars are as the squares of any lineal dimension, that is, as their transverse areas (**201**); while their weights are as the cubes of any lineal dimension. Further, if the strengths of long pillars of similar section remain constant while their lengths vary, their transverse areas will vary as their lengths, and their weights therefore will vary as the squares of their lengths.

285. Weight which will deflect a very long pillar is very near the breaking weight.—It appears from eq. (b) that, if a very long pillar be bent in different degrees, D will vary as λ, that is, as f (7); and, from eq. (a), $W = \frac{M}{D}$ which is constant since M also varies as f; hence it follows that, W, the weight which keeps the pillar bent, is nearly the same whether the flexure be greater or less. This statement would be accurately true were it not that the assumptions on which eqs. (a) and (b) are based and the law of elasticity are only approximate. It will, however, agree very closely with experiment when the pillar is long enough to allow D to be considerable, even though the curvature be small. From this it follows that any weight which produces moderate flexure in a very long pillar will also be very near the breaking weight, as a trifling additional load will bend the pillar very much more, and strain the fibres beyond what they can bear. This theoretic result is in accordance with the following observation of Mr. Hodgkinson:—"From the first experiment on long hollow pillars with rounded ends, it was evident that so little flexure of the pillar was necessary to overcome its greatest resistance (and beyond this a smaller weight would have broken it), that the elasticity of the pillars was very little injured by the pressure, if the weight was prevented from acting upon the pillar after it began to sink rapidly, through its greatest resistance being overcome."—*Phil. Trans.*, 1840, p. 411.

As all the longitudinal forces at the middle of the pillar balance, we have the following equation:—

$$C = T + W.$$

This enables us to predict how a long pillar will fail, whether by the convex side tearing asunder, or by the concave side crushing. A wrought-iron pillar, for instance, may be expected to fail on the concave side, as its power to resist compression is less than that to resist extension. A long pillar of cast-iron, on the contrary, will probably fail by the convex side tearing asunder, as the compressive strength of cast-iron greatly exceeds its tenacity. This is corroborated by Mr. Hodgkinson's experiments on long hollow cast-iron pillars which "seldom gave way by compression."—*Phil. Trans.*, 1840, p. 409.

286. Hodgkinson's laws—Three classes of pillars.—Our knowledge of the laws of the resistance of pillars to flexure, though perhaps not so satisfactory in a theoretic point of view as might be desired, is, however, owing to Mr. Hodgkinson's able investigations, aided by the liberality of Mr. Fairbairn, Mr. R. Stephenson and the Royal Society, practically far enough advanced to enable us to predict with considerable accuracy the strength of pillars of the usual forms. The results of these investigations are here given; the reader who desires more detailed information respecting the experiments, is referred to Mr. Hodgkinson's original papers.*

Mr. Hodgkinson divides pillars into three classes:—

1°. *Short pillars* whose length (if cast-iron, under four or five diameters) is so small compared with their diameter that they fail by actual crushing of the material, not by flexure.

2°. *Long flexible pillars* whose length is so great (if cast-iron, thirty diameters and upwards when both ends are flat, fifteen diameters and upwards when both ends are rounded,) that they fail by flexure, like girders subject to transverse strain, the breaking weight being far short of that required to crush the material when in short pieces.

3°. *Medium or short flexible pillars* whose length is such that, though they deflect, yet the breaking weight is a considerable portion of that required to crush *short* pillars. This class includes all pillars which are intermediate in length between those in the first two classes, and they may be said to fail partly by flexure and partly by crushing.

In the following remarks the passages in inverted commas are verbatim extracts from Mr. Hodgkinson's writings.

CLASS I.—SHORT PILLARS WHICH FAIL BY CRUSHING; LENGTH UNDER FOUR OR FIVE DIAMETERS.

387. Pillars whose height is less than their diameter.— When the lateral dimensions of a prism exceed its height, the strength

* See *Report of the British Association*, Vol. vi.—*Philosophical Transactions:* 1840, p. 385; and 1857, p. 851.—*Experimental Researches on the strength and other properties of Cast-Iron.* By E. Hodgkinson, F.R.S. London, 1846.—*Report of the Commissioners appointed to inquire into the application of Iron to Railway Structures*, 1849.

to resist crushing is somewhat indefinite, and "shorter specimens generally bear more than larger ones of the same diameter, or dimensions of base. In the shortest specimens fracture takes place by the middle becoming flattened and increased in breadth (*bulged*), so as to burst the surrounding parts, and cause them to be crumbled and broken in pieces. This is usually the case when the lateral dimensions of the prism are large compared with the height."—*Exp. Res.*, p. 319.

288. Pillars whose height is from one to four or five times the diameter—Angle of fracture—Crushing strength of short pillars equals their transverse area multiplied by the crushing unit-strain.—When the height of a solid prism is equal to or greater than its diameter—but not so great that it yields by flexure (this height for cast-iron varies from one and a-half to four or five times the least lateral dimension)—"fracture is caused by the body becoming divided diagonally in one or more directions. In this case the prism, in cast-iron at least, either does not bend before fracture, or bends very slightly; and therefore the fracture takes place by the two ends of the prism forming cones or pyramids, which split the sides and throw them out; or, as is more generally the case in cylindrical specimens, by a wedge sliding off, starting at one of the ends, and having the whole end for its base; this wedge being at an angle which is constant in the same material, though different in different materials (see Plate II.). In cast-iron the angle is such that the height of the wedge is somewhat less than $\frac{3}{4}$ of the diameter." "In timber, like as in iron and crystalline bodies generally, crushing takes place by wedges sliding off in angles with their base which may be considered constant in the same material: hence the strength to resist crushing will be as the area of fracture, and consequently as the direct transverse area; since the area of fracture would, in the same material, always be equal to the direct transverse area, multiplied by a constant quantity."—*Exp. Res.*, pp. 319, 323. In other words, eq. (1) is applicable to short pillars, and their crushing strength is equal to their transverse section multiplied by the crushing unit-strain of the material.

289. Pillars whose height exceeds four or five diameters.—If the length exceeds four or five times the diameter, "the body bends with the pressure, and though it may break by sliding off as before, the strength is much decreased. In cases where the length is much greater than as above, the body breaks across, as if bent by a transverse pressure."—*Exp. Res.*, p. 321.

290. Crushing unit-strain.—From the foregoing observations the reader will perceive that the crushing unit-strain of any material should be derived from experiments on prisms whose height is not less than the length of the wedge, nor so great that the prism will deflect. Mr. Hodgkinson seems to have preferred prisms whose height equalled two diameters, and in Table IV. (**208**), it will be seen that prisms of cast-iron whose height equalled one diameter generally bore more than those whose height equalled two diameters. If, however, the material, like glass and some limestones, do not form wedge-shaped but longitudinal splinters (**281, 282**), it seems probable that, within considerable limits, the height of the specimen will not affect its crushing strength. Experimenters on stone have generally used cubes; Mr. Hodgkinson's practice, however, seems preferable.

CLASS II.—LONG PILLARS WHICH FAIL BY FLEXURE; LENGTH, IF BOTH ENDS ARE FLAT AND FIRMLY BEDDED, EXCEEDING 30 DIAMETERS FOR CAST-IRON AND TIMBER, AND 60 DIAMETERS FOR WROUGHT-IRON; IF BOTH ENDS ARE ROUNDED, OF ONE-HALF THESE LENGTHS.

291. Long pillars with flat ends firmly bedded three times stronger than pillars with round ends.—"In all long pillars of the same dimensions, the resistance to fracture by flexure is about three times greater when the ends of the pillars are flat and firmly bedded, than when they are rounded (*like the end of an egg*) and capable of turning."—*Exp. Res.*, p. 332.

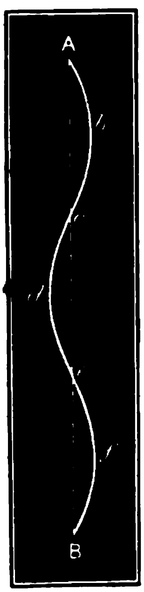
292. Strength of Pillars with one end round and the other flat a mean between that of pillars with both ends rounded

and with both ends flat.—" The strength of a pillar, with one end round and the other flat, is the arithmetical mean between that of a pillar of the same dimensions with both ends rounded, and with both ends flat. Thus, of three (*long*) cylindrical pillars, all of the same length and diameter, the first having its ends rounded, the second with one end rounded and one flat, and the third with both ends flat, the strengths are as 1, 2, 3, nearly."—*Exp. Res.*, p. 332.

298. A long pillar with ends firmly fixed as strong as a pillar of half the length with round ends.—" A long uniform pillar, with its ends firmly fixed, whether by discs or otherwise, has the same power to resist breaking as a pillar of the same diameter, and half the length, with the ends rounded or turned so that the force would pass through the axis."—*Exp. Res.*, p. 332.

Of this fact Mr. Hodgkinson offers the following explanation:—" Suppose a long uniform bar of cast-iron were bent by a pressure at its ends so as to take the form A*bcdef*B, where all the curves A*bc*, *cde*, *ef*B, separated by the straight line A*ce*B, would be equal, since the bar was supposed to be uniform. The curve having taken this form, suppose it to be rendered immoveable at the points *b* and *f*, by some firm fixings at those points. This done, it is evident we may remove the parts near to A and B, without at all altering the curve *bcdef* of the part of the pillar between *b* and *f*, and consider only that part. The part *bf*, which alone we shall have to consider, will be equally bent at all the points *b*, *d*, *f*. The points *c* and *e* too are points of contrary flexure, consequently the pillar is not bent in them. These points are unconstrained except by the pressure which forces them together, and the pillar might be reduced to any degree in them, provided they were not crushed or detruded by the compressing force. These points may then be conceived as acting like the rounded ends of the pillars, and the part *cde* of the pillar, with its ends *c* and *e* rounded, will be bearing the same weight as the whole

Fig. 88.



pillar *bcdef* of double the length with its ends *b*,*f* firmly fixed."—*Phil. Trans.*, 1857, p. 855.

294. Hodgkinson's laws apply to cast-iron, steel, wrought-iron, and wood.—" The preceding properties were found to exist in long pillars of steel, wrought-iron, and wood," as well as cast-iron.—*Exp. Res.*, p. 333. They apply to pillars whose length is so great in proportion to their diameter that the breaking unit-strain of the pillar is far short (for cast-iron not exceeding one-fourth) of the crushing unit-strain of the material.—*Exp. Res.*, p. 341.

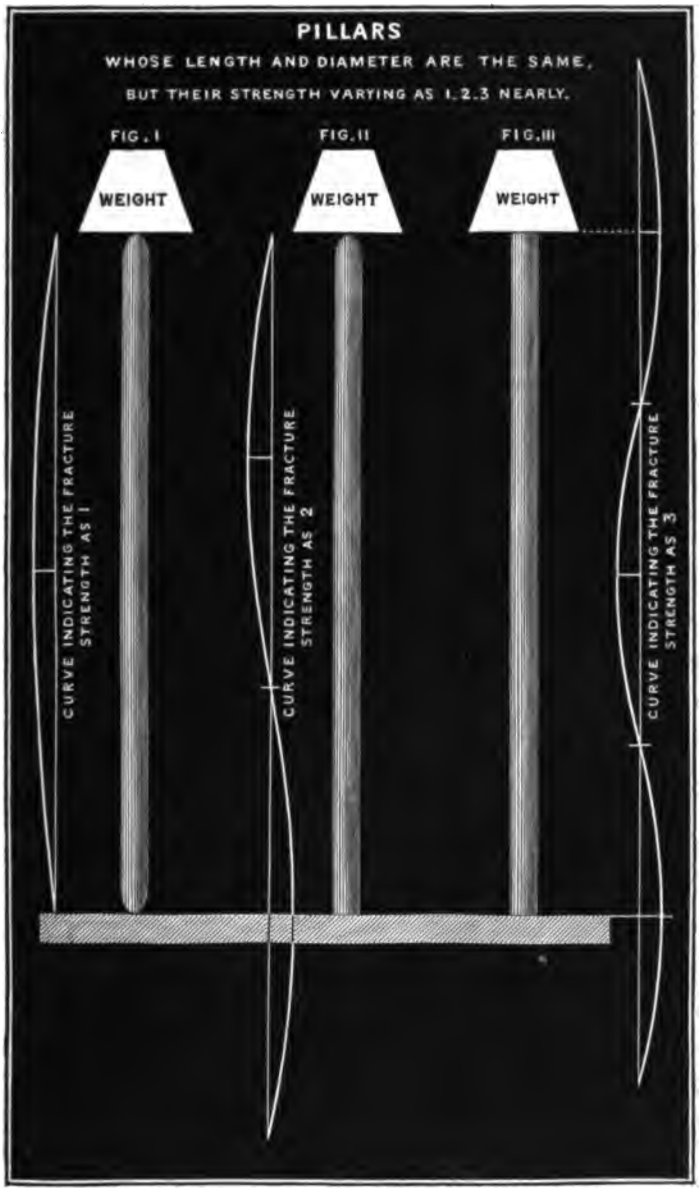
295. Discs on the ends add but little to the strength of flat-ended pillars.—Cast-iron pillars with discs on their ends are somewhat stronger than those with merely flat ends, but the difference of strength is trifling.—*Phil. Trans.*, 1840, p. 391.

296. Position of fracture.—Long uniform cast-iron pillars with both ends rounded break in one place only—the middle; those with both ends flat in three—at the middle and near each end; those with one end rounded and one flat, at about one-third of the distance from the rounded end. Plate III. represents the curves indicating the form of flexure in each class of pillar.—*Phil. Trans.*, 1857, p. 858.

297. Enlarging diameter in the middle of solid pillars with rounded ends increases their strength slightly.—" In all the (*cast-iron*) pillars with rounded ends, those with increased middles were stronger than uniform pillars of the same weight, the increase being about one-seventh of the weight borne by the former." This increase of strength was more marked in pillars with rounded ends than in those with discs, for " in the pillars with discs, those with the middle but little increased had no advantage, with regard to strength, over the uniform ones. But the pillars with the middle diameter half as great again as the end ones bore from one-eighth to one-ninth more than uniform pillars of the same weight with discs upon the ends."—*Phil. Trans.*, 1840, p. 395.

298. Enlarging diameter of hollow pillars in the middle or at one end does not increase their strength.—" In hollow (*cast-iron*) pillars of greater diameter at one end than the other,

PLATE III.


PILLARS
WHOSE LENGTH AND DIAMETER ARE THE SAME,
BUT THEIR STRENGTH VARYING AS 1.2.3 NEARLY.
FIG. I
FIG. II
FIG. III
WEIGHT
WEIGHT
WEIGHT
CURVE INDICATING THE FRACTURE STRENGTH AS 1
CURVE INDICATING THE FRACTURE STRENGTH AS 2
CURVE INDICATING THE FRACTURE STRENGTH AS 3

or in the middle than at the ends, it was not found that any additional strength was obtained over that of uniform cylindrical pillars."—*Exp. Res.*, p. 349.

299. Square pillars yield in the direction of their diagonals.—Solid "square pillars do not bend or break in a direction parallel to their sides, but to their diagonals, nearly."—*Exp. Res.*, p. 331.

300. Long Pillars irregularly fixed lose from one-half to two-thirds of their strength.—"A (*long*) pillar irregularly fixed, so that the pressure would be in the direction of the diagonal, is reduced to one-third of its strength, the case being nearly similar to that of a (*long*) pillar with rounded ends, the strength of which has been shown to be only $\frac{1}{3}$rd of that of a pillar with flat ends."—*Exp. Res.*, p. 350. And in two experiments on long solid cast-iron pillars with the ends formed so that the pressure would not pass through the axis, but in lines one-fourth of the diameter and one-eighth of the diameter respectively from one side, the breaking weights were little more than one-half that of a pillar of the same dimensions with the ends turned so that the force would pass through the axis.—*Phil. Trans.*, 1840, pp. 413, 449.

301. Strength of similar long pillars is as their transverse area.—The strength of *similar* long pillars is nearly as the area of their transverse section. As derived from Mr. Hodgkinson's experiments on cast-iron, the strength varied as the 1·865 power of the diameter or any other linear dimensions.—*Exp. Res.*, p. 346.

302. Hodgkinson's formulæ for pillars with both ends flat and well bedded, and whose length exceeds 30 diameters if cast-iron or timber, and 60 diameters if wrought-iron.—The following formulæ have been deduced by Mr. Hodgkinson from his experiments. They represent the breaking weights of pillars with both ends *flat* and *well bedded*, and whose lengths exceed 30 diameters if cast-iron or timber, and 60 diameters if wrought-iron.

Let W = the breaking weight in tons,
l = the length of the pillar in feet,
d = the external diameter (or breadth, if the pillar be square timber) in inches,
d_1 = the internal diameter of hollow pillars in inches,
m = a coefficient varying with the quality of the cast-iron, and derived from experiments.

Solid round pillars of cast-iron.—*Phil. Trans.*, 1857, p. 859.

$$W = m \frac{d^{3.5}}{l^{1.63}} \qquad (212)$$

Hollow round pillars of *Low Moor* cast-iron, No. 2.—*Phil. Trans.*, 1857, p. 862.*

$$W = 42 \cdot 347 \frac{d^{3.5} - d_1^{3.5}}{l^{1.63}} \qquad (213)$$

Solid round pillars of wrought-iron.—*Phil. Trans.*, 1840, p. 424.

$$W = 133 \cdot 75 \frac{d^{3.55}}{l^2} \qquad (214)$$

Square pillars of Dantzic oak (dry).—*Phil. Trans.*, 1840, p. 425.

$$W = 10 \cdot 95 \frac{d^4}{l^2} \qquad (215)$$

Square pillars of red deal (dry).—*Phil. Trans.*, 1840, p. 425.

$$W = 7 \cdot 8 \frac{d^4}{l^2} \qquad (216)$$

Square pillars of French oak (dry).—*Phil. Trans.*, 1840, p. 426.†

$$W = 6 \cdot 9 \frac{d^4}{l^2} \qquad (217)$$

303. The following tables contain the values of the coefficient m, derived from experiments on solid pillars of cast-iron 10 feet long and 2½ inches diameter, with their ends flat; also the powers of diameters and lengths of pillars.—*Phil. Trans.*, 1857, pp. 872 and 860.

* "The pillars from this iron were cast 10 feet long, and from 2½ to 4 inches diameter, approaching in some degree, as to size, to the smaller ones used in practice."—*Proc. Roy. Soc.*, Vol. viii., p. 319.

† The crushing strength of French oak, according to Rondelet, = 6,336 lbs. per square inch.—*Phil. Trans.*, 1840, p. 427.

TABLE I.—COEFFICIENTS m in eq. (212).

Description of Iron.	Value of coefficient m.
Old Park Iron, No. 1. Stourbridge—cold blast, - - - - - - - - -	lbs. tons. 111858 = 49·94
Derwent Iron, No. 1. Durham—hot blast, - - - - - - - - -	105079 = 46·91
Portland Iron, No. 1. Tovine, Scotland—hot blast, - - - - - - -	104098 = 46·47
Calder Iron, No. 1. Lanarkshire—hot blast, - - - - - - - -	104137 = 46·49
London Mixture. One-half old plate iron, and one-half Calder iron, - - -	92862 = 41·46
Level Iron, No. 1. Staffordshire—hot blast, - - - - - - - -	94202 = 42·05
Coltness Iron, No. 1. Edinburgh—hot blast, - - - - - - - -	90119 = 40·23
Carron Iron, No. 1. County of Stirling—hot blast, - - - - - - -	89949 = 40·16
Blaenavon Iron, No. 1. South Wales—cold blast, - - - - - - - -	86114 = 38·44
Old Hill Iron, No. 1. Staffordshire—cold blast, - - - - - - - -	75270 = 33·60
Second London Mixture. One-third No. 1 best Scotch pig-iron, and two-thirds old metal, -	104628 = 46·21
Low Moor Iron, No. 2. Yorkshire—cold blast, - - - - - - - -	90674 = 40·48
Blaenavon Iron, No. 3. South Wales—cold blast, - - - - - - - -	92329 = 41·22
Mean of 13 Irons, - - - - - - - - - -	95486 = 42·6

TABLE II.—POWERS OF DIAMETERS, OR $d^{3·5}$.

1·0$^{3·5}$ = 1·0000	4·25$^{3·5}$ = 158·26	6·8$^{3·5}$ = 819·94
1·25$^{3·5}$ = 2·1837	4·3$^{3·5}$ = 164·87	6·9$^{3·5}$ = 862·92
1·5$^{3·5}$ = 4·1335	4·4$^{3·5}$ = 178·68	7·0$^{3·5}$ = 907·49
1·75$^{3·5}$ = 7·0898	4·5$^{3·5}$ = 193·305	7·1$^{3·5}$ = 953·68
2·0$^{3·5}$ = 11·314	4·6$^{3·5}$ = 208·76	7·2$^{3·5}$ = 1001·53
2·1$^{3·5}$ = 13·4205	4·7$^{3·5}$ = 225·08	7·25$^{3·5}$ = 1026·08
2·2$^{3·5}$ = 15·7935	4·75$^{3·5}$ = 233·58	7·3$^{3·5}$ = 1051·07
2·25$^{3·5}$ = 17·086	4·8$^{3·5}$ = 242·295	7·4$^{3·5}$ = 1102·33
2·3$^{3·5}$ = 18·452	4·9$^{3·5}$ = 260·43	7·5$^{3·5}$ = 1155·35
2·4$^{3·5}$ = 21·416	5·0$^{3·5}$ = 279·51	7·6$^{3·5}$ = 1210·17
2·5$^{3·5}$ = 24·705	5·1$^{3·5}$ = 299·57	7·7$^{3·5}$ = 1266·83
2·6$^{3·5}$ = 28·340	5·2$^{3·5}$ = 320·635	7·75$^{3·5}$ = 1295·85
2·7$^{3·5}$ = 32·3425	5·25$^{3·5}$ = 331·56	7·8$^{3·5}$ = 1325·35
2·75$^{3·5}$ = 34·488	5·3$^{3·5}$ = 342·74	7·9$^{3·5}$ = 1385·78
2·8$^{3·5}$ = 36·733	5·4$^{3·5}$ = 365·91	8·0$^{3·5}$ = 1448·15
2·9$^{3·5}$ = 41·533	5·5$^{3·5}$ = 390·18	8·25$^{3·5}$ = 1612·83
3·0$^{3·5}$ = 46·765	5·6$^{3·5}$ = 415·58	8·5$^{3·5}$ = 1790·47
3·1$^{3·5}$ = 52·4525	5·7$^{3·5}$ = 442·14	8·75$^{3·5}$ = 1981·66
3·2$^{3·5}$ = 58·617	5·75$^{3·5}$ = 455·87	9·0$^{3·5}$ = 2187·00
3·25$^{3·5}$ = 61·886	5·8$^{3·5}$ = 469·89	9·25$^{3·5}$ = 2407·11
3·3$^{3·5}$ = 65·283	5·9$^{3·5}$ = 498·86	9·5$^{3·5}$ = 2642·61
3·4$^{3·5}$ = 72·473	6·0$^{3·5}$ = 529·09	9·75$^{3·5}$ = 2894·12
3·5$^{3·5}$ = 80·212	6·1$^{3·5}$ = 560·60	10·0$^{3·5}$ = 3162·28
3·6$^{3·5}$ = 88·5235	6·2$^{3·5}$ = 593·43	10·25$^{3·5}$ = 3447·73
3·7$^{3·5}$ = 97·433	6·25$^{3·5}$ = 610·35	10·5$^{3·5}$ = 3751·13
3·75$^{3·5}$ = 102·12	6·3$^{3·5}$ = 627·61	10·75$^{3·5}$ = 4073·14
3·8$^{3·5}$ = 106·965	6·4$^{3·5}$ = 663·18	11·0$^{3·5}$ = 4414·43
3·9$^{3·5}$ = 117·15	6·5$^{3·5}$ = 700·16	11·25$^{3·5}$ = 4775·66
4·0$^{3·5}$ = 128·00	6·6$^{3·5}$ = 738·59	11·5$^{3·5}$ = 5157·54
4·1$^{3·5}$ = 139·55	6·7$^{3·5}$ = 778·51	11·75$^{3·5}$ = 5560·74
4·2$^{3·5}$ = 151·835	6·75$^{3·5}$ = 799·03	12·0$^{3·5}$ = 5985·96

TABLE III.—POWERS OF LENGTHS, OR $l^{1.63}$.

$1^{1.63}$ = 1·	$7\frac{1}{2}^{1.63}$ = 26·6901	$16^{1.63}$ = 91·7731
$2^{1.63}$ = 3·0951	$8^{1.63}$ = 29·6508	$17^{1.63}$ = 101·805
$2\frac{1}{2}^{1.63}$ = 4·4529	$9^{1.63}$ = 35·9265	$18^{1.63}$ = 111·197
$3^{1.63}$ = 5·9939	$10^{1.63}$ = 42·6580	$19^{1.63}$ = 121·442
$4^{1.63}$ = 9·5798	$11^{1.63}$ = 49·8276	$20^{1.63}$ = 132·032
$5^{1.63}$ = 13·7823	$12^{1.63}$ = 57·4203	$21^{1.63}$ = 142·961
$6^{1.63}$ = 18·5518	$13^{1.63}$ = 65·4226	$22^{1.63}$ = 154·223
$6\frac{1}{2}^{1.63}$ = 19·8282	$14^{1.63}$ = 73·8225	$23^{1.63}$ = 165·812
$7^{1.63}$ = 23·8512	$15^{1.63}$ = 82·6093	$24^{1.63}$ = 177·723

304. Pillars with both ends rounded and whose length exceeds 15 diameters if cast-iron or timber, and 30 diameters if wrought-iron.—If both ends are rounded, and the length exceeds 15 diameters for cast-iron and timber, and 30 diameters for wrought-iron, the breaking weight is approximately one-third of that given by the preceding formulæ (**291**).

305. Relative strength of long pillars of cast-iron, wrought-iron, steel, Dantzic oak, and red deal.—If we call the strength of a *long* cast-iron pillar 1000, the following numbers will express the strength of similar pillars of the other materials experimented on by Mr. Hodgkinson:—

Cast-iron (Low Moor), . . .	1000
Wrought-iron (best Staffordshire), . .	1745
Cast-steel (not hardened), . . .	2518
Dantzic oak, ...	108·8
Red deal, ...	78·5

"The numbers, all but the last, were obtained from the pillars with rounded ends, and the computations made by the rules used for cast-iron."—*Exp. Res.*, p. 351.

CLASS III.—MEDIUM OR SHORT FLEXIBLE PILLARS, WHICH YIELD PARTLY BY FLEXURE, PARTLY BY CRUSHING; LENGTH, IF BOTH ENDS ARE FLAT AND FIRMLY BEDDED, LESS THAN 30 DIAMETERS FOR CAST-IRON AND TIMBER, AND 60 DIAMETERS FOR WROUGHT-IRON; IF BOTH ENDS ARE ROUNDED, OF LESS THAN ONE-HALF THESE LENGTHS.

306. Medium pillars with flat ends stronger than those with rounded ends in varying ratios.—In cast-iron pillars whose length is less than 30 diameters, and in wrought-iron pillars whose length is less than about 60 diameters, the strength of those with flat ends varies from 3 to 1·5 times that of those with rounded ends, "or less, according as we reduce the number of times which the length exceeds the diameter;" "but whatever may be the ratio of the strengths of pillars with rounded ends to those with flat ones, the strength of those with one end rounded and one flat, is nearly an arithmetical mean between the strengths of the other two."—*Phil. Trans.*, 1840, pp. 389, 421.

307. Hodgkinson's formula for medium pillars of cast-iron or timber with both ends flat and well bedded.—"The formulæ above (**302**) apply to all pillars whose length is not less than about 30 times the external diameter (*for cast-iron and timber, and* 60 *diameters for wrought-iron*); for pillars shorter than this, it will be necessary to modify the formulæ by other considerations, since in these shorter pillars the breaking weight is a considerable proportion of that necessary to crush the pillar. Thus, considering the pillar as having two functions, one to support the weight, and the other to resist flexure, it follows that when the material is incompressible (supposing such to exist), or when the pressure necessary to break the pillar is very small, on account of the greatness of its length compared with its lateral dimensions, then the strength of the whole transverse section of the pillar will be employed in resisting flexure; when the breaking pressure is half of what would be required to crush the material, one half only of the strength may be considered as available for resistance to flexure, whilst the other half is employed to resist crushing; and when, through the shortness of the pillar, the breaking weight is so great as to be nearly equal to the crushing force, we may consider that no part of the strength of the pillar is applied to resist flexure."—*Exp. Res.*, p. 337. Acting on this view Mr. Hodgkinson devised the following formula for the strength of medium pillars of cast-iron and timber, with both ends flat and well bedded. For the reasoning on which it is based the reader is referred to Mr. Hodgkinson's writings.—*Phil. Trans.*, 1840, pp. 405, 426.

$$W' = \frac{Wc}{W + \frac{3}{4}c} \tag{218}$$

where W = the breaking weight in tons derived from the formulæ for long pillars in (**202**), on the hypothesis that the pillar yields by flexure alone,

c = the crushing weight of a short length of the pillar, *i.e.*, its sectional area multiplied by the crushing unit-strain of the material in tons,

W′ = the real breaking weight of the medium pillar in tons, from the combined effects of flexure and crushing.

Ex. 1. What is the breaking weight of a solid pillar of Blaenavon iron, No. 3, with flat ends carefully bedded, whose length = 9 feet, diameter = 6 inches, and whose crushing strength = 37·3 tons per square inch?

From Table I., $m = 41{\cdot}2$ tons,

$$c = 37{\cdot}3 \times 28{\cdot}3 = 1056 \text{ tons},$$

from eq. (212), $W = 41{\cdot}2\,\frac{529}{36} = 605$ tons,

Answer, eq. (218). $W' = \frac{605 \times 1056}{605 + 792} = 457$ tons.

If intended for a warehouse, the greatest load in practice should not exceed $\frac{1}{6}$th of this, = 76 tons, and that only when the ends are adjusted with great care, so as to have a very uniform bearing; when this is not the case the effect will be the same as if the ends were rounded, in which case the breaking weight will be much less (**206**), probably only $\frac{W'}{2} = \frac{457}{2} = 228{\cdot}5$ tons, of which $\frac{1}{6}$th, or the safe working load, will = 38 tons.

Ex. 2. What is the breaking weight of a hollow flat-bedded pillar of the same iron, of the same height and external diameter, and whose internal diameter = 4 inches?

On examining Table V. (**208**), we find that the crushing strength of Blaenavon iron, No. 3, medium = 37·3 tons per inch, while that of Low Moor, No. 2, medium = 34·6 tons. We may therefore assume that the coefficient for hollow cylinders of Blaenavon iron is the same as that for Low Moor in eq. (218).

$$\text{Here,} \quad c = 37{\cdot}3 \times 15{\cdot}7 = 586 \text{ tons},$$

$$\text{from eq. (213), } W = 42{\cdot}35 \frac{529 - 128}{36} = 472 \text{ tons nearly},$$

$$\textit{Answer}, \text{ eq. (218). } W' = \frac{472 \times 586}{472 + 448} = 303 \text{ tons},$$

of which $\frac{1}{6}$th, or the working load, $= 50\frac{1}{2}$ tons, *i.e.*, when the ends are fitted with extreme care; otherwise, $\frac{W'}{12} = 25\frac{1}{4}$ tons, is a sufficient load in practice.

CAST-IRON.

308. Crushing strength of cast-iron.—Table IV. contains the "Results of experiments on the crushing strength of cylinders of cast-iron of various kinds; the diameters of the cylinders being turned to $\frac{3}{4}$ inch each, and the heights being $\frac{3}{4}$ and $1\frac{1}{2}$ inches respectively." "In both cases the height was so small that the specimen could not bend before crushing. Before each experiment was commenced, a very thin sheet of lead was laid over and under the specimen, to prevent any small and unavoidable irregularity between its flat surfaces and those of the parallel steel discs between which it was to be crushed."—*Rep. of Com.*, 1849, App. A., pp. 12, 13.

TABLE IV.—CRUSHING STRENGTH OF CAST-IRON.

Description of Iron.	Height of Specimen.	Crushing Weight per square inch of Section.	Mean.
	Inch.	lbs. tons.	lbs. tons.
Low Moor Iron, No. 1 - - - -	$\frac{3}{4}$ $1\frac{1}{2}$	64534 = 28·809 56445 = 25·198	60489 = 27·004
Do. No. 2 - - - -	$\frac{3}{4}$ $1\frac{1}{2}$	99525 = 44·430 92332 = 41·219	95928 = 42·825
Clyde Iron, No. 1 - - - - -	$\frac{3}{4}$ $1\frac{1}{2}$	92869 = 41·459 88741 = 39·616	90805 = 40·537
Do. No. 2 - - - - -	$\frac{3}{4}$ $1\frac{1}{2}$	109992 = 49·103 102030 = 45·549	106011 = 47·326
Do. No. 3 - - - - -	$\frac{3}{4}$ $1\frac{1}{2}$	107197 = 47·855 104881 = 46·821	106039 = 47·339

TABLE IV.—CRUSHING STRENGTH OF CAST-IRON—*continued.*

Description of Iron.	Height of Specimen.	Crushing Weight per square inch of Section.	Mean.
	Inch.	lbs. tons.	lbs. tons.
Blaenavon Iron, No. 1	¾ 1½	90860 = 40·562 80561 = 35·964	85710 = 38·263
Do. No. 2—1st sample	¾ 1½	117605 = 52·502 102408 = 45·717	110006 = 49·109
Do. No. 2—2nd sample	¾ 1½	68559 = 30·606 68532 = 30·594	68545 = 30·600
Calder Iron, No. 1	¾ 1½	72193 = 32·229 75983 = 33·921	74088 = 33·075
Coltness Iron, No. 3	¾ 1½	100180 = 44·723 101831 = 45·460	101005 = 45·091
Brymbo Iron, No. 1	¾ 1½	74815 = 33·399 75678 = 33·784	75246 = 33·592
Brymbo Iron, No. 3	¾ 1½	76133 = 33·988 76958 = 34·356	76545 = 34·171
Bowling Iron, No. 2	¾ 1½	76132 = 33·987 73984 = 33·028	75058 = 33·508
Ystalyfera Anthracite Iron, No. 2	¾ 1½	99926 = 44·610 95559 = 42·660	97742 = 43·635
Yniscedwyn Anthracite Iron, No. 1	¾ 1½	83509 = 37·281 78659 = 35·115	81084 = 36·198
Do. No. 2	¾ 1½	77124 = 34·430 75369 = 33·646	76246 = 34·038
Mean of the foregoing 16 irons	- -	- - -	86284 = 38·519
Mr. Morries Stirling's Iron, 2nd quality*	¾ 1½	125333 = 55·952 119457 = 53·329	122395 = 54·640
Do. 3rd quality†	¾ 1½	158653 = 70·827 129876 = 57·980	144264 = 64·403

* Composed of Calder No. 1 hot-blast, mixed and melted with about 20 per cent. of malleable iron scrap.

† Composed of No. 1 hot-blast Staffordshire iron from Ley's Works, mixed and melted with about 15 per cent. of common malleable iron scrap.

Table V. contains the "crushing weights of short cylinders of different kinds of cast-iron, cut from the bars, $2\frac{1}{2}$ inches diameter previously used (*in experiments on pillars*), and now turned to be $\frac{3}{4}$ inch diameter nearly, and $1\frac{1}{2}$ inch high. The results are means from three or four experiments on each kind of iron. The specimens were usually cut out of the iron between the centre and the circumference of the bar, denominated the medium part. In several cases they were cut out of the centre of the bar, and sometimes out of the circumference."—*Phil. Trans.*, 1857, p. 889.

TABLE V.—CRUSHING STRENGTH OF CAST-IRON.

Description of Iron.	Diameter of Specimen.	Crushing Weight per square inch of Section.
	Inch.	lbs. tons.
Old Park Iron, No. 1. Medium, ...	·747	88070 = 39·32
Old Park Iron, No. 1. Centre, ...	·747	74653 = 33·33
Derwent Iron, No. 1. Medium, ...	·747	97160 = 43·37
Coltness Iron, No. 1. Medium, ...	·747	63048 = 28·14
Blaenavon Iron, No. 1. Medium, ...	·748	70909 = 31·66
Level Iron, No. 1. Medium, ...	·749	68217 = 30·45
Carron Iron, No. 1. Medium	·750	68509 = 30·58
London Mixture. Medium, ...	·749	80923 = 36·08
Calder Iron, No. 1. Medium, ...	·750	84648 = 37·79
Portland Iron, No. 1. Medium, ...	·748	94802 = 42·32

TABLE V.—CRUSHING STRENGTH OF CAST-IRON—*continued.*

Description of Iron.	Diameter of Specimen.	Crushing Weight per square inch of Section.
	Inch.	lbs. tons.
Old Hill Iron, No. 1. Medium, - - - - - - - - - -	·749	54761 = 24·45
Low Moor Iron, No. 2. Medium, - - - - - - - - - -	·748	77489 = 34·59
Low Moor Iron, No. 2. Centre, - - - - - - - - - -	·742	66407 = 29·65
Blaenavon Iron, No. 3. Medium, - - - - - - - - - -	·737	83517 = 37·28
Blaenavon Iron, No. 3. Centre, - - - - - - - - - -	·747	76643 = 34·22
Second London Mixture. Medium. From 2½ inch pillar, as all above have been, -	·747	95338 = 42·56
Second London Mixture. Centre. From 2½ inch pillar, as all above have been, -	·747	78451 = 35·02
Second London Mixture. Medium. From 1½ inch pillar, - - - - - -	·750	111080 = 49·59
Second London Mixture. Centre. From 1½ inch pillar, - - - - - - -	·750	104071 = 46·46
Low Moor Iron, No. 2. From a hollow pillar 4 inches diameter and ½ inch thick. The height of the first two specimens was ·72 inch, and of the last 1·502 inch, - - - - - - - -	·421	87502 = 39·06
Low Moor Iron, No. 2. From the thin ring of a hollow pillar about 3¼ inches diameter. Height of specimens ·53 inch, - - - -	·299	115993 = 51·78
Low Moor Iron, No. 2. From the thin ring of a hollow pillar about 3¼ inches diameter. Height of specimens ·53 inch, - - - -	·296	110212 = 49·20
Mean of the foregoing 22 irons, - - - - - - -	-	84200 = 37·6

309. Hardness and crushing unit-strain of thin castings greater near the surface than in the heart — Crushing unit-strain of thin greater than that of thick castings.—"Of the different irons tried in the experiments on pillars, whether solid or hollow, the external part of the casting was always harder than that near to the centre, and the iron of the external ring of a hollow casting was very hard, the hardness increasing with the thinness. Thus, in solid pillars of 2½ inches diameter of Low Moor iron, No. 2 (Table V.), the crushing force per square inch of the central part was 29·65 tons, and that of the intermediate part near to the surface was 34·59 tons, whilst the external ring, ½ inch thick, of a hollow cylinder 4 inches diameter, of which the outer crust had been removed, was crushed with 39·06 tons per square inch; and external rings of the same iron, thinner than half an inch, required from 49·2 to 51·78 tons per square inch to crush them. These facts show the great superiority of hollow pillars over solid ones of the same weight and length."—*Phil. Trans.*, 1857, p. 890. Hence removing the skin of a *thin* casting reduces its strength to resist compression.

310. A slight inequality in thickness of hollow pillars does not impair their strength materially.—Referring to castings of unequal thickness, Mr. Hodgkinson remarks:—"In experiments upon hollow pillars, it is frequently found that the metal on one side is much thinner than that on the other; but this does not produce so great a diminution in the strength as might be expected, for the thinner part of a casting is much harder than the thicker, and this usually becomes the compressed side."—*Phil. Trans.*, 1857, p. 862. In practice, neither the excess or want of thickness should exceed 25 per cent. of the average thickness; if, for instance, a hollow pillar is specified to be 1 inch in thickness, then in no place should the metal be less than ¾ inch or more than 1¼ inch thick.

311. Hardness and crushing strength at the surface and in the heart of thick castings not materially different.—"To ascertain whether the internal strength of larger pillars varied in the same manner as that of smaller ones, a cylindrical casting was made 5 inches diameter and 15 inches long. It was cast vertically, from

Blaenavon iron. Through the axis of this cylinder, a slab, 15 inches long, 5 inches broad, and about 1 inch thick, was taken. Across the middle of this slab five cylinders, 1½ inch long and ¾ inch diameter, were obtained at equal distances from each other, the middle one being in the centre, and the outer ones as near as possible to the sides. Comparing the results of the experiments (*on crushing these cylinders*) it appears that the external part of the casting was somewhat stronger than the internal. But the variation was only from 62 to 66 (62,444 *to* 65,739 *lbs. per square inch*), and therefore much less than was obtained from the smaller masses." From this and other experiments on small cylinders cut out of a slab of Derwent iron, No. 1, cast 9 inches square and 12 inches long, "it appears that the difference of hardness between the external and internal parts of a large casting is much less than in a small one, and may frequently be neglected."—*Phil. Trans.*, 1857, pp. 891, 892.

219. Transverse strength of thick castings much less than that of thin castings.—" In some experiments made by Captain (now Colonel Sir Henry) James, as a member of the Royal Commission for inquiring into the application of iron to railway structures,* it was found that the central part of bars of iron planed was much weaker to bear a transverse strain than bars cast of the same size. He states that " it was found by planing out ¾-inch bars from the centre of 2-inch square and 3-inch square bars, that the central portion was little more than half the strength of that from an inch bar, the relation being as 7 to 12" (there may be some doubt whether the iron was sound at the centre). In page 111 of the same report, I (*Mr. Hodgkinson*) showed that rectangular bars of cast-iron, cast 1, 2, and 3 inches square, laid upon supports 4½ feet, 9 feet, and 13½ feet asunder, were broken by weights of 447 lbs., 1394 lbs., and 3043 lbs. respectively. These weights, divided by the squares of the lengths, should give equal results; the quotients, however, were as 447, 349, and 338 respectively. I attributed this falling off and deviation from theory partly to the defect of elasticity, which I had always found in cast-iron, but principally to the superior hardness of the smaller castings."—*Phil. Trans.*, 1857, p. 867.

* See *Report*, 1849, App. B., p. 250.

313. Working load on cast-iron pillars should not exceed one-sixth of their breaking weight.—Owing to the want of recorded information it is difficult to assign what proportion of the breaking weight eminent engineers have considered the safe working load for cast-iron pillars. The opinions elicited by the Commissioners appointed to inquire into the application of iron to railway structures throw little or no light on the matter, as the evidence was chiefly confined to the strength of girders under transverse strain. Navier* gives $\frac{1}{5}$th of the breaking weight as the safe load in practice. Francis,† an American engineer, also gives $\frac{1}{5}$th; while Morin‡ adopts $\frac{1}{6}$th. My present opinion is that cast-iron pillars supporting loads free from vibration, such as water tanks, will safely carry $\frac{1}{5}$th of their breaking weight. In factories or stores, where moderate vibrations occur, the working load should not exceed $\frac{1}{6}$th; and if the pillar be liable to transverse strains, or severe shocks, like those on the ground floors of warehouses, where loaded waggons or heavy bales are apt to strike against them, the load should not exceed $\frac{1}{10}$th of the breaking weight, or even less in some cases, where the strength of the pillar depends rather on the transverse strain to which it is liable than the weight it has to support. For instance, the effect of wind on a light open shed, supported by pillars, may produce a transverse strain which will be very severe in proportion to the weight of the roof. The same thing may occur if heavy rolling goods, such as provision kegs or loaves of sugar, are piled up in such a manner as to cause horizontal pressure, like that of a liquid. It is also necessary to take into consideration the foundations on which the pillars rest, for if these yield unequally, one pillar may sustain much more than its proper share of load (**258**). When struts are required in machinery, wrought-iron has generally superseded cast-iron. When, however, the latter material is adopted it is well that the working load should, at all events, not exceed $\frac{1}{10}$th of the calculated breaking load.

314. + and H shaped pillars.—A cast-iron pillar of the + form of section, "as in the connecting rod of a steam engine, the ends being moveable, is very weak to bear a strain as a pillar; and

* *Résumé des Leçons sur l'application de la Mécanique*, p. 204. Bruxelles. 1839.

† *On the Strength of Cast-iron Pillars*, p. 17. New York. 1865.

‡ *Résistance des Matériaux*, p. 106.

indeed less than half the strength of a hollow cylindrical pillar of the same weight and length, rounded at the ends." The ratio of the strength, according to Mr. Hodgkinson's experiments, was as 17,578 to 39,645.—*Phil. Trans.*, 1857, p. 893.—*Exp. Res.*, p. 350.

A cast-iron pillar of the **H** form of section with rounded ends was found to be "considerably stronger than the preceding, but much weaker than a hollow cylinder of the same weight." The ratio of the strength, according to Mr. Hodgkinson's experiments, was as 29,571 to 39,645.—*Phil. Trans.*, 1840, pp. 413, 449.

315. Round, square, and triangular pillars.—From a comparison of Mr. Hodgkinson's experiments it appears that long solid square cast-iron pillars are about 60 per cent. stronger than solid cylindrical pillars of diameters equal to the sides of the squares.—*Phil. Trans.*, 1840, pp. 431, 437. Solid triangular pillars of cast-iron with flat ends are somewhat stronger than those with either circular or square sections. "It appears that the strengths of (*long*) circular, square, and triangular solid pillars of the same quality, weight, and length, vary as 55,299, 51,537, and 61,056, the last being the strongest."—*Phil. Trans.*, 1857, p. 893. Reducing these to a more convenient standard, their relative strengths are as follows:—

Long solid round pillar, ...	100
„ square „	93
„ triangular „	110

WROUGHT-IRON.

316. For the laws of wrought-iron tubular pillars we are again indebted to Mr. Hodgkinson. The following are the chief results of his experiments, made both at the commencement and during the progress of the Britannia and Conway tubular bridges.*

317. Crushing strength of wrought-iron.—Wrought-iron is crushed (*i.e.*, bulged) with a compression of 16 tons per square inch, and when the pressure exceeds 12 or 13 tons, wrought-iron, "in most cases, cannot be usefully employed, as it will sink to any degree, though in hollow cylinders it will sometimes bear 15 or 16 tons per square inch."—*Rep.*, p. 121.

318. Resistance of long plates to flexure.—A plate may be regarded as a number of square pillars placed side by side. It will

* *Report of the Commissioners appointed to inquire into the application of Iron to Railway Structures*, 1849. Also, *Clark on the Tubular Bridges*. London. 1850.

therefore follow the laws of pillars as far as deflexion at right angles to its plane is concerned. The ultimate resistance of *long* plates to *flexure* is nearly as the cube (2·878 power) of the thickness into the breadth, and inversely as the square of the length. This conclusion was derived from plates which were so long as to yield with strains varying from 1 to 9 tons per square inch; probably it holds good only when their length exceeds 60 times their thickness. If, however, the plates form the sides of a tube the rule does not apply, since they yield in that case by buckling, not by flexure, being held in the line of thrust by the adjacent sides which enable them to bear a greater strain than if not so supported along their edges.—*Rep.*, pp. 119, 120, 121.

319. Strength of wrought-iron tubes independent of length within certain limits.—When the length of wrought-iron tubular pillars does not exceed 15 or 20 times their width, they fail by bulging or buckling of the plates, not by flexure of the pillar, and within this limit the strength of the tube is nearly independent of its length.—*Rep.*, pp. 121, 163.

320. Strength per square inch of wrought-iron tubes depends upon the ratio between the thickness of the plate and the width of the tube.—Of rectangular tubes of uniform thickness none but those which had thick plates compared with the width of the tube (not less than one-thirtieth of the width) resisted with 12 tons per square inch, but hollow cylinders gave better results, some attaining the limit of 16 tons.—*Rep.*, p. 121.

321. Round and rectangular tubular pillars compared—Strongest form of rectangular cell—Safe working load.—Round tubular pillars of uniform thickness are in general stronger than rectangular tubes of uniform thickness, and the crushing unit-strain of these latter is greater the thicker the plates are in proportion to the width of tube. It is to be regretted that the relation between the thickness of the plate and the lateral dimensions of the tube, on which the laws of buckling seem to depend, was not fully investigated. In most of those rectangular tubes which sustained a compression of 10 tons per square inch or upwards, the thickness of the plate was about $\frac{1}{30}$th of the breadth of the tube. In one experiment a square tube, 8 feet long, 18 inches in width, of $\frac{1}{2}$-inch plates united by angle-irons at the corners, sustained a compressive strain of 13·6 tons per square inch.* Unfortunately there were no further experiments made on tubes thus strengthened at the angles. From this experiment, and from one made during the construction of the Boyne Viaduct to test the strength of a braced pillar (see Appendix A), I infer that the strongest form of rectangular cell to resist buckling is one in whose angles the chief part of the material is concentrated, making the sides thin plates, or lattice work, merely sufficient to withstand flexure of the angles, in which case the thin plates act the part of the web, and the angles act as the flanges of a girder.

The working load on wrought-iron pillars should not exceed $\frac{1}{4}$th of their breaking weight, or $\frac{1}{8}$th if subject to shocks like the jib of a crane. In machinery, where there is rapid reciprocating action, as in the piston rod of a steam engine, the working strain should not exceed $\frac{1}{10}$th of the breaking weight.

STEEL.

322. Crushing strength of cast-steel.—The following table contains experiments on the crushing weights of cylinders of cast-steel by Major Wade, U.S. Army:—†

TABLE VI.—CRUSHING STRENGTH OF CAST-STEEL.

Kind of Cast-steel.	No. of Sample.	Length.	Diameter.	Crushing weight per square inch of section.
				lbs.
Not hardened, - - - - - -	1	1·021	·400	198,944
Hardened; low temper; chipping chisels, - -	2	·995	·402	354,544
Hardened; mean temper; turning tools, - -	3	1·016	·403	391,985
Hardened; high temper; tools for turning hard steel,	4	1·005	·405	372,598

NOTE.—All the samples of steel tested were cut from the same bar. No. 1 remained unchanged, as made at the steel factory. Nos. 2, 3, and 4, were all hardened, and the temper afterwards drawn down in different proportions.

* *Clark*, p. 364.

† *Reports of Experiments on the Strength and other Properties of Metals for Cannons*, by Officers of the Ordnance Department, U.S. Army, p. 258. Philadelphia, 1856.

DIFFERENT METALS.

393. Crushing strength of copper, brass, tin, and lead.—The following table contains experiments by Mr. G. Rennie on the crushing strength of $\frac{1}{4}$ inch cubes of different metals.—*Phil. Trans.*, 1818, p. 125.

TABLE VII.—CRUSHING STRENGTH OF DIFFERENT METALS.

Description of Metal.	Crushing Weight on a $\frac{1}{4}$ inch Cube.
	lbs.
Cast copper crumbled with - - - - -	7318
Fine yellow brass reduced $\frac{1}{10}$th, with - - - -	3213
Do. do. $\frac{1}{2}$, with - - - -	10304
Wrought-copper reduced $\frac{1}{16}$th, with - - - -	3427
Do. do. $\frac{1}{8}$th, with - - - -	6440
Cast-tin do. $\frac{1}{16}$th, with - - - -	552
Do. do. $\frac{1}{3}$rd, with - - - -	966
Cast-lead do. $\frac{1}{2}$, with - - - -	483

Alluding to these ductile metals, Mr. Rennie observes:—"The experiments on the different metals give no satisfactory results. The difficulty consists in assigning a value to the different degrees of diminution. When compressed beyond a certain thickness, the resistance becomes enormous."

TIMBER.

394. Long pillars.—The strength of long round or square timber pillars is, from Mr. Hodgkinson's experiments, nearly as the fourth power of the diameter or side, divided by the square of the length.—*Phil. Trans.*, 1840, p. 425.

395. Square the strongest form of rectangular pillar.—"Of rectangular pillars of timber it was proved experimentally that the pillar of greatest strength, where the length and quantity of material is the same, is a square."—*Exp. Res.*, p. 351.

396. Crushing strength of timber—Wet timber not half so strong as dry.—The following table contains the results of experiments, by Mr. Hodgkinson, on the crushing strength of various kinds of timber, "the force being applied in the direction of the fibre."—*Phil. Trans.*, 1840, p. 429.

TABLE VIII.—CRUSHING STRENGTH OF TIMBER.

Description of Wood.	Crushing Weight per square inch of Section.	
	Wood in the ordinary state of dryness.	Wood very dry.
	lbs.	lbs.
Alder,	6831	6960
Ash,	8683	9363
Baywood,	7518	7518
Beech,	7733	9363
Birch, American,	...	11663
Birch, English,	3297	6402
Cedar,	5674	5863
Crab,	6499	7148
Deal, red,	5748	6586
Deal, white,	6781	7293
Elder,	7451	9973
Elm,	...	10331
Fir, Spruce,	6499	6819
Hornbeam,	4533	7289
Larch (fallen two months),	3201	5568
Mahogany,	8198	8198
Oak, Quebec,	4231	5982
Oak, English,	6484	10058
Oak, Dantzic (very dry),	...	7731
Pine, pitch,	6790	6790
Pine, yellow (full of turpentine),	5375	5445
Pine, red,	5395	7518
Plum, wet,	3654	...
Plum, dry,	8241	10493
Poplar,	3107	5124
Sycamore,	7082	...
Teak,	...	12101
Walnut,	6063	7227
Willow,	2898	6128

"The results in the first column were in each case a mean from about three experiments upon cylinders of wood turned to be one inch diameter, and two inches long, flat at the ends. The wood was moderately dry, being such as is employed in making models for castings. The second column gives the mean strength, as before, from similar specimens, after being turned and kept drying in a warm place two months longer. The lengths of these latter specimens were, in some instances, only one inch, which reduction would increase the strength a little. But the great difference frequently seen in the strength, as given by the two columns, shows strongly the effect of drying upon wood, and the great weakness of wet timber, *it not having half the strength of dry*"—a consideration of much importance in works under water.

297. Rondelet's rule for timber pillars.—Rondelet deduced the following rule from his experiments on the compression of oak and fir.* Taking as unity the force which would crush a cube, the force requisite to break a post whose height is—

12 times the thickness, will be			-	-	$\frac{5}{6}$
24	,,	,,	,,	-	$\frac{1}{2}$
36	,,	,,	,,	-	$\frac{1}{3}$
48	,,	,,	,,	-	$\frac{1}{6}$
60	,,	,,	,,	-	$\frac{1}{12}$
72	,,	,,	,,	-	$\frac{1}{24}$

Rondelet also found that timber pillars do not begin to yield by flexure until their length is about ten times their least lateral dimensions.

298. Working load on timber pillars should not exceed one-tenth of their breaking weight—Load at right angles to the fibre.—Owing to the liability of timber to decay, the permanent working load on wooden pillars should not exceed $\frac{1}{10}$th of their breaking weight. For merely temporary purposes, however, the working load may be much higher than this, probably as high as $\frac{1}{4}$th of the breaking weight. Gauthey recommends that the pressure on timber at right angles to the fibres should not exceed $\frac{4}{8}$ths of that in the direction of their length.†

299 Working load on Piles—Rondelet's rule—Rankine's rule.—The working load on timber piles, surrounded on all sides by the ground, may vary, according to Rondelet, from 427 to 498 lbs. per square inch.‡ Professor Rankine§ says:—" It appears from practical examples that the limits of the safe load on piles are as follows:—

" For piles driven till they reach the firm ground, 1,000 lbs. per square inch of area of head.

" For piles standing in soft ground by friction, 200 lbs. per square

* Navier; *Résumé des Leçons sur l'application de la Mécanique*, p. 200.

† Morin; *Résistance des Matériaux*, p. 72. ‡ Idem., p. 71.

§ *A Manual of Civil Engineering*, p. 602.

inch of area of head." Professor Rankine's rule is based on sound principles, for the nature of the ground and the resistance which it offers to the penetration of the piles have generally more to do with their safe working load than the strength of the timber has. As far as the latter alone is concerned, we might safely load piles surrounded by the ground with $\frac{1}{8}$th of the crushing weight of wet timber, which rule, according to Hodgkinson's experiments, is equivalent to $\frac{1}{10}$th of the tabulated crushing weight of dry timber. When, however, piles project above the surface of the ground and support a superstructure, they act in the capacity of pillars and their strength accordingly should exceed that of piles surrounded by earth.

STONE, BRICK, CEMENT, AND GLASS.

330. Crushing strength of Stone and Brick—Safe working load.—The following table contains the crushing strength of stone and brick. The working load on brick-work, concrete, and rubble masonry, rarely exceeds $\frac{1}{8}$th of the crushing weight of the aggregate mass. Cut stone, like the voussoirs of an arch, or in pillars, should not be subjected to more than $\frac{1}{20}$th of the crushing weight of the stone. Practically this limit is seldom reached.

TABLE IX.—CRUSHING STRENGTH OF STONE AND BRICK.

Description of Stone.	Sp. Gravity.	Crushing Weight in lbs. per square inch.	Authority.
GRANITES.			
Aberdeen, blue kind,	2·625	10914	Rennie.
Peterhead, hard close grained,	...	8283	,,
Cornish,	2·662	6356	,,
Killiney, near Dublin, very felspathic,	...	10780	Wilkinson.
Kingstown, do., grey colour,	...	10115	,,
Blessington, Co. Wicklow, coarse and loosely aggregated,	...	3630	,,
Newry, slightly syenitic,	...	13440	,,
Mount Sorrel granite,	2·675	12861	Fairbairn.
SANDSTONES AND GRITS.			
Arbroath pavement,	...	7884	Buchanan.
Caithness do.	...	6493	,,
Dundee sandstone or Brescia,	2·530	6630	Rennie.
Craigleith white freestone,	2·452	5487	,,

TABLE IX.—CRUSHING STRENGTH OF STONE AND BRICK—*continued.*

Description of Stone.	Sp. Gravity.	Crushing Weight in lbs. per square inch.	Authority.
SANDSTONES AND GRITS—*continued.*			
Bramley Fall, near Leeds (with and against strata)	2·506	6059	Rennie.
Derby Grit, a red friable sandstone, - -	2·316	3142	,,
Ditto, from another quarry, - - -	2·428	4345	,,
Yorkshire paving (with and against strata), -	2·507	5714	,,
Red sandstone, Runcorn (17 feet per ton), -	...	2185	L. Clark.
Quartz rock, Holyhead (across lamination), -	...	25500	Mallet.
Ditto (parallel to lamination), - - -	...	14000	,,
OOLITES.			
Portland stone, - - - - -	2·423	3729	Rennie.
Ditto, another specimen, - - - -	2·428	4570	,,
MARBLES.			
Marble, statuary, - - - - -	...	3216	Rennie.
Ditto, white statuary, not veined, - -	2·760	6058	,,
Ditto, white Italian, veined, - - -	2·726	9681	,,
Ditto, black Brabant, - - - -	2·697	9219	,,
Ditto, Devonshire red, variegated, - -	...	7428	,,
LIMESTONES.			
Limestone, compact, - - - - -	2·584	7713	Rennie.
Ditto, black compact, Limerick, - -	2·598	8855	,,
Ditto, Purbeck, - - - - -	2·599	9160	,,
Ditto, Anglesea (13½ cubic feet per ton), -	...	7579	L. Clark.
Ditto, Kerry, Listowel quarry, - -	...	18043	Wilkinson.
Ditto, King's County, Ballyduff quarry, near Tullamore, - - - - -	...	11340	,,
Ditto, Kildare, near Athy, - - -	...	14350	,,
Ditto, Dublin, Finglas quarry, - -	...	16940	,,
Chalk, - - - - - -	...	501	Rennie.
SLATES.			
Valentia, Kerry, - - - - -	...	10943	Wilkinson.
Killaloe, Tipperary, - - - -	...	20860	,,
Wicklow, Glanmore, - - - -	...	16170	,,
BASALTS.			
Whinstone, Scotch, - - - - -	...	8270	Buchanan.
Greenstone, from Giant's Causeway, - -	...	17220	Wilkinson.
Grauwacke, from Penmaenmawr, - -	2·748	16893	Fairbairn.
BRICKS.			
Pale red, - - - - - -	2·085	562	Rennie
Red brick, - - - - - -	2·168	808	,,
Yellow-face baked Hammersmith paviors, -	...	1002	,,

TABLE IX.—CRUSHING STRENGTH OF STONE AND BRICK—*continued.*

Description of Stone.	Sp. Gravity.	Crushing Weight in lbs. per square inch.	Authority.
BRICKS—*continued.*			
Yellow-faced burnt Hammersmith paviors,	...	1441	Rennie.
Fire brick (Stourbridge)	...	1717	"
Brickwork set in cement (bricks not of a hard descripton)	...	521	L. Clark.

Buchanan, see *Practical Mechanics' Journal*, Vol. ii., p. 285.
L. Clark, see *The Britannia and Conway Tubular Bridges*, p. 365.
Fairbairn, see *The Application of Iron to Building Purposes*, 1858, p. 188.
Mallet, see *Philosophical Transactions*, 1862, p. 671.
Rennie, see *Philosophical Transactions*, 1818, p. 131.
Wilkinson, see *Practical Geology and Ancient Architecture of Ireland.*

331. Stone.—In Mr. Clark's experiments "the sandstones gave way *suddenly*, and without any previous cracking or warning. After fracture the upper portion generally retained the form of an inverted square pyramid, very symmetrical, the sides bulging away in pieces all round. The limestones formed *perpendicular* cracks and splinters a considerable time before they crushed." Mr. Rennie observes, "it is a curious fact in the rupture of amorphous stones, that pyramids are formed, having for their base the upper side of the cube next the lever, the action of which displaces the sides of the cubes, precisely as if a wedge had operated between them." Mr. Wilkinson remarks (p. 347), "The results of the cubes experimented on, show the strongest stones to be the basalts, primary limestones, and slates. Of the limestones, the primary limestones and compact hard calp are the strongest; and the light dove-coloured and fossiliferous limestones are among the weakest. The strength of the sandstones, like their mineral aggregation, is very variable." Besides the results given in the foregoing table, Mr. Wilkinson's book contains a very extensive series of experiments on the transverse and crushing strengths of Irish stones.

332. Crushing strength of Portland Cement and Mortar.—The following table contains the results of experiments by Mr. Grant on the crushing strength of Portland cement and cement mortar.*

* *Proceedings of the Institution of Civil Engineers*, Vol. xxv., p. 97.

TABLE X.—CRUSHING STRENGTH OF PORTLAND CEMENT AND CEMENT MORTAR.

Description of Cement or Mortar.		Crushing Weight in lbs. per square inch.
Neat Portland cement, - - -	made 3 months.	3795
1 Portland cement to 1 pit sand, -		2491
ditto 2 ditto, -		2004
ditto 3 ditto, -		1436
ditto 4 ditto, -		1331
ditto 5 ditto, -		959
Neat Portland cement, - - -	made 6 months.	5388
1 Portland cement to 1 sand, - -		3478
ditto 2 ditto, - -		2752
ditto 3 ditto, - -		2156
ditto 4 ditto, - -		1797
ditto 5 ditto, - -		1540
Neat Portland cement, - - -	made 9 months.	5984
1 Portland cement to 1 pit sand, -		4561
ditto 2 ditto, -		3647
ditto 3 ditto, -		2393
ditto 4 ditto, -		2208
ditto 5 ditto, -		1678

In Mr. Grant's experiments the specimens were made into bricks 9″ × 4·25″ × 2·75″, and exposed to the pressure of a hydraulic press on their flat surface of 9″ × 4·25″ = 38·25 square inches. The results would doubtless have been somewhat different if they had been cubes—probably a little less, at least for pure cement, than the figures in the foregoing table (**290**). Each specimen showed signs of giving way with considerably less pressure than that which finally crushed it, the ratio of the weight which produced the first crack to that which finally crushed being nearly as $\frac{5}{8}$.

333. Crushing strength of Glass.—The following table contains the crushing strength of glass from experiments by Messrs. Fairbairn and Tate.*

TABLE XI.—CRUSHING STRENGTH OF GLASS.

Kind of Glass.	Sp. Gravity.	Crushing Weight in lbs. per square inch.
Best flint glass annealed rod, drawn out when molten to the diameter required, - -	3·0782	27582
Common green glass ditto ditto, -	2·5284	31876
White crown glass ditto ditto, -	2·4504	31003

* *Philosophical Transactions*, 1859, p. 213.

On these experiments Messrs. Fairbairn and Tate observe (p. 221), "The specimens were crushed almost to powder from the violence of the concussion, when they gave way; it, however, appeared that the fractures occurred in vertical planes, splitting up the specimen in all directions; cracks were noticed to form some time before the specimen finally gave way; then these rapidly increased in number, splitting the glass into innumerable irregular prisms of the same height as the cube; finally, these bent or broke, and the pressure, no longer bedded on a firm surface, destroyed the specimen." Some cubes were cut from the centre of large lumps of glass, and crushed. Their resistance was less than that of the drawn rods in the ratio of $\frac{5}{8}$, possibly because the external skin was removed, and also because they were less perfectly annealed than the drawn rods (**309**).

334. Influence of the height and number of courses in stone columns.—Vicat found, from experiments on plaster prisms, that the strength of a monolithic prism, whose height is h, being represented by unity, we have the strength of prisms:—

Of 2 courses	and of the height	h	$= 0\cdot930$
Of 4	,,	,, $2h$	$= 0\cdot861$
Of 8	,,	,, $4h$	$= 0\cdot834$

even without the interposition of mortar. He concludes that the division of a column into courses, each of which is a monolith, with carefully dressed joints and properly bedded in mortar, does not sensibly diminish its resistance to crushing; but he intimates that this does not hold good when the courses are divided by vertical joints.*

335. Navier's Rules.—The following practical rules are given by M. Navier† as the results of experiments made by Duleau, Lamandé, Rondelet, Girard, Rennie, and others:—

Cast-iron.—The force requisite to crush a short piece whose height equals once or twice the thickness, ought to be reduced to $\frac{5}{6}$rds when the length is equal to 4 times the thickness, to $\frac{1}{2}$ when

* Morin. *Résistance des Matériaux*, p. 76.

† *Résumé des Leçons sur l'application de la mécanique à l'établissement des constructions*, p. 204. Brussels edition, 1839.

the length is equal to 8 times the thickness, and to $\frac{1}{15}$th when the length is equal to 36 times the thickness.

Wrought-iron.—The crushing strength of short pieces should be reduced to $\frac{5}{8}$ths when the length is equal to 12 times the thickness, and to $\frac{1}{2}$ when the length is equal to 24 times the thickness.

Timber.—The crushing strength of short pieces should be reduced to $\frac{5}{6}$ths when the length is equal to 12 times the thickness, and to $\frac{1}{2}$ when the length equals 24 times the thickness.

In practice, the breaking weight obtained by the foregoing rules should be reduced to about $\frac{1}{10}$th for wooden pillars and to $\frac{1}{4}$th or $\frac{1}{5}$th for wrought or cast-iron.

336. Crushing strength of Rollers and Spheres.—From M. Vicat's experiments it appears that the strength of cylinders employed as rollers between two horizontal planes is proportional to the product of their axis by the diameter. The strength of spheres to resist crushing is proportional to the square of their diameter. If the strength of a cube be represented by unity, that of the inscribed cylinder standing on its base will be 0·80; that of the same cylinder on its side will be 0·32; and that of the inscribed sphere will be 0·26.*

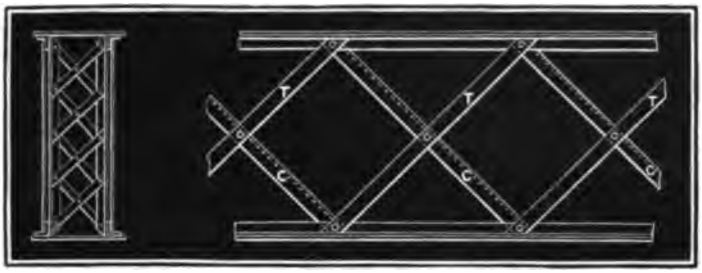
BRACED PILLARS.

337. Internal bracing—Example.—One of the chief practical difficulties which occur in bridges of large span is the combination of lightness with stiffness in long struts, such as the compression bars of the web. The arrangement represented in Fig. 89 is a modification of the bracing so familiar in scaffolding. It is now frequently used for the compression bars of lattice girders, and as it unites the requisite qualities of strength and lightness in an eminent degree, it is worth devoting some space to investigating the nature of the strains in this form of pillar.

The diagram represents the cross section and side elevation of a

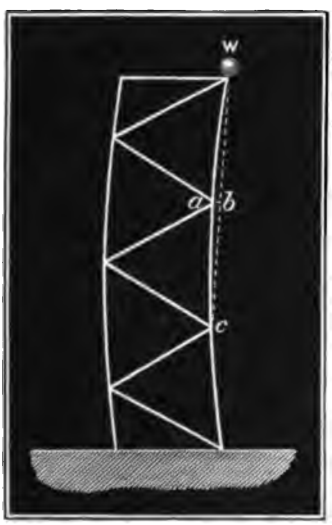
* *Résistance des Matériaux*, par Arthur Morin, pp. 75, 82.

Fig. 89.



lattice tubular girder of simple construction. The tension diagonals (marked T,) intersect the compression diagonals (marked C,) at moderate intervals, and keep them from deflecting, especially in the plane of the girder. It is obvious, however, that long compression bars, even though formed of angle or T iron, have but little stiffness in themselves, and we cannot trust to the tension bars keeping them in the line of thrust at right angles to the former direction, for the tension bars may not always be in a sufficient state of strain (**155**). Hence it is desirable, at least in long pillars, to connect each pair of compression bars by internal cross-bracing, as shown in the section.

Fig. 90.



The strains to which this braced pillar is subject may be investigated in the following manner, which, though rude, is yet sufficiently approximate for practical purposes:—

Let Fig. 90 represent a pillar which has become deflected from the weight resting more on one side than on the other, or from defective construction, or from accident.

Let W = the weight resting on one side,

$D = ab =$ the deflection in the interval of two bays,

$l = Wa = ac =$ the length of one bay,

R = the radius of curvature of the deflected pillar,

P = the resultant of the strains in Wa and ac, *i.e.*, the nearly horizontal pressure produced on the two braces intersecting at a, in consequence of the weight being transmitted through a curved pillar.

At the apex a, 3 forces balance, viz., the nearly vertical pressures (each = W) in the two adjacent bays, and their resultant P. Hence we have $P = \frac{2DW}{l}$, but $D = \frac{l^2}{2R}$, therefore

$$P = \frac{Wl}{R} \tag{219}$$

The pillar may therefore be regarded as a girder, each of whose flanges is subject to a longitudinal pressure equal to W, besides having a weight P resting on each apex. Hence the strains in the bracing may be found by the methods already explained in Chapters VI. and VII. If the pillar have a tendency to assume an S form, the strains developed in the internal bracing in one loop of the curve may, to some extent, neutralize those produced in the other. If, however, the pressure on one side exceed that on the other by any known or assumed quantity, then their difference of length and the corresponding deflection may be obtained as explained in the chapter on deflection, but in practice errors of workmanship will almost always exceed the amount of deflection produced by a difference of pressure, and experience must dictate the requisite allowance. Let, for example, a pillar with internal bracing, composed of two systems of right-angled triangles, similar to that represented in Fig. 89, be 30 feet long and 2 feet wide, and let each bay be 2 feet in length, in which case there will be 15 bays in each side, and let the total load on the pillar = 40 tons, or 20 tons on a side. Now suppose that the maximum error of workmanship amounts to half an inch deflection in the centre of the pillar, in which case R will equal 2700 feet, then the pressure P, produced at each apex by a vertical pressure of 20 tons on each side of the pillar is as follows:—

$$P = \frac{Wl}{R} = \frac{20 \times 2}{2700} \text{ tons} = 33{\cdot}2 \text{ lbs.}$$

As there are 14 apices in each system of bracing, *i.e.*, 7 on each side, the strain in each of the end braces $= \dfrac{33 \cdot 2 \times 14 \times 1 \cdot 414}{2}$ $= 328 \cdot 6$ lbs., eq. (118). We thus see that the strain in the internal bracing is comparatively trifling, and that the difficulty of providing against flexure in long compression braces is not so formidable as might have been supposed.

338. Longitudinal strains in braced pillars due to internal bracing.—It will be observed that the internal bracing develops longitudinal strains in the side bars at each apex. These increments are, however, insignificant compared with the pressure of the weight.

339. Each bay of braced pillars resembles a pillar with rounded ends.—Compression flanges of girders resemble braced pillars.—In braced pillars the side bars must be made stiff enough to resist flexure for the length of one bay between the apices of the internal bracing. Each bay cannot, however, be regarded as a pillar of this length firmly fixed at the ends, but rather as one with rounded ends, since it might assume a waved form like the letter S, consecutive bays deflecting in opposite directions. This remark also applies to the compression flanges of girders. The vertical webs preserve them from deflecting in a vertical plane; the cross bracing between the flanges performs the same service in a horizontal plane, and the compression diagonals, especially if they are braced pillars, also convey a large share of rigidity from the tension flanges and roadway to the compression flanges. The failure of the latter, therefore, as far as flexure is concerned, is thus confined to the short length of one bay.

340. Strength of braced pillars independent of length within certain limits.—From Hodgkinson's experiments on plate-iron tubular pillars (**319**), it seems highly probable that the strength of braced pillars is also within considerable limits independent of their length, for internal bracing will generally be made somewhat stronger than theory alone might require.

CHAPTER XIV.

EXTENSION.

341. Tensile strain.—The tendency of tensile strain is to draw the material into a straight line between the points of attachment, and, unless its section alter very suddenly or the mode of attachment be defective so as to produce indirect strain, each transverse section will sustain a uniform unit-strain throughout its whole area; eq. (1) is, therefore, applicable to ties without any other practical correction than this, that if the material be pierced with holes, such as rivet or bolt holes in iron, or knots in timber, the effective area for tension in any transverse section is not the gross, but the net area which remains after deducting the aggregate area of all the holes which occur in that transverse section.

CAST-IRON.

342. Tensile strength.—The following table contains the results of Mr. Hodgkinson's experiments on the tensile strength of various kinds of British cast-iron.* Those samples whose specific gravities are given are the same irons as those whose crushing strengths have been already stated in Table IV. (**208**).

TABLE I.—TENSILE STRENGTH OF CAST-IRON.

Description of Iron.	Specific Gravity.	Tearing Weight per square inch of Section.
		lbs. tons.
Carron (Scotland) Iron, No. 2, hot blast, - - - - -		13,505 = 6·03
Ditto, do., cold blast, - - - - -		13,683 = 7·45
Ditto, No. 3, hot blast, - - - - -		17,755 = 7·93
Ditto, do., cold blast, - - - - -		14,200 = 6·35
Devon (Scotland) Iron, No. 3, hot blast, - - - - -		21,907 = 9·78
Buffery (near Birmingham) Iron, No. 1, hot blast, - -		13,434 = 6·00
Ditto, do., cold blast, - -		17,466 = 7·80

* *Experimental Researches on the Strength and other Properties of Cast-Iron*, by Eaton Hodgkinson, p. 310. Also, *Report of the Commissioners appointed to inquire into the application of Iron to Railway Structures*, 1849, p. 9.

TABLE I.—TENSILE STRENGTH OF CAST-IRON—*continued.*

Description of Iron.	Specific Gravity.	Tearing Weight per square inch of Section.
		lbs. tons
Coed-Talon (North Wales) Iron, No. 2, hot blast,		16,676= 7·45
Ditto, do., cold blast,		18,855= 8·40
Low Moor (Yorkshire), No. 3,		14,535= 6·50
Mixture of Iron,		16,542= 7·39
Low Moor Iron, No. 1,	7·074	12,694= 5·667
Ditto, No. 2,	7·043	15,458= 6·901
Clyde Iron (Scotland), No. 1,	7·051	16,125= 7·198
Ditto, No. 2,	7·093	17,807= 7·949
Ditto, No. 3,	7·101	23,468=10·477
Blaenavon Iron (South Wales), No. 1,	7·042	13,938= 6·222
Ditto, No. 2, first sample,	7·113	16,724= 7·466
Ditto, No. 2, second sample,	7·051	14,291= 6·380
Calder Iron (Lanarkshire), No. 1,	7·025	13,735= 6·131
Coltness Iron (Edinburgh), No. 3,	7·024	15,278= 6·820
Brymbo Iron (North Wales), No. 1,	7·071	14,426= 6·440
Ditto, No. 3,	7·037	15,508= 6·923
Bowling Iron (Yorkshire), No. 2,	6·989	13,511= 6·032
Ystalyfera Anthracite Iron (South Wales), No. 2,	7·119	14,511= 6·478
Yniscedwyn Anthracite (South Wales), No. 1,	7·034	13,952= 6·228
Ditto, No. 2,	7·013	13,348= 5·959
Mean of the foregoing 27 irons,	- -	15,679= 7·00
Mr. Morries Stirling's Iron, denominated 2nd quality,*	7·165	25,764=11·502
Mr. Morries Stirling's Iron, denominated 3rd quality,†	7·108	23,461=10·474

* Composed of Calder, No. 1, hot blast, mixed and melted with about 20 per cent. of malleable iron scrap.

† Composed of No. 1, hot blast, Staffordshire iron, from Ley's works, mixed and melted with about 15 per cent. of common malleable iron scrap.

343. Cold-blast stronger than hot-blast iron.—On comparing the tenacity of hot and cold-blast iron in the first part of the foregoing table, it will be observed that, with one exception, the cold-blast irons are stronger than the hot-blast irons of the same make. This is confirmed by experiments made in the United States, where, since 1840, hot-blast iron has been condemned for ordnance purposes.*

344. Re-melting, within certain limits, increases the strength and density of cast-iron.—Re-melting cast-iron seems to have an important effect in increasing its density as well as in improving its tensile and transverse strength, as appears from the following experiments by Major Wade on proof bars of No. 1 Greenwood pig-iron thrice re-melted†:—

TABLE II.—EXPERIMENTS ON THE TENSILE AND TRANSVERSE STRENGTH OF RE-MELTED CAST-IRON.

	Density.	Tensile strength in lbs. per square inch.	Transverse strength, S in lbs. (64).
Crude pig iron, - - - -	7·032	15129	5290
Do. re-melted once, - -	7·086	21344	6084
Do. do. twice, - -	7·198	30107	7322
Do. do. three times, -	7·301	35786	9448

In summing up the results of his experiments on re-melting cast-iron Major Wade observes, "the softest kinds of iron will endure a greater number of meltings with advantage than the higher (*more decarbonized*) grades, and it appears that when iron is in its best condition for casting into proof bars of small bulk, it is then in a state which requires an additional fusion to bring it up to its best condition for casting into the massive bulk of cannon. In selecting, and preparing iron for cannon, we may, therefore, proceed by repeated fusions, or by varying the proportions of the different grades, until the maximum tenacity in proof

* *Report on the Strength and other Properties of Metals for Cannon.* By Officers of the Ordnance Department U. S. Army. Philadelphia, 1856, p. 236.

† *Ibid*, pp. 242, 249.

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is attained, the iron will then be in its best condition for being again melted and cast into cannon."

Experiments made by Mr. Fairbairn, for the British Association, though on a much more limited scale than those by Major Wade, also prove the advantage to be derived from repeated fusions.* One ton of No. 3 Eglinton hot-blast iron was melted 18 times successively, each time under similar conditions of fusion; proof bars, 5 feet long and 1 inch square, were cast each time, and broken by transverse strain, the distance between the supports being 4 feet 6 inches. The results are given in the following table:—

TABLE III.—EXPERIMENTS ON THE TRANSVERSE AND CRUSHING STRENGTH OF RE-MELTED CAST-IRON.

No. of meltings.	Specific gravity.	Mean breaking-weight of bars exactly 1 in. square, and 4 feet 6 inches between supports, in lbs.	Mean ultimate deflection in inches.	Power to resist impact.	Resistance to compression in tons per square inch.
1	6·969	490·0	1·440	705·6	44·0
2	6·970	441·9	1·446	630·9	43·6
3	6·886	401·6	1·486	596·7	41·1
4	6·938	413·4	1·260	520·8	40·7
5	6·842	431·6	1·503	648·6	41·1
6	6·771	438·7	1·320	579·0	41·1
7	6·879	449·1	1·440	646·7	40·9
8	7·025	491·3	1·753	861·2	41·1
9	7·102	546·5	1·620	885·3	55·1
10	7·108	566·9	1·626	921·7	57·7
11	7·113	651·9	1·636	1066·5	69·8
12	7·160	692·1	1·666	1153·0	73·1
13	7·134	634·8	1·646	1044·9	66·0†
14	7·530	603·4	1·513	912·9	95·9
15	7·248	371·1	0·643	238·6	76·7
16	7·330	351·3	0·566	198·5	70·5
17	Lost.	...	...	...	...
18	7·385	312·7	0·476	148·8	88·0

In Mr. Fairbairn's experiments the transverse strength increased up to the 12th melting, after which it fell off in a marked degree.

345. Prolonged fusion, within certain limits, increases the strength and density of cast-iron.—The improvement due to prolonged fusion is shown by the following experiments by Major Wade on Stockbridge iron of the 2nd fusion.—*Rep.*, p. 44.

* *Application of Iron to Building Purposes*, p. 60.

† The cube did not bed properly upon the steel plates, otherwise it would have resisted a much greater force—probably 80 or 85 tons per square inch.

TABLE IV.—EXPERIMENTS ON PROLONGED FUSION.

	Density.	Tensile strength in lbs. per square inch.	Transverse strength, S in lbs. (64).
Iron in fusion ½ hour, .	7·187	17,843	7,126
Do. do. 1 " .	7·217	20,127	8,778
Do. do. 1½ " .	7·250	24,387	10,083
Do. do. 2 " .	7·279	34,496	11,614

On this subject Mr. Truran makes the following observations* :—

"The composition of the original grey pig-iron doubtless influences, in a very great measure, the amount of improvement obtained with different periods of fusion. A refining of the iron takes place; and the quantity of alloyed matters oxidized and removed will vary with the character of the pig-iron. Carbon is a principal ingredient in cast-iron; and a long exposure, equally with repeated meltings, offers a ready method of burning it away. The reverberating column of gases in the re-melting furnace contains a proportion of free oxygen, which combines with the carbon to form carbonic acid; but since the oxygen is in contact only with the surface of the metal, its removal requires numerous fusions, or the maintenance in fusion for a long period. Repeated fusions of the iron are attended with a heavy waste of material, which goes far to compensate for the increase of strength. The tensile strength, as influenced by the size of the masses and rapidity of cooling, varies with the condition of the iron previous to casting. If the refining process, by lengthened fusion or numerous re-meltings, be carried too far, the resulting product will be of a hard, brittle quality; and when cast into small articles, be chilled to that extent as to be incapable of working with steel cutting-tools. Cast into larger articles, however, and cooled more slowly, a maximum tenacity may be developed, and the texture of the iron be found of a character to bear cutting-tools on its surface. Continuing the operation too long also produces a thickening of the

* *The Useful Metals and their Alloys*, pp. 215, 217. London: 1857.

molten iron, until it is of too great a consistence for the proper filling of the moulds, and the prevention of air cavities in the body of the casting. The burning away of the carbon is attended with a loss of fluidity; and this defect occurring, there is no remedy short of introducing further portions of the original crude iron, to restore, by mixing, a certain degree of fluidity."

346. Tensile strength of thick castings of highly decarbonized iron greater than that of thin ones—Annealing cast-iron diminishes its density and tensile strength.—It has been already shown (**312**) that the transverse strength of thin castings exceeds that of thick ones, and it might naturally be thought that this was always due to greater tensile strength in the smaller castings. This, however, seems to be disproved by the following experiments by Major Wade, of the United States army, who found that small castings in vertical dry sand moulds had a less tensile strength than large gun castings similarly moulded and cast at the same time.* The diminution of strength in the small bars amounted to nearly 5 per cent., while their transverse strength was 14 per cent. greater than that of bars cut from the guns, as is shown in the following table:—

TABLE V.—COMPARISON OF PROOF BARS CUT FROM THE BODY OF THE GUN, WITH THOSE CAST AT THE SAME TIME IN SEPARATE VERTICAL DRY SAND MOULDS, SHOWING THE DIFFERENCE IN THE SAME IRON, CAUSED BY SLOW COOLING IN LARGE MASSES, AND MORE RAPID COOLING IN SMALL CASTINGS.

Guns.	Transverse strength. S in lbs. (**64**).		Tensile strength.		Specific gravity.	
	Bar cut from gun.	Bar cast separate.	Bar cut from gun.	Bar cast separate.	Bar cut from gun.	Bar cast separate.
			lbs.	lbs.		
6-pounder gun,	8415	9880	30234	29143	7·196	7·263
6-pounder gun,	9233	9977	31087	30039	7·278	7·248
8-inch gun,	8575	10176	26367	24583	7·276	7·331
Mean,	8741	10011	29229	27922	7·250	7·281
Proportional,	1·000	1·145	1·000	·995	1·000	1·004

"These results," observes Major Wade, "show that the transverse

* *Report on the Strength and other Properties of Metals for Cannon.*—By Officers of the Ordnance Department, U. S. Army. Philadelphia, 1856, p. 45.

strength is augmented by rapid cooling in small castings, and that the tensile strength is increased by slow cooling in large masses. The differences in specific gravity are less marked; but it is somewhat higher in the small castings cooled rapidly." This conclusion, however, must be qualified by further statements of the same writer at pp. 234 and 268; where, in allusion to similar experiments, he says:—" Such results happen only in cases where the iron is very hard. As a general rule, the tenacity of the common sorts of foundry iron is increased by rapid cooling. In this case the condition of the iron when cool was *too high*—that is to say, the process of decarbonization had been carried too far—for a maximum strength, when cooled rapidly, in small masses; although it was in its best condition for casting into a large mass, where it must cool slowly. As iron of high density, when cast into bars of small bulk, is liable to become unsound and to contain small cavities, this cause may account, in some measure, for the diminished tensile strength in bars of high density." Major Wade found that annealing small bars of cast-iron invariably diminished both their density and tenacity.* American cannon iron, the reader will observe, is much stronger and denser than ordinary English cast-iron, the mean tensile strength of a large number of American guns cast in 1851 being 37,774 lbs. per square inch.†

247. Indirect strain greatly reduces the tensile strength of cast-iron.—Mr. Hodgkinson found "that the strength of a rectangular piece of cast-iron, drawn along the side, is about one-third, or a little more, of its strength to resist a central strain."—*Ex Res.*, p. 312. In proving specimens of cast-iron in a testing machine it is essential that the strain pass exactly through the axis of the specimen, otherwise the apparent will be much less than the real tensile strength.

248. Cast-iron not suited for tension.—Cast-iron is liable to air-holes, internal strains from unequal contraction in cooling, or other concealed defects which often seriously reduce its effective area for tension, and as its tenacity is only about one-third of that of

* *Report on the Strength and other Properties of Metals for Cannon*, p. 234.

† *Idem*, p. 276.

wrought-iron, the latter material or steel should be preferred for tensile strains whenever practicable.

349. Tensile working strain of cast-iron should not exceed one-sixth of the tearing weight.—From the causes just referred to, cast-iron is seldom used in the form of a tie-bar. It frequently occurs, however, in tension in the lower flanges of girders with continuous webs, and some engineers have considered it safe to load these with $\frac{1}{4}$th of their breaking weight. It is now, however, generally allowed that the working load of cast-iron girders, when subject to vibration, should not exceed $\frac{1}{6}$th of their ultimate strength, and as the tension flange is generally the weakest, and, therefore, the first to fail, this rule allows $1\frac{1}{4}$ tons per square inch as the safe tensile working strain for cast-iron of good quality. The student will find copious evidence on this subject in the appendix to the "Report of the Commissioners appointed to inquire into the application of Iron to Railway Structures." In my own practice I adopt the rule of $\frac{1}{6}$th, and find it satisfactory.

WROUGHT-IRON.

350. Tensile strength of wrought-iron—Fractured area—Ultimate set.—To Mr. David Kirkaldy we are indebted for an exceedingly valuable series of experiments on the tensile strength of wrought-iron and steel, made by means of a lever-testing machine at the works of Messrs. Robert Napier and Sons, Glasgow.* The following tables contain abstracts of the more important results of these experiments.

The columns headed "Tearing strain per square inch of fractured area" give the breaking weight per square inch of the area when reduced by the specimen drawing out under proof. The ratio of these to the breaking weights per square inch of original area indicate the quality of the iron, whether ductile or the reverse. The soft and ductile irons draw out to a small "fractured area," and consequently have a very high unit-strain referred to it, whereas the hard irons stretch but little under proof, and therefore have a comparatively low unit-strain

* *Experiments on Wrought-iron and Steel*, by David Kirkaldy, Glasgow, 1863.

referred to the same standard. The last column, headed "Ultimate set," gives the ratio of the increment of length or set after fracture, to the original length before fracture, in the form of a per-centage of the latter. The figures in this column are greater or less according as the material is more or less ductile, and consequently this "ultimate set" is a test of the toughness and ductility of the iron under proof. In my own practice I find that the "ultimate set" is more easily measured than the "fractured area," and that it is a very convenient test of the quality of the iron.

TABLE VI.—TENSILE STRENGTH OF IRON BARS.

NOTE—All the pieces were taken *promiscuously* from engineers' or merchants' stores, except those marked *samples*, which were received from the makers.

District.	Names of the Makers or Works.	Description.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Yorkshire.	LOW-MOOR, - -	Rolled bars, 1 inch square	60,364	117,147	24·9
	Do. - -	Rolled bars, 1 inch round	61,798	131,676	26·5
	Do. - -	Rolled bars, $\frac{11}{16}$ inch, for rivets	60,075	125,775	20·5
	Do. - -	Planed from 1 inch square bars	60,245	114,410	23·8
	Do. - -	Forged from $1\frac{1}{4}$ inch round bars	66,392	115,040	20·2
	BOWLING, - -	Rolled bars, 1 inch round	62,404	114,220	24·4
	Do. - -	Turned from $1\frac{1}{4}$ inch round bars	61,477	120,229	26·0
	FARNLEY, - -	Rolled bars, 1 inch round	62,886	127,425	25·6
Staffordshire.	J. BRADLEY and Co., (L) (Charcoal) - -	*Samples.* Rolled bars, 1 inch round	57,216	146,521	30·2
	Do. B. B., Scrap, -	*Samples.* Rolled bars, 1 inch round	59,370	123,805	26·6
	Do. S C ♛ -	Rolled bars, $\frac{3}{4}$ inch, for rivets	56,715	112,336	22·5
	Do. do. - -	Rolled bars, $\frac{7}{8}$ inch round	62,231	97,575	22·2

TABLE VI.—TENSILE STRENGTH OF IRON BARS—*continued.*

District.	Names of the Makers or Works.	Description.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Staffordshire—*con.*	G. B. THORNEYCROFT & Co., TNS	Rolled bars, 13/16 inch, for rivets	59,278	99,595	22·4
	LORD WARD, L W.R-O	Rolled bars, 11/16 inch, for rivets	59,753	95,724	18·6
	MALINSLEE, BEST -	Rolled bars, ¾ inch × 1 inch	56,289	88,300	21·4
	BAGNALL, J. B. -	Rolled bars, 1¼ inch round	55,000	75,351	17·3
	Do. do. -	Do., do., turned down to 1 inch	55,381	80,638	19·1
Lancashire.	ULVERSTON RIVET, BEST	Rolled bars, ¾ inch round	53,775	104,680	21·6
	MERSEY Co., BEST, -	Forged from ¾ inch square bars	60,110	86,295	16·9
Lanarkshire.	GOVAN, EX. B. BEST, -	Rolled bars, ¾ inch square	56,655	99,000	19·1
	Do. do. -	Rolled bars, ¾ inch round	57,591	95,248	17·3
	Do. do. -	Rolled bars, 1⅛ inch round	58,358	97,821	23·8
	Do. do. -	Rolled bars, 1 inch round	59,109	98,527	22·3
	Do. do. -	Rolled bars, ⅞ inch round	58,169	101,863	19·2
	Do. do. -	Rolled bars, ¾ inch round	57,400	92,880	17·6
	GOVAN, B. BEST, -	Rolled bars, 1⅛ inch round	60,879	84,770	17·0
	Do. do. -	Rolled bars, 1 inch round	62,849	88,550	19·1
	Do. do. -	Rolled bars, ⅞ inch round	61,341	96,442	20·0
	Do. do. -	Rolled bars, ¾ inch round	64,795	97,245	17·3
	Do. do. -	Rolled bars, ¾ inch round	59,548	95,706	16·9
	GOVAN, * -	Rolled bars, 1⅛ inch round	58,326	78,139	16·7
	Do. do. -	Rolled bars, 1 inch round	59,424	79,378	16·4
	Do. do. -	Rolled bars, ⅞ inch round	63,956	88,512	15·8
	Do. do. -	Rolled bars, ¾ inch round	61,887	95,319	18·8
	GLASGOW, B. BEST, -	Rolled bars, 1 inch round	58,885	97,548	23·2
	Do. do. -	Rolled bars, 15/16 inch round	58,910	97,559	21·3
	Do. do. -	Forged from 1 inch rolled bars	59,045	80,053	20·9

TABLE VI.—TENSILE STRENGTH OF IRON BARS—*continued.*

District.	Names of the Makers or Works.	Description.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Lanarkshire—*continued.*	GLASGOW, B. BEST, -	Rolled bars, 1¼ inch round	54,579	85,012	20·3
	Do. do. - -	Do., do., turned down to 1 inch	55,533	86,590	21·3
	Do. do. - -	Do., do., forged down to 1 inch	56,112	81,508	18·6
	Do. do. - -	Rolled bars, ⅝ inch round	59,300	99,612	20·0
	GLASGOW BEST RIVET, -	Rolled bars, ⅞ inch round	57,092	96,205	23·7
	COATBRIDGE, BEST RIVET,	Rolled bars, ¾ inch round	61,723	96,267	21·6
	ST. ROLLOX, BEST RIVET, -	Rolled bars, 13/16 inch round	56,981	77,383	16·6
	R. SOLLOCH E. BEST, -	Rolled bars, 11/16 inch, for rivets	57,425	96,959	17·7
	◇ GOVAN, ◇ -	Rolled bars, 1¼ inch round	57,598	114,866	24·8
	Do. do. - -	Do., do., turned down to 1 inch	57,288	116,869	25·6
	Do. do. - -	Do., do., forged down to 1 inch	57,095	112,705	23·1
	Do. do. - -	Rolled bars, 1 inch round	58,746	113,700	25·2
	Do. do. - -	Rolled bars, ⅝ inch round	58,199	116,549	21·4
	DEMDYVAN (Common) -	Rolled bars, 1¼ inch round	51,327	54,100	6·3
	Do. do. - -	Do., do., turned down to 1 inch	55,995	63,280	11·1
	Do. do. - -	Rolled bars, 1¼ inch, forged down	54,247	60,856	7·3
	Do. do. - -	Rolled bars, 1 inch round	53,852	58,304	6·8
	BLOCHAIRN, B. BEST, -	Rolled bars, 1 inch round	56,141	90,313	21·3
	BLOCHAIRN, BEST RIVET, -	Rolled bars, ¾ inch round	59,219	89,279	19·4
	PORT DUNDAS, EX. B. BEST,	Rolled bars, 1⅛ inch round	54,594	85,563	20·6
	GOVAN, Puddled Iron, -	Rolled bars, ½×2¼ inch, forged down	46,771	48,057	3·4
South Wales.	YSTALYFERA, Puddled Iron,	Rolled bars, ⅜×2¾ inch, forged down	29,626	29,818	0·6
	Do. do. - -	Do., do., strips cut off	38,526	39,470	2·0

TABLE VI.—Tensile Strength of Iron Bars—*continued.*

District.	Names of the Makers or Works.	Description.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Hammered.	Hammered Scrap Iron, -	— —	53,420	94,105	24·8
	Bushelled Iron from Turnings, - -	— —	55,878	72,531	16·6
	Cut out of a Crank Shaft of Hammered Scrap Iron, 14″ wide, and reduced to the required shape in the lathe, not on the anvil,	Length of shaft -	47,582	59,003	21·8
		Across shaft - -	44,758	50,971	16·8
	Do. do.	Lengthways - -	43,759	56,910	20·5
		Crossways - -	38,487	42,059	8·4
	Hammered Armour Plate, 16′ 6″ × 3′ 9″ × 4½″, cut off the end and turned down,	Crossways - ..	38,868	44,611	11·7
		Do. - -	36,824	39,085	6·4
Swedish.	Per Eckman and Co., R F, Gothenburg,	flat tilted bars, Strips cut off -	47,855	121,065	27·8
	Do. do.	flat tilted bars, Forged round -	48,232	150,760	26·4
Russian.	Prince Demidoff, C C N D,	flat tilted bars, Strips cut off -	49,564	73,118	13·3
	Do. do.	flat tilted bars, Forged round -	56,805	77,632	15·3
Foreign.	Swedish, X -	flat tilted bars, Strips cut off	48,933	141,702	17·0
	Do. ♛ ♛ W	flat tilted bars, Strips cut off	43,509	77,349	15·3
	Do. ♛ C	flat tilted bars, Strips cut off	42,421	63,632	15·2
	Russian, I O P 3	flat tilted bars, Strips cut off	59,096	68,047	6·0
	Swedish, X -	flat tilted bars, Forged down	50,262	188,731	18·7
	Do. ♛ C	flat tilted bars, Forged down	41,251	98,510	14·3
	Do. ♛ ♛ W	flat tilted bars, Forged down	44,230	83,851	15·8
	Russian, I O P 3	flat tilted bars, Forged down	51,466	67,907	7·5

TABLE VII.—TENSILE STRENGTH OF ANGLE IRON.

NOTE.—All the pieces were taken *promiscuously* from engineers' or merchants' stores, except those marked *samples*, which were received from the makers.

District.	Names of the Makers or Works.	Size.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Yorkshire, -	FARNLEY, - - -	$\frac{9}{16}$	61,260	104,468	20·9
Lanarkshire,	GLASGOW Best Scrap, -	$\frac{5}{8}$	56,094	71,764	15·0
	GLASGOW Best Best, -	$\frac{9}{16}$	55,937	70,706	15·4
	Do. do. -	$\frac{5}{8}$	55,520	62,373	8·5
	Do. do. -	$\frac{9}{16}$	53,300	65,770	12·8
	Do. do. -	$\frac{5}{8}$	51,800	64,962	12·7
Staffordshire,	ALBION ♛ Best, -	$\frac{5}{8}$	56,157	69,367	14·0
	ALBION Best, - -	$\frac{5}{8}$	52,159	67,695	14·1
	Do. do. - -	$\frac{11}{16}$	51,467	60,675	11·2
	EAGLE Best Best, - -	$\frac{5}{8}$	54,727	71,441	13·7
	EAGLE, - - -	$\frac{21}{32}$	50,056	58,545	8·8
Durham,	CONSETT Best Best, -	$\frac{1}{2}$	53,548	65,554	12·6
	CONSETT Ship Angle Iron,	$\frac{9}{16}$	50,807	58,201	5·8

TABLE VIII.—TENSILE STRENGTH OF IRON STRAPS AND BEAM IRON.

NOTE.—All the pieces were taken *promiscuously* from engineers' or merchants' stores, except those marked *samples*, which were received from the makers.

District.	Names of the Makers or Works.	Size.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Lanarkshire,	GLASGOW, Ship Beam, -	$\frac{5}{8}$ $\frac{1}{2}$	55,937	67,606	10·79
	DUNDYVAN, Ship Strap, -	$\frac{1}{2}$ $\frac{11}{16}$	55,285	63,635	8·03
	MOSSEND, Ship Strap, -	$\frac{9}{16}$ 1	45,439	50,459	5·18
Staffordshire,	THORNEYCROFT, Ship Strap,	$\frac{1}{2}$	52,789	59,918	8·03
S. Wales, -	DOWLAIS, Ship Beam, -	$\frac{1}{2}$	41,386	45,844	4·82

TABLE IX.—Tensile Strength of Iron Plates.

Note.—All the pieces were taken *promiscuously* from engineers' or merchants' stores, except those marked *samples*, which were received from the makers. L denotes that the strain was applied lengthways of the plate ; C, crossways.

District.	Names of the Makers or Works.	Thick.	See Note above.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
				lbs.	lbs.	per cent.
Yorkshire,	Lowmoor, -	5/16	L	52,000	64,746	13·2
			C	50,515	57,383	9·3
	Bowling, - -	3/8	L	52,235	61,716	11·6
			C	46,441	50,009	5·9
	Farnley, - -	3/8	L	56,005	68,763	14·1
			C	46,221	53,293	7·6
	Do. - -	1/4	L	58,487	70,538	10·9
			C	54,098	59,698	5·9
	Do. - -	3/4	Samples L	58,437	83,112	17·0
			Samples C	55,033	68,961	11·3
Durham,	Consett, - -	3/4	L	51,245	59,183	8·93
			C	46,712	52,050	6·43
	Do. Best Best, -	7/16 & 14/16	L	49,120	55,472	8·0
			C	46,755	50,000	5·2
	Do. do. -	7/16 & 9/16	L	53,559	62,346	11·5
			C	45,677	48,358	4·0
Staffordshire,	J: Bradley & Co., S. C. ♛	1/2	L	55,831	67,406	12·5
			C	50,550	55,206	5·5
	Do. L F do. -	3/8 to 1/2	L	56,996	66,858	13·0
			C	51,251	56,070	5·9
	Do. ,, do. -	3/8 to 1/2	L	55,708	65,652	10·7
			C	49,425	54,002	5·1
	T. Wells, Best Best ♛	5/16 to 8/16	L	47,410	51,521	4·0
			C	46,630	48,348	3·4
	K B M - -	5/16	L	46,404	51,896	6·1
			C	44,764	47,891	4·3
	Malinslee, Best ♛	3/8	L	52,572	62,131	8·6
			C	50,627	55,746	5·8
	G. B. ThornEycroft, Best D W Best, -	10/16	L	54,847	62,747	11·2
			C	45,585	47,712	4·6
	J. Wells ♛ B.Best,	10/16 & 12/16	Samples L	45,997	51,140	6·7
			Samples C	49,311	54,842	7·0

TABLE IX.—TENSILE STRENGTH OF IRON PLATES—*continued.*

District.	Names of the Makers or Works.	Thick.	See Note p. 237.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
				lbs.	lbs.	per cent.
Staffordshire—*continued,*	LLOYDS, FOSTER, & Co., Best, - -	$\frac{6}{16}$ to $\frac{7}{16}$	L C	44,967 44,732	49,162 48,344	5·3 4·6
	SNEDSHILL ♛ Best, -	$\frac{6}{16}$ to $\frac{7}{16}$	L C	52,362 43,036	61,581 45,300	9·6 2·8
Shropshire,	MOSSEND, Best Best, -	$\frac{3}{8}$	L C	43,433 41,456	46,038 43,622	3·3 2·9
Lanarkshire,	GLASGOW, Best Boiler, -	$\frac{3}{8}$ to $\frac{11}{16}$	L C	53,849 43,848	60,522 52,252	9·3 4·6
	Do. Ship, -	$\frac{3}{16}$ to $\frac{11}{16}$	L C	47,773 44,355	49,816 45,343	3·65 2·11
	Do. Best Best, -	$\frac{7}{16}$ to $\frac{11}{16}$	L C	45,626 41,340	48,208 42,430	4·34 2·37
	Do. do. -	$\frac{1}{4}$ to $\frac{3}{4}$	L C	53,399 41,791	59,557 43,614	8·95 2·63
	Do. Best Scrap, -	$\frac{3}{8}$	L	50,844	58,412	10·5
	Makers' Stamp uncertain,	$\frac{4}{16}$ to $\frac{11}{16}$	L C	47,598 40,682	53,182 43,426	5·9 2·5
	GOVAN, Best, - -	$\frac{1}{4}$ to $\frac{1}{2}$	L C	43,942 39,544	45,886 40,624	3·4 1·4
	◇ GOVAN ◇ -	$\frac{13}{32}$	L C	54,644 49,399	66,728 54,020	11·6 6·5

251. Tensile strength of wrought-iron, mean results.—The following short table contains the mean results of Mr. Kirkaldy's experiments on the tensile strength of wrought-iron.*

TABLE X.—TENSILE STRENGTH OF WROUGHT-IRON, MEAN RESULTS.

	lbs.	tons.	tons.
188 bars, rolled,	57,557	= $25\frac{3}{4}$	—
72 angle-iron and straps,	54,729	= $24\frac{1}{2}$	—
167 plates, lengthways,	50,737	= 22·65	= $21\frac{3}{4}$ (167 and 160 plates together)
160 plates, crossways,	46,171	= 20·6	

* *Expts.*, p. 96.

252. Kirkaldy's conclusions.—Mr. Kirkaldy sums up the results of his experimental inquiry in the following concluding observations, which the student should study carefully:—*

1. The breaking strain does *not* indicate the quality, as hitherto assumed.
2. A *high* breaking strain may be due to the iron being of superior quality, dense, fine, and moderately soft, or simply to its being very hard and unyielding.
3. A *low* breaking strain may be due to looseness and coarseness in the texture, or to extreme softness, although very close and fine in quality.
4. The contraction of area at fracture, previously overlooked, forms an essential element in estimating the quality of specimens.
5. The respective merits of various specimens can be correctly ascertained by comparing the breaking strain *jointly* with the contraction of area.
6. Inferior qualities show a much greater variation in the breaking strain than superior.
7. Greater differences exist between small and large bars in coarse than in fine varieties.
8. The prevailing opinion of a rough bar being stronger than a turned one is erroneous.
9. Rolled bars are slightly hardened by being forged down.
10. The breaking strain and contraction of area of iron plates are greater in the direction in which they are rolled than in a transverse direction.
11. A very slight difference exists between specimens from the centre and specimens from the outside of crank shafts.
12. The breaking strain and contraction of area are greater in those specimens cut lengthways out of crank shafts than in those cut crossways.
13. The breaking strain of steel, when taken alone, gives no clue to the real qualities of various kinds of that metal.
14. The contraction of area at fracture of specimens of steel must be ascertained as well as in those of iron.
15. The breaking strain, *jointly* with the contraction of area, affords the means of comparing the peculiarities in various lots of specimens.
16. Some descriptions of steel are found to be very hard, and, consequently, suitable for some purposes; whilst others are extremely soft, and equally suitable for other uses.
17. The breaking strain and contraction of area of *puddled-steel* plates, as in iron plates, are greater in the direction in which they are rolled; whereas in *cast*-steel they are less.
18. Iron, when fractured suddenly, presents invariably a crystalline appearance; when fractured slowly, its appearance is invariably fibrous.
19. The appearance may be changed from fibrous to crystalline by merely altering the shape of specimen, so as to render it more liable to snap.
20. The appearance may be changed by varying the treatment, so as to render the iron harder and more liable to snap.
21. The appearance may be changed by applying the strain so suddenly as to render the specimen more liable to snap, from having less time to stretch.
22. Iron is less liable to snap the more it is worked and rolled.
23. The "skin" or outer part of the iron is somewhat harder than the inner part, as shown by appearance of fracture in rough and turned bars.

* *Expts.*, p. 91.

24. The mixed character of the scrap-iron used in large forgings is proved by the singularly varied appearance of the fractures of specimens cut out of crank shafts.

25. The texture of various kinds of wrought-iron is beautifully developed by immersion in dilute hydrochloric acid, which, acting on the surrounding impurities, exposes the metallic portion alone for examination.

26. In the fibrous fractures the threads are drawn out, and are viewed externally, whilst in the crystalline fractures the threads are snapped across in clusters, and are viewed internally or sectionally. In the latter cases the fracture of the specimen is always at right angles to the length ; in the former it is more or less irregular.

27. Steel invariably presents, when fractured slowly, a silky fibrous appearance ; when fractured suddenly, the appearance is invariably granular, in which case also the fracture is always at right angles to the length ; when the fracture is fibrous, the angle diverges always more or less from 90°.

28. The granular appearance presented by steel suddenly fractured is nearly free of lustre, and unlike the brilliant crystalline appearance of iron suddenly fractured ; the two combined in the same specimen are shown in iron bolts partly converted into steel.

29. Steel which previously broke with a silky fibrous appearance, is changed into granular by being hardened.

30. The little additional time required in testing those specimens, whose rate of elongation was noted, had no injurious effect in lessening the amount of breaking strain, as imagined by some.

31. The rate of elongation varies not only extremely in different qualities, but also to a considerable extent in specimens of the same brand.

32. The specimens were generally found to stretch equally throughout their length until close upon rupture, when they more or less suddenly drew out, usually at one part only, sometimes at two, and, in a few exceptional cases, at three different places.

33. The ratio of ultimate elongation may be greater in short than in long bars in some descriptions of iron, whilst in others the ratio is not affected by difference in the length.

34. The lateral dimensions of specimens forms an important element in comparing either the rate of, or the ultimate, elongations—a circumstance which has been hitherto overlooked.

35. Steel is reduced in strength by being hardened in water, while the strength is vastly increased by being hardened in oil.

36. The higher steel is heated (without, of course, running the risk of being burned) the greater is the increase of strength, by being plunged into oil.

37. In a highly converted or hard steel the increase in strength and in hardness is greater than in a less converted or soft steel.

38. Heated steel, by being plunged into oil instead of water, is not only considerably *hardened*, but *toughened* by the treatment.

39. Steel plates hardened in oil, and joined together with rivets, are fully equal in strength to an unjointed soft plate, or the loss of strength by riveting is more than counterbalanced by the increase in strength by hardening in oil.

40. Steel rivets, fully larger in diameter than those used in riveting iron plates of the same thickness, being found to be greatly too small for riveting steel plates, the probability is suggested that the proper proportion for iron rivets is not, as generally assumed, a diameter equal to the thickness of the two plates to be joined.

41. The shearing strain of steel rivets is found to be about a fourth less than the tensile strain.

42. Iron bolts, case-hardened, bore a less breaking strain than when wholly iron, owing to the superior tenacity of the small proportion of steel being more than counterbalanced by the greater ductility of the remaining portion of iron.

43. Iron highly heated and suddenly cooled in water is hardened, and the breaking strain, when gradually applied, increased, but at the same time it is rendered more liable to snap.

44. Iron, like steel, is softened, and the breaking strain reduced, by being heated and allowed to cool slowly.

45. Iron subject to the cold-rolling process has its breaking strain greatly increased by being made extremely hard, and not by being "consolidated," as previously supposed."

46. Specimens cut out of crank-shaft are improved by additional hammering.

47. The galvanizing or tinning of iron plates produces no sensible effects on plates of the thickness experimented on. The result, however, may be different, should the plates be extremely thin.

48. The breaking strain is materially affected by the shape of the specimen. Thus the amount borne was much less when the diameter was uniform for some inches of the length than when confined to a small portion—a peculiarity previously unascertained, and not even suspected.

49. It is necessary to know correctly the exact conditions under which any tests are made before we can equitably compare results obtained from different quarters.

50. The startling discrepancy between experiments made at the Royal Arsenal, and by the writer, is due to the difference in the shape of the respective specimens, and not to the difference in the two testing machines.

51. In screwed bolts the breaking strain is found to be greater when old dies are used in their formation than when the dies are new, owing to the iron becoming harder by the greater pressure required in forming the screw thread when the dies are old and blunt than when new and sharp.

52. The strength of screw-bolts is found to be in proportion to their relative areas, there being only a slight difference in favour of the smaller compared with the larger sizes, instead of the very material difference previously imagined.

53. Screwed bolts are not necessarily injured, although strained nearly to their breaking point.

54. A great variation exists in the strength of iron bars which have been cut and welded; whilst some bear almost as much as the uncut bar, the strength of others is reduced fully a third.

55. The welding of steel bars, owing to their being so easily burned by slightly overheating, is a difficult and uncertain operation.

56. Iron is injured by being brought to a white or welding heat, if not at the same time hammered or rolled.

57. The breaking strain is considerably less when the strain is applied suddenly instead of gradually, though some have imagined that the reverse is the case.

58. The contraction of area is also less when the strain is suddenly applied.

59. The breaking strain is reduced when the iron is frozen; with the strain gradually applied, the difference between a frozen and unfrozen bolt is lessened, as the iron is warmed by the drawing out of the specimen.

60. The amount of heat developed is considerable when the specimen is suddenly stretched, as shown in the formation of vapour from the melting of the layer of ice on one of the specimens, and also by the surface of others assuming tints of various shades of blue and orange, not only in steel, but also, although in a less marked degree, in iron.

61. The specific gravity is found generally to indicate pretty correctly the quality of specimens.

62. The density of iron is *decreased* by the process of wire-drawing, and by the similar process of cold rolling, instead of *increased*, as previously imagined.

63. The density in some descriptions of iron is also decreased by additional hot-rolling in the ordinary way; in others the density is very slightly increased.

64. The density of iron is decreased by being drawn out under a tensile strain, instead of increased, as believed by some.

65. The most highly converted steel does not, as some may suppose, possess the greatest density.

66. In cast-steel the density is much greater than in puddled-steel, which is even less than in some of the superior descriptions of wrought-iron.

The foregoing extracts afford the reader but a meagre idea of Mr. Kirkaldy's laborious researches, and the student who seeks more detailed information regarding his experiments, or the instruments and method he adopted in testing specimens, is referred to his book on the subject.

353. Strength of iron plates lengthways 10 per cent. greater than crossways—Removing skin of wrought-iron does not injure its tensile strength.—From Table X. it appears that the average strength of iron plates drawn in the direction of their length is about ten per cent. greater than when drawn across the grain. The ultimate set is also much greater in the direction of the fibres. This agrees with Mr. Clark's experiments* and with my own experience. With reference to the effect of removing the outer skin or glaze on rolled iron, Mr. Kirkaldy observes, "The generally received opinion, that by removing the 'skin' the relative strength was greatly reduced, or that a *rough* bar was much stronger than one *turned* to the same diameter, is proved to be erroneous."†

354. Bar iron stronger than plates—Boiler plates—Ship plates—Hard iron unfit for ship-building.—From Table X. it appears that bars of ordinary sizes are nearly 14 per cent. stronger than plates; perhaps this does not apply to bars of large section, say three inches in diameter and upwards. The great demand for iron ships has given rise to the manufacture of a cheap quality of plate iron called "ship" or "boat" plates; this iron is generally inferior in strength and toughness to "boiler" plates, and is often so hard and brittle that its ultimate set does not exceed three per cent., while its tensile strength is frequently less than eighteen tons per square inch. There can be no greater mistake

* Clark *on the Tubular Bridges*, p. 377.

† *Expts.*, p. 27.

than to suppose that hard iron is fit for ships. Iron plates which are tough and ductile like copper will, when struck, often escape with a mere dint or bulge, whereas hard iron under the same circumstances will crack or tear, especially along a line of rivet holes.

255. Forgings not so strong as rolled iron—Annealing reduces the tensile strength of wrought iron, but increases its ductility.—It is generally believed that large forgings are less tenacious than small ones (**252** 46). About this, however, there is some difference of opinion, and the subject requires further experiments before it can be definitively settled.* Large forgings certainly require greater manufacturing skill than small ones, and it is probable that most forgings are somewhat weaker in tensile strength than bar or plate iron to which the rolling process imparts a fibrous structure; this view seems to be confirmed by Mr. Kirkaldy's experiments on hammered iron in Table VI. Annealing iron also reduces its tensile strength (**252** 44 and Ex. 1, p. 40), though it increases its ductility and toughness, which are sometimes more important. For instance, it is a good practice to anneal old crane chains which have become brittle by constant use, and thus render them less liable to snap from sudden jerks.

256. Tensile working strain of wrought-iron should not exceed one-fourth of its tearing strength.—For ordinary purposes, such as girder work, the maximum tensile working strain of wrought-iron should not exceed $\frac{1}{4}$th of its ultimate strength, and, as the tenacity of common plate iron is about 20 tons per square inch, this rule allows 5 tons per square inch of *net* sectional area as the safe working strain for plates of ordinary quality. Where structures, such as cranes, are subject to sudden shocks the working strain should not exceed $\frac{1}{6}$th of the ultimate strength, and in boilers or machinery it is not prudent to make the working strain exceed $\frac{1}{8}$th of the tearing strain of the iron.

IRON WIRE.

257. Tensile strength of iron wire—Annealing iron wire reduces its tensile strength.—From Mr. Telford's experiments

* See discussion on Mr. Mallet's paper on the Coefficients of Elasticity and Rupture in Massive Forgings.—*Proceedings of Inst. of C. E.*, Vol. xviii., p. 296.

it appears that the strength of iron wire $\frac{1}{10}$th inch diameter = 36 tons per square inch.* The strength of the iron wire used by Mr. Roebling at the Niagara Falls suspension bridge is nearly 100,000lbs. (= 44·6 tons) per square inch. This wire measures 18·31 feet per lb., and is "small No. 9 Gauge, 60 wires forming one square inch of solid section."†

Table XI. contains the results of experiments made by M. Seguin, senior, on iron-wire of different sizes and qualities. ‡

TABLE XI.—TENSILE STRENGTH OF IRON WIRE.

Description of Wire.	Diameter.	Tearing Strain per Square Millimetre.
	Millimetres.	Kilogrammes.
Iron wire from Bourgogne, No. 8, unequally annealed,	1·172	38·2
Idem, No. 7, carefully annealed,	1·062	36·1
Idem, No. 18, not annealed,	3·366	58·8
Idem, No. 7, not annealed,	1·062	73·7
Fil de l'Aigle, employed for carding,	0·2294	89·8
Passe-perle, rather soft,	0·5917	85·7
Wire from a factory in Besançon—		
No. 1, soft,	0·6188	86·1
2, soft,	0·7078	87·0
3, brittle,	0·7327	80·8
4, brittle,	0·838	76·6
5, very brittle,	0·9115	72·3
6	1·022	76·1
7	1·08	71·2
8, very brittle,	1·123	67·3

* Barlow *on the Strength of Materials*, p. 283.

† *Papers and Practical Illustrations of Public Works of Recent Construction, both British and American.* Weale: 1856. pp. 16, 18.

‡ *Résumé des leçons sur l'application de la Mécanique.* Par M. Navier. Bruxelles: 1839. p. 30.

TABLE XI.—TENSILE STRENGTH OF IRON WIRE—*continued.*

Description of Wire.	Diameter	Tearing Strain per Square Millimetre.
Wire from a factory in Besançon—*continued.*	Millimetres.	Kilogrammes.
No. 9, rather brittle,	1·293	69·8
10, very soft,	1·435	64·8
11, very soft,	1·476	58·6
12	1·691	55·5
13	1·8	57·2
14, very soft, without elasticity,	2·072	49·3
15	2·226	51·9
16, very soft,	2·489	63·9
17, flawed,	2·695	68·1
18	3·087	84·0
19	3·492	78·2
20	4·14	65·7
21	4·812	62·5
22, very brittle,	5·449	67·7
23, soft,	5·942	62·6

NOTE.—A millimetre equals very nearly $\frac{1}{25}$th of an inch; and kilogrammes per square millimetre may be converted into tons per square inch by multiplying by 0·635.

That annealing iron wire seriously impairs its tensile strength may be inferred from the foregoing experiments.

STEEL.

358. Tensile strength of steel.—The following tables contain the principal results of Mr. Kirkaldy's experiments on the tensile strength of steel bars and plates. His "conclusions" respecting this material will be found in (**359**).

TABLE XII.—Tensile Strength of Steel Bars.

Note.—All the pieces were taken *promiscuously* from engineers' or merchants' stores, except those marked *samples*, which were received from the makers.

District.	Names of the Makers or Works.	Description.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
			lbs.	lbs.	per cent.
Sheffield.	T. Turton and Sons, Cast Steel for tools (from Acadian Iron),	All forged from rolled bars by the same smith, reheated after hammering and allowed to cool gradually.	132,909	139,124	5·4
	Thomas Jowitt, Cast Steel for Tools,		132,402	151,857	5·2
	Do. do., Cast Steel for Chisels,		124,852	150,243	7·1
	Do. do., Cast Steel for Drifts,		115,882	147,570	13·3
	T. Jowitt, Double Shear Steel,		118,468	147,396	13·5
	Bessemer, Sheffield (tool), *samples*,		111,460	143,327	5·5
	Wilkinson, Ⓛ Blister Steel,		104,298	132,472	9·7
	T. Jowitt, Cast Steel for Taps,		101,151	142,070	10·8
	T. Jowitt, Spring Steel,	Forged from ¾ inch rolled bars,	72,529	95,490	18·0
	Moss and Gambles, Cast Steel for Rivets, *samples*,	Rolled bars, ¾ inch round,	107,286	158,013	12·4
	Naylors, Vickers, and Co., Cast Steel for Rivets,	Rolled bars, ¾ inch round,	106,615	158,785	8·7
	Shortridge, Howell, and Co., Homogeneous Metal,	Rolled bars, $\frac{9}{16}$ inch, for rivets,	90,647	142,920	13·7
	Do., do.,	Forged, - -	89,724	121,212	11·9
Liverpool.	Mersey Co., Puddled Steel,	Forged, - -	71,486	110,451	19·1
Glasgow.	Blochairn, Puddled Steel,	Rolled bars, -	70,166	84,871	11·3
	Do., do.,	Forged from slabs,	62,255	80,370	12·0
	Do., do.,	Forged from rolled bars,	62,769	71,231	9·1
Prussia.	Krupp, Dusseldorf, Cast Steel for Bolts,	Rolled bars, round,	92,015	139,434	15·3

TABLE XIII.—TENSILE STRENGTH OF STEEL PLATES.

NOTE.—All the pieces were taken *promiscuously* from engineers' or merchants' stores, except those marked *samples,* which were received from the makers. L denotes that the strain was applied *lengthways* of the plate; C, *crossways.*

District.	Names of the Makers or Works.	Thick.	Description.	Tearing Strain per square inch of original area.	Tearing Strain per square inch of fractured area.	Ultimate Set.
				lbs.	lbs.	per cent.
Sheffield.	T. TURTON AND SONS, Cast Steel,	$\frac{1}{4}$	Samples L	94,289	100,063	5·71
			C	96,308	111,811	9·64
	NAYLOR, VICKERS, AND Co., Cast Steel,	$\frac{1}{4}$	L	81,719	104,232	17·50
			C	87,150	112,018	17·32
	MOSS AND GAMBLES, Cast Steel,	$\frac{3}{16}$ & $\frac{4}{16}$	L	75,594	105,554	19·82
			C	69,082	112,546	19·64
	SHORTRIDGE, HOWELL, AND Co., Homogeneous Metal,	$\frac{3}{16}$	L	96,280	114,106	8·61
			C	97,150	114,300	8·93
	Do., do.,	$\frac{3}{8}$	C	96,989	113,305	14·4
	Do., Second Quality, -	$\frac{1}{4}$	Samples L	72,408	81,823	5·93
			C	73,580	78,245	3·21
Liverpool.	MERSEY Co., Puddled Steel (Ship Plates),	$\frac{3}{16}$ & $\frac{3}{16}$	L	101,450	109,552	2·79
			C	84,968	91,746	1·25
	MERSEY Co., Puddled Steel "Hard,"	$\frac{1}{4}$	Samples L	102,593	107,827	4·86
			C	85,365	89,116	3·30
	Do. "Mild," do.,	$\frac{1}{4}$	L	77,046	88,240	6·16
			C	67,686	73,634	5·72
	Do. do. (Ship Plates), -	$\frac{9}{32}$	L	71,532	77,520	3·57
Glasgow.	BLOCHAIRN, Puddled Steel, -	$\frac{3}{16}$	Samples L	102,234	108,079	3·60
			C	84,398	87,877	2·68
	Do., do. (Boiler plates),	$\frac{3}{16}$	L	96,320	107,614	8·22
			C	73,699	76,646	4·14

359. Steel plates often deficient in uniformity and toughness—Safe working strain.—The reader will observe that the ultimate set of steel plates is in general small compared with that of the tougher kinds of iron in Table IX. This indicates the direction to which manufacturers of steel should direct their attention, as for many purposes, especially shipbuilding, toughness and ductility are quite as essential as great tensile strength (**354**). Judging from the experiments in Table XIII., plates of cast-steel (now called "crucible" cast-steel to distinguish it from Bessemer cast-steel, Homogenous metal, or other recent inventions) are tougher than puddled steel. Sometimes steel plates are so brittle as to fly in pieces under the hammer, or split in punching, and thick plates are said to possess this undesirable quality to a greater degree than thin ones. Complaints also are made of want of uniformity of texture, some plates of a lot being all that could be desired, while others of the same lot may be hard and brittle. Owing to this uncertainty the manufacture of steel plates seems still in a transition state, and consequently engineers and naval architects have not made use of the material to the extent to which its superior tensile strength seems to destine it. We cannot, therefore, infer from extensive practice what is the safe working strain for steel. Probably one-fourth of the tearing strain, or from 8 to 10 tons per square inch for plates, is a safe tensile working strain. The most important steel girder bridge which has come under my notice is that constructed by Major Adelsköld, of the Royal Swedish Engineers, for the Herljunga and Wenersborg Railway in Sweden. The girder is an inverted bowstring, carrying the railway in one span of 137½ feet over a rapid torrent (**315**). "The dimensions are calculated for a strain of 8 tons per square inch, every portion having been tested to 16 tons per square inch before being put in place." *

DIFFERENT METALS AND ALLOYS.

360. Tensile strength of different metals and alloys.— The following table contains the tensile strength of different metals and alloys by various experimenters.

* *The Engineer*. Vol. xxii., p. 240, 1866.

TABLE XIV.—TENSILE STRENGTH OF DIFFERENT METALS AND ALLOYS.

Description of Metal.	Specific Gravity.	Initials of Experimenters.	Tearing Strain per Square Inch.
			lbs. tons.
Brass, Fine Yellow Cast,	—	R.	17,968 = 8·02
Do., Wire,	—	D.	91,325 = 40·77
Copper, Wrought, reduced per hammer,	—	R.	33,792 = 15·08
Do., do., in bolts,	—	K.	47,936 = 21·40
Do., Cast,	—	R.	19,072 = 8·51
Do., do., Lake Superior,	8,672	W.	24,252 = 10·82
Do., Sheet,	—	N.	30,016 = 13·4
Do., Wire, not annealed,	—	M. D.	77,504 = 34·6
Do., do., annealed,	—	M. D.	32,144 = 14·35
Gun Metal or Bronze, hard,	—	R.	36,368 = 16·23
Do., mean of 83 gun-heads,	8,523	W.	29,655 = 13·24
Do., mean of 5 breech-squares,	8,765	W.	46,509 = 20·76
Do., mean of 32 small bars cast in same moulds with guns,	8,584	W.	42,019 = 18·76
Do., small bars cast separately in { iron moulds,	8,953	W.	37,688 = 16·82
Do., small bars cast separately in { clay do.,	8,313	W.	25,788 = 11·51
Do., in finished guns,	—	W.	23,108 to 52,192 = 10·3 to 23·3
Yellow Metal, Patent,	—	K.	49,185 = 21·9
Lead, Cast,	—	R.	1,824 = 0·81
Do., Sheet,	—	N.	1,926 = 0·86
Tin, Cast,	—	R.	4,736 = 2·11
Do., Banco,	7,297	W.	2,122 = 0·95
Do.,	—	M. D.	2,845 = 1·27

D. Dufour, *Résumé des leçons sur l'application de la Mécanique.* Par M. Navier. Brussels, 1839, p. 35.

M. D. Minard et Desormes, *idem*, pp. 34, 36.

N. Navier, *idem*, p. 36.

K. Kingston, *Barlow on the Strength of Materials*, p. 211.

R. Rennie, *Philosophical Transactions for* 1818, p. 126.

W. Wade, *Reports of Experiments on Metals for Cannon, by Officers of the Ordnance Department, U. S. Army*, 1856, pp. 281, 288, 289, 290, 295.

361. Alloys of copper and zinc, and copper and tin.—Table XV. contains the results of experiments made by R. Mallet, Esq., on the physical properties of certain alloys of copper and zinc, and copper and tin.*

TABLE XV.—PHYSICAL PROPERTIES OF ALLOYS OF COPPER AND ZINC, AND COPPER AND TIN.

Chemical Constitution.	Composition by weight per cent.	Specific Gravity.	Tearing Strain per square inch in tons.	Commercial Title.
COPPER AND ZINC.				
Cu	100 + 0	8·667	24·6	Copper.
5 Cu + Zn	83·02 + 16·98	8·415	13·7	Bath Metal.
4 Cu + Zn	79·65 + 20·35	8·448	14·7	Dutch Brass.
3 Cu + Zn	74·58 + 25·42	8·397	13·1	Rolled Sheet Brass.
2 Cu + Zn	66·18 + 33·82	8·299	12·5	British Brass.
19 Cu + 12 Zn	60·00 + 40·00	8·200	1·9	Muntz Patent Sheathing.
Cu + Zn	49·47 + 50·53	8·230	9·2	German Brass.
Cu + 2 Zn	32·85 + 67·15	8·283	19·3	,, ,, Watchmakers.
Cu + 4 Zn	19·65 + 80·35	7·371	1·9	White Button Metal.
Zn	0 + 100	6·895	15·2	Zinc, brittle.
COPPER AND TIN.				
10 Cu + Sn	84·29 + 15·71	8·561	16·1	Gun Metal.
9 Cu + Sn	82·81 + 17·19	8·462	15·2	Gun Metal.
8 Cu + Sn	81·10 + 18·90	8·459	17·7	Gun Metal, tempers best.
7 Cu + Sn	78·97 + 21·03	8·728	13·6	Hard Mill Brasses, &c.
Cu + Sn	34·92 + 65·08	8·056	1·4	Small bells, brittle.
Cu + 3 Sn	15·17 + 84·83	7·447	3·1	Speculum Metal of Authors.
Sn	0 + 100	7·291	2·5	Tin.

NOTE.—"The ultimate cohesion was determined on prisms of 0·25 of an inch square, without having been hammered or compressed after being cast. The weights given are those which each prism just sustained for a few seconds before rupture."

* *On the Construction of Artillery.* By Robert Mallet, F.R.S. 1856, p. 82.

The tensile strength of brass wire and unannealed copper wire in Table XIV., and that of cast copper in Table XV., seem high, while that of Muntz Patent Sheathing in Table XV., seems low; further experiments, perhaps, may modify them.

362. Gun metal or bronze—High temperature at casting injurious to bronze.—The proportion of tin to copper in the bronze gun metal on which Major Wade experimented was 1 to 8, and the great diversity in its tenacity seems attributable to defective homogeneity in the alloy, some parts containing more tin than others, and consequently remaining longer fluid. A high temperature at casting is injurious to the quality of bronze, as it seems to facilitate the separation of the metals, and small bars are stronger than large castings, probably because the former solidify more suddenly and are thereby not allowed a sufficient time for a division of the alloy into separate compounds. Bronze guns are cast on end in flask moulds, with the breech downwards, and a large extra head of metal above the muzzle to ensure sufficient liquid pressure. Breech-squares, being at the bottom of the moulds, are subject to a much higher pressure than the gunheads which are at the top, and they are consequently both stronger and denser than the latter. The small bars cast in the gun mould are stronger than those cast separately, probably in consequence of their being under greater pressure, and because they were fed, as they solidified, from the mass of the gun with which they communicated. Major Wade also attributes their superiority to the annealing process they underwent after solidification, from the proximity of the large mass of the gun.*

TIMBER.

363. Tensile strength of timber.—The following table contains the results of experiments by various authorities on the tensile strength of timber in the direction of the fibres.

* *Rep. on Expts. on Metals for Cannon*, pp. 296, 299.

TABLE XVI.—Tensile Strength of Timber Lengthways.

Description of Wood.	Tearing Strain in lbs. per Square Inch.	Authority.
Mahogany,	8,000	Barlow.
Do.	21,800	Bevan.
Acacia,	16,000	Bevan.
Alder,	13,900	Muschenbroeck.
Ash,	12,000	Do.
Do.	16,700	Bevan.
Do.	17,000	Barlow.
Beech,	11,500	Barlow.
Do.	17,300	Muschenbroeck.
Do.	22,000	Bevan.
Box,	20,000	Barlow.
Cedar,	4,880	Muschenbroeck.
Do.	11,400	Bevan.
Chesnut, Spanish,	13,300	Rondelet.
Do.	10,500	Bevan.
Cypress,	6,000	Muschenbroeck.
Elder,	10,000	Do.
Elm,	13,200	Do.
Do.	14,400	Bevan.
Fir,	8,330	Muschenbroeck.
Do.	12,000	Barlow.
Jugeb,	18,500	Muschenbroeck.
Lance Wood,	23,400	Bevan.
Larch,	10,220	Rondelet.
Do.	8,900	Bevan.
Lemon,	9,250	Muschenbroeck.
Locust-tree,	20,100	Do.

TABLE XVI.—TENSILE STRENGTH OF TIMBER LENGTHWAYS—*continued.*

Description of Wood.	Tearing Strain in lbs. per Square Inch.	Authority.
Mulberry,	12,500	Muschenbroeck.
Oak,	17,300	Do.
Do., English,	10,000	Barlow.
Do., do.	19,800	Bevan.
Do.	13,950	Rondelet.
Do., Black Bog,	7,700	Bevan.
Orange,	15,500	Muschenbroeck.
Pear,	9,800	Barlow.
Pine, Pitch,	7,650	Muschenbroeck.
Do., Norway,	14,300	Bevan.
Do., do.	7,287	Rondelet.
Do., Petersburg,	13,300	Bevan.
Plum,	11,800	Muschenbroeck.
Pomegranite,	9,750	Do.
Poplar,	5,500	Do.
Do.	7,200	Bevan.
Quince,	6,750	Muschenbroeck.
Sycamore,	13,000	Bevan.
Tamarind,	8,750	Muschenbroeck.
Teak,	15,000	Barlow.
Do., old,	8,200	Bevan.
Walnut,	8,130	Muschenbroeck.
Do.	7,800	Bevan.
Willow,	12,500	Muschenbroeck.

Barlow, *Barlow on the Strength of Materials*, p. 23.
Muschenbroeck, *idem*, p. 4.
Bevan, *Tredgold's Carpentry*, 4th edition, p. 41.
Rondelet, *idem*.

364. Lateral adhesion of the fibres.—The following table gives the lateral adhesion of the fibres, that is, the tensile strength of timber across the grain, in which direction it is much weaker than lengthways.

TABLE XVII.—Tensile Strength of Timber Crossways.

Description of Wood.	Tearing Strain in lbs. per Square Inch.	Authority.
Fir, Memel, - - - -	540 to 840	Bevan.
Do., Scotch, - - - -	562	Do.
Larch, - - - - -	970 to 1,700	Tredgold.
Oak, - - - - -	2,316	Do.
Poplar, - - - - -	1,782	Do.

Bevan, *Tredgold's Carpentry*, p. 42.
Tredgold, *idem.*

365. Tensile working strain of timber should, in general, not exceed one-tenth of its tearing strain.—Owing to its liability to decay, the tensile working strain of timber in permanent structures should in general not exceed $\frac{1}{10}$th of its tearing strain. When, however, timber is used for merely temporary purposes, such as military bridges, a much higher working strain, probably $\frac{1}{4}$th of the tearing strain, may be safely adopted, as Mr. Barlow states that he "left more than three-fourths of the whole weight hanging for 24 or 48 hours, without perceiving the least change in the state of the fibres, or any diminution of their ultimate strength." *

STONE, BRICK, MORTAR, CEMENT, GLASS.

366. Tensile strength of stone.—As stone is rarely employed in direct tension, there are but few experiments on its tensile strength.

* Barlow *on the Strength of Materials*, p. 24.

TABLE XVIII.—TENSILE STRENGTH OF STONE.

Name of Material.	Tearing Strain in lbs. per Square Inch.	Authority.
Arbroath Pavement,	1,261	Buchanan.
Caithness do.	1,054	Do.
Craigleith Stone,	453	Do.
Hailes,	336	Do.
Humbie,	283	Do.
Binnie,	279	Do.
Redhall,	326	Do.
Whinstone,	1,469	Do.
Marble, White,	722	Do.
Do., do.	551	Hodgkinson.

Buchanan, see *Practical Mechanics' Journal*, Vol. I., pp. 237, 285.
Hodgkinson, see *Tredgold on the Strength of Cast-iron*, p. 287.

367. Tensile strength of Plaster of Paris and Lime mortar.

TABLE XIX.—TENSILE STRENGTH OF PLASTER OF PARIS AND LIME MORTAR.

Name of Material.	Tearing Strain in lbs. per Square Inch.	Authority.
Plaster of Paris,	71	Rondelet.
Mortar of Quartzose Sand and eminently Hydraulic Lime, well made,	136	Vicat.
Mortar of Quartzose Sand and ordinary Hydraulic Lime, well made,	85	Do.
Mortar of Quartzose Sand and ordinary Lime, well made,	51	Do.
Mortar badly made,	21	Do.

Rondelet, see *Résumé des leçons sur l'application de la Mécanique à l'établissement des Constructions.* Par M. Navier, p. 13.

Vicat, *idem.*

868. Adhesion of Plaster of Paris and Lime mortar to brick or stone.—Rondelet states that the adhesive strength of plaster of Paris to brick or stone is about ⅔rds of its tensile strength. Its adhesion is greater for millstone and brick than for limestone, and diminishes greatly with time. Rondelet also states that the adhesion of mortar to stone or brick exceeds its tensile strength.*

869. Tensile strength of Portland cement and Cement mortar—Organic matter or loam very injurious to cement mortar.—The following tables show the tensile strength of cements and cement mortar. With the exception of the French experiments in Table XX., they are all taken from Mr. Grant's valuable paper on the *Strength of Cement* in the *Proceedings of the Institution of Civil Engineers*, Vol. xxv., p. 66. Proof samples of cement are generally made into ■━■ shaped bricks, 1½ inches square at the waist; these are immersed in water as soon as the cement sets, and they remain immersed till the time of testing.

TABLE XX.—TENSILE STRENGTH OF ENGLISH AND FRENCH CEMENTS.

Description of Cement.	Tearing Strain in lbs. per Square Inch.	Authority.
English artificial Portland Cement, at the end of 7 days' immersion in water,	270	Grant.
Do., after 1 year's immersion,	478	Do.
Do., do.	427 to 498	Belgrand and Michelot.
Boulogne natural Portland, do.	640 to 711	Do.
Roman Cement, from "Septaria," - -	170 to 218	Do.

Belgrand and Michelot, see *Gillmore on Limes, Hydraulic Cements, and Mortars*, p. 268. New York, 1863.

Artificial Portland Cement is made of chalk and clay in certain definite proportions, carefully ground together in water. The mixture is then run off into reservoirs where it settles, and, after attaining sufficient consistency, it is artificially dried and then calcined in kilns at a high temperature, the calcination being carried to the verge of vitrification. The burned cement is ground in the ordinary way between millstones.

* *Résumé des Leçons*, p. 13.

TABLE XXI.—METROPOLITAN MAIN DRAINAGE—CONTRACTS ON THE SOUTH SIDE—PORTLAND CEMENT, SEVEN DAY TESTS, FROM 1859 TO 1865.

Names of Manufacturers and Agents.	Quantity in Bushels.	Average Weight per Bushel.	Number of Tests.	On area = 2·25 square Inches. 7 days old.	
				Average Breaking Tests.	Specified Standard Test.
		lbs.		lbs.	lbs.
Mr. Robins - - -	128,467	119·43	1,428	670·67	400
Burham Brick and Cement Company (Mr. Webster)	938,322	112·89	7,028	631·09	400 & 500
Messrs. Lee and Co. -	99,450	120·03	962	612·85	400
Messrs. E. Bow and Co. (Agents)	1,000	—	10	546·80	—
Messrs. J. B. White & Brothers	34,430	109·13	327	544·18	—
Mr. Hilton - - -	27,788	114·17	510	511·01	—
Messrs. Knight and Co. -	69,358	112·66	574	500·53	—
Mr. Smeed - - -	44,241	111·85	423	452·36	—
Mr. Wood (agent) - -	3,600	107·88	102	444·50	—
Messrs. Cubitt and Co. -	—	112·00	6	409·11	—
Mr. Buckwell (Agent) -	1,400	100·00	3	408·50	—
Messrs. Fletcher & Co. (Agents)	20,154	118·00	179	394·03	—
Messrs. Francis Brothers -	1,000	105·66	31	328·22	—
Mr. Tatham (Agent) - -	—	106·00	14	184·71	—
Generally - -	1,369,210	114·15	11,587	606·80 = 270 lbs. ℔ sq. in.	

NOTE—1 cubic foot = ·779 bushels.
1 bushel = 1·283 cubic feet.

TABLE XXII.—TABLE of the Results of 960 experiments with Portland Cement, weighing 112 lbs. to the Imperial Bushel, gauged neat, and with different proportions of various kinds of Sand, showing the Breaking Weight on a Sectional Area of 2·25 Square Inches. 1862 and 1863.

Age and Time of Immersion in Water.	On a Sectional Area = 2·25 Square Inches.															
	Cement Neat.	Clean Thames Sand.					Clean Pit Sand.					Loamy Pit Sand.				
		1 to 1.	1 to 2.	1 to 3.	1 to 4.	1 to 5.	1 to 1.	1 to 2.	1 to 3.	1 to 4.	1 to 5.	1 to 1.	1 to 2.	1 to 3.	1 to 4.	1 to 5.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
1 Week -	445·0	97·0	52·5	27·0	—	—	152·0	64·5	44·5	22·0	—	114·2	53·0	21·0	—	—
1 Month	679·9	309·3	123·5	58·0	32·5	21·0	326·5	166·5	91·5	71·5	49·0	274·7	130·5	68·0	60·5	31·5
3 Months	877·9	367·0	254·5	135·5	109·0	88·5	549·6	451·9	305·3	153·0	123·5	448·3	354·0	149·0	118·5	78·5
6 Do.	978·7	546·3	425·1	232·4	157·0	95·5	639·2	497·9	304·0	275·6	218·8	586·5	415·6	274·2	225·5	141·0
9 Do.	995·9	607·3	431·5	—	—	—	718·7	594·4	383·6	—	—	600·1	516·3	321·3	226·7	154·3
12 Do.	1075·7	700·3	458·5	320·6	221·6	122·3	795·9	607·5	424·4	317·6	215·6	645·5	533·2	358·4	244·4	166·2

Both organic matter and loam in the sand are very detrimental to the strength of cement mortar. Clean sharp sand, quite free from argillaceous matter, will give the best result.

TABLE XXIII.—TABLE of the Results of 160 Experiments with Portland Cement weighing 123 lbs. to the Imperial Bushel, gauged neat, and with an equal proportion of clean Thames Sand, showing the Breaking Weight on a Sectional Area of 2·25 Square Inches. These form the first portion of a Series intended to extend over 10 years. The whole of the Specimens were kept in Water form the time of their being made till the time of Testing, 1863 and 1864.

Age.	On Area = 2·25 Square Inches.	
	Neat Cement.	1 of Cement to 1 of Sand.
	Average Breaking Test of 10 Experiments.	Average Breaking Test of 10 Experiments.
	lbs.	lbs.
7 Days . .	817·1	353·2
1 Month . .	935·8	452·5
3 Months . .	1055·9	547·5
6 Ditto . .	1176·6	640·3
9 Ditto . .	1219·5	692·4
12 Ditto . .	1229·7	716·6
2 Years . .	1324·9	790·3
3 Ditto . .	1314·4	784·7

TABLE XXIV.—Southern Outfall Works, Crossness. Summary of Portland Cement Tests, from 1862 to 1866, showing generally increase of Strength with increased Specific Gravity.

Number of Bushels.	Average Weight per Bushel.	Tearing Strain on Area = 2·25 Square Inches. 7 days old.	Number of Bushels.	Average Weight per Bushel.	Tearing Strain on Area = 2·25 Square Inches. 7 days old.
	lbs.	lbs.		lbs.	lbs.
1,300	106	472·6	12,500	119	777·9
5,300	107	592·3	18,530	120	732·3
26,166	108	650·1	15,144	121	705·6
37,036	109	646·6	5,000	122	716·6
20,820	110	708·3	5,428	123	673·6
6,900	111	693·8	13,400	124	819·9
13,812	112	687·5	5,400	125	816·2
10,610	113	701·5	1,800	126	657·2
24,224	114	699·7	1,800	127	864·6
16,240	115	705·5	3,600	128	916·6
27,400	116	768·3	1,820	129	920·2
26,800	117	718·4	1,800	130	913·9
23,306	118	644·1	327,136	—	550·6

370. Tensile strength of Roman cement and Cement mortar.—The following tables contain the results of Mr. Grant's experiment's on the tensile strength of Roman cement and cement mortar. This cement is very inferior in strength to Portland, and is apt to vegetate and crumble away, especially if mixed with loamy sand.

Roman Cement is a natural cement, derived from argillo-calcareous, kidney-shaped stones, called "Septaria," belonging to the Kimmeridge and London clay, generally gathered on the sea shore, though sometimes dug out of the ground.

TABLE XXV.—TABLE of the Results of 90 Experiments with Roman Cement and Sand. Manufactured by Messrs. J. B. WHITE and BROTHERS. March, 1864.

Age and Time immersed in Water.	On Area = 2·25 Square Inches.					
	Neat Cement.			1 Cement to 1 Sand.		
	Minimum Breaking Test.	Maximum Breaking Test.	Average Breaking Test.	Minimum Breaking Test.	Maximum Breaking Test.	Average Breaking Test.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
7 Days .	170	240	202·0	15	45	26·5
14 Ditto .	160	190	173·0	—	—	—
21 Ditto .	170	205	186·5	—	—	—
1 Month	246	291	260·3	—	—	—
3 Months	307	344	322·5	—	—	—
6 Ditto .	442	502	472·7	—	—	—
9 Ditto .	313	520	471·1	—	—	—
12 Ditto .	596	680	643·1	—	—	—

TABLE XXVI.—TABLE of the Results of 250 Experiments with Roman Cement and Sand. Manufactured by Mr. JAMES R. BLASHFIELD. March, 1864.

Age and Time immersed in Water.	On a Sectional Area = 2·25 Square Inches.																	
	Neat Cement.			1 to 1 Sand.			1 to 2 Sand.			1 to 3 Sand.			1 to 4 Sand.			1 to 5 Sand.		
	Min. Breaking Test.	Max. Breaking Test.	Average Breaking Test.	Min. Breaking Test.	Max. Breaking Test.	Average Breaking Test.	Min. Breaking Test.	Max. Breaking Test.	Average Breaking Test.	Min. Breaking Test.	Max. Breaking Test.	Average Breaking Test.	Min. Breaking Test.	Max. Breaking Test.	Average Breaking Test.	Min. Breaking Test.	Max. Breaking Test.	Average Breaking Test.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
7 Days -	95	150	120·5	35	75	47·5	5*	10*	7·0*	10†	10†	10·0†	10†	10†	10†			
14 Ditto -	145	209	169·9	53	79	65·6	22	57	42·8	8*	30*	19·2*						
21 Ditto -	128	171	155·2	54	89	74·2	37	56	45·9	8	25	17·4	*	*	*	*	*	*
1 Month	343	378	358·2	75	91	81·2	8	54	41·9	*	*	*						
3 Months	160	274	220·4	32	160	121·9	72‡	126‡	91·75‡				40§	54§	45·5	29‖	48‖	37·71‖
6 Ditto -	227	300	252·5	296	337	314·3	—	—	—	—	—	—	—	—	—	—	—	—
9 Ditto -	169	300	251·5	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
12 Ditto -	216	300	268·5	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

* Five of these would not bear the minimum strain. † Eight do. ‡ Two do. § Six do. ‖ Three do.

371. Tensile strength of Keene's, Parian, and Medina cements.—The following tables contain the results of Mr. Grant's experiments on the tensile strength of Keene's, Parian, and Medina cements. The two former are chiefly used for internal decoration; Medina is a natural cement, like Roman, and is inferior in strength to artificial Portland cement, which is made of chalk and clay. Medina is useful for pointing the joints of quick-setting marine masonry which has been set in Portland cement. It hardens rapidly and prevents the rising tide from washing the slower setting Portland out of the joints before it has had time to harden sufficiently to resist the action of water in motion.

TABLE XXVII.—TABLE of the Results of 120 Experiments with Keene's Cement, manufactured by Messrs. J. B. WHITE and BROTHERS; and Parian Cement, manufactured by Messrs. FRANCIS and SONS.

In Water in Testing-house, and out of Water in Testing-house, September, 1864.

Age and Time Immersed in Water.	On Area = 2·25 Square Inches.			
	Keene's Cement.		Parian Cement.	
	In Water.	Out of Water.	In Water.	Out of Water.
	Average Breaking Test.	Average Breaking Test.	Average Breaking Test.	Average Breaking Test.
	lbs.	lbs.	lbs.	lbs.
7 Days - - -	543·9	546·0	595·1	642·3
14 Ditto - - -	486·9	585·8	600·8	671·2
21 Ditto - - -	503·0	579·4	543·4	696·6
1 Month - -	490·2	584·2	544·3	746·7
2 Months - -	454·7	648·4	500·7	725·6
3 Ditto - - -	508·8	720·5	521·1	853·7

TABLE XXVIII.—TABLE OF THE RESULTS OF 100 EXPERIMENTS WITH MEDINA CEMENT AND SAND. MANUFACTURED BY MESSRS. FRANCIS BROTHERS, 1864.

Age and Time Immersed in Water.	On Area = 2·25 Square Inches.					
	Neat Cement.			1 Cement to 1 Sand.		
	Minimum Breaking Test.	Maximum Breaking Test.	Average Breaking Test	Minimum Breaking Test.	Maximum Breaking Test.	Average Breaking Test.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
7 Days - -	83	100	92·1	—	—	—
Ditto (2nd Series) -	195	235	211·0	41	63	49
14 Days - -	238	335	303·4	—	—	—
21 Ditto - -	274	332	298·0	—	—	—
1 Month - -	210	346	306·0	—	—	—
3 Months - -	420	468	448·8	—	—	—
6 Ditto - -	376	438	412·4	—	—	—
9 Ditto - -	438	507	457·2	—	—	—
12 Ditto - -	456	527	476·9	—	—	—

372. Grant's conclusions.—The following conclusions are the result of Mr. Grant's numerous experiments on cement during the execution of the Southern Metropolitan Main Drainage Works:—

1. Portland cement, if it be preserved from moisture, does not, like Roman cement, lose its strength by being kept in casks, or sacks, but rather improves by age; a great advantage in the case of cement which has to be exported.
2. The longer it is in setting, the more its strength increases.
3. Cement mixed with an equal quantity of sand is at the end of a year approximately three-fourths of the strength of neat cement.
4. Mixed with two parts of sand, it is half the strength of neat cement.
5. With three parts of sand, the strength is a third of neat cement.
6. With four parts of sand, the strength is a fourth of neat cement.
7. With five parts of sand, the strength is about a sixth of neat cement.
8. The cleaner and sharper the sand, the greater the strength.
9. Very strong Portland cement is heavy, of a blue-grey colour, and sets slowly. Quick setting cement has, generally, too large a proportion of clay in its composition, is brownish in colour, and turns out weak, if not useless.

10. The stiffer the cement is gauged, that is, the less the amount of water used in working it up, the better.

11. It is of the greatest importance, that the bricks, or stone, with which Portland cement is used, should be thoroughly soaked with water. If under water, in a quiescent state, the cement will be stronger than out of water.

12. Blocks of brick-work, or concrete, made with Portland cement, if kept under water till required for use, would be much stronger than if kept dry.

13. Salt water is as good for mixing with Portland cement as fresh water.

14. Bricks made with neat Portland cement are as strong at from six to nine months as the best quality of Staffordshire blue brick. or similar blocks of Bramley Fall stone, or Yorkshire landings.

15. Bricks made of four parts or five parts of sand to one part of Portland cement will bear a pressure equal to the best picked stocks.

16. Wherever concrete is used under water, care must be taken that the water is still. Otherwise, a current, whether natural or caused by pumping, will carry away the cement, and leave only the clean ballast.

17. Roman cement, though about two-thirds the cost of Portland, is only about one third its strength, and is therefore double the cost, measured by strength

18. Roman cement is very ill adapted for being mixed with sand.

373. Tensile strength of glass.—

TABLE XXIX.—TENSILE STRENGTH OF GLASS.

Description of Glass.	Tearing Strain per Square Inch.	Authority.
	lbs. tons.	
Glass Tubes and Rods - -	3,527 = 1·57	Navier.
Annealed Flint Glass Rod - -	2,413 = 1·07	Fairbairn and Tate.
Common Green Glass Rod - -	2,896 = 1·29	Do.
White Crown Glass Rod - -	2,546 = 1·14	Do.

Fairbairn and Tate, *Philosophical Transactions*, 1859, p. 216.

Navier, *Résumé des leçons sur l'application de la Mécanique à l'établissement des Constructions*, p. 87.

374. Thin plates of glass stronger than stout bars—Crushing strength of glass is 12 times its tensile strength.—In their experiments on the resistance of thin glass globes to internal pressure, Messrs. Fairbairn and Tate found that the tenacity of glass in the form of thin plates is 5,000 lbs. per square inch, or about twice that of glass in the form of bars. On this they observe:—"The tensile strength is much smaller in the case of glass fractured by a direct strain in the form of bars, than when burst by internal pressure in the form of thin globes. This difference is, no doubt, mainly due to the fact that thin plates of this material generally possess a higher tenacity than stout bars, which, under the most favourable circumstances, may be but imperfectly annealed." "The ultimate resistance of glass to a crushing force is about 12 times its resistance to extension"* (**333**).

CORDAGE.

375. Tensile strength of cordage.—Table XXX. gives the sizes, weights, and strength of different kinds of best Bower cables employed in the British Navy.† The strength was determined by the chain-testing machine in Woolwich Dockyard, in which the strain is measured by levers.

TABLE XXX.—Tensile Strength of Bower Cables.

Best Bower Hempen Cables, 100 Fathoms.				Number of Threads in each.	Tearing Strain by Experiment.		
Circumference.	Weight.						
Inches.	Cwt.	qrs.	lbs.		Cwt.	qrs.	lbs.
23	96	2	27	2,736	114	0	0
22	89	0	12	2,520	89	0	0
21	80	0	22	2,268			
18	58	2	6	1,656	63	0	0
14¼	38	0	21	1,080	40	0	0

Table XXXI. "shows the mean results of 300 trials made by Captain Huddart. It shows the relative strength or cohesive power of each kind of rope, taking as a standard of comparison $\frac{1}{10}$th of a circular inch, equal to an area of ·078 or nearly $\frac{1}{13}$th of a square

* *Phil. Trans.*, 1859, pp. 216, 246.

† Barlow *on the Strength of Materials*, p. 260

inch. It shows that ropes formed by the warm register are stronger than those made up with the yarns cold; because the heated tar is more fluid, and penetrates completely between every fibre of hemp, and because the heat drives off both air and moisture, so that every fibre is brought into close contact by the twisting and compression of the strand; the tar thus fills up every interstice, and the rope becomes a firmly agglutinated elastic substance almost impermeable to water. But, although rope so made is both stronger and more durable, it is less pliable, and therefore the cold registered rope is more generally used for crane work, where the rope must be wound round barrels, or passed through pullies." *

TABLE XXXI.—TENSILE STRENGTH OF CORDAGE.

Size of Ropes.		Tearing Strain, Made by the Old Method.				Tearing Strain, Made by the Register.			
Girth.	Diameter.	Of common Staple Hemp.	Per $\frac{1}{10}$ of a Circular inch in area.	Of the best Petersburgh Hemp.	Per $\frac{1}{10}$ of a Circular inch in area.	Cold Register.	Per $\frac{1}{10}$ of a Circular inch in area.	Warm Register.	Per $\frac{1}{10}$ of a Circular inch in area.
In.	In.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
3	0·95	5,050	561	6,030	670	7,380	935	8,640	960
3½	1·11	6,784	554	8,669	707	11,165	911	11,760	906
4	1·27	8,768	548	10,454	653	13,108	819	15,360	960
4½	1·43	10,308	504	12,440	614	16,825	806	19,440	960
5	1·59	13,250	530	15,775	631	20,500	820	24,000	960
5½	1·75	15,488	512	18,604	614	24,805	820	29,040	960
6	1·91	18,144	504	21,616	600	24,520	820	33,120	920
6½	2·07	20,533	486	26,323	559	34,645	820	40,554	959
7	2·24	22,932	468	27,342	558	40,188	819	47,040	960
7½	2·39	24,975	444	30,757	546	46,125	820	54,000	960
8	2·54	26,880	421	32,000	500	52,480	820	61,430	960

NOTE.—$\frac{1}{10}$th of a circular inch = ·078, or nearly $\frac{1}{13}$th of a square inch.

* Glynn's *Rudimentary Treatise on the Construction of Cranes and Machinery*, pp. 93, 94.

376. Strength and weight of Cordage—English rule—French rule.—"The old ropemakers' rule was to square the girth of the rope in inches, which, multiplied by four, gave the ultimate or breaking strength of the rope in cwts., and it was a good rule for small cordage, up to 7 inches in circumference. The square of half the circumference was considered to represent the weight of a fathom in pounds."* The old ropemakers' rule for strength is equivalent to 2·51 tons per square inch of section.

The French rule, as given by Morin,† allows 2·79 tons per square inch for the tearing strain of tarred hemp cordage.

377. Working strain of cordage should not exceed one-fourth of its breaking weight.—Cordage rapidly deteriorates by use and exposure to the weather, and when passed round barrels or pullies the outer strands are subject to greater strains than those next the barrel. For this reason, as well as to diminish useless work, the diameters of pullies and barrels should be made as large as practicable. Experience alone can estimate the proper allowance to be made for wear and friction, and after deducting this from the original tearing strength, one-fourth of the remainder is a sufficient load in practice, though for merely temporary purposes one-third of the tearing strain may be considered safe.

CHAINS.

378. Stud-link or Cable chain — Close-link or Crane chain — Open long-link or Buoy chain.— *Stud-link chain* is chiefly used for ships' cables, and derives its name from the cast-iron stud or stay which is inserted across the shorter diameter of each oval link to keep the sides from closing together under heavy strains. It also prevents the chain from kinking, to which long links without stays are liable. *Short* or *close-link* chain is that in common use. It is well adapted for crane work where flexibility is essential to enable the chain to pass freely round barrels and pullies. Open *long-link* chain without studs is used for permanent mooring

* Glynn's *Rudimentary Treatise*, p. 92.

† *Résistance des Matériaux*, p. 41.

cables, where flexibility is a secondary object, and where lightness is desirable, as in the case of lightships or beacon buoys.

379. Tensile strength of stud-chain.—The following table contains the results of experiments on the tensile strength of stud-chain made by Mr. William Smale, leading man of the test house in Her Majesty's Dockyard, Woolwich.* Mr. Smale found that the average tearing strain of good round bars of one inch diameter was 19 tons, or 24·19 tons per square inch of section; their greatest strength being about 20 tons, or 25·33 tons per square inch of section.

TABLE XXXII.—TENSILE STRENGTH OF STUD-CHAIN.

Size of Chain.	Length of each Piece.	Number of Pieces Tested.	Mean Tearing Strain.	Government Proof Strain.	Ratio of Tearing to Proof Strain.	Area of Bar.	Tearing Strain per square inch of each side of link.	
In.	Ft. in.		Tons.	Tons.		Sq. in.	Tons.	
⅝	24 0	6	9·58	7·00	1·37	·307	15·6	Manufactured by various contractors for the Government.
¾	"	6	13·51	10·125	1·33	·442	15·3	
1	"	6	24·25	18·00	1·35	·785	15·4	
1⅛	"	6	29·54	22·75	1·30	·994	14·9	
1½	"	6	59·58	40·50	1·47	1·767	16·9	
1¾	"	6	74·125	55·125	1·34	2·405	15·4	
1⅞	"	6	92·88	63·25	1·47	2·761	16·8	
2	"	3	99·54	72·00	1·38	3·141	15·8	
⅞	2	20	20·38	13·75	1·48	·601	16·9	
1¾	Single links	30	78·70	55·125	1·42	2·405	16·3	Made in Woolwich Dockyard.
		Mean	—	—	1·39		15·9	

* *Report from the Select Committee on Anchors, &c.* (Merchant Service), 1860. Appendix, pp. 151, 152.

Messrs. Brown, Lenox, & Co., inform me that they have found by experience that the average breaking strain of stud-link chain, up to 2¼ inches, is from 900 to 1,000 lbs. per circular ⅛th of an inch of the diameter of iron—equivalent to from 16·37 to 18·19 tons per square inch of each side of the link. This is for cables of *good quality*, much chain being made of a description of iron that will stand the proof and but little more. Hence stud-chain is about ⅔rds as strong as bar iron of the same sectional area as both sides of the links together; in other words, the bar loses 33 per cent. of its strength by being converted into a link.

Ex. A one-inch stud-chain contains 64 circular ⅛ths, and, if of good quality, its tearing strain = 64 × 900 = 57,600 lbs. = 25·7 tons. The tearing strain of two round bars of good iron, each one inch diameter, = 2 × 19 = 38 tons.

380. Government Proof-strain of Stud-chain.—By a recent Act of Parliament,* which continues in force till July, 1872, no maker or dealer in chain cables or anchors may sell for the use of any vessel any chain cable whatever, or any anchor exceeding 168 lbs., unless they have been previously submitted to the same proof-strain as that adopted in Her Majesty's Naval Service. For stud-chain this proof-strain equals 630 lbs. per circular ⅛th of an inch of the diameter of iron, equivalent to 11·46 tons per square inch of each side of the link. Hence the Government proof for stud-chains is about ⅔rds of the ultimate strength of cables of *good quality*, and one-half the strength of ordinary bar iron—*i.e.*, the Government proof of a stud-chain is equal to the ultimate strength of the single bar of which it is made, supposing the latter equal to 23 tons per square inch.

Ex. A one-inch stud chain has 1·57 square inches of area in both sides of the link together, and 1·57 × 11·46 = 18 tons = the proof-strain. The ultimate strength of *best best* chain should reach $\frac{3}{2} \times 18 = 27$ tons, and the breaking weight of the single bar should not be less than 18 tons, though better still, 19 or 20 tons.

The following table gives the proof-strains and weight per 100 fathoms of stud-chain cables for Her Majesty's Naval Service.

* Chain Cables and Anchors Act, 1864.

TABLE XXXIII.—SCALE OF PROOFS SHOWING THE TENSILE STRAIN TO WHICH CHAIN CABLES ARE SUBJECTED BEFORE BEING RECEIVED FOR THE USE OF HER MAJESTY'S NAVAL SERVICE.

Diameter of Iron of Common Links.	Common Links. Mean Length 6 Diameters of the Iron; not to be over more than one-tenth of a Diameter.	Common Links. Mean Width 3·6 Diameters of the Iron; not to be over or under more than one-tenth of a Diameter.	Stay Pins, one Diameter of the Iron at the ends; 0·6 do. at the centre. Weight of each not to exceed.	Weight of 100 fathoms of Cable in 8 lengths, including 4 swivels, and 8 joining shackles, not to be exceeded by more than one-fifteenth part for sizes 2¼ inch and upwards, and not more than one-twentieth part for sizes under 2¼ inch.			Weight of 100 fathoms, with the allowance added.			Proof strain, equal to 630 lbs. per circular ⅛th inch.
In.	In.	In.	Ozs.	Cwts.	qrs.	lbs.	Cwts.	qrs.	lbs.	Tons.
2¾	16½	9·9	72	363	0	0	387	0	22	136⅛
2½	15	9·0	54·69	300	0	0	320	0	0	112½
2⅜	14¼	8·55	47·5	270	3	0	288	3	6	101½
2¼	13½	8·1	40	243	0	0	259	0	22	91⅛
2⅛	12¾	7·65	33·584	216	3	0	227	2	9	81¼
2	12	7·2	28	192	0	0	201	2	11	72
1⅞	11¼	6·75	23	168	3	0	177	0	21	63¼
1¾	10½	6·3	18·76	147	0	0	154	1	11	55⅛
1⅝	9¾	5·85	15	126	3	0	133	0	9	47½
1½	9	5·4	11·81	108	0	0	113	1	17	40½
1⅜	8¼	4·95	9	90	3	0	95	1	4	34
1¼	7½	4·5	6·836	75	0	0	78	3	0	28⅛
1⅛	6¾	4·05	4·983	60	3	0	63	3	4	22¾
1	6	3·6	3·5	48	0	0	50	1	16	18
⅞	5¼	3·15	2·344	36	3	0	38	2	10	13¾
¾	4½	2·7	1·473	27	0	0	28	1	11	10⅛
11/16	4⅛	2·475	1·137	22	2	21	23	3	8	8½
⅝	3¾	2·25	·854	18	3	0	19	2	21	7
9/16	3⅜	2·025	·622	15	0	21	15	3	22	5½
½	3	1·8	·437	12	0	0	12	2	11	4½
7/16	2⅝	1·575	·293	9	0	21	9	2	16	3½

NOTE.—The tensile strain is applied to each of the 8 lengths separately, and not to the whole length of 100 fathoms at one time.

Cables generally weigh the full weight allowed, the iron being rolled a little full to allow for waste in the manufacture. Those for the merchant service are usually made in lengths of 15 fathoms each.

381. Close-link chain—Proof-strain.—The Admiralty proof-strain for close-link chain is 420 lbs. per circular ⅛th of an inch of the diameter of iron, or ⅔rds of that for stud-chains; this is equivalent to 7·63 tons per square inch of each side of the link, or nearly one-half the breaking weight of the chain.

Table XXXIV. gives the proof-strain and weight per 100 fathoms of close-link chain, the extreme length of links not to exceed 5 diameters of the iron; it also gives the size and weight of rope of equal strength.

TABLE XXXIV.—ADMIRALTY PROOF-STRAINS FOR CLOSE-LINK CHAIN.

Diameter of Chain.	Average Weight per 100 Fathoms.	Proof strain, equal to 420 lbs. per circular ⅛th inch.	Girth of Rope of equal Strength.	Weight of Rope per Fathom.
Inches.	Cwt.	Tons.	Inches.	Lbs.
1⅝	155	31⅝	—	—
1½	125	27	—	—
1⅜	104	22⅝	—	—
1¼	86	18¾	—	—
1⅛	70	15¼	—	—
1	56	12	10	22
15/16	50	10½	9½	19½
⅞	42	9⅛	9	17½
13/16	35	7⅞	8¼	15
¾	32	6¾	7½	12
11/16	25	5⅝	7	10½
⅝	21	4⅝	6¼	8½
9/16	16	3¾	5½	7
½	13	3	4¾	5
7/16	10	2¼	4	3¾
⅜	7	1⅝	3¼	2½
5/16	5	1⅛	2½	1½
¼	3	¾	2	—
3/16	2	8½ cwt.	1½	—

The rope of the foregoing table "is such as is now generally made by machinery at most of the large rope works, but was formerly known as 'Patent Rope,' in which every yarn is made to bear its part of the strain; but if common hand-laid rope be used, the proof-strain must be reduced one-fourth, and in actual work the load should not, at any time, exceed one-half the proof."*

282. Open long-link chain — Admiralty proof-strain — Trinity proof-strain.—The links of long-link chain are not oval like those of a stud-chain, but parallel-sided, and the open-link chain of same length of link as the stud-chain is lighter by the weight of the studs. As already observed, it is suited for moorings of a permanent character, such as those of harbour mooring buoys, beacon buoys, or light ships, which are seldom shifted and where consequently flexibility is a secondary object. Besides its comparative lightness, long-link chain has another advantage over either close-link or stud-chain, for each 15-fathom length of the two latter requires long open-links at the ends for the purpose of connecting it by shackles to the adjoining lengths, and if one of these chains break, a whole length must be taken out, since there is not room for a shackle to pass through the ordinary close-link or stud-link. When, however, a long-link chain breaks, the links adjoining the fracture can be connected together without taking out a whole 15 fathom length, as a shackle will pass through any of the long links. The Admiralty proof for large open long-link chain without studs is 315 lbs. per circular ⅛th of an inch, or one-half the proof of stud-chain, as shown in the following table.

TABLE XXXV.—ADMIRALTY PROOF-STRAINS FOR PENDANT AND BRIDLE CHAINS.

Diameter of Iron.	Proof strain, equal to 315 lbs. per circular ⅛th inch.	
Inches.	Tons.	
3½	110	
3¼	95	
3	81	Permanent deflection or collapsion of link not to exceed one quarter of an inch.
2⅞	74	
2¾	68	
2⅝	62	
2½	56	
2	36	

* Glynn on the *Construction of Cranes*, p. 92.

I am indebted to the Elder Brethren of the Trinity House for the following information, obtained through the courtesy of the Secretary, respecting the proofs adopted by the Corporation in testing cable chains such as are used for mooring light-ships and beacon buoys:—

"The chains are subjected, in lengths of 15 fathoms, to a strain of 466 lbs. per circular ⅛th inch of the diameter of iron (equivalent to 8·47 tons per square inch of each side of the link, or one-half the breaking weight of the chain). They are then repeatedly struck, while the strain is on, with heavy sledge hammers; and, after careful examination, a link is broken out indiscriminately from parts of the chain to show the quality of the iron. New and somewhat larger links are substituted for those which are broken or prove defective in the application of the process for trying their strength, when the chain again undergoes a similar test. The sizes of the chains which are made to stand these tests vary from ¾ inch to 2 inches, and the length of the links equals 6 diameters of the iron. Trinity chains are manufactured of iron specially selected and prepared for the service, and the test applied to them—which was determined after numerous experiments—is the highest strain to which open-linked chain can be subjected without altering the shape of the link, and is comparatively much more severe than the usual test for chain without studs. On this account the Elder Brethren do not consider that the tests applied to the chains specially supplied for the service of the Corporation should be adopted as the standard by which to test chains of the same dimensions of ordinary manufacture."

383. French Government proof.—In the French Marine the proof for stud-chains ⅝th inch in diameter and upwards equals 10·8 tons per square inch of the bar. For chains less than ⅝th inch, without studs, the proof is 8·9 tons per square inch.*

384. Insufficiency of present system of proof.—It will be observed that, although the ordinary system of proof by a limited tensile strain may be a sufficient test of the workmanship, it does not prove the quality of the iron, much chain being made of an inferior

* Morin, *Résistance des Matériaux*, p. 42.

description of iron that will stand the proof and but little more. The strength and toughness of the iron, as well as the welding, may be tested by tearing asunder short lengths of the chain in the proving machine, and also by breaking individual links on the anvil with a heavy sledge hammer; the more the iron stretches under proof, and the more battering the links endure, the greater the toughness of the iron and the better the workmanship.

385. Working strain of chains should not exceed one-half the proof-strain.—Mr. Glynn* states that both stud and short-link chain "may safely be worked to half the strain to which they have been proved, but not to more." This for stud-chain = $\frac{11{\cdot}46}{2} = 5{\cdot}73$ tons per square inch of each side of the link, or $\frac{1}{3}$rd of the ultimate strength of *good* chain, and $\frac{1}{4}$th of the tearing strain of ordinary bar-iron.

For close-link chain this rule allows $\frac{7{\cdot}63}{2} = 3{\cdot}81$ tons, per square inch of each side of the link, or $\frac{1}{6}$th of the tearing strain of ordinary bar-iron.

386. Comparative strength of stud and close-link chain.—I am indebted for the following practical observations to the courtesy of Messrs. Brown, Lenox, & Co., the eminent manufacturers of anchors and chains:—"We are not of opinion that studs increase the strength of chain, or enable it to bear a heavier ultimate breaking strain than if made without them, both descriptions being made of the same length of link. The object of their being used is to prevent collapse of the link, which in open-link chain takes place at a strain considerably below the breaking strain, and, of course, renders the chain unserviceable. They thereby enable chains, made with them, to be used for heavier strains than open-link chain, but do not add to their ultimate strength—indeed, from the experiments we have tried, and the experience we have had, we are inclined to believe that the link without stay-pins almost invariably breaks at a higher strain than stud-chains. The proof for studded chain is the higher, only because a sufficient proof cannot be given to open-link

* *Rudimentary Treatise on the Construction of Chains*, p. 91.

chain before the link spoils its form and becomes rigid. The stay prevents collapse, by which the link is prevented elongating so much, and taking its natural position before its utmost power is exhausted and a break ensues. The link, if sound in the workmanship, will nearly always break near the stay-pin, which is caused by the nip across the stay-pin. If made without stays, it will collapse until it is rigid, and the iron will reach as near as possible the direct line of the strain, or right through the centre of the chain; the sides of the links will incline inwards, and the break will ensue at the nip across the crown of the next link."

287. Weight and strength of bar-iron, stud-chain, close-link chain, and cordage.—Stud-chain is about $3\frac{1}{2}$ times as heavy as the bar of which it is made: thus, one fathom of $1\frac{1}{2}$ inch stud-chain weighs about 125 lbs.—a bar 21 feet long would weigh about 124 lbs. Close-link chain is about 4 times as heavy as the bar: thus, one fathom of $1\frac{1}{2}$ chain weighs about 140 lbs.—a bar 24 feet long would weigh about 141 lbs. Close-link chain is about 12 per cent. heavier than stud-chain made with stay-pins of Government dimensions; large and heavy stays are introduced by some manufacturers into ordinary cables, thereby greatly increasing the useless weight of cast-iron, and enabling the chain to be sold cheaper by weight. The following table shows at a glance the relative weight and strength of bar-iron, stud-chain, close-link chain, and hemp cordage.

TABLE XXXVI.—WEIGHT AND STRENGTH OF BAR-IRON, CHAIN, AND CORDAGE.

	Weight of 100 Fathoms; (d = diameter in inches).	Tearing Strain per square inch.	Relative weight of equal lengths of the same ultimate strength; *i.e.*, each length on the point of rupture from the same load.	Safe Working Strain per square inch.	Relative weight of equal lengths of the same useful strength *i.e.*, each length strained to its limit of safe working strain from the same load.
	Tons.	Tons.		Tons.	
Bar-Iron, ...	$0{\cdot}70d^2$	24	100	6·0	100
Stud-chain, ...	$2{\cdot}45d^2$	16 on each side of link	262	5·7 on each side of link	184
Close link chain, . .	$2{\cdot}80d^2$	16	300	3·82	314
Hemp Cordage (hand made),	$0{\cdot}11d^2$	2·51	150	0·63	150

WIRE ROPE.

388. Tensile strength of round iron and steel wire rope and hemp rope.—The following table shows the strength of iron wire rope and hemp rope, by the eminent American Engineer, J. A. Roebling, Esq.* The breaking weight is given in the American ton of 2,000 lbs.

TABLE XXXVII.—STRENGTH OF ROUND IRON WIRE ROPE AND HEMP ROPE, BY J. A. ROEBLING, C.E.

	Circumference of Wire Rope in inches.	Trade Number.	Circumference of Hemp Rope of equal strength in inches.	Tearing Strain in tons of 2,000 lbs.
Fine Wire,	6·62	1	15½	74
	6·20	2	14½	65
	5·44	3	13	54
	4·90	4	12	43·6
	4·50	5	10¾	35
	3·91	6	9½	27·2
	3·36	7	8	20·2
	2·98	8	7	16
	2·56	9	6	11·4
	2·45	10	5	8·64
Coarse Wire,	4·45	11	10¾	36
	4·00	12	10	30
	3·63	13	9½	25
	3·26	14	8¼	20
	2·98	15	7¼	16
	2·68	16	6¼	12·3
	2·40	17	5½	8·8
	2·12	18	5	7·6
	1·9	19	4·75	5·8

* *Memoranda on the Strength of Materials*, by J K. Whildin, New York, p. 9.

TABLE XXXVII.—STRENGTH OF ROUND IRON WIRE ROPE AND HEMP ROPE, BY J. A. ROEBLING, C.E.—*Continued.*

	Circumference of Wire Rope in inches.	Trade Number.	Circumference of Hemp Rope of equal strength in inches.	Tearing Strain in tons of 2,000 lbs.
Coarse Wire,	1·63	20	4	4·09
	1·53	21	3·3	2·83
	1·31	22	2·80	2·13
	1·23	23	2·46	1·65
	1·11	24	2·2	1·38
	0·94	25	2·04	1·03
	0·88	26	1·75	0·81
	0·78	27	1·50	0·56

TABLE XXXVIII.—WEIGHT, STRENGTH, AND WORKING LOAD OF HEMP AND ROUND WIRE ROPES, AS STATED BY THE MAKERS, MESSRS. NEWALL AND CO. OF GATESHEAD-ON-TYNE.

Hemp.		Iron.		Steel.		Equivalent Strength.	
Circumference. Inches	Lbs. Weight per Fathom.	Circumference. Inches.	Lbs. Weight per Fathom.	Circumference. Inches.	Lbs. Weight per Fathom.	Working Load. Cwts.	Tearing Strain. Tons.
2¾	2	1	1	—	—	6	2
—	—	1¼	1½	1	1	9	3
3¾	4	1½	2	—	—	12	4
—	—	1¾	2½	1¼	1½	15	5
4½	5	1⅞	3	—	—	18	6
—	—	2	3½	1⅝	2	21	7
5½	7	2⅛	4	1¾	2½	24	8
—	—	2¼	4½	—	—	27	9
6	9	2⅜	5	1⅞	3	30	10
—	—	2½	5½	—	—	33	11
6½	10	2⅝	6	2	3½	36	12

TABLE XXXVIII.—WEIGHT, STRENGTH, AND WORKING LOAD OF HEMP AND ROUND WIRE ROPES, AS STATED BY THE MAKERS, MESSRS. NEWALL AND CO. OF GATESHEAD-ON-TYNE.—*Continued.*

Hemp.		Iron.		Steel.		Equivalent Strength.	
Circumference. Inches.	Lbs. Weight per Fathom.	Circumference. Inches.	Lbs. Weight per Fathom.	Circumference. Inches.	Lbs. Weight per Fathom.	Working Load. Cwts.	Tearing Strain. Tons.
—	—	2¾	6½	2⅛	4	39	13
7	12	2⅞	7	2¼	4½	42	14
—	—	3	7½	—	—	45	15
7½	14	3⅛	8	2⅜	5	48	16
—	—	3¼	8½	—	—	51	17
8	16	3⅜	9	2½	5½	54	18
—	—	3½	10	2⅝	6	60	20
8½	18	3⅝	11	2¾	6½	66	22
—	—	3¾	12	—	—	72	24
9½	22	3⅞	13	3¼	8	78	26
10	26	4	14	—	—	84	28
—	—	4¼	15	3⅜	9	90	30
11	30	4⅜	16	—	—	96	32
—	—	4½	18	3½	10	108	36
12	34	4⅝	20	3¾	12	120	40

389. Tensile strength of flat iron and steel wire rope and flat hemp rope.

TABLE XXXIX.—WEIGHT, STRENGTH, AND WORKING LOAD OF FLAT HEMP ROPE AND FLAT WIRE ROPE, AS STATED BY THE SAME MAKERS.

Hemp.		Iron.		Steel.		Equivalent Strength.	
Size in Inches.	Lbs. Weight per Fathom.	Size in Inches.	Lbs. Weight per Fathom.	Size in Inches.	Lbs. Weight per Fathom.	Working Load. Cwts.	Tearing Strain. Tons.
4 +1⅛	20	2¼+ ½	11	—	—	44	20
5 +1¼	24	2½+ „	13	—	—	52	23

TABLE XXXIX.—WEIGHT, STRENGTH, AND WORKING LOAD OF FLAT HEMP ROPE AND FLAT WIRE ROPE, AS STATED BY THE SAME MAKERS.—*Continued.*

Hemp.		Iron.		Steel.		Equivalent Strength.	
Size in Inches.	Lbs. Weight per Fathom.	Size in Inches.	Lbs. Weight per Fathom.	Size in Inches.	Lbs. Weight per Fathom.	Working Load. Cwts	Tearing Strain. Tons.
5½+1⅝	26	2½+ ⅝	15	—	—	60	27
5¾+1¾	28	3 + „	16	2 + ½	10	64	28
6 +1½	30	3¼+ „	18	2¼+ ½	11	72	32
7 +1⅞	36	3⅜+ „	20	„ „	12	80	36
8¼+2⅛	40	3½+ 11/16	22	2½+ ⅝	13	88	40
8½+2¼	45	4 + „	25	2¾+ ⅝	15	100	45
9 +2⅜	50	4¼+ ¾	28	3 + „	16	112	50
9½+2⅝	55	4½+ „	32	3¼+ „	18	128	56
10 +2⅞	60	4⅝+ „	34	3½+ „	20	136	60

390. Safe working strain of wire rope.—From Table XXXVIII. the safe working strain of round wire rope is a little more than 1/7th of its tearing strain; and from Table XXXIX. the working strain of flat wire rope is 1/9th of its tearing strain, and Messrs. Newall and Co. state that "round rope in pit-shafts must be worked to the same load as flat ropes."

CHAPTER XV.

SHEARING-STRAIN.

291. Shearing in detail — Simultaneous shearing.—The nature of shearing-strain* in the vertical web of girders has been already investigated in (**14**), and we have frequent examples of the same kind of strain, though on a smaller scale, in rivets or similar connexions which sustain forces tending to cut them across at right angles to their length. For example, the rivet joining the blades of a pair of scissors is subject to a shearing-strain equal to the pressure applied to the handles plus the resistance of the fabric which is being cut. The latter also is subject to a shearing-strain, differing however in character from that which the rivet sustains in consequence of the oblique action of the blades which sever only a short length of the fabric at a time. Machines for shearing metals act on this principle, their cutting edges being generally set at an acute angle to each other, so that they shear plates in detail, and thus diminish the effort exerted at each instant of time; in punching machines, however, the whole circumference of the hole is cut at the first effort, and subsequent pressure is merely necessary to overcome friction and push out the burr. The shearing-strains which occur in engineering structures generally resemble that which rivets sustain, where the whole transverse area simultaneously resists shearing. In this case it is clear that the strength of the rivets is proportional to their sectional area; in other words, if F and f represent the total and the unit shearing-strains, eq. (1) will apply to shearing as well as to tensile and compressive forces, provided always that the cutting edges act simultaneously on the whole transverse section of the rivet or material under strain.

292. Experiments on punching wrought iron.—Table I. exhibits the results of experiments made at Bristol by Mr. Jones, "on the force required for punching different sized holes in different thicknesses of plates, up to 1 inch diameter and 1 inch thickness; the force was applied by means of dead weights with a pair of

* Called *Detrusion* by some authors.

levers giving a total leverage of 60 to 1, so that 1 cwt. in the scale gave a pressure of 3 tons on the punch; the weights were added gradually by a few lbs. at a time until the hole was punched." *

TABLE I.—EXPERIMENTS ON PUNCHING PLATE IRON.

Diameter of Hole.	Thickness of Plate.	Sectional area cut through.	Total Pressure on Punch.	Pressure per square inch of area cut.
Inch.	Inch.	Square inch.	Tons.	Tons.
0·250	0·437	0·344	8·384	24·4
0·500	0·625	0·982	26·678	27·2
0·750	0·625	1·472	34·768	23·6
0·875	0·875	2·405	55·500	23·1
1·000	1·000	3·142	77·170	24·6

Table II. contains experiments by Mr. C. Little on punching holes in hammered scrap iron with Eastwood's hydraulic shearing press, the force applied being measured by weights hung on the end of the force pump handle. This method of measurement is not so accurate as that by direct leverage, since the friction of the press is rather an uncertain element in the calculation.†

TABLE II.—EXPERIMENTS ON PUNCHING HAMMERED SCRAP IRON.

No. of Experiment.	Diameter of Punch.	Sectional area cut.		Pressure on Punch.		Remarks.
		Thickness and Circumference.	Area.	Total.	Tons per inch of area cut.	
	Ins.	Ins. Ins.	Sq. ins.	Tons.	Tons.	
1	1	0·51×3·14	1·60	35·8	22·4	22·5 mean.
2	1	0·98×3·14	3·08	69·3	22·6	
3	2	0·52×6·28	3·27	59·7	18·3	19·4 mean.
4	2	0·57×6·28	3·58	70·5	19·7	
5	2	1·06×6·28	6·66	132·8	19·9	
6	2	1·52×6·28	9·55	186·7	19·5	

* *Proc. Inst. Mech. Eng.*, 1858, p. 76. † *Idem*, p. 73.

393. Experiments on shearing wrought-iron.—Table III. contains experiments, also by Mr. Little, with Eastwood's hydraulic shearing press, on the force required to shear bars of hammered scrap and rolled iron presented edgeways and flatways to the cutter.

TABLE III.—EXPERIMENTS ON SHEARING HAMMERED SCRAP BARS AND ROLLED IRON.

No. of Experiment.	Direction of Shearing.	Sectional area cut.		Pressure on Cutters.		Remarks.	
		Thickness and Breadth.	Area.	Total.	Tons per inch of area cut.		
		Ins. Ins.	Sq. ins.	Tons.	Tons.		
7	Flat	0·50×3·00	1·50	33·4	22·3	22·7 mean.	Parallel Cutters.
8	Edge	0·50×3·00	1·50	34·6	23·1		
9	Flat	1·00×3·00	3·00	69·2	23·1	21·5 mean.	
10	Edge	1·00×3·00	3·00	68·1	22·7		
11	Flat	1·00×3·02	3·02	59·7	19·8		
12	Edge	1·00×3·02	3·02	62·1	20·6		
13	Edge	1·80×5·00	10·20	210·6	20·6	Flanged tyre.	
14	Flat	0·56×3·00	1·68	21·2	12·6		Inclined Cutters (angle 1 in 8).
15	Edge	0·56×3·00	1·68	33·2	19·7		
16	Flat	0·90×3·37	3·03	27·4	9·0		
17	Edge	0·87×3·32	2·89	57·4	19·8		
18	Flat	1·06×3·02	3·20	50·2	15·7		
19	Edge	1·06×3·02	3·20	67·5	21·1		
20	Flat	1·52×3·03	4·61	83·7	18·2		
21	Edge	1·53×3·03	4·64	93·3	20·1		
22	Flat	1·39×4·50	6·25	89·7	14·3		
23	Edge	1·38×4·50	6·21	111·2	17·9		
24	Flat	1·73×5·30	9·17	153·1	16·7		
25	Edge	1·73×5·30	9·17	207·0	22·6		
26	Flat	1·56×6·00	9·36	140·0	15·0		
27	Edge	1·56×6·00	9·36	172·3	18·4		

TABLE III.—Experiments on Shearing Bar Iron.—*Continued.*

No. of Experiment.	Direction of Shearing.	Sectional area cut.		Pressure on Cutters.		Remarks.
		Thickness and Breadth.	Area.	Total.	Tons per inch of area cut.	
28	Square	3·10×3·10	9·61	165·1	17·2	Hammered iron.
29	Square	3·10×3·10	9·61	155·5	16·2	Rolled iron.
30	Flat	1·80×5·00	10·20	99·3	9·7	Flanged tyre.
31	Edge	1·80×5·00	10·20	185·5	18·2	Flanged tyre.
32	Edge	1·70×5·25	10·57	179·5	17·0	Flanged tyre.

"In the above experiments of shearing (Nos. 7 to 13 inclusive), cutters with parallel edges were used; but when the ordinary cutter with edges inclined to one another at an angle of 1 in 8 were employed (Nos. 14 to 32 inclusive), the force required in shearing was diminished, and considerably so in the case of the thinner sections when sheared flatways; and as bars are usually sheared flatways, a decided advantage is shown in favour of inclined over parallel cutters. The force in tons per square inch of section cut with the bars:

		Flatways tons.		Edgeways tons.		
3 ×1½ inch	was	18·2	and	20·1	or	10 per cent. less flatways.
4½×1⅝	"	14·3	"	17·9	"	20 "
3 ×1	"	15·7	"	21·1	"	26 "
5¼×1¾	"	16·7	"	22·6	"	26 "
6 ×1½	"	15·0	"	18·4	"	18 "

"A trial was also made of the force required to shear some hard railway tyres 1¾ inch thick, and the result was 185 tons total edgeways, and 99 tons flatways (No. 30 and 31). A 3 inch square bar of rolled iron was also tried, and the force required was 155 tons total, against a total of 165 tons required for a hammered bar of the same section (Nos. 28 and 29)."*

During the construction of the Britannia and Conway tubular bridges several experiments were made by means of a lever on the

* *Proc. Inst. Mech. Eng.* 1858, p. 74.

shearing strength of bars of rivet iron ⅞th inch diameter. "The mean result from these experiments gives 23·3 tons per square inch as the weight requisite to shear a single rod of rivet iron of good quality. The ultimate tensile strength of these same bars was also found to be 24 tons; hence their resistance to single shearing was nearly the same as their ultimate resistance to a tensile strain." Two plates ⅝th inch thick were also "riveted together by a single rivet ⅞th inch diameter, and the rivet was sheared by suspending actual weights from the plate; the rivet thus sustained 12,267 tons, or 20·4 tons per square inch. Three plates were then united by a similar rivet, and the rivet was sheared in two places by the centre plate. The ultimate weight suspended from the rivet was 26·8 tons, or 22·3 tons per square inch of section." *

394. Shearing strength of wrought-iron equals its tensile strength.—From these various experiments on punching and shearing, we may infer that the shearing strength of wrought-iron is practically equal to its tensile strength.

395. Shearing strength of rivet steel is three-fourths of its tensile strength.—From Mr. Kirkaldy's experiments it appears that the shearing strength of rivet steel is 63,796 lbs. per square inch, the tensile strength of the bar employed being 86,450 lbs. per square inch of area.† Hence the shearing strength of rivet steel is about three-fourths of its tensile strength.

396. Shearing strength of copper.—From experiments by Mr. Joseph Colthurst on punching plates of wrought-iron and copper with a lever apparatus, it appears that the force required to punch copper is two-thirds of that required to punch iron. "It was observed, that duration of pressure lessened considerably the ultimate force necessary to punch through metal, and that the use of oil on the punch reduced the pressure about 8 per cent." ‡

397. Shearing strength of fir in the direction of the grain.—From Mr. Barlow's experiments on the resistance of Fir to drawing out, *i.e.*, punching, in the direction of the grain, it appears that this

* Clark *on the Britannia and Conway Tubular Bridges*, p. 392.

† *Experimental Inquiry*, p. 71.

‡ *Proc. of Inst. of C. E.*, Vol. i., p. 60.

amounts to 592 lbs. per square inch, or nearly one-twentieth of the tensile strength of the timber lengthways * (**363**).

398. Shearing strength of oak treenails.—The following table contains experiments by Mr. Parsons of H.M. dockyard service, on the "transverse strength of Treenails of English oak, used as fastening for planks of 3 and of 6 inches in thickness, and subjected to a cross strain."

TABLE IV.—STRENGTH OF TREENAILS OF ENGLISH OAK.

Number of the Experiment.	DIAMETER OF THE TREENAILS.							
	1 Inch.		1¼ Inch.		1½ Inch.		1¾ Inch.	
	THICKNESS OF THE PLANK.							
	3 Inches.	6 Inches.	3 Inches.	6 Inches.	3 Inches.	6 Inches.	3 Inches.	6 Inches.
	T. C.	T. C.	T. C.	T. C.	T. C.	T. C.	T. C.	T. C.
1	1 8	1 7	1 14	2 8	2 0	3 12	3 0	5 10
2	1 7	1 15	2 2	2 2	2 6	2 10	2 10	3 13
3	1 2	1 8	1 17	2 19	2 15	2 10	4 0	4 0
4	1 5½	1 8	2 2	2 2	2 4	3 12	2 8	3 8
5	2 12	1 3	2 2	1 15	2 18	2 5	3 10	4 0
6	2 2	1 7	2 9	2 10	2 6	2 5	3 10	5 8
7	2 4	1 10	2 8	2 10	3 7	2 5	3 5	3 12
8	1 6	2 3	2 7	2 0	2 5	3 0	3 5	3 13
9	1 8	1 8	2 12	2 10	3 0	4 0	4 6	4 13
10	1 2	2 3	2 10	2 15	3 0	4 10	3 8	4 0
11	2 0	2 0	2 7	2 0	3 9	2 18	4 0	3 8
12	1 8	1 7	2 10	2 0	4 2	3 0	4 10	5 0
13	1 16	2 8	2 17	2 0	3 2	3 18	4 2	5 5
Average	1 11	1 13	2 6	2 6	2 16	3 2	3 10	4 6

"In all these experiments on treenails, when the treenails were

* Barlow *on the Strength of Materials*, p. 23.

evidently good, they gave way gradually. In some of the rejected experiments, however, the treenails certainly did break off suddenly, but then they were evidently, on examination, either of bad or over-seasoned material. In the experiments on treenails, the plank generally moved about half an inch previous to the fracture of the treenail." *

* Murray *on Shipbuilding in Iron and Wood*, p. 94.

CHAPTER XVI.

ELASTICITY AND SET.

399. Limit of Elasticity—Set—Hooke's law of elasticity practically true.—It has been already stated in (5) that Mr. Hodgkinson's experiments led him to infer the non-existence of a definite *elastic limit*, or a limit within which, if the particles of a substance be displaced, they will return exactly to their original relative positions after the disturbing force is removed. The opposite view was held by Professor Robison, whose opinions are also entitled to great respect. In the article on the "Strength of Materials" in the *Encyclopædia Britannica*, he writes as follows:—"It is a matter of fact that all bodies are in a certain degree perfectly elastic; that is, when their form or bulk is changed by certain moderate compressions or distractions, it requires the continuance of the changing force to continue the body in this new state; and when the force is removed, the body recovers its original form. We limit the assertion to *certain moderate* changes. For instance, take a lead wire of one-fifteenth of an inch in diameter and ten feet long; fix one end firmly to the ceiling, and let the wire hang perpendicular; affix to the lower end an index like the hand of a watch; on some stand immediately below, let there be a circle divided into degrees, with its centre corresponding to the lower point of the wire; now turn this index twice round, and thus twist the wire. When the index is let go, it will turn backwards again, by the wire untwisting itself, and make almost four revolutions before it stops; after which it twists and untwists many times, the index going backwards and forwards round the circle, diminishing, however, its arch of twist each time, till at last it settles precisely in its original position. This may be repeated for ever. Now, in this motion, every part of the wire partakes equally of the twist. The particles are stretched, require force to keep them in their state of extension, and recover completely their relative positions. These are all the characters of what the mechanician calls *perfect* elasticity. This is a quality quite familiar in many cases, as in glass, tempered steel, &c., but was thought incompetent to lead, which is generally considered as having little or no elasticity. But we make the assertion in the most general terms, with the limitation to moderate derangement of form. We have made the same experiment on a thread of pipe-clay, made by forcing soft clay through the small hole of a syringe by means of a screw, and we found it more elastic than the lead wire; for a thread of one-twentieth of an inch diameter and seven feet long allowed the index to make two turns, and yet completely recovered its first position. But if we turn the index of the lead wire four times round and let it go again, it untwists again in the same manner, but it makes little more than four turns back again; and after many oscillations, it finally stops in a position almost two revolutions removed from its original position. It has now acquired a new arrangement of parts, and this new arrangement is permanent like the former; and what is of particular moment, it is perfectly elastic. This change is familiarly known by the denomination of a set." *

Whatever opinion the reader may hold regarding the existence or non-existence of a definite *elastic limit*, experiments prove that Hooke's *Law of Elasticity*, namely, that the elastic reaction of the fibres is proportional to their increment or decrement of length according as they are subject to tension or compression, is for all practical purposes true of many of the materials used in construction, in some cases over a very considerable range of strain, amounting even to half the breaking weight of the material (**7**).

CAST-IRON.

400. Decrement of length and set of cast-iron in compression—Coefficient of compressive elasticity.—We are indebted to Mr. Hodgkinson for some valuable experiments on the decrement of length and compressive set of eight bars of cast-iron, each 10 feet long and 1 inch square nearly. The first pair of bars were Low Moor iron No. 2; the second pair, Blænavon iron

* *Enc. Brit.*, 8th Ed, Vol. xx., p. 749, Art. "Strength of Materials."

No. 2; the third pair, Gartsherrie iron No. 3; and the fourth pair, a mixture of Leeswood iron No. 3, and Glengarnock iron No. 3, in equal proportions. Table I. contains the mean of these experiments reduced to a convenient unit-strain by Mr. Clark,* and I have added in the last column the coefficients of compressive elasticity per square inch, obtained by dividing the original length, viz., 10 feet, by the decrement of length per ton in the second column (δ).

TABLE I.—DECREMENT OF LENGTH AND COMPRESSIVE SET OF A CAST-IRON BAR 10 FEET LONG AND 1 INCH SQUARE.

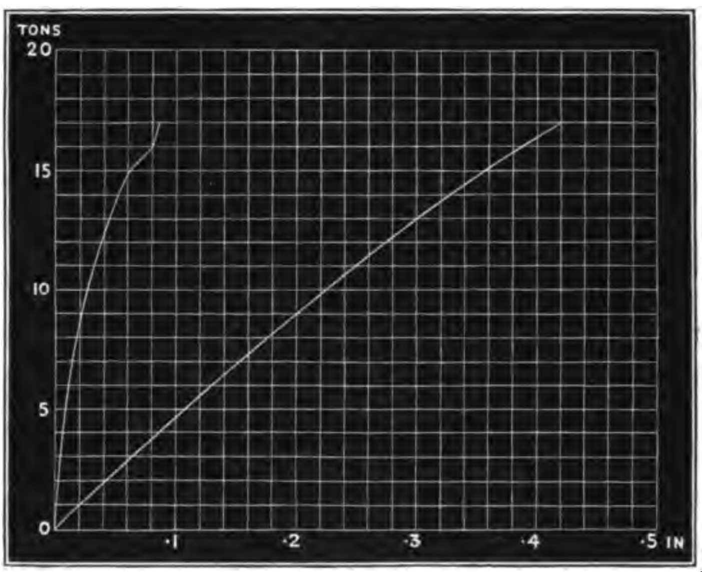
Tons, per square Inch.	Decrement of Length per ton.	Total Decrement of Length.	Set.	E′ The coefficient of Compressive Elasticity per square inch.
	Inch.	Inch.	Inch.	Tons.
1	·020338	·020338	·000510	5900
2	·021038	·042077	·002452	5704
3	·021618	·064855	·004340	5551
4	·021369	·085479	·006998	5615
5	·021594	·107872	·009188	5557
6	·021752	·130518	·011798	5517
7	·021950	·153654	·015243	5467
8	·022154	·177235	·018572	5416
9	·022374	·201373	·024254	5363
10	·022477	·224774	·028126	5339
11	·022567	·248237	·032023	5317
12	·022802	·273632	·037653	5262
13	·023014	·299187	·043318	5214
14	·023523	·329330	·052640	5101
15	·023539	·353092	·060905	5098
16	·024409	·390558	·080256	4916
17	·024805	·421695	·086298	4838

* *Rep. of Com. App.*, p. 63; and Clark *on the Tubular Bridges*, p. 312.

Mr. Hodgkinson makes the following remarks on these experiments:—" The great difficulty of obtaining accurately the decrements and sets from the small weights in the commencement of the experiments, rendered those decrements and sets, particularly the latter, very anomalous; it was found, too, that some of the bars which had been strained by 16 or 18 tons had become very perceptibly undulated. In has not been thought prudent, therefore, to draw any conclusion from bars which have been loaded with more than 14 to 16 tons; and it may be mentioned that the results from 2 to 14 tons are those only which ought to be used in seeking for general conclusions."*

The results of Table I. are exhibited graphically in Fig. 91, where the longer curve refers to the total decrements of length, and the shorter one to the sets. The ordinates represent the weights in column 1, and the abscissas the total decrements of length and sets in columns 3 and 4 respectively.

Fig. 91.



* *Rep. of Com. App* p. 64.

401. Hodgkinson's formulæ for the decrement of length and set of cast-iron in compression.—The following formula was deduced by Mr. Hodgkinson from his experiments on the four different irons just described to express the relation between the load and the corresponding decrements of length in cast-iron bars 1 inch square and of any length.*

$$\lambda' = l\{\cdot012363359 - \sqrt{\cdot000152853 - \cdot00000000191212W}\} \qquad (220)$$

Where λ' = the decrement of length in inches,
l = the length in inches,
W = the weight in lbs. compressing the bar.

The compressive set of Low Moor cast-iron Mr. Hodgkinson expressed by the following equation† :—

$$\text{Compressive set in inches} = \cdot543\lambda'^2 + \cdot0013 \qquad (221)$$

402. Increment of length and set of cast-iron in tension—Coefficient of tensile elasticity.—The following table shows the increment of length and tensile set of cast-iron bars 10 feet long and 1 inch square, reduced by Mr. Clark from Mr. Hodgkinson's experiments "upon round bars of iron, united together at the ends, so that the whole length, exclusive of the couplings, was 50 feet, except in two instances, where the length was 48 feet 3 inches. There were nine experiments upon these connected lengths, and the experiments were upon four kinds of cast-iron—the Low Moor No. 2, the Blænavon No. 2, Gartsherrie No. 3, and a mixture of iron, composed of Leeswood No. 3, and Glengarnock No. 3, in equal proportions. There were two experiments upon each of the simple irons, and three upon the mixture, and the mean results were afterwards reduced to those of 10 feet and 1 square inch exactly." "The bars were suspended vertically, and acted upon directly by weights attached at their lower ends."‡

* *Rep. of Com. App.*, p. 109.

† *Rep. of Com. App.*, p. 123.

‡ *Rep. of Com. App.*, pp. 59, 51; and Clark *on the Tubular Bridges*, p. 379.

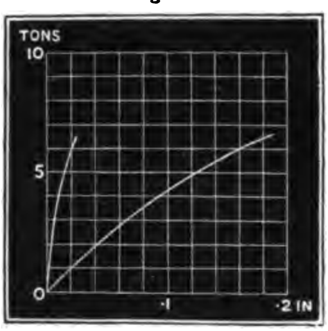
TABLE II.—INCREMENT OF LENGTH AND TENSILE SET OF A CAST-IRON BAR 10 FEET LONG AND 1 INCH SQUARE.

Tons, per square Inch.	Increment of Length per Ton.	Total Increment of Length.	Set.	E, The coefficient of Tensile Elasticity per square Inch.
	Inch.	Inch.	Inch.	Tons. lbs.
1	·01976	·01976	·000579	6073 = 13,603,520
2	·02027	·04155	·00186	5920 = 13,260,800
3	·02171	·06515	·003954	5528 = 12,382,720
4	·02318	·09274	·007543	5177 = 11,596,480
5	·02479	·12397	·012619	4841 = 10,843,840
6	·02727	·16363	·020571	4400 = 9,856,000
6½	·02815	·18297	·023720	4263 = 9,549,120

The mean increment of length per ton for the first 3 tons per square inch equals ·0001715 of the length, and the reader will perceive that the coefficients of tensile elasticity differ considerably from those of compressive elasticity in Table I.

The results of Table II. are exhibited graphically in Fig. 92 by curves which give the laws that connect the total increments of length and the sets respectively with the load. The ordinates represent the weights in column 1, and the abscissas the total increments of length and sets in columns 3 and 4.

Fig. 92.



By the aid of Tables I. and II. we can easily find the decrement, increment, or set of cast-iron bars of any section.

Ex. The compression flange of a new cast-iron girder, 40 feet long, which has not been previously strained, will be shortened by an inch-strain of 6 tons by an amount equal to $40 \times 0{\cdot}0130513 = 0{\cdot}522052$ inch, and its set, or residual decrement of length after the load has been removed, will equal $40 \times 0{\cdot}0011798 = 0{\cdot}047192$ inch. If the whole of this set were permanent, which however is problematical, the flange would be permanently shortened by this amount, and on any subsequent application of the same load its new decrement of length would $= 0{\cdot}522052 - 0{\cdot}047192 = 0{\cdot}474860$ inch.

403. Hodgkinson's formulæ for the increment of length and set of cast-iron in tension.—The following formulæ were deduced by Mr. Hodgkinson from his experiments on the extension of the four different irons just described, to express the relation between the increments of length and sets respectively and the weights producing them, in cast-iron bars 1 inch square, and of any length.*

$$\lambda = l\{\cdot 00239628 - \sqrt{\cdot 00000574215 - \cdot 000000000343946W}\} \qquad (222)$$

Where λ = the increment of length in inches,
l = the length in inches,
W = the weight in lbs. extending the bar.

$$\text{Tensile set in inches} = \cdot 0193\lambda + \cdot 64\lambda^2 \qquad (223)$$

404. Coefficients of tensile and compressive elasticity of cast-iron different.—The mean ultimate tenacity of the four irons from which the preceding formulæ were derived was 7·014 tons per square inch of section, and their mean ultimate extension under this strain equalled $\frac{1}{600}$th of the length, which is equivalent to $\frac{1}{4208}$th, = ·00024, of the length for one ton per square inch. The mean compression of the same metal by 7·014 tons per inch (the tensile breaking weight) was $\frac{1}{775}$th of the length, which is equivalent to $\frac{1}{5436}$th, = ·00018, of the length for one ton per square inch. Hence the coefficient of tensile elasticity at the tearing limit of 7·014 tons per square inch = 4208 tons = 9,425,920 lbs.; and the coefficient of compressive elasticity with the same strain of 7·014 tons per square inch = 5436 tons = 12,176,640 lbs., or rather more than $1\frac{1}{4}$ times the tensile coefficient. The coefficient of elasticity derived from experiments on the ultimate deflection of a rectangular bar of Blænavon iron, broken

* *Rep. of Com. App.*, pp. 60, 108.

by transverse strain is somewhat less than this; see ex. (**226**). Tredgold's coefficient given in the table in (**8**) is, doubtless, far too high.

405. Increment of length and set of cast-iron extended a second time—Relaxation of set—Viscid elasticity.—Mr. Hodgkinson made a second series of experiments on the extension of some parts of the coupled bars which were strained nearly to their breaking point, but had escaped actual rupture at the first trial.* Their increments of length on the second trial, though very nearly the same as before, were slightly less for the higher loads. It might perhaps be supposed that bars once stretched would not again take a set, provided the second load did not exceed that previously applied. This, however, was not the case; all the bars took sets again, though in general less than before, their mean ultimate set being nearly half that on the first trial. It is very probable that cast-iron, and also other materials, recover a portion of the set when the strain producing it is relaxed for some time—in fact, that there exists a sort of sluggish elasticity, due perhaps to a certain viscidity of the material. Possibly constant repetitions or long continuation of strain would render the set permanent. Experiments alone can settle these points, which, however, have more interest for the physicist than practical importance for the engineer.

406. Set of cast-iron from transverse strain nearly proportional to square of deflection.—The set of cast-iron bars subject to *transverse strain* is nearly proportional to the square of their deflection, though somewhat less, and may be expressed approximately by the following formula deduced by Mr. Hodgkinson from his experiments on rectangular bars of Blænavon cast-iron bent transversely by a load in the middle.†

$$\text{Transverse set in inches} = \frac{D^2}{31\cdot5} \qquad (224)$$

in which D represents the deflection of the bar in inches.

WROUGHT-IRON.

407. Decrement of length of wrought-iron in compression.—The following table contains the results of experiments by

* *Rep. of Com. App.*, p. 61. † *Ibid.*, p. 69.

Mr. Hodgkinson on the compression of two wrought-iron bars 10 feet long and 1 inch square nearly.*

TABLE III.—DECREMENT OF LENGTH OF WROUGHT-IRON BARS 10 FEET LONG AND 1 INCH SQUARE NEARLY.

Bar 1. Area of Section = 1·025×1·025 = 1·0506 square inches.		Bar 2. Area of Section = 1 016×1·02 = 1·0363 square inches.	
Weight compressing Bar.	Decrement of Length	Weight compressing Bar.	Decrement of Length.
Lbs.	Inches.	Lbs.	Inches.
5098	·028	5098	·027
9578	·052	9578	·047
14058	·073	14058	·067
16298	·085	—	—
18538	·096	18538	·089
20778	·107	20778	·100
23018	·119	23018	·113
25258	·130	25258	·128
27498	·142	27498	·143
29738	·154	29738	·163
31978	·174	31978	·190
34218	·214	in ½ hour.	·261
—	—	31,978	·269
—	—	in ½ hour.	·282
—	—	repeated.	·328

From the foregoing table it appears that the decrement of length of wrought-iron in compression increases with remarkable uniformity in proportion to the weight, and equals very nearly ·0001 of the length for each ton per square inch until the pressure reaches 11 or 12 tons per inch, after which irregular bulging begins, the amount of which, no doubt, varies considerably according to

* *Rep. of Com., App.* p. 122.

the quality of the iron, the hard and brittle irons bulging less than the tough and ductile kinds (**817**).

408. Increment of length and set of wrought-iron in tension.—Table IV. contains the results of experiments, also by Mr. Hodgkinson, on the extension and set of two bars of *annealed* wrought-iron of the quality denominated *best*. Their dimensions were as follows:—

	Bar 1.	Bar 2.
Length, . . .	49 feet 2 inches, .	50 feet.
Mean diameter, .	·517 inch, . .	·7517 inch.
Mean area of section,	·2099 square inch,	·44379 square inch.

In the following table the results are reduced to the standard of bars 10 feet long and 1 inch square.*

TABLE IV.—INCREMENT OF LENGTH AND TENSILE SET OF TWO ANNEALED WROUGHT-IRON BARS, 10 FEET LONG AND 1 INCH SQUARE.

Bar 1.			Bar 2.		
Weight per square inch of Section.	Increment of Length.	Set.	Weight per square inch of Section.	Increment of Length.	Set.
Lbs.	Inches.	Inches.	Lbs.	Inches.	Inches.
—	—	—	1262	·00520	—
2668	·00986	—	2524	·01150	—
5335	·02227	—	3786	·01690	·00050
8003	·03407	·000305	5047	·02240	·00060
10670	·04556	·000407	6309	·02772	·00050
13338	·05705	·000509	7571	·03298	·00045
16005	·06854	·000610	8833	·03790	·00050 ?
18673	·07993	·000813	10095	·04300	·00050 ?

* *Rep. of Com., App.* pp. 47, 49.

TABLE IV.—INCREMENT OF LENGTH AND TENSILE SET OF TWO ANNEALED WROUGHT-IRON BARS, 10 FEET LONG AND 1 INCH SQUARE.—*Continued.*

Bar 1.			Bar 2.		
Weight per square inch of Section	Increment of Length.	Set.	Weight per square inch of Section.	Increment of Length.	Set.
Lbs.	Inches.	Inches.	Lbs.	Inches.	Inches.
21340	·09193	·001525	11357	·04854	—
24008	·10485	·003966	12619	·05370	·00070
26676	·12163	·009966	13880	·05950	—
29343	·15458	·031424	15142	·06480	—
32011	·26744	—	16404	·06980	—
—	·28271 in 5 minutes.	·13566	17666	·07530	·00130
34678	·5148	·36864	18928	·08170	—
37346	1·0995	1·01695	20190	·08740	·00270
Repeated.	1·1949	1·02966	21452	·09310	—
40013	1·220 in 5 minutes.	1·093	22713	·09920	·00410
Repeated and left on.	1·411 after 1 hour.	—	23975	·10570	—
"	1·424 after 2 hours.	—	25237	·11250	·00680
"	1·433 after 3 hours.	—	26499	·12040	—
"	1·434 after 4 hours.	—	27761	·12880	·0120
"	1·436 after 5 hours.	—	29023	·14500	—
"	1·437 after 6 hours.	—	30285	·1991	—
"	1·443 after 7 hours.	—	"	·2007 after 5 minutes.	—
"	1·443 after 8 hours.	—	"	·2018 after 10 minutes.	·0736
"	1·443 after 9 hours.	—	"	·2054 after 15 minutes.	·0774
"	1·443 after 10 hours.	—	Repeated.	·2080 nearly, after 20 minutes.	·0796
42681	2·148 in 5 minutes.	1·983	"	·2096 after 1 hour.	·0814
Repeated.	2·339 in 5 minutes.	—	"	·2366 after bearing the weight 17 hours	·1082
"	2·383 in 10 minutes.	2·212 after 1 hour	—	—	—

TABLE IV.—INCREMENT OF LENGTH AND TENSILE SET OF TWO ANNEALED WROUGHT-IRON BARS, 10 FEET LONG AND 1 INCH SQUARE.—*Continued.*

Bar 1.			Bar 2		
Weight per square inch of Section.	Increment of Length.	Set.	Weight per square inch of Section.	Increment of Length.	Set.
Lbs.	Inches.	Inches.	Lbs.	Inches.	Inches.
Repeated.	2·428 after 46 hours.	2·237	31546	·242 after 5 minutes.	·1083
45348	2·580 after 5 minutes.	2·377	Repeated.	·2449 after 5 minutes.	·1111
Repeated.	2·605 after 1 hour.	—	32808	·5506	·4141
,,	2·606 after 2 hours.	—	Repeated.	·7024 after 5 minutes.	·5635
,,	2·606 after 19 hours.	2·408	,,	·7966 after 10 minutes.	·6558
48016	2·975 after 5 minutes.	2·733 after 10 minutes.	,,	1·014 after about ½ hour.	·866
Repeated.	3·019 after 1 hour.	—	34070	1·346 after 1 minute.	—
,,	3·029 after 11 hours.	—	,,	1·400 after 2 minutes.	—
50684	4·195 in 10 minutes.	3·941 in 10 minutes.	,,	1·600	1·44
Repeated.	4·226	—	Repeated.	1·65 after 1 minute.	—
,,	4·227 in 7 hours.	—	34070	1·786 after 1 hour or less.	1·628
,,	4·227 in 12 hours.	—	—	—	—
53351 = 23·817 tons.	Broke at one of the "weldings" where there was a slight defect; perhaps a rather smaller weight would have broken it.		35332	2·04 after 5 minutes.	1·874
			Repeated.	2·18 after 5 minutes.	2·01
			,,	2·254	2·08
			36594	2·54 after 6 minutes.	—
			37856 = 16·9 tons.	2·894 The loop at the lower end of the rod having broken, the experiment was discontinued.	—

The following table is given by Mr. Clark at p. 373 of his work on the Britannia and Conway tubular bridges. Though not expressly stated so, it is probably reduced from Mr. Hodgkinson's experiment on Bar 1 in the foregoing table.

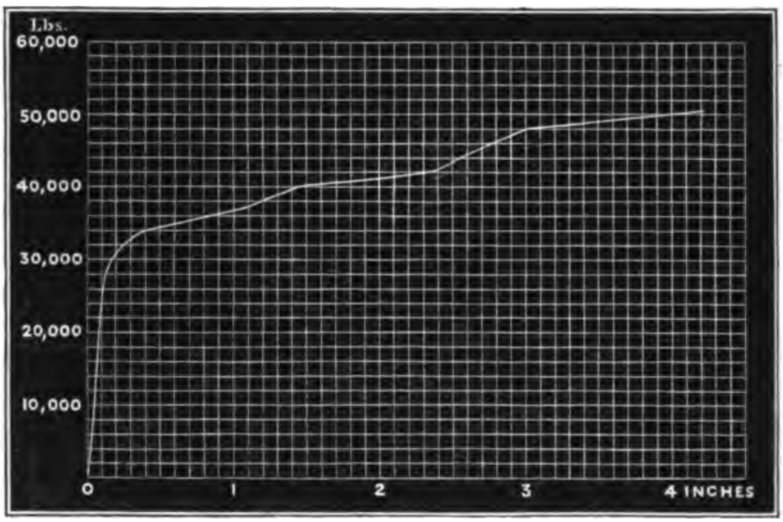
TABLE V.—INCREMENT OF LENGTH AND TENSILE SET OF A NEW WROUGHT-IRON BAR, 10 FEET LONG AND 1 INCH SQUARE.

Tons per square inch.	Observed extension in terms of the length.	Computed extension assumed uniform at $\frac{8}{100000}$ of the length per ton per square inch.	Corresponding extension in fractional parts of the length computed at $\frac{8}{100000}$ per ton per square inch.	Observed set in terms of the length.	Observed set in fractional parts of the length.
1	·0000689	·00008	$\frac{1}{12500}$		
2	·000156	·00016	$\frac{1}{6250}$		
3	·000238	·00024	$\frac{1}{4166}$	·00000213	$\frac{1}{469483}$
4	·000319	·00032	$\frac{1}{3125}$	·00000283	$\frac{1}{353357}$
5	·000399	·00040	$\frac{1}{2500}$	·00000356	$\frac{1}{280898}$
6	·00048	·00048	$\frac{1}{2083}$	·00000427	$\frac{1}{234192}$
7	·00056	·00056	$\frac{1}{1785}$	·00000497	$\frac{1}{201207}$
8	·00064	·00064	$\frac{1}{1562}$	·00000650	$\frac{1}{153846}$
9	·00072	·00072	$\frac{1}{1389}$	·00001201	$\frac{1}{83264}$
10	·00080	·00080	$\frac{1}{1250}$	·00001334	$\frac{1}{74962}$
11	·000896	·00088	$\frac{1}{1136}$	·00003392	$\frac{1}{29481}$
12	·00102	·00096	$\frac{1}{1040}$	·00008868	$\frac{1}{11276}$
13	·00128	·00104	$\frac{1}{961}$	·0002598	$\frac{1}{3849}$
14	·00218 in ten minutes ·00231	·00112	$\frac{1}{893}$	·0011075	$\frac{1}{903}$
15	·00416	·00120	Beyond this weight the permanent set is greater than the computed extension as above.	·002976	$\frac{1}{336}$
16	·00443	·00128		·003175	$\frac{1}{315}$
17	·00934 in ten minutes ·01015	·00136		·008750	$\frac{1}{114}$
18	·01024 in ten minutes ·01212	·00144		·009170	$\frac{1}{109}$
19	·01785 in ten minutes ·02017	·00152		·018590	$\frac{1}{54}$
20	·02124 in ten minutes ·02146	·00160		·019790	$\frac{1}{50}$
21	·02429 in ten minutes ·02472	·00168		·022810	$\frac{1}{44}$
22	·03400 in ten minutes ·03425	·00176		·031933	$\frac{1}{31}$

From the foregoing tables it would appear that the increment of length of annealed wrought-iron in tension increases with great uniformity in proportion to the weight, and equals ·00008, = $\frac{1}{12500}$th, of the length for each ton per square inch up to 11 or 12 tons, after which the law suddenly changes, and rapid and irregular stretching begins, the amount depending, no doubt, on the quality of the iron.

The relation between the weights and corresponding increments of length of the first bar in Table IV. are exhibited graphically in Fig. 93, in which the ordinates represent the weights per square inch of section, and the abscissa the corresponding increments of length.

Fig. 93.



Mr. Barlow also made several experiments on bars of wrought-iron, from which he inferred that its limit of elasticity is about 10 tons per square inch, and that it extends ·000096 of its length for each ton within this limit.*

409. Coefficients of elasticity of wrought-iron.—From Mr. Hodgkinson's experiments it appears that the coefficient of compressive elasticity of wrought-iron = 23,243,179 lbs = 10,376 tons

* *Strength of Materials*, p. 315.

per square inch. The coefficient of tensile elasticity for *annealed* wrought-iron = 27,691,200 lbs. = 12,362 tons.* The coefficient of elasticity usually adopted for both tension and compression is 24,000,000 lbs. per square inch.

410. Stiffness of imperfectly elastic materials improved by stretching—Practical method of stiffening wrought-iron bars—Limit of elasticity of wrought-iron equals 12 tons per square inch—Proof strain should not exceed the limit of elasticity.—When an imperfectly elastic material has received a permanent set from the application of any weight which is subsequently removed, the material becomes more perfectly elastic than before within the range of strain which first produced the set, and its alteration of length per unit of strain is less than at first. When, for instance, a girder is tested for the first time, its deflection exceeds that produced by a subsequent application of the same load. Hence the laudable custom of "stretching" a railway girder by a heavy load before it is viewed by the Government Inspector. In compound structures, such as lattice girders, some of the initial deflection may, perhaps, be attributed to the separating or closing together of the numerous joints on the first application of a heavy load, though probably the greater portion is due to the straightening of parts in tension originally constructed a little out of line. The ultimate deflection of a bar of soft wrought-iron subject to transverse strain is very considerable, and when the useful load which such a bar will carry is determined by the amount of deflection rather than by its breaking weight, its useful strength, *i.e.*, its stiffness, may be much increased by giving it a considerable camber when at a dull red heat, and afterwards straightening it when cold. Such a bar, as far as deflection is concerned, is stronger than before.†

For practical purposes the *limit of elasticity* of wrought-iron does not exceed 12 tons per square inch, and though higher strains than this may not in the least diminish its ultimate strength, yet they will take the "stretch" out of it and thus may render iron which was originally tough and ductile so hard and brittle as to be seriously

* *Rep. of Com.*, App., p. 172. † *Tubular Bridges*, p. 449.

injured for many purposes. A tough quality of iron, for instance, will evidently sustain sudden shocks with greater impunity than brittle iron, and previous over-straining may perhaps thus explain the unexpected rupture of chains with sudden loads considerably below their breaking weight. These considerations show that the proof strain of this material should not exceed its limit of elasticity.

411. Elastic flexibility of cast-iron twice that of wrought-iron.—The *elastic flexibility* of cast-iron is nearly twice as great as that of wrought-iron, that is, the alteration of length from the same unit-strain is nearly twice as great in cast as in wrought-iron; in other words, wrought-iron is nearly twice as stiff as cast-iron. On this account a girder of cast-iron will deflect nearly twice as much as a similar one of wrought-iron, provided the flanges of both girders be subject to the same unit-strains.

412. Experiments on elasticity liable to error—Sluggish elasticity.—Scientific conclusions derived from experiments on the elasticity of materials in which the effect of previous strain is overlooked are evidently worthless, and it should be recollected that time ought to be allowed after each experiment in order to let the material adjust itself to the new condition of strain, especially when the load approaches the limits of rupture, in which case the extension of ductile materials, such as wrought-iron, may continue for a considerable time after the load is laid on, especially if aided by vibration. Referring to the Britannia and Conway Tubular Bridges Mr. Clark observes, "In all the tubes a considerable time elapsed before they attained a deflection which remained constant. Time is an important element in producing the ultimate permanent set in any elastic material; but when the permanent set due to the strain is once attained, the continuance of the same strain induces no further deflection, which is confirmed by the fact, that no subsequent change has occurred in the deflection of the Conway Bridge from two years of use, nor has any increase in the versed sine of the Menai Suspension-bridge taken place in twenty-five years, where the strain is greater than in the plates of the Conway Bridge, and liable to be considerably varied from the oscillation which occurs in gales of wind. The permanent strain in the

Britannia Bridge is under three-fifths of that in the Suspension-bridge. The effect of time in producing permanent elongation has been also observed at the High Level Bridge (*Newcastle-upon-Tyne*), where the wrought-iron tie-chains, which resist the thrust of the arches, although under much less strain than the above, continued to extend for a considerable period before they attained a set at which they remained constant. These motions are so extremely minute that they are only ascertainable in large rigid structures, where they are measured by the corresponding increase of deflection."*

The residual set, after the strain has been removed, also takes time to adjust itself to a permanent condition, and some crude experiments of my own tend to prove that the set of wrought-iron relaxes to a considerable extent, even after the lapse of several days after the strain has been removed.

STONE.

418. Vitreous materials take no set—Hookes' law of elasticity apparently does not apply to some kinds of stone in compression.—It is stated by Dr. Robinson that "hard bodies of an uniform glassy structure, or granulated like stones, are elastic through the whole extent of their cohesion, and take no set, but break at once when overloaded."† It may be doubted whether this is true of granulated bodies like stones, for Mr. Mallet, referring to his experiments on crushing small cubes of quartz and slate rock from Holyhead, 0·707 inch upon each edge, observes, "the *per-saltum* way in which all the specimens of both rocks yield, in whatever direction pressed, is another noteworthy circumstance. The compressions do not constantly advance with the pressure, but, on the contrary, the rock occasionally suffers almost no sensible compression for several successive increments of pressure, and then gives way all at once (though without having lost cohesion, or having its elasticity permanently impaired) and compresses thence more or less for three or four or more

* *The Britannia and Conway Tubular Bridges*, p. 671.

† *Encyc. Metr.*, 8th ed., art. "Strength of Materials," Vol. xx., p. 756.

successive increments of pressure, and then holds fast again, and so on. This phenomenon is probably due to the mass of the rock being made up of intermixed particles of several different simple minerals, having each specific differences of hardness, cohesion, and mutual adhesion, and which are, in the order of their resistances to pressure, in succession broken down, before the final disruption of the whole mass (weakened by these minute internal dislocations) takes place. Thus it would appear that the micaceous plates and aluminous clay-particles interspersed through the mass give way first. The chlorite in the slate, and probably felspar-crystals in the quartz-rock, next, and so on in order, until finally the elastic skeleton of silex gives way, and the rock is crushed. It is observable, also, that this successive disintegration does not occur at equal pressures, in the same quality and kind of rock, when compressed transverse and parallel to the lamination." *

* *Phil. Trans.*, 1862, p. 669.

CHAPTER XVII.

TEMPERATURE.

414. Arches camber, suspension bridges deflect, and girders elongate from elevation of temperature—Expansion rollers.—Changes of temperature affect bridges very differently, according to their mode of construction. An increase of temperature causes the crowns of iron arches which are confined between fixed abutments to rise, and the spandrils to extend lengthways, chiefly along their upper flanges; hence, room for longitudinal expansion should be provided by leaving a vertical space between the masonry of the abutments above springing level and the ends of the arch spandrils. When iron arches extend over two or more spans, their spandrils should not be rigidly connected together like continuous girders; for then their expansion may cause a dangerous crushing strain along the line of junction and throughout the top flanges, a portion of which strain will, no doubt, be transmitted to the arches themselves. When, therefore, it is considered desirable to connect together the spandrils of consecutive iron arches, it should be effected by sliding covers or some similar contrivance, which, while restraining lateral motion, will allow perfect freedom for changes of length. The rise in the crown of one of the cast-iron arches of Southwark bridge for a change of temperature of 50° F. was observed by Mr. Rennie to be about 1·25 inch; the length of the chord of the extrados is 246 feet, and its versed sine 23 feet 1 inch, and accordingly the length of the arch, which is segmental, is 3020·8 inches.*

Stone arches are affected in the same way as iron arches. With increased temperature the crown rises and joints in the parapets over the crown open, while others over the springing close up. The reverse takes place in cold weather; the crown descends, joints over the springing open and those over the crown close. When

* *Trans. Inst. C. E.*, Vol. iii., p. 201.

stone or iron arches are of large span their movements from changes of temperature will generally dislocate to a slight degree the flagging and pavement of the roadway above.

An increase of temperature causes suspension bridges to deflect, just the reverse of what happens to arches; girders, which exert only a vertical pressure on the points of support, extend longitudinally under the same influence, and on this account it is usual to provide rollers, or, if the span be moderate, sliding metallic surfaces, under one end of each girder. It may be questioned, however, whether sliding surfaces long remain in order, and some engineers prefer timber or stone wall-plates beneath the ends of the girder, even when the span exceeds 100 feet. In place of being supported by rollers, girders are sometimes hung from suspension links, the pendulous motion of the links affording the requisite horizontal movement due to change of temperature. The chains of suspension bridges are generally attached to saddles which rest on rollers on top of the towers; the object of these, however, is rather to compensate for unequal loading than for changes of temperature.

415. Alteration of length from change of temperature—Coefficients of linear expansion.—The coefficient of linear expansion of any material is the fractional part of its length at zero centigrade, which it elongates or shortens from a change of one unit of temperature, generally 1° C. The alteration of length for other changes of temperature is expressed by the following equation.

$$\lambda = nkl \qquad (225)$$

Where l = the length of the bar at (Fig.) 0°C.,

k = the coefficient of linear expansion of the material for one degree,

n = the number of degrees through which the temperature of the bar is raised or lowered,

λ = the increment or decrement of length due to a change of temperature equal n degrees.

Ex. The total length of the Britannia wrought-iron tubular bridge is 1,510 feet, and an increase of temperature of 26°F. caused an increase of length of $3\frac{1}{4}$ inches, what is the coefficient of linear expansion of plate iron for 1°C.?—(*Clark*, p. 715.)

Here, $l = 1510$ feet $= 18120$ inches,
$n = 26°F. = 14{\cdot}44°C.$,
$\lambda = 3{\cdot}25$ inches.

Answer, $k = \dfrac{\lambda}{nl} = \dfrac{3{\cdot}25}{14{\cdot}44 \times 18120} = 0{\cdot}000012421$ inch.

The following table contains the coefficients of linear expansion of various materials for one degree centigrade.

TABLE I.—COEFFICIENTS OF LINEAR EXPANSION FOR 1°C.

Description of Material.	Authority.	Coefficients of linear expansion for 1°C.
METALS.		
Antimony,	Smeaton,	·000010833
Bismuth,	Do.	·000013916
Brass (supposed to be Hamburg plate brass),	Ramsden,	·000018554
Do. (English plate, in form of a rod),	Do.	·000018928
Do. (English plate, in form of a trough),	Do.	·000018949
Do. (cast),	Smeaton,	·000018750
Do. (wire),	Do.	·000019333
Copper,	Laplace & Lavoisier,	·000017122
Do.	Do.	·000017224
Gold (de départ),	Do.	·000014660
Do. (standard of Paris, not annealed),	Do.	·000015515
Do. (do. annealed),	Do.	·000015136
Iron (cast),	Ramsden,	·000011094
Do. (from a bar cast 2 inches square),	Adie,	·000011467
Do. (do. ½ an inch square),	Do.	·000011022
Do. (soft forged),	Laplace & Lavoisier,	·000012204
Do. (round wire),	Do.	·000012350
Do. (wire),	Troughton,	·000014401

NOTE.—One degree Fahrenheit = $\frac{5}{9}$ths of one degree centigrade. To convert a given temperature on Fahrenheit's scale to the corresponding temperature centigrade, subtract 32°., and multiply the remainder by $\frac{5}{9}$. Thus, the *temperature* of 86°F. = 30°C., but a *range* of 86°F. = 48°C. nearly.

TABLE I.—COEFFICIENTS OF LINEAR EXPANSION FOR 1°C.—*Continued.*

Description of Material.	Authority.	Coefficients of linear expansion for 1°C.
METALS.		
Lead,	Laplace & Lavoisier,	·000028484
Do.,	Smeaton,	·000028666
Palladium,	Wollaston,	·000010000
Platina,	Dulong and Petit,	·000008842
Do.,	Troughton,	·000009918
Silver (of Cupel),	Laplace & Lavoisier,	·000019097
Do. (Paris standard),	Do.	·000019086
Do.,	Troughton,	·000020826
Solder (white; lead 2, tin 1),	Smeaton,	·000025053
Do. (spelter; copper 2, zinc 1),	Do.	·000020583
Speculum metal,	Do.	·000019333
Steel (untempered),	Laplace & Lavoisier,	·000010788
Do. (tempered yellow, annealed at 65°C.),	Do.	·000012395
Do. (blistered),	Smeaton,	·000011500
Do. (rod),	Ramsden,	·000011447
Tin (from Malacca),	Laplace & Lavoisier,	·000019376
Do. (from Falmouth),	Do.	·000021729
Zinc,	Smeaton,	·000029416
TIMBER.		
Baywood, in the direction of the grain, dry,	Joule,	·00000461 to ·00000566
Deal, do. do. do.	Do.	·00000428 to ·00000438
STONE, BRICK, GLASS, CEMENT.		
Arbroath pavement,	Adie,	·000008985
Brick (best stock),	Do.	·000005502
Do. (fire),	Do.	·000004928
Caithness pavement,	Do.	·000008947

TABLE I.—COEFFICIENTS OF LINEAR EXPANSION FOR 1°C.—*Continued.*

Description of Material.	Authority.	Coefficients of linear expansion for 1°C.
STONE, BRICK, GLASS, CEMENT.		
Cement (Roman),	Do.	·000014349
Glass (English flint),	Laplace & Lavoisier.	·000008117
Do. (French, with lead),	Do.	·000008720
Granite (Aberdeen grey),	Adie,	·000007894
Do. (Peterhead red, dry),	Do.	·000008968
Do. (do. do. moist),	Do.	·000009588
Greenstone (from Ratho),	Do.	·000008089
Marble (Carrara, moist),	Do.	·000011928
Do. (do. dry),	Do.	·000006539
Do. (black Galway),	Do.	·000004452
Do. (do. softer specimen, containing more fossils),	Do.	·000004793
Do. (Sicilian white, moist),	Do.	·000014147
Do. (do. do. dry),	Do.	·000011041
Sandstone (from Craigleith quarry),	Do.	·000011743
Slate (from Penrhyn quarry, Wales),	Do.	·000010376

Adie; *Dixon's Treatise on Heat*, p. 35.
Dulong and Petit; *Pouillet, Éléments de Physique*, p. 208.
Joule; *Proc. Roy. Soc.*, Vol. ix., No. 28, p. 3.
Laplace and Lavoisier; *Dixon's Treatise on Heat*, p. 29.
Ramsden; *idem*, p. 27.
Smeaton; *Pouillet, Éléments de Physique*, p. 207.
Troughton; *idem*.
Wollaston; *idem*.

416. Expansibility of timber diminished, or even reversed, by moisture.—Mr. Joule found that moisture occasioned a marked diminution in the expansibility of timber by heat. After a rod of bay-wood on which he experimented "had been immersed in water until it had taken up 150 grains, making its total weight 882 grains, its coefficient of expansion was found to be only ·000000436. Experiments with the rod of deal, weighing when dry 425 grains, gave similar results; when made to absorb water its coefficient of expansion gradually decreased, until, when it weighed 874 grains, indicating an absorption of 449 grains of water, expansion by heat ceased altogether, and, on the contrary, a *contraction* by heat equal to ·000000636 was experienced.*

417. Moisture increases the expansibility of some stones—Raising the temperature produces a permanent set in some stones.—"In the case of greenstone, and some descriptions of marble, the effect of moisture was to increase the amount of expansion; in other instances no effect of this kind was perceptible. Mr. Adie also found that in white Sicilian marble a permanent increase in length was produced every time that its temperature was raised, the amount of increase diminishing each time." †

418. A change of temperature of 15°C. in cast-iron, and 8°C. in wrought-iron, are capable of producing a strain of one ton per square inch—Open girders in the United Kingdom liable to a range of 45°C.—The alteration of length of a cast-iron bar within the range of three tons tension and seven tons compression per square inch is about ·000175 of the original length for each ton per square inch, and its coefficient of linear expansion for 1°C. = ·000011467 according to Adie; consequently, an alteration of temperature of about 15°C. (= 27°F.) is capable of developing a force equal to one ton per square inch. Again, if we assume that the alteration of length of a bar of wrought-iron for both tensile and compressive strains = ·0001 of its length for each ton per square inch, its coefficient of expansion for 1°C. being ·000012204, an alteration of temperature of about 8°C. (= 14·4°F.) is capable of developing a force equal to one ton per square inch. The range of temperature to which open-work bridges through which the air has free access are subject in this country seldom exceeds 45°C. (= 81°F.) for which range wrought-iron alters ·000549, or nearly $\frac{1}{1800}$th of its original length. This change of length is

* *Proc. Roy. Soc.*, Vol. ix., No. 28, p. 3. † Dixon's *Treatise on Heat*, p. 34.

nearly equivalent to that which would be produced by a strain of 5½ tons per square inch. The range of temperature of cellular flanges may, however, exceed that mentioned above, as Mr. Clark mentions that the temperature of the Britannia tubular bridge, before it was roofed over, differed "widely from that of the atmosphere in the interior, for the top during hot sunshine has been observed to reach 120° F., and even considerably more; and, on the other hand, a thermometer on the surface of the snow on the tube has registered as low as 16° F." *

A familiar instance of the contractile force of wrought-iron in cooling is exhibited in the tires of wheels. "An ingenious application of this force was also made in the case of a gallery in the Conservatoire des Arts et Metiers in Paris, whose walls were forced outwards by some horizontal pressure. To draw them together M. Molard, formerly director of the Museum in that establishment, had iron bars passed across the building, and through large plates of metal bearing on a considerable surface of the external walls. The ends of these bars were formed into screws, and provided with nuts, which were first screwed close home against the plates. Each alternate bar was then elongated by means of the heat of oil lamps suspended from it, and when expanded the nuts were again screwed home. The lamps being removed, the bars contracted, and in doing so drew the walls together. The other set of bars was then expanded in the same manner, their nuts screwed home, and the wall drawn in through an additional space by their contraction. And this series of operations was repeated until the walls were completely restored to the vertical, in which position the bars then served permanently to secure them." †

419. Tubular plate girders subject to vertical and lateral motions from changes of temperature—Open-work girders nearly quite free from these movements.—In addition to the longitudinal movements to which all girders are subject from changes of temperature, tubular plate girders move vertically or

* *Britannia and Conway Tubular Bridges*, p. 714.

† Dixon's *Treatise on Heat*, p. 121.

laterally whenever the top or one side becomes hotter than the rest of the tube. Referring to the Britannia tubular bridge, Mr. Clark states that "even in the dullest and most rainy weather, when the sun is totally invisible, the tube rises slightly, showing that heat as well as light is radiated through the clouds. On very hot sunny days the lateral motion has been as much as 3 inches, and the rise and fall 2 inches and $\frac{5}{10}$ths." *

These vertical and lateral motions have not been much observed in lattice or open-work girders; no doubt because the air and sunshine have free access to all parts, and thus produce an equable temperature throughout the whole structure.

420. Transverse strength of cast-iron not affected by changes of temperature between 16°F. and 600°F.— It appears from Mr. Fairbairn's experiments on the transverse strength of cast-iron at various temperatures from 16°F. upwards, that its strength "is not reduced when its temperature is raised to 600°F., which is nearly the melting point of lead; and it does not differ very widely, whatever the temperature may be, provided the bar be not heated so as to be red hot." †

421. Tensile strength of plate-iron uniform from 0°F. to 400°F.—It also appears from Mr. Fairbairn's experiments on wrought-iron at various temperatures that the tensile strength of plates is substantially uniform between 0°F. and 400°F. This result is corroborated by the experiments of the committee of the Franklin Institute appointed to report on the strength of materials employed in the construction of steam boilers. Mr. Fairbairn also found that the strength of the best bar-iron was increased about one-third when the temperature reached 320°F., after which it again diminished.‡ This, however, seems anomalous, and further confirmation would be desirable.

* *Britannia and Conway Tubular Bridges*, p. 717.

† Hodgkinson's *Exp. Res.*, p. 378; and Fairbairn's *Useful Information for Engineers*, first series, p. 81.

‡ *Useful Information for Engineers*, second series, pp. 114, 124.

CHAPTER XVIII.

FLANGES.

422. Cast-iron girders.—The compression flange of cast-iron girders is frequently made stronger than is theoretically necessary for the purpose of rendering it sufficiently stiff to resist side pressure, vibration, or other disturbing causes; in a word, to resist flexure. As the average crushing strength of cast-iron is about 5 times its tensile strength, theory indicates the most economical proportion of the compression to the tension flange, when both are horizontal, to be also 1 to 5 (**17**), whereas it is generally made much stronger than this, its area being sometimes as high as one-third of that of the tension flange. Hence cast-iron girders rarely fail in the compression flange, and it is a common practice to calculate their strength, as well as that of wrought-iron girders, from the leverage of the tension flange by the following well-known modification of eq. (19):—

$$W = \frac{adc}{l} \tag{226}$$

in which W = the breaking weight in the middle in tons,

a = the *net* area of the tension flange in square inches,

d = the depth of the web in the middle in inches,

l = the length between bearings in inches,

c = a coefficient depending on the material.

For cast-iron the coefficient $c = 4 \times 7 = 28$, the average tensile strength of cast-iron being about 7 tons per inch. For wrought-iron $c = 4 \times 20 = 80$, the tensile strength of ordinary plate iron being about 20 tons per inch. This equation omits any strength derived from the vertical web acting as an independent rectangular girder (**99**); it gives, therefore, too low a result when the area of the web forms a large portion of the total cross section.

423. Cellular flanges.—The box or closed cell was for some years a favourite form for the compression flange of tubular plate girders, whereas the tension flange was generally made of one or several plates riveted together so as to form practically one thick plate.

Fig. 94.

The adoption of the cell in this instance arose from the impression that it is better adapted than other forms of pillar for resisting flexure, and so no doubt it is when used as a pillar without extraneous support. Its connexion with the continuous web, however, prevents the flange from deflecting in a vertical direction, for at each point along its length it is held rigidly in the direction of the thrust, nor can it escape from this without separating from the side plates, and it is obvious that a very moderate force will hold a pillar in the line of thrust when the flexure is of trifling amount (**155**). It should also be kept in view that the stiffness of an unsupported plate to resist flexure is proportional to the cube of its thickness (**218**), and consequently, if the top and bottom plates of the cell be riveted together, we have a plate 8 times as stiff as either separately. If to these we add the central plate and the upper half of each side of the cell (so as to leave the depth of girder measured from the centre of the cell to the lower flange unaltered), and the spare angle irons, we have a top flange at least 3 times as thick, and therefore 27 times as stiff to resist vertical flexure as the unsupported top of the original cell. Though we do not thoroughly know the laws which govern the buckling of tubes (290), it is evident that the pile of plates possesses a superiority over the cell in this respect. It is, moreover, clear that the lateral stiffness of the flange is scarcely, if at all, affected by using one thick plate of the same width and sectional area as the cell, for, regarding the pile as a girder on its side, we have the adjacent parts of the double web performing the duty of flanges in place of the sides of the cell. In addition to this it will be shown in the chapter on connexions that considerable economy in the joint covers may be effected with plates in piles as compared with cells. One great objection to the closed cell is this: a large extent of surface is exposed to corrosion, and is at the same time difficult of access and therefore liable to be neglected; at the best its preservation is costly, and depends on the amount of care which the painter may feel inclined to bestow on an irksome task, for the completion of which he feels but little responsibility since his work is rarely inspected, while during its tedious and unhealthy performance he is obliged to assume an unnatural and fatiguing posture.* The corrosion inside a cell, however, may be greatly diminished by stopping up the ends so as to exclude change of air and moisture.

424. Piled flanges—Long rivets not objectionable.—When several plates are built into one pile it may be objected that great length of rivet is required, and that the workmanship is in consequence less sound; but this objection has no real value so far as the riveting is concerned. In parts of the Britannia Tubular Bridge rivets passed through six layers of iron of an aggregate thickness of nearly $3\frac{1}{2}$ inches,† and in the Boyne Viaduct many rivets passed through six and seven plates, and in some parts even nine. As I had forgotten the exact method of manipulating these long rivets at the Boyne Viaduct, I obtained from Mr. Colville, the intelligent superintendent of the iron-work, the following details:—

* A painful soreness of the eyes and tendency to faint are experienced in close cells whenever the stifling vapour of new lead paint is not removed by constant currents of fresh air passing through them. Hence, when the ventilation is defective, the painter must come out at short intervals to breathe the fresh air.

† *Britannia and Conway Tubular Bridges*, p. 575.

"The longest rivet we had was about 8 inches long, and the holes must be well rimed out. The rivets were kept cool, head and point, by dipping in water, and the body of the rivet made very hot, which enabled the workmen to use the cup tool and the heavy hammer at once. Some of the long rivets I had cut out, after being riveted, to see what they looked like, and I must say they filled better than I expected, being at top of the piers, which was very difficult to get to. I see no difficulty in riveting such thickness as was at the Boyne Bridge, but it must be with care in the heating of the rivets, and using about a 14lb. hammer and cup tools. Common light riveting hammers would only upset the rivet at the point, and would not fill in the body in such thickness as $4\frac{1}{2}$ to 5 inches."

425. Punching and drilling tools.—Careful attention is doubtless required in punching plates, so that the holes in the successive layers may coincide, and without proper precaution much trouble and expense would be incurred in subsequent riming out the holes; but this labour may, to a great extent, be avoided by using accurate templates, or when the magnitude of the work warrants such an outlay, by punching machines similar to the Jacquard machine used at the Conway Bridge, and subsequently at the Boyne Viaduct and Canada Works, and constructed expressly for the purpose of producing accurate repetitions of any required pattern.* Drilling tools for boring several holes at once have been occasionally used with much success, as at Charing-cross Bridge. Such tools will often repay their first cost by the saving of manual labour in punching and plating, besides insuring more accurate work.

426. Position of roadway.—The roadway is generally attached to one or other of the flanges, but is sometimes placed midway, or even suspended below the girder as in suspension bridges. Both the latter positions are objectionable, since with either arrangement we lose the advantage of horizontal rigidity which the roadway imparts to the flange to which it is attached. Moreover, less material is generally required for forming the connexion between

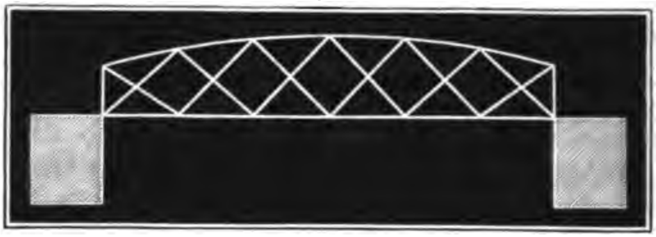
* For a description of this machine see Part 121 of the *Civil Engineers' and Architects' Journal.*

the road girders and the main girders at the flanges than elsewhere. When local circumstances do not determine the level of the road it may at first sight appear desirable to connect it with the upper or compression flanges, so as to stiffen them against horizontal flexure, and this is probably the best position with shallow girders. It allows the load to be placed more immediately over the longitudinal axis of each girder, and thus, besides dispensing with heavy cross girders, it removes any tendency to unequal strain, which a one-sided load on the lower flanges might produce. But with large and deep girders, independently of the consideration that the lower the centre of gravity the more stable the structure, some slight advantage results from connecting the road with the lower flange, as the expense of a parapet is saved, and there is a greater appearance of security when a train travels through instead of over a tubular bridge.

427. Compression flange stiffened by the compression bracing of web.—When the road is attached to the lower flanges and the depth of girder is not enough to admit of cross-bracing between the upper flanges, the horizontal stiffness of the road is communicated to the upper flanges by the internal bracing of the compression braces (Fig. 89, **237**). Triangular gussets between the latter and the cross girders are sometimes introduced; or, if these are found insufficient, the cross girders may be prolonged like cantilevers, and their extremities connected by raking struts with the upper flanges, as in the parapets of wooden bridges.

428. Waste of material in flanges of uniform section—Arched upper flange—Waste of material in continuous girders crossing unequal spans.—It frequently happens that the flanges have a greater sectional area near their ends than theory requires, in order to preserve the symmetry of the girder throughout its entire length and to avoid injudiciously thinning the material. This source of loss does not exist in the bowstring girder, as in it the strain is nearly uniform throughout each flange. A compromise may be effected between the bowstring girder and that with parallel flanges by arching the upper flange, as in Fig. 95. In this form of girder the strains near the ends of each flange

Fig. 95.



are increased, and thus the extra material is utilized, at the same time that the strains in the end braces are diminished in consequence of the oblique flange taking a share of their shearing-strain (15). The mode of calculation is the same as for the bow-string girder.

For a similar cause to that just mentioned there is frequently a waste of material in the flanges of continuous girders of uniform depth crossing spans of very unequal length. In this case the segments over the smaller spans are much deeper in proportion to their length than those over the larger spans, and hence a considerable waste of material may arise from carrying the section of the flanges symmetrically throughout.

499. An excess of strength in one flange does not increase the strength of braced girders, though it may slightly increase the strength of girders with continuous webs.—If the flanges of a braced girder be well proportioned both will fail simultaneously with the breaking load, and any increase of strength in one flange only does not increase the strength of the girder, but rather diminishes its useful strength by the excess of dead weight. When, however, the web is continuous an increase of strength is produced by enlarging one of the flanges beyond its due proportion, for the following reason:—The unit-strain in the re-enforced flange is less than before; consequently, there is less alteration in its length from strain, and the neutral surface approaches closer to it than if the flanges were duly proportioned; hence a larger portion of the web aids the weaker flange. The useful strength of the girder, however, is not necessarily increased, since the extra strength thus obtained may merely suffice to support the extra weight of the re-enforced flange.

CHAPTER XIX.

WEB.

430. Plate web—Calculation of strains.—In lattice girders the flanges and the compression braces are intersected at short intervals, and thus divided into short pillars as far as their tendency to flexure in the plane of the girder is concerned (**339**); this support is carried to its extreme limit in plate girders, the characteristic feature of which is the continuity of the vertical connexion (single or double, as the case may be) between the flanges. As the thin web of plate girders is ill adapted to resist buckling or flexure under compression it is usual to stiffen it by vertical T or angle irons reaching from flange to flange, like the frames of a ship. On a little consideration it will be obvious that these stiffening frames make the web more rigid at short intervals in vertical lines; thus this method of constructing plate girders resembles the vertical and diagonal bracing investigated in the seventh chapter, and the strains in the web may be approximately calculated in the manner there described. If these frames be placed diagonally in place of vertically the web will resemble the class of bracing investigated in the sixth chapter.

431. Strength of a continuous web calculated from shearing-strain.—The theoretic thickness of a continuous web may also be calculated from the shearing-strain by the following rule:—

$$\text{Sectional area of web} = \frac{\text{Shearing-strain}}{\text{Unit-strain}} \qquad (227)$$

in which the unit-strain is the safe unit-strain for shearing (**46**). This gives the minimum thickness, which, however, is often much less than a due regard for durability requires, neither does this rule give an adequate idea of the additional material required for stiffening the web against buckling.

Ex. The iron-work of one of the Conway tubes weighs 1,112 tons between the supports; adding 400 tons for weight of permanent way and a passing train, we have a total distributed load of 1,512 tons, of which one-fourth, or 378 tons, is the shearing-strain at each end of each side. The end side plates are 19 feet high and $\frac{5}{8}$ inch thick; hence their gross section = 142·5 square inches, but as their edges are pierced by one inch rivet holes, three inches apart centres, their net section is one-third less, or 95 square inches. The shearing-strain at the joints is therefore about 4 tons per inch of net section. In this example no credit has been given to the outsides of the cellular flanges, which, doubtless, contribute their quota of strength to resist shearing-strains.

432. Ambiguity respecting direction of strains in continuous webs—Bracing generally more economical than plating—Minimum thickness of plating in practice—Relative corrosion of metals.—Besides these compressive strains acting in directions more or less defined, there exist in the web of every plate girder diagonal tensile strains which cross the stiffening frames, and whose directions assume a very undefined character, doubtless varying with every position of the load. It thus appears that some portions of the web of plate girders are simultaneously sustaining tension and compression, and it might hence seem at first sight that a continuous web is more economical than one formed of diagonal bracing, since in the former arrangement the same piece of material performs a double duty, which in the diagonal system requires two distinct braces. Theoretically this view is correct if it be conceded that one and the same portion of material is capable of sustaining, without injury, both tensile and compressive strains transmitted through it simultaneously at an angle with each other, and, in the absence of direct experiment, there seems some reason for believing this to be the case within the limits of strain which are considered safe in practice. For instance, the shell and ends of a cylindrical boiler with internal flue are subject to tensile strains, the former in two directions at right angles to each other, the latter in various directions, while the flue is subject to tension in the direction of its length and compression at right angles to it. Again, experiments on the strength of riveted joints have not indicated any source of weakness in the plates other than that due to the reduction of area by the rivet holes or the mode of punching, and, if moderate compression does reduce the tensile strength, closely riveted joints, such as those of boilers, would be perceptibly weakened by the compression due to the contraction of the rivets in cooling. It seems, therefore, reasonable to infer that a moderate strain (three or four tons per square inch at all events) of either kind does not affect the ultimate strength of iron to sustain a strain of the other kind at right angles. However this may be, practical reasons prevent the plate-iron web from being so economical as that formed of bracing, except in very small or very shallow girders, or girders which sustain unusually heavy loads and in which therefore the shearing-strain is very considerable, or near the ends of girders of very large span; for unless the plating be reduced in thickness to the extent which theory indicates as sufficient, but which is quite unsuitable for practical reasons, the bars of the braced web will require so much less material than the continuous web of a plate girder as to make the former really the more economical.

One quarter inch may be assumed to be the minimum thickness that experience sanctions for the plating of permanent structures. A thinner plate than this may with care last for years, but few engineers would wish to risk the stability of any important structure on the chance of such frequent attention to prevent corrosion as so great a degree of tenuity would require. Mr. Mallet gives the relative oxidation of certain metals in moist air as follows*:

Cast-iron, - - - - -	·42
Wrought-iron, - - -	·54
Steel, - - - - - -	·56

He also states at p. 27 of his third report on the action of air and water upon iron to the British Association in 1843, that in one century the depth of corrosion of Low Moor plates, as deduced from his experiments, would be—

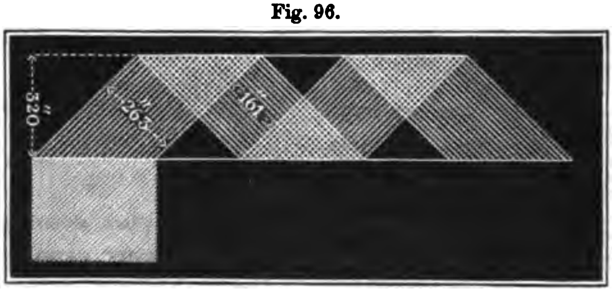
	Inch.
In clear sea water, - -	0·215
In foul sea water, - - -	0·404
In clear fresh water only, -	0·035

433. Plating more economical than bracing near the ends of very long girders—Continuous webs more economical in shallow than in deep girders.—When, however, the span is of

* *On the Construction of Artillery*, p. 138.

great extent the opens between the braces towards the ends become smaller from the increased width of the bars, and therefore nearly equal to their overlap; hence there is a certain length of girder beyond which it may be found more economical to form the ends of the web of continuous plates, and the intermediate portion of diagonal bracing. The length of girder at whose extremities the same amount of material is required for the web, whether formed of bracing or of plates, depends, among other things, on the ratio of depth to span. In large railway girders, in which this ratio is frequently about 1 to 15, the span beyond which it becomes more economical to substitute plating near the ends in place of bracing lies between 300 and 400 feet. Take, for example, the single-line railway bridge of 400 feet span, whose weight is calculated in the chapter on estimating girder-work. The length = 400 feet, the depth = 26·67 feet, or 1-15th of the length, and the maximum weight, including the permanent load, which the bridge has to support = 1,490 tons distributed uniformly. One-fourth of this, or 372·5 tons, is the shearing-strain supported by the web at the end of one main girder. Now if the bracing be at an angle of 45°, which is the most economical angle, the strain in the end diagonals will equal the shearing-strain multiplied by 1·414 = 526·7 tons, requiring, at 4 tons per inch, a gross section of 131·7 square inches.* If the iron be half-inch, the width of the end diagonal will equal 263 inches, as in Fig. 96, in which for simplicity only one system of triangulation is represented, since the overlap will be the same whether one or several systems be adopted.

Fig. 96.



* In consequence of the rivet holes, 4 tons per square inch of gross section is for tensile strain assumed equivalent to 5 tons per square inch of net section.

It is evident that the overlap of the bars considerably exceeds the open spaces. This example, therefore, has attained the span beyond which it would be more economical to employ plating for the end portions of the web. If $\frac{3}{8}$-inch plating be considered sufficiently thick the limit would of course happen sooner. If, however, the depth were greater than 1-15th of the length, the limit would be greater than in our example. It is obvious also, from what has just been stated, that the relative economy of plate webs is greater in shallow than in deep girders; for if bracing were used the opens between the braces would be much smaller in the former than in the latter case, and, consequently, if these opens be filled up by continuous plating, there will be less waste of material in the shallow than in the deep girder.

434. Greater proportion of a continuous web available for flange strains in shallow than in deep girders.—That plate girders derive from the continuity of the web some increase of strength over that due to the sectional area of the flanges is certain (**99**), but the amount of horizontal strain which a thin web is capable of transmitting is, in large girders, generally too indefinite to admit of any considerable reduction in the area of the flanges on this account, and is, therefore, practically of slight importance, for it seems unlikely that horizontal strains of compression can be transmitted with much energy through the thin continuous web of a deep girder, except in that portion which is close to the flange, and therefore stiffened against buckling by its connexion therewith. In shallow plate girders, however, such as those used for the cross-girders of bridges, deck-beams of ships, fire-proof floors, &c., the web generally forms a large portion of the whole, possesses considerable strength by itself, and is therefore available for horizontal as well as vertical strains.

These considerations show that the compression flange of a shallow plate girder derives a greater per-centage of aid from the web than that of a deep girder.

435. Deflection of plate girders substantially the same as that of lattice girders.—From these considerations it would also appear that the deflection of plate girders is little, if at all, less than that of lattice girders, the length, depth, and flange area being the same in both; for if their flanges be subject to the same unit-strains, their deflections will be alike (**224**). Even assuming that the web does relieve the flanges of horizontal strain to the full extent which theory indicates, the deflection will not be very materially diminished thereby, for it appears from eq. (149) that a continuous web is for horizontal strain equivalent to only $\frac{1}{6}$th of its area placed in each flange. Plate girders, it is true, are generally thought to be stiffer than those with braced webs, and closely latticed girders than those with only one or two systems of triangulation, but I am not aware of accurate comparative experiments on this subject. It is quite possible that when the compression flange has but few points supported by intersecting braces it may assume under strain a slightly undulating line, and therefore be a little shorter than a similar flange held straight at short intervals by close latticing or a plate web. This would of course increase the deflection.

436. Webs of cast-iron girders add materially to their strength.—The webs of cast-iron girders are usually made much stronger than is required for the mere transmission of the shearing-strain. Hence they rarely require stiffening ribs, and the web should add to the strength of such girders, calculated merely from the leverage of either flange round the other as a fulcrum, by an amount nearly equal to the breaking weight of the web taken separately (**99**).

437. Minute theoretic accuracy undesirable.—In constructing wrought-iron girders of small span, say under 30 or 40 feet, it is generally more economical to make the lattice bars of one, or at most, of two sizes throughout, even though they might be safely reduced in section as they approach the centre. This arises from the expense and trouble of having different templates and a stock of bars of various sizes. It is, therefore, cheaper to have a slight excess of material than go to the nicety of sizes which would be theoretically strong enough. For a similar reason $2\frac{1}{2}$ inches may be assumed to be the minimum useful width for a lattice bar of ordinary railway girders. When of less width it is generally necessary to swell out the rivet holes in the forge, so as to avoid reducing the effective section of the bar, and, independently of the bad effect produced by heating the iron, this process is of course more expensive than cold punching. One result of this is that the central bracing is generally stronger than theory requires.

488. Multiple and single systems of triangulation compared—Simplicity of design desirable.—This leads to another consideration, viz., the number of systems employed in bracing. It has been already stated (**155**) that the practical advantage of a multiple over a single system of triangulation consists in the more frequent support given to the compression bars by those in tension, and by both to the flanges, thus subdividing the parts which are subject to compression into a number of short pillars, and restraining them from deflection chiefly in the plane of the girder. It may also be urged in favour of close latticing, that if an accident, such as an engine running off the line, occurs on a bridge with the braces few and far apart, that in such a case the safety of the whole structure is menaced by the fracture of a single bar, whereas a closely latticed or plate girder is not only comparatively free from this danger, but affords greater security in case of one bar being originally defective, while to the public eye it has the semblance of greater safety, a consideration not altogether to be despised.

The number of systems adopted will also depend on the distance between the cross-girders, which generally occur at an apex, and on the practical consideration of what sized material is the most economical; and this again will depend on two things, the first cost of iron of small and large scantlings, and the subsequent cost of workmanship; which latter item varies much according to the simplicity or complexity of the design. No definite rule can be laid down for all cases, but one consideration of importance should not be overlooked in seeking after apparent economy at the outset. The larger the scantlings and the more *simple the method of construction* the smaller is the surface exposed to atmospheric influences, and the more easily detected is any corrosion or decay when such does occur. The chief advantage of masonry is its permanent character. No rust or decay in it requires constant attention or painting, and, if well executed at the outset, masonry truly deserves the title of permanent.

439. Ordinary sizes of iron.—It will be useful to recollect that bars or strips are not rolled wider than 9 inches; when a greater width than this is required narrow plates with shorn edges must be used. Plates exceeding 4 feet in width or 15 feet in length, or containing more than 32 square feet, or weighing more than 4 cwt., are generally charged extra; also T or angle iron, the sum of whose breadth and depth exceeds 9 inches. Plates are rolled up to 7 feet wide or 30 feet long, and 60 square feet in area; they increase in thickness by sixteenths of an inch, and are generally called *sheet* iron when less than $\frac{3}{16}$ inch thick. Ordinary angle iron can be got in lengths of from 30 to 36 feet, and up to $6 \times 6 \times \frac{7}{8}$ inches.

440. Testing small girders by a central weight equal to half the uniform load not accurate.—Small girders are frequently tested by a central weight equal to half the uniform or passing load which they are expected to carry with safety. Though convenient this is not altogether a fair trial of the web. Let $W =$ the proof load in the centre, and $2W =$ the uniform load. The web of a girder designed to support the central load, W, should be of uniform strength, for it sustains throughout a shearing-strain equal to $\frac{W}{2}$ (**34**). The web of a girder designed for the uniform load, 2W, should increase from the centre where the strain is slight, towards the ends, where the strain $= W$, in proportion to the distance from the centre (**46**); and the web of a girder designed to support a passing load of the same density as the uniform load, should increase from the centre towards the ends, where the shearing-strain $= W$, in the ratio of the square of the distance from the further end (**50**). Consequently the strain in the centre of the web $= \frac{W}{4}$. It is obvious, therefore, that the web near the centre is subject to a much greater strain from a central load than from a uniform or passing load of twice its weight, whereas at the ends the reverse of this takes place. The importance of these remarks may be practically lessened by the considerations referred to in (437).

441. Connexion between web and flanges—Uniform strain in flanges—Trough and ⊢⊣ sections—Rivets preferable to pins.—In wrought-iron girders the shearing area of the rivets connecting each brace with the flanges should equal the net section of the brace; otherwise there is a risk of its separating from the flanges at a much lower strain than would destroy the brace. If the web be a continuous plate, the shearing area of the connecting rivets should equal its theoretic horizontal section, *i.e.*, the horizontal net section of a plate whose thickness is that which theory demands; in practice, however, the plate area is generally considerably in excess of what theory requires, and hence the rivet area seldom equals its horizontal net section. The ┳┳ shaped or trough section (Fig. 89, p. 221), is a favourite form for the flanges of tubular braced girders, as it affords great facilities for attaching the bracing to the flanges. Objections have been raised to the trough with deep vertical plates, on the ground that the unit-strain is not constant throughout its whole area, the unconnected edges of the vertical plates being subject to a severer unit-strain than the horizontal plates in consequence of each brace giving off its horizontal component of strain at a point which generally lies nearer the free edge of the vertical plate than the centre of gravity of the whole section. Let us confine our attention to the compression flange, as similar reasoning applies to that in tension. This tendency to excessive strain is sometimes supposed to show itself by a slight undulation or buckling of the free edge of the vertical plate endeavouring to escape from the line of thrust. This buckling, however, is not necessarily a sign of excessive compression, but rather of defective stiffness in the lower part of the plate, for if it were stiffened laterally so that it could not escape from the line of thrust, and if the unit-strain along this edge were greater than that in the horizontal plates, the result would be that the whole flange would camber from the shortening of its lower edge. This, however, does not take place, and hence it is reasonable to suppose that the strain is not very unequally distributed throughout the whole section. Undulation certainly is a defect, and proves that the plate is not standing up to its work, and therefore not subject to excessive compressive strain; it rather indicates that a small portion of the vertical plate at each apex on the side remote from the centre may be in tension, pulling, instead of thrusting, the flange towards the centre. Vertical plates ought therefore to be thick enough to resist buckling, say $\frac{1}{15}$th of their depth (**321**), or else be stiffened by an angle iron along their free edges. The weight of the flange itself acting as a series of short girders between the apices tends to produce tension in the lower edges of the vertical plates, and so far counteracts excessive compressive strain, and the whole flange being held at short intervals by the bracing resembles a long thin pillar inside a tube; the pillar may undulate slightly and press here and there against the sides of the tube, but the compressive strain may for all practical purposes be considered as being distributed uniformly throughout the whole section of the pillar. The ⊢⊣ section of flange has its advocates, who maintain that it is free from the objections alleged to lie against the trough section. The practical convenience of the latter, however, will probably enable it to hold its ground against its rival. The student who wishes to learn the views of eminent engineers on this subject is referred to the discussions on "The Charing Cross Bridge" and "Uniform Stress in Girder Work," in the 22nd and 24th Vols. of the *Proceedings of the Institution of Civil Engineers.* The main bracing is sometimes connected to the vertical plates by pins, like those of suspension bridges. Judging, however, from the experience gained at the Crumlin Viaduct—where riveting was substituted for pins, after some years' wear and vibration had loosened the latter—it seems desirable to make rigid connexions, and for this purpose riveting is at once the most convenient and effective method.*

Each pair of tension and compression braces should intersect somewhere in the vertical plate. In very faulty designs the braces

* *The Engineer*, 1866, Vol. xxii., p. 384.

are sometimes arranged so that they do not intersect in the flange at all, but would, if produced, meet considerably above it, in which case the flange is subject to an injurious cross-strain, and is liable to become broken-backed from the compression braces thrusting it upwards while the tension braces pull it down. In some instances this has produced disastrous results. When the vertical plate is deep enough to give a choice of position, the apex may be either in the middle or rather closer to the upper edge, the latter position being perhaps the better of the two.

CHAPTER XX.

CROSS-BRACING.

442. Weather-bracing—Maximum force of wind.—Cross-bracing generally fulfills two functions; it acts as a horizontal web, holding the compression flanges at short intervals in the line of thrust and thus preserving them from lateral flexure to which all long pillars are liable; it also braces the whole structure in a horizontal plane, strengthening it to resist the side pressure of the wind just as the vertical web enables the main-girders to sustain the downward pressure of the load. When the roadway is attached to the lower flange, and the depth of the main-girders is not sufficient to admit of cross-bracing between the upper flanges, the latter must be made sufficiently wide to resist any tendency they may have to deflect sideways under longitudinal compression, and their lateral stiffness may be calculated by the laws of pillars, though they are much aided by the internal bracing of the compression braces, or the angle iron stiffners of plate webs, which convey a large share of rigidity from the roadway to the upper flanges. Under these circumstances the roadway and cross-bracing between the lower flanges have to resist the greater portion of the lateral pressure of the wind, whose maximum force in this country may, for the purpose of calculation, be assumed equivalent to a uniform pressure of 25 lbs. per square foot of side surface exposed to its influence.

443. Pressure of wind practically uniformly distributed.—The pressure of the wind is not always, as might be supposed, uniformly exerted along the whole length of a girder. With reference to the effect of violent gales on the Britannia Bridge, Mr. Clark remarks:—"The blow struck by the gale was not simultaneous throughout the length of the tube, but impinged locally and at unequal intervals on all parts of the length which presented a broadside to the gale."*

A little further on he remarks:—"The tube, however, on no

* Clark *on the Tubular Bridges*, p. 455.

occasion attained any serious oscillation, but appeared, to some extent, permanently sustained in a state of lateral deflection, without time to oscillate in the opposite direction." Hence the effect of the wind may be assumed not very different from that of a uniformly distributed load; as a precautionary measure, however, it is desirable to make the central bracing somewhat stronger than would be requisite if the pressure were really uniform.

444. Smeaton's table of velocity and force of wind.— The following table of the velocity and corresponding pressure of the wind by Mr. Rouse is given by Smeaton in the *Philosophical Transactions* for the year 1759:—

Velocity of the Wind.		Perpendicular force on a square foot in lbs. Avoirdupois.	Common appellations of the Wind.
Miles per hour.	Feet per second.		
1	1·47	·005	Hardly perceptible.
2	2·93	·020	Just perceptible.
3	4·40	·044	
4	5·87	·079	Gentle pleasant gale.
5	7·33	·123	
10	14·67	·492	Pleasant brisk gale.
15	22·00	1·207	
20	29·34	1·968	Very brisk.
25	36·67	3·075	
30	44·01	4·429	High winds.
35	51·34	6·027	
40	58·68	7·873	Very high.
45	66·01	9·963	
50	73·35	12·300	A storm or tempest.
60	88·02	17·715	A great storm.
80	117·36	31·490	A hurricane.
100	146·70	49·200	A hurricane that tears up trees, and carries buildings before it, &c.

445. Strains produced in the flanges by cross-bracing.—When there are both upper and lower cross-bracings, each has to sustain one-half the pressure of the wind; consequently, in every gale the compression flange on the weather and the tension flange on the lee side have their normal strains somewhat increased, while those in the other flanges are diminished to the same extent. This increase and diminution of strain are, however, generally insignificant compared to the strains produced by the load, and are, of course, less in open-work girders than in those with solid sides, which present a larger unbroken surface to the action of the wind.

446. Cross-bracing must be counterbraced—Best form of cross-bracing—Initial strain advantageous.—As the wind may blow on either side of a bridge it is necessary to counterbrace the cross-bracing throughout; hence the description of bracing described in Chap. VII., with transverse struts and diagonal ties, is well suited for cross-bracing, and in order to make it stiff and come into action before much lateral movement takes place, it is desirable to put a small initial strain on the diagonals. This will tend also to stiffen the whole structure against lateral vibration from moving loads. The initial strain may be produced by coupling screws, cotters, or similar appliances. When the design does not admit of these the transverse struts may be first riveted in place, and then the diagonals may be riveted while they are temporarily expanded by heat; when cold the whole will be in a state of slight strain. The same effect may be produced in small tubes by laying them on their side, so that the cross-bracing may be in a vertical plane; a few weights will then stretch one system of diagonals, and when thus strained the second series may be riveted in place. After the removal of the weights the required degree of initial strain will be produced if the operation be carefully performed. The sagging of the cross-bracing from its own weight between girders placed far apart, will also aid in producing the required amount of stiffness, provided the bars be supported in a horizontal position while riveting up.

The absence of the initial strain alluded to was strongly marked in the Britannia Bridge. At p. 717, Mr. Clark remarks:—"The effect of pressure against the side of the tube is very striking; a single person, by pushing against the tube, can bend them to an extent which is quite visible to the eye; and ten men, by acting in unison, and keeping time with the vibrations, can easily produce an oscillation of 1¼ inch, the tube making 67 double vibrations per minute." A severe storm on the 14th of January, 1850, produced oscillations not exceeding one inch. This was before the two tubes were connected together, side by side.

447. End pillars subject to transverse strain.—When there is cross-bracing between the upper flanges the pressure of the wind against the upper half of the girder is transmitted to the abutments or piers through the end pillars which form the terminations of the web immediately over the points of support, at least so much of it as is not conveyed by the web stiffeners to the lower flanges and thence to the abutments. These pillars are therefore semi-girders as well as pillars, as they are subject not only to vertical compression from the shearing-strains in the main bracing, but to lateral pressures at top tending to overthrow them, and nearly equal in amount to one-half the total pressure of the wind. Thus, if there be two main girders and four end pillars, each of the latter sustains a transverse pressure at top nearly equal to one-eighth of the pressure of the wind. It is, therefore, desirable to fix the lower ends of these pillars very securely by means of strong gussets attached to the masonry, or, if these be inadmissible from the longitudinal expansion of the bridge, to a cross road girder, which may be made stronger and stiffer than usual for this purpose.

448. Bow of bowstring girders subject to transverse strains.—The bowstring girder, with roadway attached to the string, does not admit of cross-bracing between the bows throughout their entire length, but only near the centre, where there is sufficient headway for carriages beneath. The ends of the bows are consequently subject to transverse strains similar to those just described in the case of end pillars of girders with horizontal flanges.

CHAPTER XXI.

CROSS-GIRDERS.

449. Maximum weight on cross-girders—Economical distance between cross-girders.—The cross-girders of railway bridges support the platform, ballast, sleepers, and rails; and when the interval between them does not exceed that between two adjacent axles of a locomotive, say 6 or 7 feet, the greatest load which each cross-girder has to support is determined by the weight resting on one pair of driving wheels, which rarely, if ever, exceeds 16 tons or 8 tons per wheel. Consequently, if the effect on the rails, sleepers, and platform in spreading the load over several girders be neglected, each road girder, however close they may be together, ought to be capable of sustaining 16 tons if the bridge be made for a single line, and twice this if made for a double line, in addition to the permanent weight of platform, ballast, &c., and as a train of locomotives and tenders, that is, the load of maximum density, does not exceed 1½ tons per running foot, it would obviously be the most economical arrangement to place the cross-girders, at all events, not closer together than the above stated distance of 6 or 7 feet.* It may, perhaps, be supposed that cross-girders placed at shorter distances need not be so strong in consequence of the rails, sleepers, and platform distributing the load over several cross-girders, and this, no doubt, is to a certain extent correct, and numerous bridges have been constructed on this principle. Government Inspectors are now, however, more particular than formerly, and each cross-girder should be strong enough to sustain the load on the driving wheels of the heaviest engine which can come on the line, inasmuch as the sleepers may decay, joints may occur in the rails close to a cross-girder, or the platform may

* The cross-girders of the Boyne Viaduct are 7 feet 5 inches apart, equal to the diagonal of the square formed by the lattice bars of the main-girders. The interval between those of the Britannia and Conway Tubular Bridges is 6 feet.

require renewal, and perhaps be altogether removed for this purpose.

450. Rail girders or keelsons—Permanent load of roadway.—When the cross-girders are further apart than from 2 to 3 feet (the distance from sleeper to sleeper) the rails must be supported by longitudinal girders resting on the cross-girders or framed in between them, and in certain cases, especially when the levels permit the cross-girders to be of great depth, these rail girders may be economically made of considerable length, with the cross-girders placed at long intervals apart, in some cases 20 feet asunder; but care must be taken not to strain the lattice bars of the main-girders beyond their safe limit, by bringing too great local pressure on those which occur at the cross-girders. The rail girders may be conveniently made of plating or lattice work, similar in general design to the main-girders of small bridges and framed in between the cross-girders. In some cases these rail girders run above the latter in unbroken lines from end to end of the bridge like the keelsons of a ship. This arrangement requires greater depth from soffit of bridge to rail than the former, and cannot therefore be so frequently adopted. The reader is referred to a valuable paper by Wm. Anderson, Esq., in the eighth volume of the *Transactions of the Institution of Civil Engineers of Ireland*, in which he shows by a tentative process that great economy is effected by placing the cross-girders 12 feet apart and upwards, especially with double lines. The permanent load of the roadway per running foot, including cross-girders 3 feet apart, platform, ballast, sleepers and rails for a single-line bridge, 14 feet wide between main girders (Irish gauge 5′ 3″), he estimates as follows:—

	Weight in tons per running foot.
Cross-girders, 3 feet apart, - - -	·18
Platform of 4-inch planks and bolts for same,	·10
Rails, chairs, spikes, and sleepers, - - -	·06
Ballast (from 3 to 4 inches deep), - -	·20
	0·54

This 0·54 ton is the permanent load of roadway for a single line per running foot, and is exclusive of main girders and cross-bracing, which vary with the span. The similar permanent load of roadway for a double line, 25 feet 6 inches between main girders, is about 1·08 ton per running foot, or double that for a single line. In bridges of small span it is frequently more economical to place the main-girders immediately beneath the rails; they then act as the rail bearers, and thus dispense with cross-girders (426). When, however, there is little head-room beneath the rails, a modification of the trough girder has been devised by Mr. Anderson, as in Figs. 97 and 98, which represent one of the bridges on the Dublin, Wicklow, and Wexford Railway.

Fig. 97.
Half Longitudinal Section and half Elevation of Bridge.

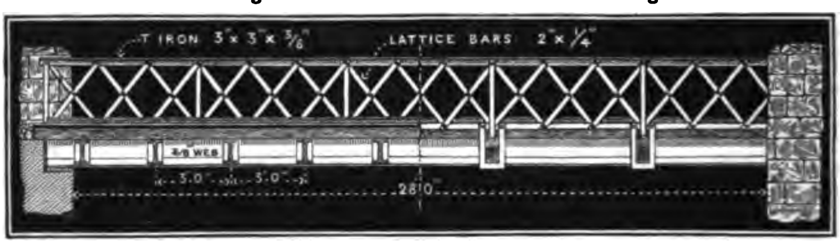


Fig. 98.
Cross Section of Bridge.

Each rail is carried between a pair of plate girders connected by short cast-iron saddles on which the sleeper and rail are laid, and to which they can be securely bolted. The girders are thus accessible in every part for cleansing and painting, without disturbing the permanent way, and at the same time water cannot lodge in the iron-work.*

* *Trans. of Inst. of C. E. of Ireland,* Vol. viii., p. 45.

450*. Requirements of Board of Trade.—The following requirements of the Board of Trade respecting railway bridges apply to the cross girders and platform.

1. The heaviest engines in use on railways afford a measure of the greatest moving loads to which a bridge can be subjected. This rule applies equally to the main and the transverse girders. The latter should be so proportioned as to carry the heaviest weights on the driving wheels of locomotive engines.

2. The upper surfaces of the wooden platforms of bridges and viaducts should be protected from fire.

3. No standing work should be nearer to the side of the widest carriage in use on the line than 2 feet 4 inches at any point between the level of 2 feet 6 inches above the rails and the level of the upper parts of the highest carriage doors. This applies to all arches, abutments, piers, supports, girders, tunnels, bridges, roofs, walls, posts, tanks, signals, fences and other works, and to all projections at the side of a railway constructed to any gauge.

4. The intervals between adjacent lines of rails, or between lines of rails and sidings, should not be less than 6 feet.

CHAPTER XXII.

COUNTERBRACING.

451. Permanent load—Passing load.—The strains in the web of a braced girder are constant both in amount and kind so long as the load remains stationary. If, however, the load change its position the strain will alter in amount, and perhaps in kind also, and it is to meet this latter change in the character of the strain that counterbracing is required (**140**). Now a certain portion of the load which every girder sustains is fixed, and consists of what I have elsewhere called the "permanent load," including in this term the weight of the whole superstructure, viz., the main girders, cross-girders, cross-bracing, platform, rails, sleepers, and ballast. This permanent load produces definite strains in the bracing which remain constant, both in amount and kind, until a further load comes upon the bridge. Let us consider the effect of a moving load of uniform density, say a train of carriages, traversing a girder with horizontal flanges, and we may chiefly confine our attention to the strains developed in the bracing at either end of the train as it has been shown in (**51**) and (**174**), that the maximum strains in the bracing from passing loads occur at these points. As the advancing train approaches the centre the normal strains in the bracing between the centre of the girder and the front of the train are diminished or even reversed by the passing load. In the latter case each brace attains its maximum reverse strain as the front of the train passes it, and counterbracing must be provided accordingly. During the same period, *i.e.*, while the train advances towards the centre, the permanent strains in the second half girder are receiving gradual increments of their own kind, but each brace in this half does not attain its state of maximum strain until the train has crossed the centre, and is so far advanced that its front is passing that particular brace, after which the strain again diminishes till the other end of the train is passing, when the strain is either at its minimum, or, if altered, attains its maximum of the reverse kind to that produced by the permanent load, in which case therefore the brace requires counterbracing.

459. Passing loads require centre of web to be counterbraced—Large girders require less counterbracing in proportion to their size than small girders.—The permanent load is usually disposed symmetrically on either side of the centre; consequently the normal strains in the bracing near the centre are less in amount than in other parts, and it is in the central braces alone that strains of a reverse character are produced by a moving load, requiring counterbracing for some distance on either side of the centre. It is evident that the heavier the permanent load is, the less will be the amount of counterbracing required for a given passing load: It has been already shown in (**50**) that the shearing-strain (to which the strain in the bracing is proportional) at the end of a passing train $= \frac{w'n^2}{2l}$, where w' = the passing load per linear unit, l = the length of the girder, and n = the length covered by the advancing load. But the shearing-strain due to the permanent load $= w\left(\frac{l}{2} - n\right)$, where w = the permanent load per linear unit and n and l as before, n being supposed less than $\frac{l}{2}$. Now, if n be proportional to l in girders of different lengths, the shearing-strain from the passing load will vary as $w'l$, and that from the permanent load as wl; and since w increases in large girders as some high power of the length, while w' may be considered constant for girders of all sizes, the shearing-strain due to the permanent load will bear a considerably greater ratio to that from the passing load in long than in short girders. Consequently the proportion which the counterbracing bears to the whole amount of material diminishes rapidly with the span of the girder. The counterbracing terminates where the two shearing forces are equal, and the point where this occurs may be determined by equating them to each other, and solving the resulting equation for n as follows:—

$$\frac{w'n^2}{2l} = w\left(\frac{l}{2} - n\right)$$

Arranging according to powers of n,

$$w'n^2 + 2wln - wl^2 = 0$$

solving for n,

$$n = l\frac{-w \pm \sqrt{w^2 + ww'}}{w'}$$

If, for example, $w = w'$,

$$n = l(-1 \pm \sqrt{2}) = \cdot 414l$$

453. Vertical and diagonal bracing.—Girders with vertical and diagonal bracing, such as that investigated in Chapter VII., may be counterbraced either by making the bracing near the centre capable of acting both as struts and ties, or by adding a second system of diagonals crossing the first. If this counterbracing be carried throughout the whole length of the girder (as in cross bracing), it is possible by tightening it up to produce an initial strain in the bracing proper, in which case the effect of a load will be to diminish the strain in the counterbracing, which, however, will relapse into its former state of strain as soon as the load is removed.

454. Large bowstring girders require little counterbracing.—I cannot close these observations on counterbracing without drawing attention to one important merit which bowstring girders possess. When the load is uniformly distributed the strains in the bracing are tensile, for then the lower flange and load are merely suspended from the bow which differs but slightly from the curve of equal horizontal thrust and therefore requires but little bracing to keep it in form. Hence compression strains are produced in the bracing only under the influence of passing loads; and in large girders, where the permanent load of string and roadway is great compared with the passing load, it may happen that the compressive strains produced by the latter do not exceed the tensile strains which the bracing sustains in its normal state. If, for instance, the permanent load of the lower flange and roadway in the example worked out in (**212**) were twice as heavy as the passing load, the strains in all the diagonals would be tensile under all circumstances; even if the permanent load were only one and a half times the passing load, diagonal 6 alone would sustain slight compression. In this case the difficulty of providing against flexure in long compression bars does not arise, and the only part of the structure subject to compression is the bow, which from its large sectional area can be constructed of a form suited to resist buckling or flexure.

CHAPTER XXIII.

DEFLECTION AND CAMBER.

455. Deflection curve of girders with horizontal flanges of uniform strength is circular.—It has been already shown in Chap. IX. that the deflection curve of girders with horizontal flanges of uniform strength, that is, girders whose flanges vary in sectional area so that they are subject to the same unit-strain throughout the whole length of each flange respectively, is circular and easily calculated by a simple formula (eq. 130). When, however, the flanges are of uniform section throughout their whole length, and their strength therefore excessive near the ends, the deflection will be somewhat less, and may be calculated by the method explained in the 226th and following articles. When the strength of a girder is not uniform there is of course a certain waste of material, which, however, cannot always be avoided, although some methods of construction—the cellular flanges of tubular bridges for instance—are more liable to this objection than others.

456. Deflection an incorrect measure of strength.—Since the deflection depends not only on the unit-strains in the flanges, but also on the length and depth of girder, the co-efficient of elasticity of the material, and to some extent the mode of construction, the popular rule by which the strength is estimated from the deflection alone, though possessing the merit of simplicity, is extremely vague and liable to lead to false conclusions unless when comparing girders of the same length, depth, and material.

457. Camber ornamental rather than useful—Permanent set after construction.—As the amount of deflection is always in practice very small compared with the length of a girder, no appreciable diminution of strength is produced merely by the change from a horizontal line to the deflection curve, for deflection, unless so excessive as to change the vertical reaction of the abutments, into an oblique one, is the result, not the cause, of increased strain. A downward curve, or even a truly horizontal line, is, however, less pleasing to the eye than a slight camber; hence it is desirable to give an initial camber somewhat in excess of the calculated deflection, so that when the girder is loaded no perceptible sag may suggest the idea of weakness, even though imaginary. It should also be borne in mind that the various parts of a built girder are put together free from strain, and are frequently a little out of line; consequently when a large girder first supports its own weight, and again, but in a less degree, when it is tested with a heavy load for the first time, there is a certain slight motion from the closing up or stretching out of the various parts accommodating themselves to their new state. A permanent set is the result, which, however, is not necessarily indicative of weakness, provided it is not increased by subsequent loads which should only produce a temporary deflection. This congenital set sometimes doubles the calculated deflection.

458. Loads in rapid motion produce greater deflection than stationary or slow loads—Less perceptible in large than small bridges—Deflection increased by road being out of order.—The Commissioners appointed to inquire into the application of iron to railway structures "carried on a series of experiments to compare the mechanical effect produced by weights passing with more or less velocity over bridges, with their effect when placed at rest upon them. For this purpose, amongst other methods, an apparatus was constructed, by means of which a car loaded at pleasure with various weights was allowed to run down an inclined plane, the iron bars which were the subject of the experiment were fixed horizontally at the bottom of the plane, in such a manner that the loaded car would pass over them with the velocity acquired in its descent. Thus the effects of giving different velocities to the loaded car, in depressing or fracturing the bars, could be observed and compared with the effects of the same loads placed at rest upon the bar. This apparatus was on a sufficiently large scale to give a practical value to the results; the upper end of the inclined plane was nearly 40 feet above the horizontal portion, and a pair of rails, 3 feet asunder, were laid along its whole length for the guidance of the car, which was capable of being loaded to about 2 tons; the trial bars, 9 feet in length, were laid in continuation of this railway at the horizontal part, and the inclined and horizontal portions of the railway were connected by a gentle curve. Contrivances were adapted to the trial bars, by means of which the deflections produced by the passage of the loaded car were registered; the velocity given to the car was also measured, but that velocity was, of course, limited by the height of the plane, and the greatest that could be obtained was 43 feet per second, or about 30 miles an hour. A great number of experiments were tried with this apparatus, for the purpose of comparing the effects of different loads and velocities upon bars of various dimensions, and the general result obtained was that the deflection produced by a load passing along the bar was greater than that which was produced by placing the same load at rest upon the middle of the bar, and that this deflection was increased when the velocity was increased. Thus, for example, when the carriage loaded to 1,120 lbs. was placed at rest upon a pair of cast-iron bars, 9 feet long, 4 inches broad, and $1\frac{1}{2}$ inch deep, it produced a deflection of $\frac{6}{10}$ths of an inch; but when the carriage was caused to pass over the bars at the rate of 10 miles an hour, the deflection was increased to $\frac{8}{10}$ths, and went on increasing as the velocity was increased, so that at 30 miles per hour the deflection became $1\frac{1}{2}$ inch; that is, more than double the statical deflection. Since the velocity so greatly increases the effect of a given load in deflecting the bars, it follows that a much less load will break the bar when it passes over it than when it is placed at rest upon it, and accordingly, in the example above selected, a weight of 4,150 lbs. is required to break the bars if applied at rest upon their centres; but a weight of 1,778 lbs. is sufficient to produce fracture if passed over them at the rate of 30 miles an hour. It also appeared that when motion was given to the load, the points of greatest deflection, and, still more, of the greatest strains, did not remain in the centre of the bars, but were removed nearer to the remote extremity of the bar. The bars, when broken by a travelling load, were always fractured at points beyond their centres, and often broken into four or five pieces, thus indicating the great and unusual strains they had been subjected to."* These experiments show that a load in rapid motion causes greater deflection than the same load at rest or moving but slowly, especially when the moving load is very large compared with the dead weight of the girder. The increase, however, is generally slight in railway practice, and the greater the weight of the structure is to that of the passing train the less will be the increment of deflection due to rapid motion. The difference of deflection caused by a locomotive crossing the central span of the Boyne Viaduct, 264 feet in the clear between supports, at a very slow speed and at 50 miles an hour was scarcely perceptible, and did not exceed the width of a very fine pencil stroke, but the increase of deflection is more marked in bridges of small span, as appears from the following experiments made on the Godstone Bridge, South Eastern Railway, by the Commissioners appointed to inquire into the application of iron to railway structures.† The Godstone is a cast-iron girder bridge, 30 feet in span, with two lines of railway.

	Tons.
Weight of two girders, - - - - -	15
Weight of platform between these girders, - -	10
Weight of half the bridge, *i.e.*, permanent load,	25
Weight of engine, - - - - - -	21
Weight of tender, - - - - - -	12
Moving load, - - - - -	33

Velocity in feet per second.	Deflection in decimals of an inch.
0,	·19
22 = 15 miles per hour, - - -	·23
40 = 27·3 do. do. - - -	·22
73 = 49·8 do. do. - - -	·25

* *Report*, p. xi.

† *Report, App.*, p. 250.

Similar results were obtained from the Ewell Bridge, upon the Croydon and Epsom Line. The span of the Ewell Bridge is 48 feet, the permanent weight of one-half is 30 tons, and the statical deflection due to an engine and tender, weighing 39 tons, was rather more than one-fifth of an inch. "This was slightly but decidedly increased when the engine was made to pass over the bridge, and at a velocity of about 50 miles per hour an increase of one-seventh was observed. As it is known that the strain upon a girder is nearly proportional to the deflection, it must be inferred that in this case the velocity of the load enabled it to exercise the same pressure as if it had been increased by one-seventh, and placed at rest upon the centre of the bridge. The weight of the engine and tender was 39 tons, and the velocity enabled it to exercise a pressure upon the girder equal to a weight of about 45 tons."*

The fact of slightly increased deflection from rapidly moving loads is also confirmed by Mr. Hawkshaw's experiments with an engine and tender run at a speed of about 25 miles an hour over five compound iron girder bridges on the Wakefield and Goole Railway. These girders varied in span from 55 feet 7 inches to 88 feet 6 inches, and were therefore less affected by rapid loads than the smaller bridges just described. Mr. Hawkshaw inferred that "where the road is in good order the deflection is not much increased by speed, but that where the road is out of order, then there is an increase of deflection." For instance, the road immediately leading on to one of the bridges in question "was considerably depressed in level, so that in running the train over the bridge at speed the whole weight of the train had to be suddenly lifted, and this of course had to be sustained by the girders as well as the ordinary weight of the train."†

The conclusions of the Commissioners, as given at p. xviii. of their report, is as follows:—"That as it has appeared that the effect of velocity communicated to a load is to increase the deflection that it would produce if set at rest upon the bridge; also that

* *Report*, p. xiv.

† *Report, App.*, p. 412.

the dynamical increase in bridges of less than 40 feet in length is of sufficient importance to demand attention, and may even for lengths of 20 feet become more than one-half of the statical deflection at high velocities, but can be diminished by increasing the stiffness of the bridge; it is advisable that, for short bridges especially, the increased deflection should be calculated from the greatest load and highest velocity to which the bridge may be liable; and that a weight which would statically produce the same deflection should, in estimating the strength of the structure, be considered as the greatest load to which the bridge is subject."

459. Effect of centrifugal force.—Centrifugal force produces a very slight but appreciable increase of pressure when the load passes rapidly across girders which, though ordinarily level, become deflected by the load, and still more so if they happen to have been built originally hollow in place of being cambered. The increased pressure due to this cause is expressed by the following well known equation:—

$$P = \frac{v^2 W}{gR} \tag{228}$$

Where P = the pressure due to centrifugal force,
W = the load,
v = the velocity in feet per second,
g = the acceleration due to gravity = 32 feet per second,
R = the radius of curvature in feet.

Ex. 1. A girder bridge 200 feet in span is deflected ·25 foot below the horizontal line by a certain load, W, at rest; what is the increased pressure due to centrifugal force if W traverses the bridge at the rate of 60 miles an hour?

$$\text{Here } v = \frac{60 \times 5280}{60 \times 60} = 88 \text{ feet per second.}$$

$$R = \frac{100 \times 100}{\cdot 5} = 20{,}000 \text{ feet.}$$

$$\textit{Answer.} \quad P = \frac{W \times 88 \times 88}{32 \times 20{,}000} = \cdot 0121\ W.$$

Ex. 2. If the span were only 100 feet, and the deflection and velocity as before, we would have,

$$\textit{Answer.} \quad P = \cdot 0484 W = \frac{W}{20} \text{ nearly.}$$

460. Practical methods of producing camber and measuring deflection.—The deflection of a girder supported at both ends is the result of the lower flange being extended while the upper one is shortened. Camber may be produced by the reverse of this, that is, by making the bays of the upper flange slightly longer than those of the lower one when the girder is in process of construction (**294**).

When small girders are under proof their deflection may be conveniently measured by means of a fine wire fastened to one end of the girder and passing over a pulley attached to the other end, where a small weight will keep it in a state of constant tension. The deflections should be read on a scale attached to the girder itself; when measured from an object fixed outside the girder they cannot be depended on, owing to the supports on which the ends of the girder rest being liable to compression or sinking under the weight of the testing load.

When great accuracy is not required the deflection of a girder bridge from passing loads may be measured by means of two wooden rods, the lower end of one of which rests on the ground while the upper end of the second rod is pressed against the soffit of the girder, so that they overlap each other midway; a pencil line is then ruled across both rods, and when the upper one is depressed by a passing load its line will descend slightly, the distance between the two lines giving the deflection of the girder.

CHAPTER XXIV.

DEPTH OF GIRDERS.

461. Depth of girders varies generally from one-eighth to one-sixteenth of the span—Depth determined by practical considerations.—The depth of large girders, with the exception of roof principals, seldom exceed $\frac{1}{8}$th or is less than $\frac{1}{16}$th of the span. For many years the common rule for cast-iron girders was to make the depth $\frac{1}{15}$th of the span, and this established a precedent for wrought-iron girders, but modern practice has with great advantage increased the ratio, so that $\frac{1}{12}$ and $\frac{1}{10}$ are now common proportions for braced girders. As the leverage of the flanges is directly as the depth while the quantity of material in the web is theoretically independent of it (**18**), it might be inferred that the deeper the girder the greater the economy. The practical limit, however, is defined by the extra material required to stiffen long compression bars or thin deep plate webs, nor should we overlook the necessity of having sufficient thickness in the latter for durability, and sufficient material in the compression flange to keep it from flexure or buckling.

462. Economical proportion of web to flange.—When a given quantity of material is to be distributed in the most advantageous manner, the thinner the web and the more the material is concentrated in the flanges, the stronger will the girder be, provided the web retains sufficient material for transmitting the shearing-strain; but when, as is frequently the case in small girders, the girder derives a considerable portion of its strength from the web acting as an independent rectangular girder, its thickness being determined from practical considerations (**484**), there is a certain depth, depending on the thickness of the web and the relation between the flanges, which will produce a girder of maximum strength.* If the flanges be equal in area this depth may be formed as follows.

Let l = the length of the girder,

b = the thickness of the web as determined by practical considerations,

d = the depth of the girder,

a = the area of either flange,

$a' = bd$ = the area of the web,

$A = 2a+a'$ = the total sectional area, which is a given quantity.

From equation (70) we have for the weight which an equal-flanged semi-girder fixed at one end and loaded at the other will support,

$$W = \frac{f}{l} d \left(a + \frac{a'}{6}\right)$$

in which f is the unit-strain in either flange. W is maximum when $d\left(a+\frac{a'}{6}\right)$ is maximum, and in order to find what value of d will produce this result we must substitute for a and a' their values in terms of d and the constant A, and then equate the differential coefficient of $d\left(a+\frac{a'}{6}\right)$ to cipher. Substituting, we have

$$W = \frac{f}{l}\left(\frac{A}{2} d - \frac{bd^2}{3}\right)$$

Equating the differential coefficient of the term within the bracket to cipher, we have

$$\frac{A}{2} - \frac{2}{3} bd = 0$$

whence
$$bd = \frac{3}{4} A \qquad (229)$$

The depth therefore should be such that the web may contain $\frac{3}{4}$ths of the whole amount of material.

* The thickness of the web of cast-iron girders, such as those used for bridges, generally varies from 1 to $2\frac{1}{2}$ inches. The thickness of boiler plate webs for railway or public bridges should not be less than $\frac{1}{2}$ inch (432).

CHAPTER XXV.

CONNEXIONS.

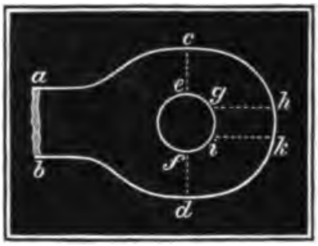
463. Appliances for connecting iron-work—Strength of joints should equal that of adjoining parts—Screws.—One general rule applies to all structures, namely, that the strength of the whole is limited by that of its weakest part, and accordingly the strength of joints should not be less than that of the parts which they connect. The usual appliances for connecting iron-work may be divided into four classes.

1. Screws.	3. Gibs and cotters.
2. Bolts or pins.	4. Rivets.

The strain to which the above mentioned connecters are subject is generally a shearing-strain, and as the strength of iron to resist shearing is practically equal to its tensile strength (**394**), the strength of an iron rivet, bolt, cotter, or screw, is measured by the product of the area subject to shearing multiplied by the tearing unit-strain of the iron. The thread of a screw subject to longitudinal tension may be "stripped" or shorn off by the nut; in the case of V threaded screws the shearing area is measured by the circumference of the bottom of the thread multiplied by the length grasped by the nut; in the case of square threads the shearing area is one-half of this.

464. Bolts or pins—Proportions of eye and pin in chains formed of flat links upsetting—Sir C. Fox's conclusions.—The bolt or pin is the simplest appliance for connecting together two pieces of iron, and, as the principal considerations connected with a bolt joint also apply to other and more complex forms, I shall devote a short space to its investigation. Take, for example, the joint of a suspension bridge, the chains of which are formed of long flat links connected by pins passing through their ends. Such a joint may fail in five ways.

Fig. 99.



1. By the link tearing through the eye at *cd* for want of sufficient material to withstand the longitudinal tensile strain. Hence the sectional area at *cd* should theoretically equal that of the link at *ab*; in practice it may be somewhat stronger, as the strain is less direct round the eye than in the body of the link.

2. By the end of the link being split along one or two lines such as *gh* and *ik* for want of sufficient section to resist the shearing action of the pin. Hence the combined sections at *gh* and *ik* should theoretically equal that of the link at *ab*, but in practice be somewhat greater, as this part of the eye acts as a short girder whose abutments are *ce* and *fd*; this causes the circumference from *c* to *d* to be in severe tension, and therefore very liable to tear.

3. By the pin being shorn across. This arises from its diameter being too small. Hence, if the pin be iron and in double shear, its area should not be less than one-half that of the link at *ab*.

4. By the pin bending. This also arises from its diameter being too small to afford requisite stiffness; the tendency may be diminished by the links being kept from spreading asunder by a head and nut on the pin, at the loss, however, of freedom of motion.

5. By the crown of the eye being upset between *g* and *i*. This arises from the bearing surface of the pin being too small in proportion to the longitudinal strain, in which case there is an excessive pressure on each superficial unit at the crown of the eye whereby the material there is upset, and the sides of the eye at *e* and *f* become first unduly attenuated and then torn, the rent extending from the inside towards the circumference. Sir C. Fox has drawn attention to this latter source of failure in a valuable

communication to the Royal Society,* in which the following remarks occur:—"If the pin be too small, the first result on the application of a heavy pull on the chain will be to alter the position of the hole through which it passes, and also to change it from a circular to a pear-shaped form, in which operation the portions of the metal in the bearing upon the pin becomes thickened in the effort to increase its bearing surface to the extent required. But while this is going on, the metal round the other portions of the hole will be thinned by being stretched, until at last, unable to bear the undue strains thus brought to bear upon it, its thin edge begins to tear, and will, by the continuance of the same strain, undoubtedly go on to do so until the head of the link be broken through, no matter how large the head may be; for it has been proved by experiment that by increasing the size of the head, without adding to its thickness (which, from the additional room it would occupy in the width of the bridge, is quite inadmissible), no additional strength is obtained. The practical result arrived at by the many experiments made on this very interesting subject is simply that, with a view to obtaining the full efficiency of a link, *the area of its semi-cylindrical surface bearing on the pin must be a little more than equal to the smallest transverse sectional area of its body*; and as this cannot, for the reasons stated, be obtained by increased thickness of the head, it can only be secured by giving a sufficient diameter to the pins. That as the rule for arriving at the proper size of pin proportionate to the body of a link may be as simple and easy to remember as possible, and bearing in mind that from circumstances connected with its manufacture the iron in the head of a link is perhaps never quite so well able to bear strain as that in the body, I think it desirable to have the size of the hole a little in excess, and accordingly for a 10″ link I would make the pin 6⅞″, in diameter, instead of 6⅔″, that dimension being exactly ⅔rds of the width of the body, which proportion may be taken to apply to every case (where the body and heads are of uniform thickness).

* "On the Size of Pins for connecting Flat Links in the Chains of Suspension Bridges." *Proc. Roy. Soc.*, Vol. xiv., No. 73, p. 139.

As the strain upon the iron in the heads of a link is less direct than in its body, I think it right to have the sum of the widths of the iron on the two sides of the hole 10 per cent. greater than that of the body itself. As the pins, if solid, would be of a much larger section than is necessary to resist the effect of shearing, there would accrue some convenience, and a considerable saving in weight would be effected, by having them made hollow and of steel."

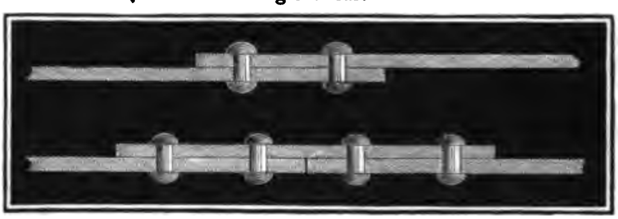
465. Rivets in single and double shear—Rules for riveting tension and compression joints—Hodgkinson's conclusions respecting strength of ordinary riveting.—The strength of a riveted joint, so far as the rivets are concerned, is proportional to the number of shears to which they are subject, a rivet in double shear, Fig. 100, being twice as strong as a rivet in single shear, Fig. 101; so that to make the joints of equal strength, the single shear joint must have twice as many rivets as the other.

Fig. 100.

Double Shear.

Fig. 101.

Single Shear.



When a joint connects tension plates the aggregate shearing area of the rivets on each side of the joint line, multiplied by the safe shearing unit-strain of the rivets, should equal the total working *strain transmitted through the plates.* It thus happens in iron-work that the shearing area of the rivets at a tension joint is nearly equal to the effective plate area, *i.e.*, the net area of the plates after deducting rivet holes. In steel-work the rivet area should be one-third greater (**394, 395**). When a joint connects compression plates whose ends do not butt closely against each other, the thrust is transmitted through the covers and tends to shear the rivets across exactly in the same manner as when a tensile strain is transmitted, and the foregoing rule applies here also. If, however, the compression plates have their ends planed square, and then brought very carefully into close contact so as to form a "jump" joint, a short cover and one, or at most two transverse rows of rivets on each side of the joint line will suffice, as the use of the cover in this case is merely to keep the joint in line but not to transmit the thrust. A jump compression joint is erroneously supposed to be stronger than one in which the plates are slightly apart with the covers and rivets duly proportioned as for a tension joint, and engineers who do not make out their own specifications and designs are sometimes over-exacting in this respect, expecting water-tight joints when the contractor gets only 16s. or 17s. per cwt. for the girder. My own impression is that a real jump joint, with plates butting along their whole width, is rare, as the process of riveting often draws the plates slightly apart, and an interval of a hundredth of an inch is nearly as bad as a quarter inch. A little caulking of the edges, however, makes all smooth to the eye, and the so-called "jump" joint passes muster.

With respect to the ordinary method of riveting in transverse rows, each row containing the same number of rivets, Mr. Hodgkinson deduced from his experiments that "the strength of plates however riveted together with one row of rivets, is reduced to about one-half the tensile strength of the plates themselves; and if the rivets be somewhat increased in number, and disposed alternately in two rows, the strength is increased from one-half to two-thirds or three-fourths at the utmost." *

* *Commissioners' Report*, App. p. 116.

466. Covers—Pitch of rivets.—*The strength of the covers of tension joints, and compression joints where the plates do not butt closely, should equal that of the plates;* hence a single cover should resemble a short length of the plate and be twice as thick as each half of a double cover.

As the quantity of material required for covers forms a very considerable per centage of the plates (12 per cent. and upwards, depending on the length of the plates), it is of great importance that the joints be as few as possible and arranged in the very best manner. This is more especially the case in large girders, where every ton of useless weight requires perhaps several tons in the main girders for its support, as will be shown in a succeeding chapter. For this reason large plates, with few joints, though they may cost extra per ton, will often make a cheaper girder than plates of ordinary sizes with more numerous joints. In the usual method of riveting, two or three transverse rows of rivets are placed on each side of the joint line, each row containing the same number of rivets, and the effective area of the plate, if in tension, is reduced by the aggregate section of the rivet holes in any one row. Hence it would appear that the fewer rivet holes there are in each transverse row the less is the plate weakened and the more is its material economized. But this again requires several successive rows of rivets in order to provide sufficient rivet area, thus introducing the necessity of long covers which may more than counterbalance the saving in the plates. The size of the plates therefore will determine to some extent the economical length of the covers as well as the transverse pitch of the rivets.*

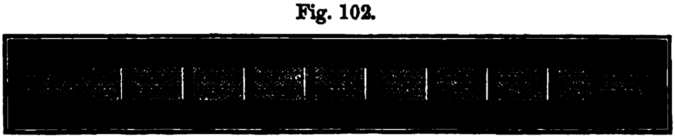
467. Single and double covers compared—Lap-joint.—The few experiments described in (**398**) seem to indicate that rivets in single shear do not withstand so great a unit-strain as rivets in double shear; this, however, requires confirmation, and good experiments on the strength of various forms of rivet joints are much wanted. From those recorded by Mr. Fairbairn in the appendix to the first series of "Useful Information for Engineers," it appears that as far as the plates are concerned a lap or single-cover joint with only one

* The "Pitch" is the distance measured from centre to centre of rivets.

transverse row of rivets in the lap is considerably weaker (in the experiments about 25 per cent. weaker) than a double-cover joint of the same theoretic strength, *i.e.*, with the same net area of plates taken across the rivet holes. This arises from the distortion of the single-cover or lap joint which, yielding in its effort to assume a straight line between the points of traction, bends the plates slightly, and makes them liable to tear across the line of rivet holes. When, however, a lap or a single-cover joint had two or more transverse rows of rivets in the lap its strength was not less than that of a double-cover joint of equal plate area. If the plates are kept in a straight line by being riveted to an angle iron or web, like the flange plates of a girder, it is still more likely that the strength of a single-cover joint will be fully equal to that of a double-cover joint of the same theoretic strength, but whenever convenient, the double-cover should be adopted from economical motives, as it gives double shear to the rivets, and need therefore be only half as long as a single cover with the same rivet area. The common lap-joint, represented above in Fig. 101, is, however, an exception to this, as the lap need be no longer than half the single cover.

468. Piled plates more economical than cells—Compression joints of piles require no covers if the plates are well butted—Cast-zinc joints.—I have already advocated the piling of plates over each other in preference to closed cells formed of single plates when a large flange area is required, and I have shown that long rivets form no practical objection to the former arrangement (**423, 424**). Besides other merits the pile requires less material in covers. When several plates are riveted together with their joints arranged in steps, the length of the covers equals the lap of one plate multiplied by the number of plates + 1. Thus, in Fig. 102, the pile consists of three plates and the length of each cover equals four laps.

Fig. 102.



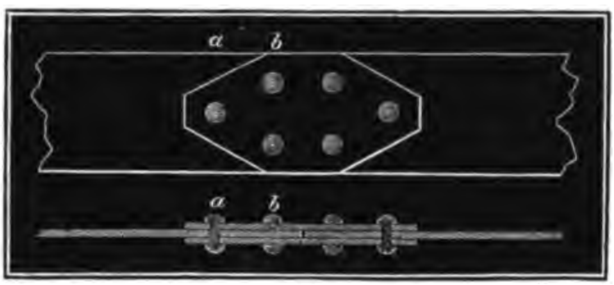
If, however, the plates be single, forming the sides of a cell, there will be as many separate joints as there are plates, so that the aggregate length of covers will equal the same lap as before multiplied by twice the number of plates; in this example the number would be six. A considerable saving therefore is effected by forming the joints in the manner described. If a pile of several plates be in compression and closely fitted, so as to butt against each other, no covers will be required, and great economy will result in very large girders, so much so as amply to repay the extra expense of planing the ends of the plates and bringing them carefully into close contact. To ensure this, however, requires considerable attention, for the riveting process has, as already observed, a tendency to open the joints slightly, but cast-zinc, which is a very hard substance, may be usefully employed for running into the compression joints of wrought as well as cast-iron, provided they are sufficiently open to let the molten metal flow freely. The joints of the cast-iron voussoirs of the Bridge of Austerlitz in Paris, finished in 1806, were thus formed,* and in my own practice I have used cast-zinc for filling up the irregular intervals between the ends of the arched ribs of a cast-iron bridge of 96 feet span and the wall-plates from which they sprung; in this case accurate fitting would have been extremely difficult, if not impossible, and a very satisfactory and close joint was made by slightly warming the parts with a fire of chips "to expel the cold air," as the workmen say, before pouring in the molten zinc. It probably expels moisture and assists the flowing of the metal into the narrower crevices. I have also used cast-zinc very successfully for securing crane posts (both cast and wrought-iron) in their foundation plates, where it ensures close contact without the cost of fitting. The following description of this method of forming the joints of a cast-iron arch of 133 feet span on the Pennsylvania Central Railroad occurs at p. 244 of *Haupt on Bridge Construction*:—"The joints were separated to the distance of one-fourth of an inch, and filled with spelter (cast-zinc) poured into them in a melted state; this

* *Enc. Brit.*, 8th Ed., art. "Iron Bridges," Vol. xii., p. 111.

was very conveniently done by binding a piece of sheet-iron around each joint, and covering it with clay. The material introduced being nearly as hard as the iron itself, and filling all the inequalities of the surface, rendered the connexion perfect." If the space between two plates be very narrow, the joint should be placed in a vertical position so that gravity may aid the flow of the metal, and a little tin added to the zinc will render the latter more fluid.

469. Economical arrangement of tension joints.—The following method of riveting reduces the tensile strength of the parts connected less than that in common use, and possesses the merit of being applicable to plates as well as bars. Its peculiarity consists in diminishing the number of rivets in each row as they recede from the joint-line, and at the same time slightly increasing the thickness of the cover or covers beyond that of the parts connected. Fig. 103 represents this arrangement applied to a bar

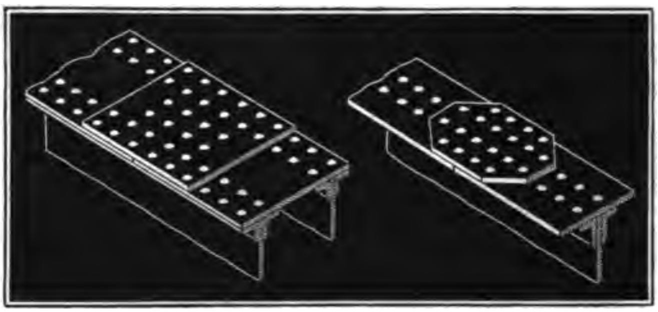
Fig. 103.



or narrow plate with double covers. There are eight different ways in which the joint may fail. 1°. By the bar tearing at *a*, where its area is reduced by only one rivet hole. 2°. By both covers tearing at *b*, where each is weakened by two rivet holes; this, however, is compensated for by their united section being somewhat greater than that of the bar. 3°. By the bar tearing at *b* at the same time that the rivet at *a* is double shorn. 4°. By the rivets on one side of the joint line double shearing. 5°. By the rivets on the alternate half-faces single shearing. 6°. By the rivets on one half-face single shearing while the opposite cover tears at *b*. 7°. By both covers tearing at *a* simultaneously with the rivets double shearing at *b*.

8°. By both covers tearing at *a* simultaneously with the bar tearing at *b*. If, for example, the plates are 10 inch × ½ inch, connected by two $\frac{5}{16}$th inch covers with 1 inch rivet holes, the effective section of the plates at *a* is 4·5 square inches; the double shearing area of the rivets at one side of the joint line equals 4·7 inches, and the area of both covers together at *b* is 5 inches. Finally, the effective section of the plate at *b* together with the double shearing area of the rivet at *a* equals 5·6 inches. This joint is therefore well proportioned, while the effective strength of the plates is really reduced by only one rivet hole, viz., that at *a*. A similar plan of joint is applicable to broad plates, Fig. 104.

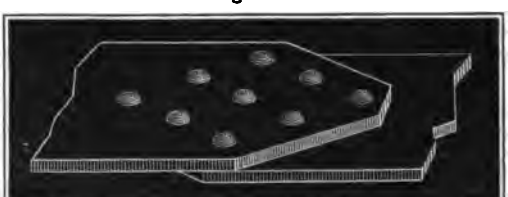
Fig. 104.



If applied to a pile of plates the extra thickness of the covers must be sufficient to compensate for the reduction in the strength of the whole pile by close transverse riveting.

When bars or plates are lap-jointed the arrangement proposed by Mr. Barton, and represented in Fig. 105, cannot be excelled.

Fig. 105.



470. Contraction of rivets and resulting friction of plates—Ultimate strength of rivet-joints not increased by friction.— Rivets contract in cooling and draw the plates together with such

* *Proc. Inst. C.E.*, Vol. xiv., p. 450.

force that the friction produced between their surfaces is sufficient to prevent them from slipping over each other so long as the strain lies within limits which are frequently not exceeded in practice. When this occurs the rivets are, of course, not subject to shearing strain. From experiments made during the construction of the Britannia Tubular Bridge it appears that the value of this friction is rather variable.* In one experiment with a $\frac{7}{8}$th inch rivet passing through three plates and therefore in double shear it amounted to 5·59 tons, in another with a $\frac{7}{8}$th inch rivet and two plates lap-jointed with $\frac{5}{16}$th inch washers next the rivet heads it reached 4·73 tons, while in a third experiment with three plates and $\frac{7}{8}$th inch rivet with $\frac{1}{2}$ inch washers next the rivet heads, making the shank of the rivet $2\frac{7}{8}$ inch long, the middle plate supported 7·94 tons before it slipped. In these experiments the hole in one or both plates was made oval and the sliding took place abruptly. Though the friction of riveted plates may be sufficient to convey the working-strain without subjecting the rivets to shearing, it does not follow, nor do experiments indicate, that the *ultimate* strength of a rivet joint is increased by this friction.

471. Dimensions of rivets—Injurious effect of punching holes—Boiler-makers' and Shipbuilders' rules—Chain riveting.—Joints may fail by each rivet splitting or shearing out the piece of plate in front of itself. Consequently the minimum theoretic distance of the rivets from the edge of an iron plate or from each other lengthways should be determined by the consideration that the shearing section of the plate (along two lines) between each rivet and the one behind it, or between each rivet in the first row and the edge of the plate, be not less than that of the rivet. If, for example, the rivets in Fig. 103 be $\frac{3}{4}$ inch and the plates $\frac{1}{2}$ inch thick, the shearing area of each rivet (in double shear) equals 1 square inch nearly,† and the distance of the edge of the rivet holes from the joint line should theoretically not be less than $\frac{1}{2}$ an inch. Practically, however, this is insufficient, for punching tends

* *Clark*, p. 393.

† Rivet holes are generally from $\frac{1}{32}$nd to $\frac{1}{16}$th inch larger than the nominal size of the rivet, in order to let the latter pass freely through when red hot. Hence the area of a $\frac{3}{4}$ inch rivet, after riveting, is nearly half a square inch.

to burst the edges of the holes if placed so close to each other or to the edge of the plate, especially if the plate be thick or of brittle quality, and in boilers the distance between the holes and the edge of the plate is usually about once the diameter of the rivet. If the distance exceed this it is difficult to make, the seam steam-tight by caulking. In girder-work, which does not require caulking like a boiler, this distance is seldom less than $1\frac{1}{2}$ times the diameter of the rivet, and the pitch may vary from $2\frac{1}{2}$ to 5 or even 7 inches. The rivet holes in first-class work are now frequently bored out with drilling machines, so as to avoid the weakening effect of punching on the plates. The great majority of girder-work, however, will probably always be done by the punch, as it would not pay to have the holes drilled unless in large girders where there are frequent repetitions of the same pattern (**425**).

The following table, taken from *Latham on Wrought-iron Bridges*, shows the usual practice in riveting boilers. It is nearly identical with Mr. Fairbairn's table.

TABLE I.—DIMENSIONS OF RIVETS FOR BOILERS.

Thickness of Plate.	Diameter of Rivet	Length of Rivet from Head.	Central Distance of Rivets (Pitch).	Lap in Single Joints.	Lap in Double Joints.	Equivalent Length of Head.
Inch.	Inch.	Inch.	Inch.	Inch.	Inch.	Inch.
$\frac{3}{16}$ = ·19	$\frac{3}{8}$ = ·38	$\frac{7}{8}$	$1\frac{1}{4}$	$1\frac{1}{4}$	$2\frac{1}{16}$	$\frac{1}{2}$
$\frac{1}{4}$ = ·25	$\frac{1}{2}$ = ·50	$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$2\frac{1}{2}$	$\frac{5}{8}$
$\frac{5}{16}$ = ·31	$\frac{5}{8}$ = ·63	$1\frac{3}{8}$	$1\frac{5}{8}$	$1\frac{7}{8}$	$3\frac{1}{8}$	$\frac{3}{4}$
$\frac{3}{8}$ = ·38	$\frac{3}{4}$ = ·75	$1\frac{5}{8}$	$1\frac{3}{4}$	2	$3\frac{5}{8}$	$\frac{7}{8}$
$\frac{1}{2}$ = ·50	$\frac{13}{16}$ = ·81	$2\frac{1}{4}$	2	$2\frac{1}{4}$	$3\frac{3}{4}$	$1\frac{1}{4}$
$\frac{9}{16}$ = ·56	$\frac{7}{8}$ = ·88	$2\frac{1}{2}$	$2\frac{1}{4}$	$2\frac{1}{2}$	$4\frac{1}{8}$	$1\frac{3}{8}$
$\frac{5}{8}$ = ·63	$\frac{15}{16}$ = ·94	$2\frac{3}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	$4\frac{5}{8}$	$1\frac{1}{2}$
$\frac{11}{16}$ = ·69	1 = 1·00	3	$2\frac{3}{4}$	3	5	$1\frac{5}{8}$
$\frac{3}{4}$ = ·75	$1\frac{1}{8}$ = 1·13	$3\frac{1}{4}$	3	$3\frac{1}{4}$	$5\frac{5}{8}$	$1\frac{3}{4}$

NOTE.—If the rivets have cup heads like those in Fig. 103, as is usual in girder-work, the equivalent length of head must be about one-half more than the amount given in the last column. The pitch in girder-work is generally one and a-half times or twice that in column 4.

The boiler-maker's rule, though not constant for all thickness of plate, is nearly as follows:—The diameter of the rivet = twice the thickness of the plate. The pitch = $2\frac{1}{2}$ to 3 diameters of the rivet. The lap for single joints = 3 diameters, and that for double joints = 5 diameters of the rivet.

Lloyd's rules for the dimensions of rivets in ship-building are as follows:—

TABLE II.—DIMENSIONS OF RIVETS FOR SHIPS.

Diameter of Rivets.	$\frac{5}{8}$ of an inch.			$\frac{3}{4}$ of an inch.			$\frac{7}{8}$ of an inch.			1 inch.			Rivets to be $\frac{1}{4}$ of an inch larger in diameter in the stem, stern-post and keel.
Thickness of Plates.	$\frac{5}{16}$	$\frac{6}{16}$	$\frac{7}{16}$	$\frac{8}{16}$	$\frac{9}{16}$	$\frac{10}{16}$	$\frac{11}{16}$	$\frac{12}{16}$	$\frac{13}{16}$	$\frac{14}{16}$	$\frac{15}{16}$	$\frac{16}{16}$	

"The rivets not to be nearer to the butts or edges of the plating, lining pieces to butts, or of any angle iron, than a space not less than their own diameter, and not to be farther apart from each other than four times their diameter, or nearer than three times their diameter, and to be spaced through the frames and outside plating, and in reversed angle iron, a distance equal to eight times their diameter apart. The overlaps of plating, where double riveting is required, not to be less than five and a-half times the diameter of the rivets; and where single riveting is admitted, to be not less in breadth than three and a quarter times the diameter of the rivets." It will be observed that in water joints the pitch may be one-third as great again as in steam joints.

The term "chain-riveting" has been applied to riveting in several transverse rows, the rivets being placed longitudinally one behind the other like the links of a chain. It merely means that both the longitudinal and transverse rows of rivets form straight lines.

479. Adhesion of iron and copper bolts—Strength of clenches and forelocks.—The shearing strength of oak treenails has been already given in (**398**). The two following tables are also the results of Mr. Parson's experiments.* "The first of these

* *Murray on Shipbuilding*, p. 94.

tables exhibits the adhesion of iron and copper bolts, driven into sound oak, with the usual drift, not clenched, and subject to a direct tensile strain. By drift is meant the allowance made to insure sufficient tightness in a fastening; it is therefore the quantity by which the diameter of a fastening exceeds the diameter of the hole bored for its reception."

TABLE III.—Table of the Adhesion of Iron and Copper Bolts driven into sound Oak with the usual Drift, not clenched, and subjected to a direct Tensile Strain.

Diameter of the Bolt.	Number of the Experiment.	Iron. Length of the Bolt driven into the Wood. Four Inches.		Iron. Six Inches.		Copper. Four Inches.		Copper. Six Inches.	
Inches.		Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.
¼	1	1	18	—		0	18½	—	
	2	2	0	—		0	18	—	
	3	2	2	—		0	19	—	
	4	1	13	—		0	18	—	
⅜	1	2	6	2	12	1	7	2	2
	2	2	4	2	11	1	8	2	2
	3	2	4	2	16	1	10	2	2
	4	2	0	2	10	1	13	2	0
½	1	3	2	3	12	2	10	2	15
	2	3	4	4	0	1	17	3	10
	3	3	0	4	0	2	2	3	1
	4	2	10	4	0	2	5	2	15
⅝	1	3	2	5	5	3	0	4	5
	2	3	0	4	8	3	6	3	18
	3	3	1	4	8	3	6	3	15
	4	3	1	5	0	2	9	3	5

TABLE III.—TABLE OF THE ADHESION OF IRON AND COPPER BOLTS DRIVEN INTO SOUND OAK WITH THE USUAL DRIFT, NOT CLENCHED, AND SUBJECTED TO A DIRECT TENSILE STRAIN.—*Continued.*

Diameter of the Bolt.	Number of the Experiment.	Iron.				Copper.			
		Length of the Bolt driven into the Wood.							
		Four Inches.		Six Inches.		Four Inches.		Six Inches.	
Inches.		Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.
$\frac{3}{4}$	1	3	3	6	0	3	10	5	5
	2	3	2	6	0	3	10	5	5
	3	3	10	5	0	3	10	5	8
	4	3	10	6	0	3	18	4	18
$\frac{7}{8}$	1	4	10	6	2	4	0	4	13
	2	5	12	5	10	4	0	4	13
	3	3	10	6	11	4	5	4	19
	4	4	10	6	4	4	2	4	19
1	1	5	0	7	2	4	2	5	19
	2	4	7	8	1	4	8	5	0
	3	4	11	6	5	3	15	6	5
	4	4	0	7	0	4	10	5	0

"In Riga fir the adhesion was, on an average, about one-third of that in oak, and in good sound Canada elm it was about three-fourths of that in oak.

"The following table exhibits the strength of clenches and of forelocks as securities to iron and copper bolts, driven six inches, without drift, into sound oak, either clenched or forelocked on rings, and subjected to a direct tensile strain. It gives the diameter of the bolt on which the experiment was made, as well as the number of the experiment:—

TABLE IV.—TABLE OF THE STRENGTH OF CLENCHES AND OF FORELOCKS, AS SECURITIES TO IRON AND COPPER BOLTS, DRIVEN SIX INCHES, WITHOUT DRIFT, INTO SOUND OAK, EITHER CLENCHED OR FORELOCKED ON RINGS, AND SUBJECTED TO A DIRECT TENSILE STRAIN.

Diameter of the Bolt.	Number of the Experiment.	Iron.				Copper.			
		Clench.		Forelock.		Clench.		Forelock.	
Inch.		Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.
¼	1	1	16	0	16	1	0	0	8
	2	1	13	0	14	0	19	0	8
	3	1	9	0	20	1	0	0	7
	4	1	9	0	18	1	0	0	6
⅜	1	3	0	1	15	2	10	1	4
	2	3	0	1	8	2	10	1	0
	3	2	16	1	9	2	5	1	2
	4	2	15	1	14	2	9	1	4
½	1	4	15	2	11	3	10	1	18
	2	4	10	2	15	3	15	1	18
	3	4	5	2	10	4	0	2	4
	4	4	12	2	12	4	10	1	16
⅝	1	5	18	3	15	6	0	2	13
	2	6	8	3	6	5	15	2	10
	3	6	8	3	0	6	5	2	16
	4	6	0	3	7	5	10	2	10
¾	1	7	10	3	10	7	0	—	
	2	7	10	3	15	7	0	—	
	3	8	0	3	10	7	5	—	
	4	8	15	3	15	7	8	—	
⅞	1	11	11	5	1	7	16	—	
	2	11	15	5	10	7	16	—	
	3	8	11	4	6	7	12	—	
	4	8	6	4	15	7	5	—	

TABLE IV.—TABLE OF THE STRENGTH OF CLENCHES AND OF FORELOCKS, AS SECURITIES TO IRON AND COPPER BOLTS, DRIVEN SIX INCHES, WITHOUT DRIFT, INTO SOUND OAK, EITHER CLENCHED OR FORELOCKED ON RINGS, AND SUBJECTED TO A DIRECT TENSILE STRAIN.—*Continued.*

Diameter of the Bolt.	Number of the Experiment.	Iron.				Copper.			
		Clench.		Forelock.		Clench.		Forelock.	
Inch.		Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.	Tons.	Cwts.
1	1	12	0	5	18	7	1	—	
	2	12	3	6	18	7	1	—	
	3	11	3	5	12	7	14	—	
	4	11	1	5	2	8	14	—	

"In the experiments on the clenches, the clenches always gave way; but with the forelocks it as frequently occurred that the forelock was cut off as that the bolt broke; and in the cases of the bolt breaking, it was invariably across the forelock hole. According to the tables, the security of a forelock is about half that of a clench.

"It appears an anomaly that the strength of a clench on copper should be equal to that of one on iron. But, in consequence of the greater ductility of copper, a better clench is formed on it than on iron. Generally the thickness of the fractured clench in the copper was double that in the iron. With rings of the usual width for the clenches, the wood will break away under the ring, and the ring be imbedded for two or more inches before the clench will give way.

"With the inch copper-bolts, all the rings under the clenches turned up into the shape of the frustum of a cone, and allowed the clench to slip through at the weights specified.

"Experiments with ring-bolts were made to ascertain the strength of the rings in comparison with the clenches. The rings were of the usual size, viz.: the iron of the ring one-eighth inch less in diameter than that of the bolt. It was found that the rings always carried away the clenches, but that they were drawn into the form of a link with perfectly straight sides. The rings bore, before any change of form took place, not quite one-half the weight which tore off the clenches. It appears that the rings are well proportioned to the strength of the clenches."

473. Adhesion of nails and wood-screws.—"The following abstract of Mr. Bevan's experiments exhibits the relative adhesion of nails of various kinds, when forced into dry Christiana deal, at right angles to the grain of the wood." *

TABLE V.—ADHESION OF NAILS OF VARIOUS KINDS IN DRY CHRISTIANA DEAL.

Kind of nails.	Number to the pound avoirdupois.	Inches long.	Inches forced into the wood.	Pounds required to extract.
Fine sprigs,	4,560	0·44	0·0	22
Ditto,	3,200	0·53	0·44	37
Threepenny brads,	618	1·25	0·50	58
Cast-iron nails,	380	1·00	0·50	72
Sixpenny nails,	73	2·50	1·00	187
Ditto,	—	—	1·50	327
Ditto,	—	—	2·00	530
Fivepenny,	139	2·00	1·50	320

"The force required to draw the same sized nail from different woods averaged as under:—

TABLE VI.—RELATIVE ADHESION OF SAME NAIL IN DIFFERENT WOODS.

Kind of wood.	Weight in lbs. required to draw a sixpenny nail, driven in one inch.
Dry Christiana deal,	187 lbs.
Dry oak,	507 „
Dry elm,	327 „
Dry beech,	667 „
Green sycamore,	312 „
Dry Christiana deal, driven in endways,	87 „
Dry elm, driven in endways,	257 „

* *Tredgold's Carpentry*, p. 189.

"It was further desirable to ascertain the degree of dependence that might be placed on nailing two pieces together, and Mr. Bevan kindly undertook to make some trials. Two pieces of Christiana deal, seven-eighths of an inch thick, were nailed together with two sixpenny nails; and a longitudinal force in the plane of the joint, and consequently at right angles to the direction of the nails, was applied to cause the joint to slide; it required a force of 712 lbs., and the time was 15 minutes; the nails curved a little and were then drawn.

"Another experiment was made in the same manner with dry oak, an inch thick, in which the force required was 1,009 lbs.: the sixpenny nails curved, and were drawn by that force.

"Dry sound ash, an inch thick, joined in the same manner by two sixpenny nails, bore 1,220 lbs. 30 minutes without sensibly yielding; but when the stress was increased to 1,420 lbs. the pieces separated with an easy and gradual slide; curving and drawing the nails as before, one of which broke.

"The following experiments on the force necessary to draw screws of iron, commonly called wood screws, out of given depths of wood, were made by Mr. Bevan. The screws he used were about two inches in length, $\frac{22}{100}$ diameter at the exterior of the threads, $\frac{15}{100}$ diameter at the bottom, the depth of the worm or thread being $\frac{35}{1000}$, and the number of threads in one inch = 12. They were passed through pieces of wood, exactly half an inch in thickness, and drawn out by the weights stated in the following tables:—

TABLE VII.—RELATIVE ADHESION OF SCREWS IN DIFFERENT WOODS.

Kind of Wood.	Weight required to draw out screws passed through half-inch boards.
Dry beech,	460 lbs.
Ditto ditto,	790 „
Dry sound ash,	790 „
Dry oak,	760 „
Dry mahogany,	770 „
Dry elm,	665 „
Dry sycamore,	830 „

"The weights were supported about two minutes before the screws were extracted. He found the force required to draw similar screws out of deal and the softer woods about half the above.

"The force necessary to cause pieces screwed together to slide at the joining, was also determined; the pieces being joined by two screws; the resultant of the force coinciding with the plane of the joint, and in line with the places of the screws.

"With Christiana deal, seven-eighths of an inch thick, joined by two screws one and five-eighths of an inch in length, and five-fortieths of an inch in diameter within the worm, a load of 1,009 lbs. gradually applied broke both the screws at the line of joint, after elongating the interior of the hole and sliding about six-tenths.

"With very dry seasoned oak, 1 inch thick, two screws one and five-eighths long, and six-fortieths diameter within the thread, bore 1,009 lbs. for ten minutes without any signs of yielding: with 1,137 lbs. both screws broke in two places; each screw about two-tenths of an inch within each piece of wood; the holes were a little elongated.

"With dry and sound ash, 1 inch thick, with screws 2¼ inches long, passing one quarter of an inch through one of the pieces, the diameter at bottom of the worm seven-fortieths; the load began with was 1,224 lbs.; gradually increased for two hours to 2,661 lbs.; they produced a slow and moderate sliding, not separation, the screws being neither drawn nor broken; but probably would, if not removed on account of night coming on, and putting an end to the experiment."

CHAPTER XXVI.

WORKING STRAIN AND WORKING LOAD.

474. Working strain defined—Fatigue—English rule for working strain—Factor of safety.—The *working strain* is the strain to which any material is subject in actual practice, but the term, unless accurately defined, is somewhat ambiguous, as it is applied to strains which the material sustains on rare occasions from extraordinary loads, as well as to those to which it is liable in ordinary every-day use. For instance, a railway girder may sustain a constant strain of $3\frac{1}{2}$ tons per square inch from the permanent bridge-load, which rises to $4\frac{1}{2}$ tons when an ordinary train passes, but reaches a maximum of 5 tons with a train of the greatest possible density, such as locomotives; or again, the chains of a suspension bridge may sustain only $2\frac{1}{2}$ tons per square inch from the permanent weight of the structure, while a dense crowd of people may raise this to 6 tons per square inch. In such cases we have three classes of strains. 1°. The permanent strain due to the weight of the structure itself, and from which the material suffers what has been termed *fatigue*. 2°. The ordinary working strain due to ordinary loads. 3°. The maximum working strain due to the greatest load possible in practice, and it is this latter which defines the strength of any structure, and which therefore we have to consider in this chapter. As might have been anticipated, different opinions are held respecting the safe unit-strain for each kind of material. English practice generally makes the working strain some submultiple of the tearing or crushing strength of the material, while General Morin recommends the working strain to be such that the resulting alteration of length shall not exceed one-half that which corresponds to the limit of elasticity.* Neither rule should be adopted to the exclusion of the other, but as we know the limit of elasticity of but few materials, and as those which are not ductile seem to have no very definite limit at all (**400, 418**), the English rule seems more generally applicable than that of General Morin and it has the sanction of extensive experience in its favour. The term *factor of safety* has been applied to the ratio of the breaking to the working strain. If, for instance, the tearing inch-strain of plate-iron is 20 tons and the working inch-strain 5 tons, the factor of safety will be 4.

CAST-IRON.

475. Effects of long-continued pressure on cast-iron pillars and bars—Fairbairn's experiments.—To determine the effect of long-continued pressure upon cast-iron, Mr. Fairbairn had four pillars cast of Low-Moor iron; the length of each was 6 feet, and the diameter 1 inch, and they were rounded at the ends. The first was loaded with 4 cwt., the second with 7 cwt., the third with 10 cwt., and the fourth with 13 cwt. These weights are respectively 30, 52, 75, and 97 per cent. of the weight which had previously broken another pillar of the same dimensions when the weight was carefully laid on without loss of time. The pillar loaded with 13 cwt. bore the weight between five and six months, and then broke; that loaded with 10 cwt. was increasing slightly in flexure at the end of three years; when first taken its deflexion was ·230 inch, and after each succeeding year it was ·380, ·380, and ·409. The other pillars, though a little bent, did not alter. In these experiments we see that a cast-iron pillar bore a steady load of one-half its breaking weight for three years without alteration, while the deflection of another pillar with three-fourths of its breaking weight was increasing slightly at the end of the same period.†

To ascertain how far cast-iron bars might be trusted with permanent loads, Mr. Fairbairn made the following experiments

* *Résistance des Materiaux*, p. 45.

† *Experimental Researches* by E. Hodgkinson, p. 351.

also:—"He took bars, both of cold and hot blast iron (Coed Talon, No. 2), each 5 feet long, and cast from a model 1 inch square; and having loaded them in the middle with different weights, with their ends supported on props 4 feet 6 inches asunder, they were left in this position to determine how long they would sustain the loads without breaking. They bore the weights, with one exception, upwards of five years, with small increase of deflexion, though some of them were loaded nearly to the breaking point. Since that time, however, less care was taken to protect them from accident, and three others were found broken. They were examined, and had their deflexions taken occasionally, which are set down in the following Table, which contains the exact dimensions of the bars, with the load upon each." *

* *Experimental Researches*, p. 374.

TABLE I.—EXPERIMENTS ON THE STRENGTH OF CAST-IRON BARS TO RESIST LONG-CONTINUED PRESSURE.

Date of observation.	Temperature of the air at time of observation.	Experiment 1.—Cold blast iron. Depth of bar 1·050 Breadth of bar 1·030	Experiment 2.—Hot blast iron. Depth of bar 1·050 Breadth of bar 1·010	Experiment 3.—Cold blast iron. Depth of bar 1·030 Breadth of bar 1·020	Experiment 4.—Hot blast iron. Depth of bar 1·040 Breadth of bar 1·020	Experiment 5.—Cold blast iron. Depth of bar 1·039 Breadth of bar 1·020	Experiment 6. Hot blast iron. Depth of bar 1·050 Breadth of bar 1·000	Experiment 7.—Cold blast iron. Depth of bar 1·000 Breadth of bar 1·010	Experiment 8.—Cold blast iron. Depth of bar 1·020 Breadth of bar 1·030	Experiment 9.—Hot blast iron. Depth of bar 1·040 Breadth of bar 1·010
	Fah.	Deflections with a permanent load of 280 lbs. laid upon each.		Deflections with a permanent load of 336 lbs. laid upon each.		Deflections with a permanent load of 392 lbs. laid upon each.		Deflections with a permanent load of 448 lbs. laid upon each.		
1837.										
Mar. 6		—	—	1·267	—	1·684	1·715	1·964	1·410	This broke with 392 lbs.; other hot blast bars were tried, but they were successively broken with 448 lbs.
„ 9	49°	·916	1·048	1·270	1·454	1·694	1·758	2·005	1·413	
„ 11		·930	1·064	1·270	1·461	1·694	1·760	2·005	1·413	
„ 17		—	—	—	—	—	—	2·010	1·418	
April 15	47°	·930	1·078	1·271	1·475	1·716	1·767	2·014	1·422	
May 31	62°	·932	1·082	1·274	1·481	1·725	1·775	Broke after bearing the weight 37 days	1·424	
Aug. 22	70°	·937	1·086	1·288	1·504	1·737	1·783		1·438	
Nov. 18	45°	·942	1·083	1·286	1·499	1·724	1·773		1·431	
1838.										
Jan. 8	38°	·941	1·086	1·288	1·502	1·722	1·773		1·430	
Mar. 12	51°	·945	1·091	1·298	1·505	1·801	1·784		1·439	
June 23	78°	·963	1·107	1·316	1·538	1·824	1·803		1.457	
1839.										
Feb. 7	54°	·950	1·093	1·293	1·524	1·815	1·784		1·433	
July 5	72°	·959	1·104	1·305	1·533	1·824	1·798		1·446	
Nov. 7	50°	·955	1·102	1·303	1·531	1·824	1·796		1·445	
Dec. 9	39°	·956	1·102	1·308	1·531	1·823	1·796		1·445	
1840.										
Feb. 14	50°	·955	1·104	1·305	1·531	1·824	1·797		1·446	
April 27	63°	·954	1·116	1·309	1·519	1·818	1·802		1·445	
June 6	61°	·951	1·112	1·303	1·520	1·825	1·798		1·445	
Aug. 3	74°	·953	1·115	1·305	1·523	1·826	1·801		1·447	
Sept. 14	55°	*1·047	1·115	1·305	*1·613	1·826	1·802		1·447	
1841.										
Nov. 22	50°	1·045	1·115	1·306	1·620	1·829	1·804		1·449	
1842.										
April 19	58°	—	—	1·308	1·620	1·828	1·812		1·449	

* After August 3, 1840, a body seems to have fallen upon the bars of the 1st and 4th Experiment, and this may have increased their deflections.

On these experiments Mr. Hodgkinson made the following observations:—"Looking at the results of these experiments, and the note upon the first and fourth, it appears that the deflection in each of the beams increased considerably for the first twelve or fifteen months; after which time there has been, usually, a smaller increase in their deflections, though from four to five years have elapsed. The beam, in Experiment 8, which was loaded nearest to its breaking weight, and which would have been broken by a few additional pounds laid on at first, had not, perhaps, up to the time of its fracture, a greater deflection than it had three or four years before; and the change in deflection in Experiment 1, where the load is less than ⅔rds of the breaking weight, seems to have been almost as great as in any other; rendering it not improbable that the deflexion will, in each beam, go on increasing till it becomes a certain quantity, beyond which, as in that of Experiment 8, it will increase no longer, but remain stationary. The unfortunate fracture of this last beam, probably through accident, has left this conclusion in doubt."* Mr. Hodgkinson concluded from these experiments that cast-iron girders might be safely trusted with one-third of their breaking weight. This conclusion, however, he seems to have subsequently modified, when a member of the Iron Commission in 1849, which reported in favour of not less than one-sixth (477).

476. Effects of long-continued impact and frequent deflections on cast-iron bars—Iron Commissioners' report.— The Commissioners appointed to inquire into the application of iron to railway structures, reported as follows on the effects of long-continued impacts and frequent deflections of cast-iron bars:—"A bar of cast-iron, 3 inches square, was placed on supports about 14 feet asunder. A heavy ball was suspended by a wire 18 feet long, from the roof, so as to touch the centre of the side of the bar. By drawing this ball out of the vertical position at right angles to the length of the bar, in the manner of a pendulum, to any required distance, and suddenly releasing it, it could be made to strike a horizontal blow upon the bar, the magnitude of which could be adjusted at pleasure either by varying the size of the ball or the distance from which it was released. Various bars (some of smaller size than the above) were subjected by means of this apparatus to successions of blows, numbering in most cases as many as 4,000. The magnitude of the blow in each set of experiments being made greater or smaller, as

* *Experimental Researches*, p. 376.

occasion required. The general result obtained was, that when the blow was powerful enough to bend the bars through one-half of their ultimate deflection (that is to say, the deflection which corresponds to their fracture by dead pressure), no bar was able to stand 4,000 of such blows in succession; but all the bars (when sound) resisted the effects of 4,000 blows, each bending them through one-third of their ultimate deflection.

"Other cast-iron bars, of similar dimensions, were subjected to the action of a revolving cam, driven by a steam-engine. By this they were quietly depressed in the centre, and allowed to restore themselves, the process being continued to the extent, even in some cases, of an hundred thousand successive periodic depressions for each bar, and at a rate of about four per minute. Another contrivance was tried by which the whole bar was also, during the depression, thrown into a violent tremor. The results of these experiments were, that when the depression was equal to one-third of the ultimate deflection, the bars were not weakened. This was ascertained by breaking them in the usual manner with stationary loads in the centre. When, however, the depressions produced by the machine were made equal to one-half of the ultimate deflection, the bars were actually broken by less than nine hundred depressions. This result corresponds with and confirms the former.

"By other machinery a weight equal to half of the breaking weight was slowly and continually dragged backwards and forwards from one end to the other of a bar of similar dimensions to the above. A sound bar was not apparently weakened by ninety-six thousand transits of the weight.

"It may, on the whole, therefore, be said, that as far as the effects of reiterated flexure are concerned, cast-iron beams should be so proportioned as scarcely to suffer a deflection of one-third of their ultimate deflection. And as it will presently appear, that the deflection produced by a given load, if laid on the beam at rest, is liable to be considerably increased by the effect of percussion, as well as by motion imparted to the load, it follows, that to allow the greatest load to be one-sixth of the breaking weight, is hardly a sufficient limit for safety even upon the supposition that the beam is perfectly sound.

"In wrought-iron bars no very perceptible effect was produced by 10,000 successive deflections by means of a revolving cam, each deflection being due to half the weight which, when applied statically, produced a large permanent flexure.

"Under the second head, namely, the inquiry into the mechanical effects of percussions and moving weights, a great number of experiments have been made to illustrate the impact of heavy bodies on beams. From these, it appears, that bars of cast-iron of the same length and weight struck horizontally by the same ball (by means of the apparatus above described for long-continued impact), offer the same resistance to impact, whatever be the form of their transverse section, provided the sectional area be the same. Thus a bar, $6 \times 1\frac{1}{2}$ inches in section, placed on supports about 14 feet asunder, required the same magnitude of blow to break it in the middle, whether it was struck on the broad side or the narrow one, and similar blows were required to break a bar of the same length, the section of which was a square of three inches, and, therefore of the same sectional area and weight as the first.

"Another course of experiments tried with the same apparatus showed, amongst other results, that the deflections of wrought-iron bars produced by the striking ball were nearly as the velocity of impact. The deflections in cast-iron are greater than in proportion to the velocity.

"A set of experiments was undertaken to obtain the effects of additional loads spread uniformly over a beam, in increasing its power of bearing impacts from the same ball falling perpendicularly upon it. It was found that beams of cast-iron, loaded to a certain degree with weights spread over their whole length, and so attached to them as not to prevent the flexure of the bar, resisted greater impacts from the same body falling on them than when the beams were unloaded, in the ratio of two to one. The bars in this case were struck in the middle by the same ball, falling vertically through different heights, and the deflections were nearly as the velocity of impact."*

* *Rep. of Com.*, p. x.

477. Working strain of cast-iron girders subject to vibration should not exceed one-sixth of their breaking strain—If free from vibration, it should not exceed one-fourth of their breaking strain—If subject to sudden severe shocks it should not exceed one-eighth of their breaking strain—Rule of Board of Trade for cast-iron Railway Bridges—One and a quarter ton per square inch safe tensile working strain for good cast-iron—Six tons per square inch safe compressive working strain for sections which cannot deflect.—The reader will observe that the Commissioners considered one-sixth of the breaking strain hardly a sufficient limit of safety for cast-iron girders when liable to percussion and deflection from moving loads. This inference was, no doubt, influenced by their experiments on bars which were much lighter in proportion to their trial loads than ordinary bridge girders are compared with the loads which in practice traverse them. As a general rule, *one-sixth* of the breaking strain may be taken as the safe working strain for cast-iron girders which are liable to vibration, as in railway or public bridges, but when the load is stationary and free from all vibration, such as water tanks, *one-fourth* of the breaking strain is safe. When, however, cast-iron girders are liable to sudden severe shocks, as in crane posts or machinery, their working strain load should not exceed *one-eighth* of their breaking strain. The railway department of the Board of Trade has laid down the following rule for the guidance of engineers in the construction of railways:—"In a cast-iron bridge the breaking weight of the girders should be not less than three times the permanent load due to the weight of the superstructure, added to six times the greatest moving load that can be brought upon it." Notwithstanding this rule, engineers will do well not to design cast-iron girders for railway bridges of less strength than six times the total load, that is, six times the permanent load added to six times the greatest moving load. The reader who desires detailed information respecting the practice of our most eminent engineers during the reign of cast-iron is referred to the evidence laid before the Commissioners in 1849.

It seems certain that the transverse strength of thick rectangular cast-iron bars is less than that of thin ones (**212**), but it does not necessarily follow that the strength of large flanged girders is diminished by the massiveness of the casting, or that they are relatively weaker than smaller girders of similar section, for the quality of the iron will, no doubt, materially affect their strength (**346**). Experiments on a large scale can only decide these questions, which, however, have less importance now than when the Iron Commission sat in 1849, as it is unlikely that large cast-iron girders will be often employed in important works when wrought-iron is available.

Cast-iron can be got, on specification, to stand 7½ tons tension per square inch; consequently the rule of one-sixth allows 1¼ tons per inch for the ordinary limit of safe tensile strain for good cast-iron in tension flanges, but this material is quite unfitted for tie-bars for the reasons referred to in **347** and **348**. In compression cast-iron will safely bear 6 or 7 tons per square inch, provided it be in a form suited to resist flexure; but the effects of flexure will seriously diminish the safe unit-strain for pillars, as already explained in the Chapter XIII. (**212**). The same remark applies to untrussed cast-iron arches in which the line of pressure may vary so as to alter the strain materially, perhaps as much as 50 or even 100 per cent.

WROUGHT-IRON.

478. Effects of repeated deflections on wrought-iron bars and plate girders—Portsmouth experiments on the deflection of bars—Fairbairn's experiment on a plate girder.— Sir Henry James and Captain Galton made some experiments in Portsmouth Dockyard for determining the effects produced by repeated deflections on wrought-iron bars.* These experiments were made with cams caused to revolve by steam machinery, which depressed the bars and allowed them to resume their natural position for a great number of times. Two cams were used; one was toothed on the edge so as to communicate a highly vibratory motion to the bar during the deflection; the other, a step cam, gently depressed the bar and released it suddenly when the full deflection had been obtained. The rate of the depressions was

* *Rep. of Com., App. B.*, p. 259.

from four to seven per minute. The following table gives the principal results:—

TABLE II.—EXPERIMENTS ON REPEATED DEFLECTIONS OF WROUGHT-IRON BARS 2 INCHES SQUARE AND 9 FEET LONG BETWEEN POINTS OF SUPPORT.

No. of Experiment.	Amount of Deflection.	Number of Depressions.	Permanent Set.	Remarks.
	Inches.		Inches.	
1	·833	100,000	0·015	Rough cam.
2	·83	10,000	0·	Step cam.
3	1·00	10,000	0·06	Do.
4	2·00	10	0·30	Do.
		50	0·54	Do.
		100	0·69	Do.
		150	0·84	Do.
		200	0·98	Do.
		300	1·34	Do.

For the purpose of comparison the following experiments were made to determine the deflections due to statical loads at the centre of a similar bar.

TABLE III.—EXPERIMENTS ON A WROUGHT-IRON BAR, 2 INCHES SQUARE AND 9 FEET LONG BETWEEN POINTS OF SUPPORT, SHOWING THE STATICAL WEIGHTS DUE TO GIVEN DEFLECTIONS, THE WEIGHTS BEING APPLIED AND THE DEFLECTIONS MEASURED AT THE CENTRE.

Deflections in inches.	Weights in lbs.	Permanent Set.	Remarks.
·333	507	0	After the bar had 1,950 lbs. on, it suddenly gave way, and although it did not break, no further weight could be applied with certainty.
·666	926	0	
·833	1,121	0	
1·00	1,364	0·054	
1·30	1,950	0·36	

In these experiments two things are worthy of note; first, the largest deflection which did not produce a permanent set appears to be that due to rather more than one-half the statical weight which crippled the bar: secondly, 10,000 depressions with the step cam, causing a deflection of 1 inch, produced almost exactly the same permanent set as the statical weight due to the same deflection of 1 inch. With the view of arriving "at the extent to which a bridge or girder of wrought-iron may be strained without injury to its ultimate powers of resistance, and to imitate as nearly as possible the strain to which bridges are subjected by the passage of heavy railway trains," Mr. Fairbairn caused a weighted lever to be lifted off and replaced alternately, by means of a water-wheel, upon the centre of a wrought-iron single-webbed plate girder of the usual construction, with double angle-irons and a flange-plate riveted on top and bottom respectively. The dimensions of the girder were as follows:—*

Extreme length, ...	22 feet.
Length between supports, . .	20 feet.
Extreme depth, ...	16 inches.
Weight of girder, ...	7 cwt. 3 qrs.

	Inches.	
Area of top flange, 1 plate 4 inches × ½ inch, .	2·00	
,, ,, 2 angle irons 2 × 2 × $\frac{5}{16}$, .	2·30	
		4·30
Area of bottom flange, 1 plate 4 inches × ¼ inch,	1·00	
,, ,, 2 angle-irons 2 × 2 × $\frac{5}{16}$,	1·40	
		2·40
Web, 1 plate 15¼ × ⅛ inch, ...		1·90
Total sectional area, ...		8·60

The area of the ½ inch rivet holes in the bottom flange, two in each angle-iron and two in the plate, is equal to ·625 square inches, which reduces the effective flange area for tension from 2·4 to 1·775 square inches. The web being continuous gave some aid to the flanges, but as it was composed of 9 short plates with vertical joints and single riveted covering strips, the amount of aid given to the tension flange probably did not exceed one-half the theoretic aid of a perfectly continuous web (**99**), that is, it equalled one-twelfth of the gross area of the web, or 0·158 square

* *Useful Information for Engineers*; third series, p. 301.

inches; adding this to the net area of the bottom flange, we have a total of $1{\cdot}775 \times 0{\cdot}158 = 1{\cdot}933$ square inches available for tension, and assuming the tearing strength of the iron to have been 20 tons per square inch, and the depth for calculation to be taken from inside to inside of the angle-iron flanges, which measures 14¾ inches, we have the breaking weight in the centre, from eq. 19, as follows,

$$W = \frac{4Fd}{l} = \frac{4 \times (20 \times 1{\cdot}933) \times 14{\cdot}75}{240} = 9{\cdot}5 \text{ tons.}$$

The compression flange, it will be observed, was much stronger than that in tension, and hence it may be supposed that a larger fraction than one-twelfth of the web should be added to the lower flange (**429**). The extra strength on this account must, however, have been very small and could scarcely raise the breaking weight beyond 10 tons. Mr. Fairbairn, however, calculates the breaking weight at 12·8 tons by an empirical formula derived from the model tube at Millwall. The following table contains a summary of the experiments with the corresponding tensile strains, calculated on the supposition that 10 tons was the true statical breaking weight.

TABLE IV.—EXPERIMENTS ON REPEATED DEFLECTIONS OF A SINGLE-WEBBED PLATE-IRON GIRDER, 16 INCHES DEEP, AND 20 FEET LONG BETWEEN POINTS OF SUPPORT.

No. of Experiment.	Weight on Middle of Girder.	No. of Changes.	Deflection.	Strain per square inch of net section on Bottom Flange.	Remarks.
	Tons.		Inches.	Tons.	
1	2·96	596,790	0·17	5·92	Above half a million changes, working continuously for two months, night and day, at the rate of about eight changes per minute, produced no visible alteration.
2	3·50	403,210	0·23	7·00	One million changes and no apparent injury.
3	4·68	5,175	0·35	9·36	Permanent set of ·05 inches; broke by the tension flange tearing across a short distance from the middle. None of the rivets loosened or broken.

Girder repaired by replacing the broken angle-irons on each side, and putting a patch over the broken plate equal in area to the broken plate itself.

No. of Experiment.	Weight on Middle of Girder.	No. of Changes.	Deflection.	Strain per square inch of net section on Bottom Flange.	Remarks.
	Tons.		Inches.	Tons.	
4	4·68	158	—	9·36	Apparatus accidentally set in motion; took a large but unmeasured set.
5	3·58	25,742	0·22	7·16	———
6	2·96	3,124,100	0·18	5·92	No increase of deflection or permanent set.
7	4·00	313,000	0·20	8·00	Broke by failure of the tension flange as before, close to the plate riveted over the previous fracture. Total number of changes after repair = 3,463,000.

These experiments seem to indicate that a constantly repeated tensile strain of 6 tons per square inch, with vibration, will not injure wrought-iron, but, as the actual breaking weight of the girder was not determined after each experiment, we cannot be quite certain whether the strength was really impaired or not by the lesser strains. To carry out the experiment scientifically would have required several girders to be broken by dead weight—one when new, as a standard for comparison; and each of the others after a few million changes of the same amount in any one girder, but of different amounts in successive girders.

479. Rule of Board of Trade for wrought-iron railway bridges—Working strain of wrought-iron girders subject to vibration should not exceed one-fourth of their breaking strain—If free from vibration it should not exceed one-third of their breaking strain—If subject to sudden severe shocks it should not exceed one-sixth of their breaking strain—French rule for wrought-iron railway bridges.—The following rule has been laid down by the Board of Trade for the strength of railway bridges. "In a wrought-iron bridge the greatest load which can be brought upon it, added to the weight of the superstructure, should not produce a greater strain on any part of the material than five tons per square inch." This rule is generally confined to parts in tension while the usual limit of strain in the compression flanges is 4 tons per square inch, and as the tearing and crushing strengths of ordinary plate iron are respectively 20 and 16 tons per square inch the foregoing rules are equivalent to stating that one-fourth of the breaking strain is the maximum safe working strain for wrought-iron girders which are subject to vibration, and this is now the recognized English practice. When supporting a dead load, like water tanks, it is likely that wrought-iron girders will bear safely one-third of their breaking strain, but when liable to sudden severe shocks, as in gantries or cranes, it will be prudent to limit the working strain to one-sixth of the breaking strain. The French rule for wrought-iron railway bridges is that in no part shall the strain exceed 6 kilogrammes per square millimètre, *i.e.*, 3·81 tons per square inch.

480. Five tons per square inch safe tensile working strain for ordinary plate iron—Net area only effective for tension—Punching reduces tensile strength of iron, especially if it be brittle—Five tons per square inch safe tensile working strain for ordinary bar and angle-iron—Six tons per square inch for bar iron of extra quality like the links of suspension chains.—The reader will recollect that the whole area of a riveted plate is not available for tension, but only the unpierced portion which lies between the rivet holes in any line of section; this is generally called the *net* area of the plate, and on this net area alone the working tensile strain of 5 tons per square inch should be calculated. The *effective* tensile area of a punched plate is, indeed, somewhat less than its net area, for the tearing strength of iron is generally injured by punching, especially if there be too great a clearance between the die and the punch or if the iron be brittle, and, though it is not the practice, it would be more correct to diminish the gross section by the sum of the rivet holes multiplied by a factor greater than unity, perhaps 1·1 or 1·2. It may, perhaps, be supposed more accurate to add a constant quantity, say $\frac{1}{8}$th inch, to the diameter of each hole in place of adding a percentage, but it is probable that the weakening effect of punching is greater the thicker the plate, and as thick plates have generally larger rivet holes than thin plates the percentage allowance will be more accurate in practice. Good experiments on this subject are much wanted. Meantime the weakening effect of punching affords an argument in favour of drilling holes, especially in hard and brittle materials (425). Punching will probably do little injury to soft and ductile iron or to annealed steel.

The safe tensile working strain for ordinary bar, angle, or T iron is generally taken the same as for plates, namely, 5 tons per square inch of net section, but bar iron of extra quality, such as the links of suspension bridges, will safely bear 6 tons per square inch for the maximum working strain. Special care is taken with the manufacture of this class of iron, and it is customary to prove each link individually to a strain of 9 tons per square inch before it is admitted into the suspension chain, the tearing strength of the iron being not less than 24 tons per square inch.

481. Gross area available for compression—Four tons per square inch safe compressive working strain for wrought-iron in large sections like flanges—Three tons per square inch for small sections like lattice-bars—Flanges of wrought-iron girders generally of equal area.—The total sectional area of a riveted plate is available for compression (flexure being duly provided against), since the thrust is transmitted through the rivet just as if it were a portion of the solid plate, for if the rivet head be properly hammered up its shank will upset and fill the hole completely. Even supposing that the rivet do not perfectly fill the hole, an exceedingly small motion of the parts, which must take place before crushing commences, will cause the strain to pass through the shank. In practice, however, the longitudinal contraction of each rivet in cooling will generally produce an amount of friction between the surfaces riveted together sufficient to resist any movement so long as the strain lies within the usual working limits (470). The crushing strength of wrought-iron is generally taken at 16 tons per square inch, and the safe limit of compressive working strain is, according to ordinary English practice, 4 tons per square inch over the gross area, provided the section is so large that it can without extra material be put into a form suitable for resisting flexure or buckling. This is generally the case with the compression flanges of girders. When, however, a thin sheet like the web of a plate girder sustains compression, or when the section of a strut is small, as in the compression bars of braced girders, it is necessary to add additional material to prevent flexure or buckling. Angle, T, or channel iron are suitable for plate stiffeners or for short struts; for long struts the plan of internal cross bracing, represented in Fig. 89 (**337**), may be advantageously adopted, the internal cross bracing, of course, not being measured as effective area to resist crushing, since it merely keeps the sides in line, but sustains none of the longitudinal thrust, and in small scantlings it will be prudent to limit the maximum compressive working strain to 3 tons per square inch.

The flanges of wrought-iron girders are generally of equal or nearly equal area, for the loss of rivet area in the tension flange is compensated by the higher unit-strain in the unpunched part between the rivet holes which is effective for tensile strain.

482. Working load of boilers should not exceed one-eighth of their bursting pressure—French Rule—Working strain of engine-work should not exceed one-tenth of the breaking strain.—The working load of boilers should not exceed one-eighth of their bursting pressure, though locomotive boilers are occasionally worked (very unsafely) to one-fourth of their bursting pressure.* General Morin states that according to a French royal decree the working strain of plate iron in boilers shall not exceed 1·9 tons per square inch.†

In constructing engine-work it is safe practice to proportion the moving parts, so that they shall not have to bear more than one-tenth of the strain that would break or cripple them.‡

483. Examples of working strain in wrought-iron girder bridges—In suspension bridges.—The following tables contain short summaries of the strains in some important girder and suspension bridges:—

* *Bourne's Handbook of the Steam Engine*, p. 464, and *Trans. Society of Engineers*, 1868, p. 55.

† *Résistance des Matériaux*, p. 20.

‡ *Drewry on Suspension Bridges*, p. 195.

TABLE V.—EXAMPLES OF WORKING STRAINS IN WROUGHT-IRON GIRDER BRIDGES.

No.	Name of Bridge.	Date.	Engineer.	Working Inch-Strain in Flanges, from Permanent Weight of Structure.		Working Inch-Strain in Flanges, when Loaded with 1 ton per running foot on each line.		Clear Span, or largest Span if more than one.	Tearing Strength of the iron per square Inch.	Observations.
				Tension.	Compression.	Tension.	Compression.			
				Tons.	Tons.	Tons.	Tons.	Feet.	Tons.	
1	Conway.	1849	R. Stephenson.	4·385	3·063	6·85	5·03	400	19·6	Tubular bridge with plate webs. In calculating the flange strains the webs are not taken into account and rivet area is not subtracted; these omissions may be assumed to balance each other in the tension flange.[1]
2	Britannia.	1850	Do.	3·1	4·6	—	—	460	19·6	Continuous tubular bridge in 4 spans with plate webs. Strains calculated over land towers, taking webs into account but not omitting rivet area in the tension flange.
3	Do.	—	—	2·9	2·7	—	—	—	—	Strains calculated at centre of large span do., do.
4	Newark Dyke.	1852	J. Cubitt.	—	—	5·0	5·0	240½	—	Warren's girders; top flanges are cast-iron tubes; lower flanges wrought-iron links; diagonal struts and ties of cast and wrought-iron respectively.
5	Boyne Viaduct.	1855	MacNeill and Barton.	—	—	5·0	4·5	264	20	Wrought-iron lattice girders in 3 continuous spans. Tensile strain is calculated for net area of tension flanges.
6	Crumlin Viaduct.	1857	Liddell and Gordon	—	—	6·65*	4·31	148	—	Warren's girder; top flanges are rectangular tubes of plate iron; lower flanges wrought-iron bars. Diagonals wrought-iron bars and angle iron. * Tensile strain is calculated for the net section of one of the diagonals.
7	Charing Cross.	1863	Hawkshaw.	—	—	5	4	154	23 ?	Lattice girders with 4 lines of rail between. Strain calculated with 1¼ tons per foot on each of four lines. Tearing strength of the iron was derived from experiments made with a hydraulic press, and are, therefore, perhaps not quite reliable.

1 Clark *on the Tubular Bridges*, pp. 376, 555, 584, 661, 748.
2 3 *Idem*, pp. 376, 587, 785.
4 *Proc. Inst. C.E.*, Vol. xii., p. 601.
5 *Idem*, Vol. xiv., p. 443.
6 *Trans. Inst. C.E. of Ireland*, Vol. vii., p. 97.
7 *Proc. Inst. C.E.*, Vol. xxii., p. 512., and *Trans. Soc. Eng.*, 1864, p. 170.

TABLE VI.—EXAMPLES OF WORKING STRAINS IN SUSPENSION BRIDGES.

No.	Name of Bridge.	Date.	Engineer.	Working Inch-Strain. From Permanent Weight of Structure.	Working Inch-Strain. When Loaded.	Chord of Catenary.	Tearing Strength of the Iron per square In.	Proof Strain, per square In.	Observations.
				Tons.	Tons.	Feet.	Tons.	Tons.	
[1]	Menai.	1826	Telford.	4·21	8·0 if loaded with 80lbs. per square foot of platform.	580	—	11	Formed of flat wrought-iron links.
[2]	Do. altered after storm of 1839.	1840	Provis.	5·06	8·86 do. do.	—	—	—	About 130 tons of timber and iron-work added for the purpose of strengthening the platform.
[3]	Hammersmith.	1827	Tierney Clark.	5·38	9·36 do. do.	422¼	—	9	Formed of flat wrought-iron links.
[4]	Pesth.	1849	Do.	5·01	8·11 do. do.	666	—	—	Formed of flat wrought-iron links made by Howard and Ravenhill's Patent.
[5]	Chelsea.	1858	Page.	4·365	8·07 do. do.	348	—	13⅓	Formed of flat wrought-iron links made by Howard and Ravenhill's Patent.
[6]	Clifton.	1864	Hawkshaw and W. H. Barlow.	2·9	5·03 do. do.	702¼	—	10	Formed of flat wrought-iron links. Strain on the suspension rods from maximum load = 4¼ tons per square inch.
[7]	Argentat.	1829	Vicat.	—	13·4 if loaded with 41lbs. per square foot of platform.	350½	47	—	12 cables of iron wire No. 18, each wire ·1181 inch in diameter.
[8]	Niagara.	1855	Roebling.	6.7	8·4 if loaded with 250 tons—viz., a railway train weighing 200 tons, and people and teams weighing 50 tns.	821	44·6	—	Four cables of 10 inches diameter, each composed of 3,640 iron wires of small No. 9 gauge, 18·3 feet to the lb., 60 wires forming one square inch of solid section. Platform is a tube formed of trussed girders, with two floors. The lower floor is used for common travel and the upper one for a single line of railway and side-walks.

[1] Drewry on *Suspension Bridges*, pp. 54, 183.
[2] *Trans. Inst. C.E.*, Vol. iii., p. 371.
[3] Drewry, p. 82, and *Parliamentary Return of Engineers' Reports upon Chelsea Bridge*, 1862, p. 19.
[4] *Reports upon Chelsea Bridge*, p. 19, and *Supplement to Theory and Practice of Bridges*. Weale, London.
[5] *Reports upon Chelsea Bridge*, pp. 7, 19.
[6] *Pro. Inst. C.E.*, Vol. xxvi., p. 243.
[7] Drewry, p. 120.
[8] *Papers and Practical Illustrations of Public Works of Recent Construction*. Weale.

484. Strength and quality of materials should be stated in specifications—Proof strain of suspension links and chains.—In drawing up specifications for girders, ships, or boiler-work, it is well to specify the tearing strength and quality of the materials. These may be tested by tearing asunder samples of the following section  in a proving machine, several of which are now to be found throughout the kingdom. The amount of elongation of wrought-iron or steel under strain is a test of toughness, a most desirable quality for many purposes, though of little importance in the compression flanges of girders. In my own specifications I require the set after fracture (ultimate elongation) of ship plates and tension plates of girders when torn with the grain to be not less than 5 per cent. of their original length; at right angles to the grain the set is generally much less, perhaps only 1 or 2 per cent. (**250, 254, 259**). In proving cast-iron, care should be taken to round off the arrises of the pin-holes of the sample, so that the strain may pass accurately through its axis (**247**). Ships' chains are now tested in proving machines sanctioned by the Board of Trade (**280**) and it is customary also to prove all the flat links of suspension chains to 9 or 10 tons per square inch. The proof, however, should not pass the limit of elasticity, say 10 to 12 tons per square inch, lest the ductility of the iron be impaired and brittleness result (**410**).

STEEL.

485. Punching reduces tensile strength of steel plates one-third as compared with drilling—Strength restored by annealing—Annealing equalizes different qualities of steel plates—8 tons per square inch safe tensile working strain for steel plates—Steel pillars.—It appears from papers on the treatment of steel, read at the annual meeting of the Institution of Naval Architects, in April, 1868,* that steel plates, such as are now being introduced for shipbuilding, may be obtained of a tensile strength of from 33 to 35 tons per square inch. Punching, as compared with drilling, reduced the strength of $\frac{5}{16}$ inch plates from 26·4 to 37·8 per cent., the average being 33 per

* *The Engineer* for April 3, 1868, pp. 248, 250, Vol. xxv.

cent. It was found, however, that annealing punched steel plates restored them to their original strength. Annealing also was recommended to equalize their strength, as in a batch of plates sent in by the same manufacturer the plates sometimes greatly differ, and a bath of molten lead was recommended as a cheap and certain mode of annealing. It was also stated that enlarging the die, so as to give it a large clearance round the punch and make a taper hole, gave a great advantage with Bessemer steel, amounting to 7 or 8 per cent., but the same advantage was not obtained with puddled steel. In experiments on iron plates it was found that a greater clearance than the usual one of a sixteenth of an inch injured the iron. Mr. Rochussen's experience lead him to prefer puddled steel for plates and Bessemer steel for angle, T, or bulb beams. The safe tensile working strain of mild steel plates, such as those described may probably be taken at eight tons per square inch (**259**).

The crushing strength of steel is so high that 12 or even 15 tons per square inch is perhaps a safe compressive working strain when the material is not permitted to deflect. Its stiffness in the form of a *long* solid pillar, judging from Mr. Hodgkinson's experiments (**205**), is $2\frac{1}{2}$ times that of cast-iron and $1\frac{1}{2}$ times that of wrought-iron, and these results seem corroborated to a certain degree by the relative values of the coefficients of elasticity of the respective materials (**282**). Experiments are, however, still wanting to determine the safe working compression of steel when subject to flexure in such sections as those usual in pillars and the struts of girders. Till such are made it will probably be safe to adopt one and a-half times the strain that a similar section of wrought-iron would safely carry.

TIMBER.

486. English, American, and French practice—Permanent working strain of timber should not exceed one-tenth of the breaking strain—Temporary working strain may reach one-fourth.—The use of timber in important structures is now so rare in the United Kingdom that it is difficult to assign the working strain which English engineers consider safe. In the

Landore Trussed Viaduct, constructed by the late Mr. Brunel of creasoted American pine, with wrought-iron ties, whether red or white pine is not stated, the timber in compression was generally calculated to bear 373 lbs. per square inch, though in some parts of the structure the strain was allowed to reach 560lbs., or 50 per cent. more.*

In the Innoshannon lattice timber bridge, erected by Mr. Nixon on the Cork and Bandon railway, the *ordinary* working strains in the flanges were 484 lbs. compression, and 847 lbs. tension per square inch. After 16 years' life this bridge was so decayed that it became unsafe and was replaced by a wrought-iron structure in 1862.† In America large timber bridges are still common, and Mr. Haupt, an American authority on this subject, "has not considered it safe to assign more than 800 lbs. per square inch as a permanent load, and 1,000 lbs. as an accidental load,"‡ and in a paper on American timber bridges, read by Mr. Mosse at the Institution of Civil Engineers in 1863, it is stated that about 900 lbs. per square inch is usually considered by American engineers to be the limit of safe compression for timber framing.§ Navier and Morin, French authorities, recommend that the working strain of timber should not exceed $\frac{1}{10}$th of the breaking strain,‖ and this rule seems safe practice for permanent structures. For merely temporary purposes a strain of $\frac{1}{4}$th of the breaking weight is probably safe (**298**). With reference to transverse strain, Tredgold states that "one-fifth of the breaking weight causes the deflexion to increase with time, and finally produces a permanent set."¶

487. Short life of timber bridges—Risk of fire.—In the paper on American timber bridges already referred to, Mr. Mosse states that they do not last in good condition more than 12 or 15 years, the timber being generally unseasoned and shrinking

* *Proc. Inst. C.E.*, Vol. xiv., p. 500.

† *Trans. Inst. C.E. Ireland*, Vol. viii., p. 1.

‡ *General Theory of Bridge Construction*, by Herman Haupt, A.M., p. 62.

§ *Proc. Inst. C.E.*, Vol. xxii., p. 310.

‖ *Navier*, p. 103, and *Morin*, pp. 51, 64, 68.

¶ *Tredgold's Carpentry*, p. 57.

much after being framed. When covered in to protect them from the weather "and cared for, any shrinkage of the braces being immediately remedied, it is believed these bridges will remain in good condition double the usual time, or about twenty-five years." Some of the old continental bridges, however, lasted much longer than this, but fire seems to be as common an agent of destruction as time in America, where, no doubt, the dry summers give it every advantage.

488. Weight which piles will support depends more upon the nature of the ground than the actual strength of the timber.—As piles in foundations beneath masonry are buried in the ground which itself supports an uncertain share of the weight of the superstructure it is impossible to say exactly what weight rests on the pile and how much on the surrounding soil. The piles in the foundations of the High Level Bridge at Newcastle, erected by Mr. R. Stephenson, were 40 feet long and driven through hard sand and gravel till they reached the solid rock. At certain states of tide, however, the sand became a quick-sand which facilitated the driving. One of these foundation piles was tested with a load of 150 tons which was allowed to remain several days, and upon its removal no settlement whatever had taken place. The piles are four feet from centre to centre, filled in between with concrete made of broken stone and Roman cement, and the utmost pressure that can come upon one of them is 70 tons, supposing none of the weight to be carried by the intervening planking and concrete.* The piles in the Royal Border Bridge, erected by Mr. Stephenson over the river Tweed, in 1850, are American elm driven from 30 to 40 feet into gravel and sand; the pressure on each of these is also 70 tons, neglecting any support derived from the intervening soil.†

Assuming the piles in these two instances to be 15 inches square, the pressure does not exceed $\frac{70}{1\cdot56} = 45$ tons per square foot, or 700 lbs. per square inch. Some of the uprights in the lofty scaffolding on which the land spans of the Britannia Bridge were built carried

* *Encycl. Brit.*, Art. "Iron Bridges," Vol. xii., Part iii., p. 604.

† *Proc. Inst. C.E.*, Vol. x., p. 224.

28 tons per square foot, or 435½ lbs. per square inch. The horizontal timbers, however, were somewhat compressed under this strain (**228**).* As already mentioned in **229**, the nature of the ground has more to do with the safe load on piles than the actual strength of the timber; 70 tons per pile, however, is the severest load I find recorded.

FOUNDATIONS, STONE, BRICK, MASONRY, CONCRETE.

489. Foundations of earth, clay, gravel.—Professor Rankine states that "the greatest intensity of pressure on foundations in firm earth is usually from 2,500 lbs. to 3,500 lbs. per square foot, or from 17 lbs. to 23 lbs. per square inch."† The depth of foundations should be sufficient to place them below the influence of frost or running water, nor should it be forgotten that deep excavations or drainage-works in the neighbourhood of buildings frequently cause subsidence of the foundations and superstructure. The following table contains a few examples of heavy pressures on foundations.

* Clark *on the Tubular Bridges*, p. 549.
† *Civil Engineering*, p. 380.

TABLE VII.—EXAMPLES OF WORKING PRESSURES ON FOUNDATIONS.

No.	Name of Bridge.	Date.	Engineer.	Material in foundation.	Pressure per square foot.	Observations.
					Tons.	
1	Leven and Kent Viaducts, Morecambe Bay, on the Ulverston and Lancaster Railway.	1857	Brunlees.	Sand beneath cast-iron disc piles.	4 to 5	Lattice girders resting on cast-iron disc piles, the discs being 2 feet 6 inches diameter and sunk by forcing water down the centre of the pile. Rubble stone tipped in round the piles prevents sand from being scoured away by the current.
2	Lóch Ken Viaduct on the Portpatrick Railway.	1861	Blyth.	Gravel.	$6\frac{1}{4}$	Bowstring girders resting on cast-iron cylinders, 8 feet in diameter, and filled with concrete and masonry. Rubble stone tipped in round the cylinders.
3	Charing Cross Bridge.	1863	Hawkshaw.	London clay.	8	Lattice girders resting on cast-iron cylinders 14 feet diameter below the ground and 10 feet diameter above, filled with Portland cement concrete and brickwork, and sunk from 50 to 70 feet below Trinity high water into the solid London clay. The pressure is calculated on the supposition that each of the four lines of railway is loaded with locomotives, and that there is no relief afforded by the friction of the sides of the cylinders against the material through which they penetrate. The friction in sinking each cylinder amounted to 150 tons. If this be taken into account, it would reduce the pressure on the clay by about 1 ton per square foot.

[1] *Proc. Inst. C.E.*, Vol. xvii., p. 442. [2] *Ibid*, Vol. xxi., p. 258. [3] *Ibid*, Vol. xxii., p. 512.

490. Working pressure on masonry, brickwork, or concrete, should not exceed one-sixth of the crushing weight of the mass—Working pressure on ashlar-work should not exceed one-twentieth of the crushing strength of the stone.— Professor Rankine states that "the resistance of *good coursed rubble* masonry to crushing is about four-tenths of that of single blocks of the stone that it is built with. The resistance of *common rubble* to crushing is not much greater than that of the mortar which it contains."* The working load on rubble masonry, brickwork, or concrete, as already observed in (**330**), rarely exceeds one-sixth of the crushing weight of the aggregate mass and this seems a safe limit. General Morin, however, states that mortar should not be subject to a greater pressure than one-tenth of its crushing weight.† The ashlar voussoirs of an arch, where the line of thrust may vary considerably from the calculated direction, should not be subjected to a greater (calculated) pressure than $\frac{1}{20}$th of that which would crush the stone. It is safe to apply the same rule to all ashlar-work, as it is very difficult, if not impossible, to command a perfectly uniform pressure throughout the whole bed of each stone, and a slight inequality in the line of pressure may cause splintering or flushing at the joints. Vicat's experiments (**334**) and the examples of pressure given in the following table, seem to show that the weight on stone columns may sometimes reach as high as $\frac{1}{10}$th of the crushing strength of the stone. This, however, is a much severer load than is usual in modern engineering practice, and cannot be recommended as very safe.

* *Civil Engineering*, p. 387.

† *Résistance des Matériaux*, p. 51.

TABLE VIII.—EXAMPLES OF WORKING PRESSURES ON MASONRY.

No.	Name of the Structure.	Date.	Engineer.	Material.	Pressure per square foot.	Observations.
1	Pont y tu Pridd Bridge, over the river Taff, in Glamorganshire.	1750	W. Edwards.	Hard grey sandstone rubble, set in Aberthaw lime.	Tons. $20\frac{2}{3}$	Single arch 140 feet span. Pressure calculated at crown of arch.
2	Barentin Viaduct, on the Havre Railway.	1845	J. Locke.	Rubble limestone, set in chalk lime.	$8\frac{1}{3}$	Fell from the crushing of the piers, the mortar in the interior being as soft as the first day it was laid. Afterwards rebuilt with brick in hydraulic lime.
3	Lockwood Viaduct, on the Huddersfield and Sheffield Railway.	1849	Hawkshaw.	Flat bedded sandstone rubble, set in Weldon Wood lime.	8	Semicircular arches 30 feet span, with two oblique arches of 40 and 70 feet span respectively. Pressure calculated at base of deepest piers, 7 feet 7 inches thick at the base.
4	Britannia Bridge, Chester and Holyhead Railway.	1850	R. Stephenson.	Ashlar Anglesey limestone, set in lime of the same stone.	16	Pressure calculated at base of the Britannia Tower, founded on chlorite schist rock. Crushing weight of the limestone is about 500 tons per square foot.
5	Saltash Bridge, Cornwall Railway.	1858	Brunel.	Granite ashlar, set in cement.	10	Pressure calculated at base of middle pier, constructed of ashlar masonry in a wrought-iron cylinder 37 feet diameter and 96 feet high, founded upon rock.
6	Pillars of the dome of St. Peter's (*Rome*).	—	—	Calcareous Tufa, called *Travertin*.	33,330lbs.=14·9	Travertin is crushed by about 536,000 lbs., = 239 tons, per square foot, or 16 times the working pressure.
7	Pillars of the dome of St. Paul's (*London*).	—	Sir C. Wren.	Portland stone (oolite limestone).	39,450lbs.=17·6	Crushing weight of Portland stone is 537,000 lbs., = 240 tons, per square foot, or 13·3 times the working pressure.
8	Pillars of the dome of St. Geneviève (*Paris*).	—	—	Limestone.	60,000lbs.=26·8	Crushing weight of this stone is 456,000 lbs., = 203·5 tons, per square foot, or 7·6 times the working pressure.
9	Pillars of the Church of Toussaint (*Angers*).	—	—	Very hard shell limestone of a reddish colour.	90,000lbs.=40·2	Crushing weight of this stone is 900,000 lbs., = 402 tons, per square foot, or 10 times the working pressure.
10	Lower courses of the Piers of the Bridge of Neuilly.	—	Perronet.	Limestone.	3,600 lbs. = 1·6	Crushing weight of this stone is 570,000 lbs., = 254 tons, per square foot, or 158 times the working pressure.
11	Pillars of the Church of All-Saints (*Angers*).	—	—	Fourneaux stone.	86,000lbs.=38·4	Crushing weight of this stone is 880,000 lbs., = 393 tons, per square foot, or $10\frac{1}{4}$ times the working pressure.
12	Pillars of the dome of the Pantheon (*Paris*).	—	—	Bagneux stone.	60,000lbs.=26·8	Crushing weight of this stone is 496,000 lbs., = 221 tons, per square foot, or $8\frac{1}{4}$ times the working pressure. Some of the stones split under the load, which probably is not quite uniformly distributed.
13	Pillar in Chapter House (*Elgin*).	—	—	Red sandstone.	40,000lbs.=17·9	

[1] *Proc. Inst. C.E.*, Vol. x., p. 241.
[2] *Ibid*, Vol. xvi., p. 438, and *Weale's Supplement to Bridges*, p. 118.
[3] *Ibid*, Vol. x., p. 296.
[4] Clark *on the Britannia and Conway Tubular Bridges*, p. 542.
[5] *Proc. Inst. C.E.*, Vol. xxi., p. 270.
[6 7 8 9 10] Mahan's *Elementary Course of Civil Engineering*, p. 28.
[11 12 13] *Encyc. Brit.*, Art. "Stone Masonry," Vol. xx., Part iii., p. 718.

TABLE IX.—EXAMPLES OF WORKING PRESSURES ON BRICKWORK.

No.	Name of the Structure.	Date.	Engineer.	Material.	Pressure per square foot.	Observations.
					Tons.	
1	Railway Viaduct in Birmingham.	1849	Brunel.	Red Birmingham brick, set in Lias lime.	7	Pressure calculated at top of footing of one of the piers.
2	Charing Cross Bridge.	1863	Hawkshaw.	London paviour bricks, set in mortar made of 1 Portland cement + 2½ sand.	12	Pressure calculated at bottom of the upper length of cylinder 10 ft. in diameter, on the supposition that each of the four lines of rail is loaded with 1½ tons per running foot; see Ex. 3, Table VII.
3	Clifton Suspension Bridge.	1864	Hawkshaw and W. H. Barlow.	Staffordshire blue brick, set in Portland cement.	10	Pressure calculated where chain anchor-plates bear against the brickwork forming the anchorage.
4	Chimney at Adkin's soap works near Birmingham.	—	—	Brick.	6	Pressure calculated at base. Height 312 feet.
5	Chimney at patent tube works, near Birmingham.	—	—	Do.	8½	Six large flues cut through base. Height 145 feet.
6	Glass House Cone.	—	—	Brickwork exposed to great heat.	4	Large arches in each of the four sides. Height 75 feet.
7	Chimney at West Cumberland Hæmatite Iron Works.	1867	—	White bricks, set in hydraulic lime.	8	Pressure calculated 2 feet above ground line. Height above ground line 250 feet. Total height 267 feet.
8	U. S. Light House, "1st order," 160 feet high.	—	—	Brick.	3·7	Pressure calculated on lower courses. Weight that would crush the material = 31·7 tons per square foot, or 8½ times the working load.
9	Merchant's Shot Tower, Baltimore.	—	—	Do.	6·5	246 feet high, 39 feet 10 inches base; pressure calculated on lower courses. Crushing weight = 31·7 tons, or 4·8 times the working pressure.

[1] *Proc. Inst. C.E.*, Vol. xi., p. 76.
[2] *Ibid*, Vol. xxii., p. 512, and *Trans. Soc. of Eng.*, 1864, pp. 162, 171.
[3] *Ibid*, Vol. xxvi., p. 248.
[4 5 6] *Ibid*, Vol. x., p. 242.
[7] *Trans. Inst. Eng., in Scotland*, Vol. xi., p. 157.
[8 9] *Strength of Materials*, by J. K. Whildin, New York, p. 23.

From Mr. Grant's experiments, made with a hydraulic press, it appears that the crushing strength of five 12-inch cubes of concrete made of Portland cement and Thames ballast in the proportion of 1+6 and 1+8, and 10 months old, varied from 52·4 to 83·2 tons per square foot, or from 815 to 1,294 lbs. per square inch, those that were kept in water giving the highest results. One twelve-inch cube of concrete, made with blue Lias lime and Thames ballast 1+6, also 10 months old and kept in water, bore 6 tons per square foot or 93 lbs. per square inch. A similar cube of Lias concrete, but made with Bramley Fall chippings 1+6 in place of ballast, and also kept in water 10 months, bore 20·4 tons per square foot or 317 lbs. per square inch.*

TABLE X.—EXAMPLES OF WORKING PRESSURES ON CONCRETE.

No.	Name of the Structure.	Date.	Engineer.	Material.	Pressure per square foot.	Observations.
					Tons.	
1	Charing Cross Bridge.	1863	Hawkshaw.	Concrete made of Portland cement and Thames gravel, 1+7.	8	See Ex. 3, Table VII.
2	Chimney at West Cumberland Hæmatite Iron Works.	1867	—	Concrete made with hydraulic lime.	2	Concrete base 8 feet thick beneath brick chimney. See Ex. 7, Table IX. Pressure on ground below = 1·6 tons per square foot.

[1] *Proc. Inst. C.E.*, Vol. xxii., p. 515. [2] *Trans. Inst. Eng. in Scotland*, Vol. xi., p. 157.

WORKING LOAD ON RAILWAYS.

491. A train of engines varying from one ton to one and one-third ton per running foot is the heaviest working load on 100 feet railway girders—Three-fourths ton per running foot the heaviest working load on 400 feet girders—Weight

* *Proc. Inst. C.E.*, Vol. xxv., p. 110.

of Engines—Girders under 40 feet liable to concentrated working loads.—A train of locomotives, the weight of which generally varies from 1 to 1⅓ tons per running foot, is the heaviest passing load to which a single-line railway bridge is liable, but it rarely happens in practice that girders are subject to a uniform load of this density except in short bridges whose length does not exceed that of two engines with their tenders which may collectively cover from 80 to 100 feet of line. We may therefore safely assume that the maximum strain to which the flanges of railway girders 100 feet in length are subject does not exceed that due to the permanent bridge-load plus a train-load of from 1 to 1⅓ tons (according to size of engines) per running foot on each line of way. In longer bridges than 100 feet the train-load per running foot will be less, and in bridges of 400 feet span or upwards, the greatest occasional load can scarcely exceed ¾ ton per running foot on each line, as this is a denser load than that of an ordinary goods train.*

Until lately it has been usual to take one ton per running foot on each line as the ruling load for engines. This, however, is scarcely safe practice since many engines now exceed this, as shown by the three following tables, for which I am indebted to A. M'Donnell, Esq., Locomotive Engineer of the Great Southern and Western Railway, Ireland, and to J. Ramsbottom, Esq., Locomotive Engineer of the London and North Western Railway.

* The following memorandum by Mr. J. C. Smith, Resident Engineer on the Dublin, Wicklow, and Wexford Railway, shows the weight of a train of wagons loaded with sulphur ore :—

"Weight of mineral engine loaded, 27 tons.

——————— tender do. 17 do.

Length of engine and tender, buffer to buffer, 44 feet.

Wagon empty 4 tons, loaded 12 tons. Length 18 feet, out to out of buffers. Two other descriptions of wagons, one 12 feet, and the other 14 feet 6 inches long, taking one ton less and weighing about 5 cwt. less.

A mineral train, of engine, 20 wagons and van, will weigh about 280 tons, and its length will be about 400 feet when buffers are close up; when running, somewhat longer."

TABLE XI.—EXAMPLES OF ENGINES ON THE GREAT SOUTHERN AND WESTERN RAILWAY, IRELAND—GAUGE 5′ 3″.

Class of Engine.	No. of Wheels.	Distance. From Middle to Trailing Wheels.		Distance. From Leading to Middle Wheels.		Wheel Base.		Weight on Wheels. Leading.		Weight on Wheels. Middle.		Weight on Wheels. Trailing.		Total Weight.		Length over Buffers.		Observations.
		Ft.	In.	Ft.	In.	Ft.	In.	Tns.	Cts.	Tns.	Cts.	Tns.	Cts.	Tns.	Cts.	Ft.	In.	
Single passenger No. 55.	6	7	6	7	4	14	10	9	6	11	8	5	0	25	14	23	0	2 cwt. of coal in firebox and 6 inches of water in glass.
Passenger No. 11, 4 wheels coupled.	6	6	0	7	11	13	11	9	7	10	0	9	0	28	7	23	3	Middle and trailing wheels coupled; 2 cwt. of coal in firebox and 6½ inches of water in glass.
Passenger No. 56, 4 wheels coupled.	6	7	0	7	9	14	9	9	12	12	8	10	6	32	6	24	9	Middle and trailing wheels coupled; 2 cwt. of coal in firebox and 7 inches of water in glass.
Goods No. 137, 4 wheels coupled.	6	8	3	6	11½	15	2½	11	6	11	2	5	11	27	19	24	2½	Leading and middle wheels coupled; 2 cwt. of coal in firebox and 6 inches of water in glass.
Goods No. 147, 6 wheels coupled.	6	7	3	8	3	15	6	11	4	10	18	10	18	33	0	24	10½	3 cwt. of coal in firebox and 6 inches of water in glass.
Tank No. 145, 6 wheels coupled.	6	7	3	7	7	14	10	9	16	12	0	14	14	36	10	29	2	Full boiler and tank and 30 cwt. of coal in bunker, but weight in ordinary working order is about 34 tons, as the tanks and coal box are seldom quite full, the engine being used for banking where it need not carry much coal.
Do., empty.	—	—		—		—		9	12	8	7	10	6	28	5	—		—

TABLE XII.—EXAMPLES OF TENDERS ON THE GREAT SOUTHERN AND WESTERN RAILWAY, IRELAND—GAUGE 5′ 3″.

Class of Tender.	Distance: From Leading to Middle Wheels.	Distance: From Middle to Trailing Wheels.	Wheel Base.	Weight on Wheels: Leading.	Weight on Wheels: Middle.	Weight on Wheels: Trailing.	Total Weight.	Length over Buffer.	Weight of Engine and Tender.	Length of Engine and Tender.	Observations.
	Ft. In.	Ft. In.	Ft. In.	Tns. Cts.	Tns. Cts.	Tns. Cts.	Tns. Cts.	Ft. In.	Tns. Cts.	Ft. In.	
For engine No. 56.	5 2	5 2	10 4	6 9	8 12	6 9	21 10	18 4	53 16	43 1	Full tank and 45 cwt. of coal.
For engine No. 147.	5 2	5 2	10 4	7 12	6 15	6 11	20 18	18 6	53 18	43 4½	Full tank and 3 tons of coals, weight of empty tender 11 tons, 2 cwt.

There are some engines on the Midland Great Western Railway of Ireland which weigh 28 tons, and stand on a 12 feet base. Mr. Price, the Resident Engineer, informs me that they overhang behind and kick dreadfully, and he regards them as the most trying of their stock on the road and girder bridges, especially on small ones.

TABLE XIII.—EXAMPLES OF ENGINES ON THE LONDON AND NORTH WESTERN RAILWAY—GAUGE 4′ 8½″.

Class of Engine.	No. of Wheels.	Distance.				Wheel Base.		Weight on Wheels.						Total Weight.		Length over Buffers.		Total Length of Engine and Tender.		Total Weight of Engine and Tender.		Remarks.
		From Leading to Middle Wheels.		From Middle to Trailing Wheels.				Leading.		Middle.		Trailing.										
		Ft.	In.	Ft.	In.	Ft.	In.	Tns.	Cts.	Tns.	Cts.	Tns.	Cts.	Tns.	Cts.	Ft.	In.	Ft.	In.	Tns.	Cts.	
Single Passenger 7′ 6″ wheel.	6	7	7	7	10	15	5	9	8	11	10	6	2	27	0	23	8½	44	7	44	8½	Engine and tender A in working order.
	—	—		—		—		9	9	8	0	6	8	23	17	—		—		—		Do. Empty.
4 wheels coupled passenger 6′ 6″ wheel.	6	7	5	8	3	15	8	9	9	11	0	8	15	29	4	25	11½	46	10	46	12½	Engine and tender A in working order.
	—	—		—		—		9	4	9	6	8	15	27	5	—		—		—		Do. Empty.
Single passenger 7′ wheel.	6	8	4	8	6	16	10	10	19	12	10	6	9	29	18	26	0½	45	6½	47	17	Engine and tender C in working order.
	—	—		—		—		9	9	11	1	5	18	26	8	—		—		—		Do. Empty.
Single passenger 6′ 6″ wheel.	6	7	9	7	9	15	6	9	0	12	0	6	10	27	10	23	4½	42	10½	45	9	Engine and tender C in working order.
	—	—		—		—		8	0	11	0	5	3	24	3	—		—		—		Do. Empty.
Single passenger 6′ wheel.	6	6	10	6	6	13	4	7	13	9	3	3	19	20	15	21	7	42	5½	38	3½	Engine and tender A in working order.
	—	—		—		—		7	0	8	12	3	10	19	2	—		—		—		Do. Empty.
Goods 6 wheels coupled 5′ wheels.	6	7	3	8	3	15	6	9	14	10	0	7	6	27	0	24	5½	45	4	48	8	Engine and tender B in working order.
	—	—		—		—		8	8	9	14	6	6	24	8	—		—		—		Do. Empty.
Tank 6 wheels coupled 5′ wheels.	6	6	11	8	7	15	6	10	11	12	16	9	0	32	7	29	2	—		—		Engine in working order, tank filled partially.
	—	—		—		—		10	0	9	17	8	5	28	2	—		—		—		Do. & tank empty.
Tank 4 wheels coupled 5′ wheels.	6	4	8	7	4	12	0	7	7	10	0	9	8	26	15	24	9½	—		—		Engine in working order, tank filled.
	—	—		—		—		5	3	9	16	6	15	21	14	—		—		—		Do. & tank empty.
Tender A 1,500 gals. of water.	6	6	6	6	0	12	6	6	0	5	7	6	1½	17	8½	20	10½	—		—		Tender in working order.
	—	—		—		—		3	10	3	2½	2	17	9	9½	—		—		—		Do. Empty.
Tender B 2,000 gals. of water.	6	6	6	6	0	12	6	7	5	7	1	7	2	21	8	20	10½	—		—		Tender in working order.
	—	—		—		—		3	18½	3	7	3	3½	10	9	—		—		—		Do. Empty.
Tender C 1,500 gals. of water.	6	5	6	5	6	11	0	6	4	5	5	6	10	17	19	19	6	—		—		Tender in working order.
	—	—		—		—		3	15	3	19	3	5	10	19	—		—		—		Do. Empty.

Occasional monster engines are also found on railways, generally where the gradients are unusually steep, as illustrated in the following table:—

TABLE XIV.—EXAMPLES OF HEAVY ENGINES ON VARIOUS RAILWAYS.

No.	Railway.	No. of Wheels.	Wheel Base.		Weight.	Observations.
			Ft.	In.	Tons.	
1	North London.	4	—		42	Four wheels coupled.
2	Oldham.	6	—		49	Goods engine with 6 wheels coupled; gradient 1 in 27.
3	Brecon and Merthyr.	6	12	0	38	Tank engine; gradient 1 in 38.
4	Vale of Neath.	8	—		56	Tank engine with 8 wheels coupled, afterwards altered into engine with tender in consequence of the destruction to the permanent way; gradient 1 in 47.
5	Mauritius Railway.	8	15	6	47	8 wheels coupled. Weight of engine 47 tons exclusive of tender, to work gradients of 1 in 27.
6	Northern Railway of France.	12	19	8	67½	Tank engine with 4 outside cylinders; wheels coupled together, as in two separate six-wheeled coupled engines.
7	Semmering.	8	—		55½	—
8	Giovi.	8	—		55½	Four Cylinders.
9	Cologne Minder.	—	11	2	32	Weight of tender 18 tons.
10	Rhenish.	—	13	0	39½	No tender.
11	Do.	—	11	0	29	Tender 17½ tons.

1 2 *Proc. Inst. C. E.*, Vol. xxvi., p. 343, 383.
3 *Ibid*, p. 335.
4 *Ibid*, pp. 372, 374.
5 *Ibid*, p. 384.
6 7 8 *Ibid*, p. 373, 343.
9 10 11 *Ibid*, Vol. xxv., p. 436.

Short railway girders, say under 40 feet span, are liable to considerably heavier strains than those due to uniform loads of 1, 1¼, or 1⅓ tons per running foot on each line, and their strength should accordingly be greater in proportion than that of girders which exceed this span. If, for instance, a six-wheeled engine, 24 feet long and weighing 32 tons on a twelve-feet wheel base, rest on the centre of a bridge 32 feet in length, the strain in the flanges is obviously greater than would occur if 42·7 tons ($= 32 \times 1\frac{1}{3}$) were distributed uniformly. A 40-feet bridge would, it is true, have the weight of only one such engine on the centre at a time, and if the load on the middle pair of wheels equal 16 tons and that on the leading and trailing pairs (6 feet on either side of the centre) equal 8 tons respectively, the equivalent load concentrated at the centre of the bridge is 27·2 tons, or 54·4 tons distributed. If there were three such engines in a row, the pressure might be slightly increased by the weight on the leading and trailing wheels of the extreme engines, each of which would have one pair of wheels, or 8 tons, resting on the bridge within 2 feet of the abutments. This is equivalent to 1·6 tons concentrated at the centre, or 3·2 tons distributed over the bridge. Adding this to the 54·4 tons due to the central engine, we have a total weight equivalent to a distributed load of 57·6 tons, or 1·44 tons per running foot. This arrangement of engines produces the greatest strain at the centre of the flanges. Again, two such engines might stand with their ends in contact at the centre of the 40-feet bridge, and though their outer ends would project beyond each abutment their collective wheel base would cover only 36 feet of the bridge. This arrangement of engines produces greater strains than the former near the ends of the flanges. Indeed, these end strains will in some cases slightly exceed those given by the following rules, but this is compensated for by the flanges being generally made heavier near the ends than theory requires (**428**).

492. Standard working loads for railway bridges of various spans.—The following tables are intended to give the results of the preceding observations in a concise form. They are based on six assumptions:—

1. The working load for railway bridges 400 feet in length and upwards does not exceed $\frac{3}{4}$ ton per running foot on each line.

2. No more locomotives than will cover 100 feet in length follow each other without interruption; hence the working load per

foot diminishes as the span increases from 100 feet up to 400 feet.

3. Engines may be arranged on bridges less than 100 feet long so as to produce greater strains than would be due to the engine load if it were of uniform density; hence the equivalent working load per foot increases as the span diminishes from 100 feet downwards.

4. Bridges less than 40 feet in span are subject to concentrated loads from single engines.

5. The standard locomotive is assumed to be 24 feet long and to have 6 wheels with a 12 feet base; to have half its weight resting on the middle wheels and one-fourth on the leading and trailing pairs respectively, which are supposed to be at equal distances on either side of the middle wheels.

6. Standard engines are assumed to weigh 24 tons, 30 tons, and 32 tons, according to their construction. This makes the standard load 1 ton, $1\frac{1}{4}$ ton, or $1\frac{1}{3}$ ton per foot of single line, according to the weight of the engines which work it, but it is safest to take the higher standards for the railways in Great Britain, as they are so interlaced that engines may pass from one line to another, and it is quite possible that we have not yet arrived at the limit of weight.

BRIDGES FROM 40 TO 400 FEET IN LENGTH.

If the standard load (the heaviest engine) on a 100 feet bridge weigh 1 ton per foot while that on a 400 feet bridge weighs ·75 tons per foot, the difference (= ·25 ton per foot) must be gradually distributed among the intervening 300 feet; in other words, the difference for each 10 feet in length $= \frac{\cdot 25}{30} = \cdot 0083$ tons.

The differences for the other standards may be found in a similar way, and the following table contains the values of the working loads corresponding to the three standards for bridges of various lengths between 40 and 400 feet.

TABLE XV.—WORKING LOADS FOR RAILWAY BRIDGES FROM 40 TO 400 FEET IN LENGTH.

Length of Bridge in feet.	Working Load in tons per Running Foot of Single Line.		
	Standard Load on a 100 feet Bridge = 1 ton per foot.	Standard Load on a 100 feet Bridge = 1¼ ton per foot.	Standard Load on a 100 feet Bridge = 1⅓ ton per foot.
40	1·05	1·35	1·45
50	1·04	1·33	1·43
60	1·03	1·32	1·41
70	1·03	1·30	1·39
80	1·02	1·28	1·37
90	1·01	1·27	1·35
100	**1·00**	**1·25**	**1·33**
120	·98	1·22	1·30
140	·97	1·18	1·26
160	·95	1·15	1·22
180	·93	1·12	1·18
200	·92	1·08	1·14
250	·88	1·00	1·04
300	·83	·92	·94
350	·79	·83	·85
400	·75	·75	·75

BRIDGES UNDER 40 FEET IN LENGTH.

Bridges under 40 feet in length should be strong enough to support a standard engine resting at the centre of the bridge. The following is an approximate method of calculating the value of the working load corresponding to each standard. First, find what load concentrated at the centre of the bridge will produce a strain in the centre of the flanges equivalent to that due to the standard engine. Twice this may be taken as the equivalent uniformly distributed load, which again divided by the span gives the working load per running foot required.

TABLE XVI.—WORKING LOAD FOR RAILWAY BRIDGES UNDER 40 FEET IN LENGTH.

Length of Bridge in feet.	Working Load in tons per Running Foot of Single Line.		
	Standard Load on a 100 feet Bridge = 1 ton per foot.	Standard Load on a 100 feet Bridge = 1¼ ton per foot.	Standard Load on a 100 feet Bridge = 1⅓ ton per foot.
12	2·0	2·5	2·67
16	1·88	2·34	2·5
20	1·68	2·1	2·24
24	1·5	1·87	2·0
28	1·35	1·68	1·79
32	1·22	1·53	1·62
36	1·11	1·39	1·48

Short railway girder bridges are so light in proportion to the passing load that it is a good plan to bed the main girders on timber wall plates to prevent the masonry of the abutments from being shaken to pieces by the vibration of heavy engines.

493. Effect of concentrated loads upon the web.—The weight of a heavy engine may, as already explained, be concentrated within a 12-feet wheel base and thus produce a great local pressure on one or two cross-girders, which they again will transmit to one or two points in each main girder. It might even happen in a lattice girder that the intervals of the bracing and cross-girders were such as to throw the load from several successive pairs of wheels on one system of diagonals which would thus be liable to excessive strain (**450**). We have, it is true, some compensation for this; first, in the rigidity of the flanges, platform, sleepers, and rails, all of which help to distribute the weight; and secondly, in the fact that the bracing of the central parts of small girders is for practical reasons generally stronger than theory requires (**487**), and it will generally be found sufficient to calculate the web strains on the supposition that the passing load is of uniform density, and equal in weight per running foot to the working loads given in the two previous tables.

494. Proof load of railway bridges—English practice—French Government rule.—No definite rule has been yet made by the Board of Trade for the proof load of railway girder bridges, but the general practice on inspection is to load each line with as many engines and tenders as the bridge will hold, and measure the deflection in the second way described in **460**. This proof is generally assumed to vary from 1 ton per running foot on the longer bridges to 1½ ton on the shorter ones; but when a bridge exceeds a certain span, say 150 feet, it is obviously unreasonable to cover it with heavy engines, and ballast wagons may be used along with two or three engines so as to bring the proof load more in accordance with Table XV.

I am indebted to M. Husquin de Rhèville, Secretary of the French Society of Civil Engineers, for the following information respecting the French Ministerial regulations for the proof loads of wrought-iron railway bridges:—

a. For bridges under 20 mètres each span, a dead load of 5,000 kilogrammes per running mètre of each line (= 1·5 tons per running foot).

b. For bridges exceeding 20 mètres each span, a dead load of 4,000 kilogrammes per running mètre of each line (= 1·2 tons per running foot). In some cases permission is given to reduce the dead load to 3,500 kilogrammes per mètre (= 1·05 tons per foot).

c. In addition to the foregoing proof by dead weight, a train composed of two engines (each weighing with its tender at least 60 tons), and wagons (each loaded with 12 tons), in sufficient number to cover at least one span, is driven across at a speed of from 20 to 39 kilomètres (12 to 24 miles) per hour.

d. A second trial is made by driving at a speed of from 40 to 70 kilomètres (25 to 43 miles) per hour, a train composed of two engines (each with its tender weighing 35 tons), and wagons loaded as in ordinary passenger trains, in sufficient number to cover at least one span.

e. For bridges with two lines the trains are made to traverse each line, at first in parallel, and then in opposite directions so that the trains meet at the centre.

WORKING LOAD ON PUBLIC BRIDGES.

495. Men marching in step and running cattle are the severest loads on suspension bridges—A crowd of people the greatest distributed load on a public bridge—Portions of highly excited crowds pack at the rate of 147 lbs. per square foot—French and English practice—100 lbs. per square foot recommended as the standard working load on public bridges—Sometimes liable to concentrated loads as high as 12 tons on one wheel.—It is well known that infantry marching in step will strain suspension bridges far more severely than any other form of passing load. On this subject Drewry came to the following conclusions:—"1st, that any body of men marching in step, say at three to three and a half miles per hour, will strain a bridge at least as much as double their weight at rest; and, 2nd, that the strain they produce increases much faster than their speed, but in what precise ratio is not determined. In prudence not more than one-sixth of the number of infantry that would fill a bridge should be permitted to march over it in step; and if they do march in step, it should be at a slow pace. The march of cavalry or of cattle is not so dangerous—first, because they take more room in proportion to their weight; and secondly, because their step is not simultaneous."* Referring to the Niagara Falls Suspension Bridge Mr. Roebling observes—"In my opinion a heavy train, running at a speed of twenty miles an hour, does less injury to the structure than is caused by twenty heavy cattle under a full trot. Public processions marching to the sound of music, or bodies of soldiers keeping regular step, will produce a still more injurious effect."† A crowd of people therefore constitutes the greatest distributed load on a public bridge, and 15 adults are generally estimated to weigh 1 ton, which gives an average of 149·3 lbs. to each adult. Different statements, however, have been made respecting the number of people that can stand in a given space, and in order to test this I *packed* twenty-nine Irish artisans

* *Drewry on Suspension Bridges*, p. 190.

† *Papers and Practical Illustrations of Public Works*, p. 29. Weale, London.

and 1 boy, taken from a forge and fitting shop, and weighing collectively 4,382 lbs. or 146 lbs. per individual, on a weigh-bridge $6' \ 1'' \times 4' \ 10'' = 29{\cdot}4$ square feet. In this experiment the men overhung the edges of the weigh-bridge to a slight extent and gave too high a result; I therefore on another occasion packed 58 Irish labourers, weighing 8,404 lbs. or 145 lbs. a man, in the empty deck-house of a ship, $9' \ 6'' \times 6' \ 0'' = 57$ square feet; this gives a load of 147·4 lbs., or very nearly one man per square foot, and is I believe a perfectly reliable experiment. Such cramming, however, could scarcely occur in practice except in portions of a strongly excited crowd, but I have no doubt that it does occasionally so occur. The standard working load for suspension bridges in France was formerly 200 kilogrammes per square mètre, = 41 lbs. per square foot.* This may be a sufficient standard for bridges with gatekeepers at the ends to prevent overcrowding, but it is obviously insufficient for bridges which are free to the public, especially in the vicinity of towns, and modern French practice seems to have raised the standard to 82 lbs. per square foot.† Drewry adopted 70 lbs. per square foot of platform as the greatest load that a public bridge would sustain if covered with people.‡ Tredgold and Professor Rankine estimate the weight of a dense crowd at 120 lbs. per square foot,§ and the late Mr. Brunel is said to have used 100 lbs. in his calculations for Hungerford Suspension Bridge. Mr. Hawkshaw adopted 80 lbs. per square foot for the footpaths of Charing Cross Bridge;‖ and (in conjunction with Mr. W. H. Barlow) 70 lbs. for the Clifton Suspension Bridge.¶ My present opinion is that the standard working load of public bridges for calculation should not be less than 100 lbs. per square foot of platform, especially for bridges near cities. Public bridges also are subject to concentrated loads at single points of quite as severe a character

* *Drewry on Suspension Bridges*, p. 113.

† *Trans. Soc. of Eng. for* 1866, p. 197.

‡ *Drewry on Suspension Bridges*, p. 189.

§ *Tredgold's Carpentry*, p. 169, and *Rankine's Civil Engineering*, p. 466.

‖ *Proc. Inst. C. E.*, Vol. xxii., p. 534.

¶ *Idem*, Vol. xxvi., p. 248.

as those to which railway bridges are liable; if, for instance, a marine boiler, a large cannon, an iron girder, a heavy forging or casting be conveyed across a public bridge, the weight resting on a single pair of wheels may reach or even exceed 16 tons. For example, the crank shaft of H.M. armour-plated ship Hercules—weighing, shaft and lorry, about 45 tons on four wheels—was refused a passage across Westminster iron bridge in 1866 for fear of injury to the bridge, and had to be conveyed across Waterloo stone bridge.* It is necessary therefore not only to make the cross girders, but every part of the sheeting on which the road material rests, strong enough to bear heavy local loads which, as we have seen in the foregoing instance, may sometimes reach nearly 12 tons on a single wheel.

496. Weights of roofing materials—Weight of snow.— The weights of various roofing materials are given by Tredgold as follows.†

TABLE XVII.—WEIGHTS OF VARIOUS ROOFING MATERIALS.

Kind of Covering.	Inclination to the Horizon, in Degrees.		Height of Roof in parts of Span.	Weight upon a Square Foot of Roofing.	
	Deg.	min.			
Copper or lead,	3	50	$\frac{1}{48}$	Copper, Lead,	1·00 lbs. 7·00 lbs.
Slates, large,	22	0	$\frac{1}{5}$		11·20
Ditto, ordinary,	26	33	$\frac{1}{4}$	From to	9·00 5·00
Stone slate,	29	41	$\frac{2}{7}$		23·80
Plain tiles,	29	41	$\frac{2}{7}$		17·80
Pantiles,	24	0	$\frac{2}{9}$		6·50
Thatch of straw, reeds, or heath,	45	0	$\frac{1}{2}$	Straw,	6·50
Force of wind does not generally exceed,	—		—		40·00

* *Engineer*, Vol. xxii., No. 564, p. 298.

† *Tredgold's Carpentry*, p. 96.

Morin states that snow weighs ten times less than water, and that it may accumulate on roofs to half a mètre, or nearly 20 inches in depth, when it will weigh 10 lbs. per square foot.* Mr. Zerah Colburn estimates that the weight of saturated snow on bridges in America is equal to 6 inches of water, or 30 lbs. per square foot over the whole floor of a bridge.† The pressure of wind has been already given in Chap. XX. Tredgold's estimate in Table XV. seems far too high, since the slope of a roof must greatly diminish the pressure of the wind on each square foot of surface.

* *Résistance des Matériaux*, p. 382.

† *Proc. Inst. C. E.*, Vol. xxii., p. 546.

CHAPTER XXVII.

ESTIMATION OF GIRDER-WORK.

497. Theoretic and empirical quantities—Allowance for rivet holes in parts in tension generally varies from one-third to one-fifth of the net section.—Chapter XI. contains formulæ for calculating the theoretic amount of material required for braced girders with horizontal flanges, when their length, depth, load and unit-strain are known. In order to render these formulæ of practical use in estimating girder-work, certain large additions derived from experience must be added to the theoretic quantities. If, for instance, the girder be made of wrought-iron, the formulæ are based on the supposition that the material is in one continuous piece whose whole section is equally effective for resisting strain. This is not the case in reality, for rivet holes in parts subject to tension, stiffners in those subject to compression, covers, packing, rivet heads and waste—all require certain additions to the theoretic quantities which experience alone can supply. When the general design is arranged it is easy to estimate the increased percentage of material arising from the weakening effect of rivet holes in parts subject to tension (**480**). In girder-work the allowance for rivet holes generally varies from one-third to one-fifth of the net sectional area according to the design; the larger allowance of one-third may be required for the tension diagonals of small girders; a medium allowance of one-fourth for the tension diagonals of large girders and the tension flanges of small ones; and an allowance of one-fifth for the tension flanges of large girders.

498. Allowance for stiffners in parts in compression varies according to their sectional area—Large compression flanges seldom require any allowance for stiffening—Compression bracing requires large percentages.—The additional percentage of material required to withstand flexure or buckling in parts subject to compression is not so easily estimated. It will generally be found to diminish in proportion as the area of the part increases, for when the area is considerable a stiff form of cross section may be given with little or no extra material. This is frequently the case with the compression flange, especially in large girders. Long compression braces, however, require much extra stiffening, and the amount of this varies within considerable limits. In the Boyne Lattice Bridge the extra material required to stiffen the compression braces varied from 60 to 128 per cent. of the theoretic amount (calculated at 4 tons per square inch) which would have been required to resist crushing merely if flexure had been left out of consideration, the higher percentages being required in the central diagonals whose scantlings were small, since they had to sustain but slight strains. In bridges above 250 feet span, with two main girders and a double line of railway, a sufficiently close approximation will generally be made if we assume the extra quantity of material to resist flexure in the compression bracing equal to as much again as the theoretic quantity calculated by the formulæ, but when the bridge is designed for a single line of railway this percentage is insufficient; perhaps in this case twice the theoretic quantity would generally be a safe allowance, as the extra quantity required for stiffening the compression bracing of a single line bridge is not widely different from that required for the double line.

499. Allowance for covers in flanges varies from 12 to 15 per cent. of the gross section.—The allowance for covers will also vary much with the design, long flange-plates requiring fewer covers than short ones (**466**), and piles of plates requiring a smaller percentage than cells (**468**). In the piled flanges of the Boyne lattice girders the covers formed about 12 per cent., or nearly $\frac{1}{8}$th, of the plates and angle iron. In the cellular flanges of the Conway tubular bridge the covers of the compression flange formed 5 per cent. of the plates and angle iron, and in the tension flange 28 per cent.; adding both flanges together the covers formed about 15 per cent.* of the plates and angle iron.

* *Clark on the Tubular Bridges*, p. 586.

500. Estimating girder-work a tentative process.—The process of estimating the quantities in any proposed bridge is tentative and depends upon experience, for it is necessary to assume a weight for the permanent bridge-load, and then make the calculations with the various practical allowances above mentioned. Now the resulting weight from this calculation may not agree with that which has been assumed. In this case the first estimate gives an approximation for a second calculation, and even a third may be necessary where great nicety is required. The following examples will illustrate this method of forming estimates:—

EXAMPLE 1.

501. Double-line lattice bridge 267 feet long.—I shall select for the first example a wrought-iron lattice bridge for a double line of railroad of the same length, depth and width as the central span of the Boyne Lattice Bridge, the weight of which is given in detail in the appendix. As the Boyne Bridge is a continuous girder in three spans, its central span, of course, requires less material than a bridge of equal dimensions in one unconnected span.

Let $l = 267$ feet $=$ the length measured from centre to centre of end pillars,

$d = \frac{l}{12} = 22{\cdot}25$ feet $=$ the depth,

$\theta = 45° =$ the angle of the bracing, whence $\sec\theta.\ \mathrm{cosec}\,\theta = 2$ (**276**),

$f = 5$ tons tensile inch-strain of net section,

$f' = 4$ tons compressive inch-strain of gross section,

and let the width of platform between the main girders equal 24 feet as in the Boyne Bridge. Let the maximum passing load equal 1 ton per running foot on each line, equal 534 tons when covering both lines together, and let us assume that the permanent bridge-load equals 490 tons, which gives the total load supported by the girders,

$$W = 534 + 490 = 1{,}024 \text{ tons.}$$

With this load uniformly distributed, the theoretic quantities of material (eqs. 202 and 204) are as follows, 4·6 cubic feet of wrought-iron being assumed equal to 1 ton.

	Tons.
Tension bracing $= \dfrac{1024 \times 267}{4 \times 5 \times 144} = 94{\cdot}93$ cubic feet,* -	20·64
Compression bracing ($\frac{5}{4}$ths of the tension bracing),	25·80
Tension flange $\left(\dfrac{l}{3d} \times \text{ tension bracing, eq. } 204\right)$, -	82·56
Compression flange ($\frac{5}{4}$ths of the tension flange), -	103·20
Total theoretic weight, - -	**232·20**

The true quantities are obtained from the foregoing by adding the percentages derived from experience, as follows.

	Tons.	Tons.
Theoretic tension bracing, - - - -	20·64	25·80
Rivet holes, say $\frac{1}{4}$th of net section, - -	5·16	
Theoretic compression bracing, - - -	25·80	51·60
Add as much again for stiffening, - - -	25·80	
Theoretic tension flange, - - - -	82·56	99·07
Rivet holes, say $\frac{1}{5}$th of net section, - -	16·51	
Covers of tension flange, say $\frac{1}{8}$th of flange, - -		12·38
Theoretic compression flange, - - - - -		103·20
Covers of compression flange, say $\frac{1}{8}$th of flange, -		12·90
		304·95
Rivet heads, packings, waste (**428, 427**), say 10 per cent.,		30·49
Weight of iron in the main girders, - - -		**335·44**
35 road girders, 7 feet 5 inches apart, each 1·32 tons (see Appendix, "Boyne Viaduct"),	46·20	63·86
Cross bracing (do. do.),	17·66	
Weight of iron between end pillars, - - -		**399·30**
6-inch planking of platform 24 feet wide, = 3,204 cubic feet, @ 50 cubic feet per ton,	64·08	90·65
Longitudinal timbers under rails, 12 inches × 6 inches = 534 cubic feet, - - -	10·68	
Barlow rails, 356 yards @ 100 lbs. per yard, -	15·89	
Permanent bridge-load between end pillars, -		**489·95**

* NOTE.—The theoretic quantity of material in the tension bracing is only one-half that given by eq. 202, which represents the quantity for the whole web.

being 0·05 tons less than that assumed. In order to obtain the total weight of wrought-iron in the bridge we must add the weight of the 4 end pillars with their 2 road girders, 2 top cross girders and gussets (**447**), say 30 tons in all, to the weight of iron between end pillars; this makes the **total weight of wrought-iron** in the structure = 399·30 + 30 = **429·30 tons.**

In this example we find that 335·44 tons of iron are required in the main girders to support themselves and an additional load of 688·56 tons uniformly distributed. Consequently each ton of additional load uniformly distributed requires $\frac{335 \cdot 44}{688 \cdot 56} = 0 \cdot 487$ tons of iron in the main girders, and if an additional load of 100 tons of ballast were spread over the platform we should add 48·7 tons of iron to the main girders to support the weight of this ballast without the unit-strains being increased.

502. Permanent strains — Strains from train-load — Economy due to continuity.—The permanent inch-strains, that is, the strains due to the permanent bridge-load of 489·95 tons are 2·39 tons tension and 1·91 tons compression; those due to the main girders alone, weighing 335·44 tons, are 1·64 tons tension and 1·31 tons compression, and those due to a train-load of one ton per running foot on each line uniformly distributed are 2·61 tons tension and 2·09 tons compression. The actual weight of iron in the main girders of the long central span of the Boyne Bridge = 297·41 tons; the difference between this and our example = 335·44 − 297·41 = 38·03 tons, which represents the saving effected in the central span of the Boyne Bridge by its connexion over the piers with the side spans. As, however, this connexion causes a certain loss of material in the shorter side spans, the total amount of economy produced by continuity is probably less than that above stated (**258**).

EXAMPLE 2.

503. Single-line lattice bridge 400 feet long.—A wrought-iron lattice bridge for a single line of railway, 400 feet long from centre to centre of end pillars, 25 feet deep and 14 feet wide between main girders, with the bracing at an angle of 45°. Using the same symbols as before, we have

$$l = 400 \text{ feet},$$
$$d = \frac{l}{16} = 25 \text{ feet},$$
$$\theta = 45°,$$
$$f = 5 \text{ tons tensile inch-strain of net section},$$
$$f' = 4 \text{ tons compressive inch-strain of gross section}.$$

Let the maximum train-load equal $\frac{3}{4}$ ton per running foot (**491**), and assuming that the permanent bridge-load equals 1,300 tons, we have the total distributed load,

$$W = 300 + 1{,}300 = 1{,}600 \text{ tons}.$$

The theoretic quantities with their empirical percentages are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{1{,}600 \times 400}{4 \times 5 \times 144} =$		
222·2 cubic feet, @ 4·6 cubic feet per ton, -	48·3	60·4
Rivet holes, say one-fourth of net section, -	12·1	
Theoretic compression bracing ($\frac{5}{4}$ths of theoretic tension bracing), - - - - - -	60·4	181·2
Add twice as much for stiffening, - - -	120·8	
Theoretic tension flange $= \dfrac{1{,}600 \times 400 \times 16}{12 \times 5 \times 144} =$		
1,185·18 cubic feet, @ 4·6 cubic feet per ton,	257·6	309·1
Rivet holes, say $\frac{1}{5}$th of net section, - - -	51·5	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		38·6
Theoretic compression flange ($\frac{5}{4}$ths of theoretic tension flange), - - - - - - - -		322·0
Covers, say $\frac{1}{8}$th of flange, - - - - - -		40·5
		951·8
Rivet heads, packings, waste, say 10 per cent., -		95·2
Iron in main girders, - - - - - - -		**1,047·0**

	Tons.
Road girders = $400 \times 0{\cdot}18$ tons (**450**), - - - -	72·0
Cross bracing, say - - - - - - - - -	35·0
Weight of iron between end pillars, - - -	**1,154·0**
Platform, rails, sleepers, and ballast = $400 \times 0{\cdot}36$ tons (**450**), - - - - - - - -	144·0
Permanent bridge-load between end pillars, -	**1,298·0**

being 2 tons less than that assumed. If the 4 end pillars and cross girders over the abutments weigh 40 tons, the **total weight of wrought-iron** in the bridge = $1{,}154 + 40 =$ **1,194 tons.** From this estimate it appears that 1,047 tons of iron are required in the main girders to support themselves and 553 tons in addition uniformly distributed; consequently each ton of additional load uniformly distributed requires for its support $\frac{1{,}047}{553} = 1{\cdot}89$ tons in the main girders. If, for instance, the maximum train-load be 1 ton in place of $\frac{3}{4}$ ton per running foot, this uniformly distributed load will amount to 400 tons in place of 300 tons, that is, 100 tons more than has been assumed, and this will require $100 \times 1{\cdot}89 = 189$ tons extra iron in the main girders for its support, and the increased total load on the bridge will be 289 tons, or nearly three times the useful addition.

504. Strains due to permanent load — Strains due to occasional load.—The iron in the flanges, including the 10 per cent. for rivet heads, packings and waste, weighs 781·2 tons; the iron in the web, also including the percentage for rivet heads, &c., weighs 265·8 tons; consequently, each ton of useful load uniformly distributed requires $\frac{781{\cdot}2}{553} = 1{\cdot}41$ tons of iron in the flanges, and $\frac{265{\cdot}8}{553} = 0{\cdot}48$ tons in the webs. The inch-strains due to the permanent bridge-load of 1,300 tons between end pillars are 4·06 tons tension and 3·25 tons compression, while those due to a uniformly distributed train-load of $\frac{3}{4}$ ton per running foot are 0·94 tons tension and 0·75 tons compression.

EXAMPLE 3.

505. Single-line lattice bridge 400 feet long, as in Ex. 2, but with higher unit-strains.—A wrought-iron lattice bridge of the same dimensions as the last, but in place of the inch-strains being 5 and 4 tons, let

$f = 6$ tons tensile inch-strain of net section,

$f' = 5$ tons compressive inch-strain of gross section.

Assuming that the permanent bridge-load = 960 tons, we have the total distributed load,

$$W = 300 + 960 = 1{,}260 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204),

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{1260 \times 400}{4 \times 6 \times 144} =$		
145·83 cubic feet, @ 4·6 feet per ton, - -	31·7	39·6
Rivet holes, say $\frac{1}{4}$th of net section, - -	7·9	
Theoretic compression bracing ($\frac{5}{6}$ths of theoretic tension bracing), - - - - - -	38·0	152·0
Add three times as much for stiffening,* - -	114·0	
Theoretic tension flange $= \dfrac{1260 \times 400 \times 16}{12 \times 6 \times 144} =$		
777·8 cubic feet, @ 4·6 cubic feet per ton, -	169·1	202·9
Rivet holes, say $\frac{1}{5}$th of net section, - -	33·8	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		25·4
Theoretic compression flange ($\frac{6}{5}$ths of theoretic tension flange), - - - - - - - -		202·9
Covers, say $\frac{1}{8}$th of flange - - - - - -		25·4
		648·2
Rivet heads, packings, waste, say 10 per cent., -		64·8
Iron in main girders, - - - - - - -		**713·0**

* In this example I allow three times, in place of twice, the theoretic amount, because the extra quantity of material required for stiffening the compression bracing is but slightly affected by the adoption of higher unit-strains.

	Tons.
Road girders, as in last example, - - - - -	72·0
Cross bracing, say, - - - - - - - -	30·0
Weight of iron between end pillars, - - -	**815·0**
Platform, rails, sleepers, and ballast, as in last, -	144·0
Permanent bridge-load between end pillars, -	**959·0**

being 1 ton less than that assumed.

If the 4 end pillars and cross girders over abutments weigh 35 tons, the **total weight of wrought-iron** in the bridge $= 815 + 35 =$ **850 tons**. The main girders in this example, weighing 713 tons, support themselves and an additional load of 547 tons uniformly distributed. Consequently each to nof useful load uniformly distributed requires for its support $\frac{713}{547} = 1{\cdot}304$ tons in the main girders. The inch-strains due to the permanent bridge-load of 960 tons between end pillars $= \frac{6 \times 960}{1260} = 4{\cdot}57$ tons tension, and $\frac{5 \times 960}{1260} = 3{\cdot}81$ tons compression, while those produced by a uniformly distributed train-load of $\frac{3}{4}$ ton per running foot are 1·43 tons tension and 1·19 tons compression. Comparing this with the preceding example, we find a saving in the main girders equal to 1,047—713 = 334 tons, or nearly 47 per cent. of the lighter bridge. The saving may even be greater than this, since I have neglected any reduction in the weight of the road-girders due to higher unit-strains.

506. Great economy from high unit-strains in long girders—Steel plates.—The two last examples illustrate the great economy produced in large girders by adopting high unit-strains. In place of the weights of the main girders being in the inverse ratio of the unit-strains, as might be supposed at first sight, we find that they vary in a much higher ratio, at least in large bridges where the main girders form a large proportion of the total load. Economy from the adoption of high unit-strains will be chiefly marked in the flanges and tension bracing, owing to the necessity of having a certain amount of material to stiffen the compression bracing, no matter how high the ultimate crushing strength of the material may be. Even a better method of riveting or jointing may produce a very important saving in a large girder by not requiring so many holes in the tension plates, or such large covers at the joints (**466**). Steel plates, which are now manufactured at a cost not much exceeding that of the better kinds of iron, but from once and a half to twice as strong as the latter, will, doubtless, enable the engineer to construct girders over spans which have been hitherto impracticable. The tensile strength of steel is known; it is to be hoped that satisfactory experiments will be made to determine its stiffness, that is, its strength to resist flexure when in the form of long pillars—an essential element in its application to girder-work (**485**).

507. Suspension principle applicable to larger spans than girders.—We are now in a position to understand how suspension bridges can be built over spans far exceeding those to which rigid girders are applicable, for not only are there no compressive strains in the webs of suspension bridges, but the compression flange the girder is superseded by land chains, and the structure between the piers is thus relieved of the weight of one flange. Moreover, the material used is generally of such an excellent quality that it is capable of sustaining with safety a higher unit-strain than ordinary plate-iron (**480**), and there is also a less percentage of material required for the joints of suspension chains, as pins passing through enlarged eyes in the ends of long links supersede the ever-recurring rivets of plated work, and the whole intermediate section of the link is thus available for tension without waste.

EXAMPLE 4.

508. Single-line lattice bridge 400 feet long, with increased depth.—The preceding example illustrates the great economy effected in large girders by the adoption of a high unit-strain. Let us now examine the result of a slight increase of depth, all the other dimensions and the unit-strains remaining the same as in Example 2, but in place of the depth being 25 feet, *i.e.*, one-sixteenth of the length, let

$$d = \frac{l}{15} = 26{\cdot}67 \text{ feet.}$$

Assuming the permanent bridge-load to be 1190 tons, we have the total distributed load,

$$W = 300 + 1190 = 1490 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{1490 \times 400}{4 \times 5 \times 144} =$		
207 cubic feet, @ 4·6 feet per ton, - -	45·0	56·2
Rivet holes, say $\frac{1}{4}$th of net section, - - -	11·2	
Theoretic compression bracing ($\frac{5}{4}$ths of theoretic tension bracing), - - - - - -	56·2	176·0
Add for stiffening the same as in Ex. 2,* -	120·8	
Theoretic tension flange $= \dfrac{1490 \times 400 \times 15}{12 \times 5 \times 144} =$		
1034·7 cubic feet, @ 4·6 feet per ton, - -	225·0	270·0
Rivet holes, say $\frac{1}{5}$th of net section, - - -	45·0	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		33·7
Theoretic compression flange ($\frac{5}{4}$ths of theoretic tension flange), - - - - - - - -		281·2
Covers, say $\frac{1}{8}$th of flange, - - - - - -		35·1
		852·2
Rivet heads, packings, waste, say 10 per cent., -		85·2
Iron in main girders, - - - - - - -		**937·4**
Road girders, as in Ex. 2, - - - - - -		72·0
Cross-bracing, do., - - - - - - -		35·0
Weight of iron between end pillars, - - -		**1044·4**
Platform, rails, sleepers, and ballast, as in Ex. 2, -		144·0
Permanent bridge-load between end pillars, -		**1188·4**

being 1·6 tons less than that assumed.

If the four end pillars and cross girders over the abutments weigh

* In place of adding, as usual, twice the theoretic amount for stiffening, viz., $2 \times 56 \cdot 2 = 112 \cdot 4$ tons, I have assumed that this example requires the same quantity as Ex. 2, for though the load in this example is less, yet the length of the compression bracing is greater than in Ex. 2, and the assumption in the text, therefore, will be probably not far from the truth.

40 tons, the **total weight of wrought-iron** in the bridge $= 1044 \cdot 4 + 40 =$ **1084·4 tons.** In this example the main girders, weighing 937·4 tons, support themselves and 552·6 tons uniformly distributed. Consequently each ton of useful load uniformly distributed requires for its support $\frac{937 \cdot 4}{552 \cdot 6} = 1 \cdot 7$ tons nearly in the main girders.

509. Permanent strains—Strains from train-load.—The inch-strains due to the permanent bridge-load of 1190 tons between end pillars $= \frac{5 \times 1190}{1490} = 4$ tons tension, and $\frac{4 \times 1190}{1490} = 3 \cdot 2$ tons compression. The inch-strains due to the main girders, weighing 937·4 tons, $= \frac{5 \times 937 \cdot 4}{1490} = 3 \cdot 14$ tons tension, and $\frac{4 \times 937 \cdot 4}{1490} = 2 \cdot 52$ tons compression. The inch-strains due to a train-load of $\frac{3}{4}$ tons per running foot over the whole bridge $= \frac{5 \times 300}{1490} = 1 \cdot 0$ ton tension, and $\frac{4 \times 300}{1490} = 0 \cdot 8$ tons compression.

510. Weights of large girders do not vary inversely as their depth.—Comparing this with Ex. 2, the saving of material in the main girders $= 1047 - 937 \cdot 4 = 109 \cdot 6$ tons. We find therefore that the weights of the girders in these two examples are inversely as the 1·7 power of the depths, but this particular proportion is accidental (**519**).

EXAMPLE 5.

511. Single-line lattice bridge 480 feet long.—A wrought-iron lattice bridge for a single line of railway, 480 feet long from centre to centre of end pillars, 30 feet deep, and 14 feet wide between main girders. Using the same symbols as in Ex. 1, we have,

$l =$ the length $= 480$ feet,

$d =$ the depth $= \frac{l}{16} = 30$ feet,

$\theta = 45° =$ the angle the diagonals make with a vertical line,

$f = 5$ tons tensile inch-strain of net section,

$f' = 4$ tons compressive inch-strain of gross section.

Let the maximum passing load $= \frac{3}{4}$ ton per running foot (**491**), and assuming that the permanent bridge-load weighs 2760 tons, we have the total distributed load,

$$W = 360 + 2760 = 3120 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{3120 \times 480}{4 \times 5 \times 144} = 520$		
cubic feet, @ 4·6 feet per ton, - -	113·0	141·3
Rivet holes, say $\frac{1}{4}$th of net section, - -	28·3	
Theoretic compression bracing ($\frac{5}{4}$ths of theoretic tension bracing), - - - - - -	141·3	423·9
Add twice as much for stiffening,* - -	282·6	
Theoretic tension flange $= \dfrac{3120 \times 480 \times 16}{12 \times 5 \times 144} =$		
2773·3 cubic feet, @ 4·6 feet per ton, -	602·9	723·5
Rivet holes, say $\frac{1}{5}$th of net section, - -	120·6	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		90·4
Theoretic compression flange ($\frac{5}{4}$ths of theoretic tension flange), - - - - - - - -		753·6
Covers, say $\frac{1}{8}$th of flange, - - - - - -		94·2
		2226·9
Rivet heads, packings, waste, say 10 per cent., -		222·7
Iron in main girders, - - - - - - -		**2449·6**
Road girders $= 480 \times 0·18$ tons (**450**), - -		86·4
Cross-bracing,† - - - - - - - - -		50·4
Weight of iron between end pillars, - - -		**2586·4**

* This allowance for stiffening is probably excessive.

† The quantity of cross-bracing is proportional to Wl (eq. 202), where W represents the pressure of the wind against the side of the bridge; if this pressure be assumed proportional to the product of length and depth, which is the case in plate girders, the quantity of cross-bracing in similar girders will vary as l^3. As, however, the side surface of similar lattice girders does not in general increase so rapidly as l^2, and as also the empirical percentages are somewhat less in large than in small bridges, it will probably be nearer the truth to assume that the quantity of cross-bracing is proportional to the square of the length. If, therefore, a bridge 400 feet long (Ex. 2), require 35 tons, one 480 feet long will require $35 \times \frac{36}{25} = 50·4$ tons.

	Tons.
Weight of iron between end pillars, - -	**2586·4**
Platform, rails, sleepers, and ballast = 480 × 0·36 tons (**450**), - - - - - - -	172·8
Permanent bridge-load between end pillars, -	**2759·2**

being 0·8 tons less than that assumed. If the weight of the four pillars and cross girders at the ends be assumed equal to 70 tons, the **total weight of wrought-iron** in the bridge will equal 70 + 2586·4 = **2656·4 tons** (**447**).

512. Permanent strains — Strains from train-load. — The inch-strains due to the permanent bridge-load of 2760 tons between end pillars are $\frac{5 \times 2760}{3120} = 4{\cdot}42$ tons tension, and $\frac{4 \times 2760}{3120} = 3{\cdot}54$ tons compression. The inch-strains due to the main girders, weighing 2449·6 tons, are $\frac{5 \times 2449{\cdot}6}{3120} = 3{\cdot}92$ tons tension, and $\frac{4 \times 2449{\cdot}6}{3120} = 3{\cdot}14$ tons compression. The inch-strains due to a train-load of $\frac{3}{4}$ ton per running foot over the whole bridge $= \frac{5 \times 360}{3120} = 0{\cdot}576$ tons tension, and $\frac{4 \times 360}{3120} = 0{\cdot}46$ tons compression.

513. Waste of material in defective designs.—In this example 2449·6 tons of iron in the main girders support themselves and an additional load of 670·4 tons uniformly distributed over the bridge. Consequently each ton of useful load requires for its support $\frac{2449{\cdot}6}{670{\cdot}4} = 3{\cdot}65$ tons of iron in the main girders. This illustrates the great waste of material produced by defective designs for large bridges, since every ton of iron uselessly added involves the necessity of adding 3·65 other tons for its support, making collectively upwards of $4\frac{1}{2}$ tons which might be saved were the design skilfully planned.

Example 6.

514. Single-line lattice bridge 480 feet long, as in Ex. 5, but with higher unit-strains.—A wrought-iron lattice bridge of the same dimensions as the last, but in place of the inch-strains being 5 and 4 tons respectively,

Let $f = 6$ tons tensile inch-strain of net section,

$f' = 5$ tons compressive inch-strain of gross section.

Assuming that the permanent bridge-load equals 1710 tons, we have the total distributed load,

$$W = 360 + 1710 = 2070 \text{ tons.}$$

The quantities are as follows (eq. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{2070 \times 480}{4 \times 6 \times 144} =$		
287·5 cubic feet, @ 4·6 feet per ton, -	62·5	78·1
Rivet holes, say $\frac{1}{4}$th of net section, - -	15·6	
Theoretic compression bracing ($\frac{6}{5}$ths of theoretic tension bracing), - - - - - -	75·0	300·0
Add three times as much for stiffening,* -	225·0	
Theoretic tension flange $= \dfrac{2070 \times 480 \times 16}{12 \times 6 \times 144} =$		
1533·3 cubic feet, @ 4·6 feet per ton, -	333·3	400·0
Rivet holes, say $\frac{1}{5}$th of net section, - -	66·7	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		50·0
Theoretic compression flange ($\frac{6}{5}$ths of theoretic tension flange), - - - - - - - -		400·0
Covers, say $\frac{1}{8}$th of flange, - - - - - -		50·0
		1278·1
Rivet heads, packings, waste, say 10 per cent., -		127·8
Iron in main girders, - - - - - - -		**1405·9**
Road girders, as in last example, - - - -		86·4
Cross-bracing, say, - - - - - - -		45·0
Weight of iron between end pillars, - - -		**1537·3**
Platform, rails, sleepers, and ballast, as in last, -		172·8
Permanent bridge-load between end pillars, -		**1710·1**

being 0·1 ton greater than that assumed.

* See note to Ex. 3, p. 420.

If the four pillars and cross girders at the ends weigh 50 tons, the **total weight of wrought-iron** in the bridge will equal $50 + 1537 \cdot 3 =$ **1587·3 tons**. In this example the main girders, weighing 1405·9 tons, support themselves and an additional load of 664·1 tons uniformly distributed. Consequently each ton of useful load requires for its support $\frac{1405 \cdot 9}{664 \cdot 1} = 2 \cdot 117$ tons in the main girders.

515. Permanent strains—Strains from train-load.—The inch-strains due to the permanent bridge-load of 1710 tons between end pillars $= \frac{6 \times 1710}{2070} = 4 \cdot 96$ tons tension, and $\frac{5 \times 1710}{2070} = 4 \cdot 13$ tons compression. The inch-strains due to the main girders, weighing 1405·9 tons, are $\frac{6 \times 1405 \cdot 9}{2070} = 4 \cdot 08$ tons tension, and $\frac{5 \times 1405 \cdot 9}{2070} = 3 \cdot 4$ tons compression. The inch-strains due to a uniformly distributed train-load of $\frac{3}{4}$ ton per running foot over the whole bridge are $\frac{6 \times 360}{2070} = 1 \cdot 04$ tons tension, and $\frac{5 \times 360}{2070} = 0 \cdot 87$ tons compression.

516. Great economy from high unit-strains in large girders.—The economy effected in large girders by the adoption of high unit-strains is very marked in this example. Compared with the preceding example, the saving amounts to $2656 \cdot 4 - 1587 \cdot 3 = 1069 \cdot 1$ tons, or nearly 68 per cent. of the lighter bridge.

EXAMPLE 7.

517. Single-line lattice bridge 480 feet long, as in Ex. 5, but with increased depth.—The previous example illustrates the great economy in large bridges due to the use of a material capable of sustaining high unit-strains with safety. We shall now examine the effect of a slight increase of depth, all the other dimensions and the unit-strains remaining the same as in Ex. 5. In place of the depth being 30 feet, or $\frac{1}{16}$th of the length, let

$$d = \frac{l}{15} = 32 \text{ feet.}$$

Assuming the permanent bridge-load to be 2435 tons, we have the total distributed load,

$$W = 360 + 2435 = 2795 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{2795 \times 480}{4 \times 5 \times 144} =$		
465·8 cubic feet, @ 4·6 feet per ton, - -	101·3	126·6
Rivet holes, say $\frac{1}{4}$th of net section, - -	25·3	
Theoretic compression bracing ($\frac{5}{4}$ths of theoretic tension bracing), - - - - - -	126·6	409·2
Add for stiffening the same as in Ex. 5,* -	282·6	
Theoretic tension flange $= \dfrac{2795 \times 480 \times 15}{12 \times 5 \times 144} =$		
2329 cubic feet, @ 4·6 feet per ton, - -	506·3	607·6
Rivet holes, say $\frac{1}{5}$th of net section, - - -	101·3	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		76·0
Theoretic compression flange ($\frac{5}{4}$ths of theoretic tension flange), - - - - - - - -		632·9
Covers, say $\frac{1}{8}$th of flange, - - - - - -		79·1
		1931·4
Rivet heads, packings, waste, say 10 per cent., -		193·1
Iron in main girders, - - - - - - -		**2124·5**
Road girders, as in Ex. 5, - - - - - -		86·4
Cross-bracing, do., - - - - - -		50·4
Weight of iron between end pillars, - - -		**2261·3**
Platform, rails, sleepers, and ballast, as in Ex. 5, -		172·8
Permanent bridge-load between end pillars, -		**2434·1**

being 0·9 ton less than that assumed.

If the four pillars and cross girders at the ends weigh 70 tons, the **total weight of wrought-iron** in the bridge will equal $70 + 2261·3 =$ **2331·3 tons**. The main girders, weighing 2124·5 tons, support themselves and 670·5 tons uniformly distributed.

* See note to Ex. 4, p. 423.

Consequently each ton of useful load uniformly distributed requires for its support $\frac{2124 \cdot 5}{670 \cdot 5} = 3 \cdot 17$ tons in the main girders.

518. Permanent strains—Strains due to train-load.—The inch-strains due to the permanent bridge-load of 2434 tons between end pillars $= \frac{5 \times 2434}{2795} = 4 \cdot 35$ tons tension, and $\frac{4 \times 2434}{2795} = 3 \cdot 48$ tons compression. The inch-strains due to the main girders, weighing 2124·5 tons $= \frac{5 \times 2124 \cdot 5}{2795} = 3 \cdot 8$ tons tension, and $\frac{4 \times 2124 \cdot 5}{2795} = 3 \cdot 04$ tons compression. The inch-strains due to a train-load of $\frac{3}{4}$ ton per running foot over the whole bridge $= \frac{5 \times 360}{2797} = 0 \cdot 64$ tons tension, and $\frac{4 \times 360}{2797} = 0 \cdot 51$ tons compression.

519. Weights of large girders do not vary inversely as their depth.—Comparing this with Ex. 5, the saving effected in the main girders by a slight increase of depth $= 2449 \cdot 6 - 2124 \cdot 5 = 325 \cdot 1$ tons. We find also that the weights of the girders in these two examples are inversely as the 2·2 power of their depths (**509**).

Example 8.

520. Single-line lattice bridge 600 feet long.—A wrought-iron bridge for a single-line of railway, 600 feet long between centres of end pillars, 37·5 feet deep, and 14 feet wide between main girders. Using the same symbols as in Ex. 1, we have,

$$l = 600 \text{ feet,}$$

$$d = \frac{l}{16} = 37 \cdot 5 \text{ feet,}$$

$$\theta = 45^\circ,$$

$$f = 5 \text{ tons tensile inch-strain of net section,}$$

$$f' = 4 \text{ tons compressive inch-strain of gross section.}$$

Let the maximum passing load $= \frac{3}{4}$ ton per running foot, and assuming that the permanent bridge-load weighs 9100 tons, we have the total distributed load,

$$W = 450 + 9100 = 9550 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \dfrac{9550 \times 600}{4 \times 5 \times 144} =$		
1989·6 cubic feet, @ 4·6 feet per ton,	432·5	540·6
Rivet holes, say $\frac{1}{4}$th of net section,	108·1	
Theoretic compression bracing ($\frac{5}{4}$ths of theoretic tension bracing),	540·6	1081·2
Add as much again for stiffening,*	540·6	
Theoretic tension flange $= \dfrac{9550 \times 600 \times 16}{12 \times 5 \times 144} =$		
10,611 cubic feet, @ 4·6 feet per ton,	2306·7	2768·0
Rivet holes, say $\frac{1}{5}$th of net section,	461·3	
Covers, say $\frac{1}{8}$th of flange,		346·0
Theoretic compression flange ($\frac{5}{4}$ths of the theoretic tension flange),		2883·4
Covers, say $\frac{1}{8}$th of flange,		360·8
		7980·0
Rivet heads, packings, waste, say 10 per cent.,		798·0
Iron in main girders,		**8778·0**
Road girders $= 600 \times 0·18$ tons (**450**),		108·0
Weight of iron between end pillars,		**8886·0**
Platform, rails, sleepers, and ballast $= 600 \times 0·36$ tons (**450**),		216·0
Permanent bridge-load between end pillars,		**9102·0**

being 2 tons in excess of that assumed.

No allowance has been made for cross-bracing, for the sectional area of the flanges is so great that they would probably extend over the whole space between the main girders so as to form a tubular bridge (**18**), and thus supersede the usual cross-bracing formed of cross-beams and diagonal tension bars. If the four end

* The quantity of material in the web is so large that it can be thrown into a form suitable for resisting flexure without much extra stiffening; I have therefore added only half the percentage for stiffening that was adopted in most of the preceding cases.

pillars and cross-girders at the ends be assumed equal to 200 tons, the **total weight of wrought-iron** in the bridge will equal 200 + 8886 = **9086 tons**. In this example 8778 tons of iron in the main girders support themselves and an additional load of 772 tons uniformly distributed over the bridge. Consequently each ton of useful load requires for its support $\frac{8778}{772} = 11{\cdot}37$ tons of iron in the main girders.

521. Permanent strains—Strains due to train-load.—The inch-strains due to the permanent bridge-load of 9100 tons between end pillars are $\frac{5 \times 9100}{9550} = 4{\cdot}76$ tons tension, and $\frac{4 \times 9100}{9550} = 3{\cdot}81$ tons compression. The inch-strains due to the main girders, weighing 8778 tons, are $\frac{5 \times 8778}{9550} = 4{\cdot}6$ tons tension, and $\frac{4 \times 8778}{9550} = 3{\cdot}67$ tons compression. The inch-strains due to a train-load of $\frac{3}{4}$ ton per running foot over the whole bridge $= \frac{5 \times 450}{9550} = 0{\cdot}235$ tons tension, and $\frac{4 \times 450}{9550} = 0{\cdot}188$ tons compression.

EXAMPLE 9.

522. Single-line lattice bridge 600 feet long, as in Ex. 8, but with higher unit-strains.—A wrought-iron bridge of the same dimensions as the last, but in place of the inch-strains being 5 and 4 tons,

Let $f = 6$ tons tensile inch-strain of net section,

$f' = 5$ tons compressive inch-strain of gross section.

Assuming that the permanent bridge-load = 3800 tons, we have the total distributed load,

$$W = 450 + 3800 = 4250 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \frac{4250 \times 600}{4 \times 6 \times 144} =$		
737·8 cubic feet, @ 4·6 feet per ton, - -	160·4	200·5
Rivet holes, say $\frac{1}{4}$th of net section. - -	40·1	

	Tons.	Tons.
Theoretic compression bracing ($\frac{6}{5}$ths of the theoretic tension bracing) - - - -	192·5	577·5
Add twice as much for stiffening, - - -	385·0	
Theoretic tension flange $= \frac{4250 \times 600 \times 16}{12 \times 6 \times 144} =$ 3935·2 cubic feet, @ 4·6 feet per ton, -	855·5	1026·6
Rivet holes, say $\frac{1}{5}$th of net section, - -	171·1	
Covers, say $\frac{1}{8}$th of flange, - - - - - -		128·3
Theoretic compression flange ($\frac{6}{5}$ths of theoretic tension flange), - - - - - - -		1026·6
Covers, say $\frac{1}{8}$th of flange, - - - - - -		128·3
		3087·8
Rivet heads, packings, waste, say 10 per cent., -		308·8
Iron in main girders, - - - - - - -		**3396·6**
Road girders, as in last example, - - - - -		108·0
Cross-bracing,* - - - - - - - - -		78·8
Weight of iron between end pillars, - - -		**3583·4**
Platform, rails, sleepers, and ballast, as in last example, - - - - - - - - -		216·0
Permanent bridge-load between end pillars, -		**3799·4**

being 0·6 tons less than that assumed.

If the four pillars and cross girders at the ends weigh 100 tons, the **total weight of wrought-iron** in the bridge will equal $100 \times 3583{\cdot}4 =$ **3683·4 tons.** In this example the main girders, weighing 3396·6 tons, support themselves and an additional load of 853·4 tons uniformly distributed. Consequently each ton of useful load requires for its support $\frac{3396{\cdot}6}{853{\cdot}4} = 3{\cdot}98$ tons in the main girders.

528. Permanent strains—Strains due to train-load.—The inch-strains due to the permanent bridge-load of 3800 tons between end pillars $= \frac{6 \times 3800}{4250} = 5{\cdot}36$ tons tension, and $\frac{5 \times 3800}{4250} = 4{\cdot}47$ tons compression. The inch-strains due to the main

* See note to Example 5, p. 425.

girders, weighing 3396·6 tons, are $\frac{6 \times 3396 \cdot 6}{4250} = 4 \cdot 8$ tons tension, and $\frac{5 \times 3396 \cdot 6}{4250} = 4 \cdot 0$ tons compression. The inch-strains due to a uniformly distributed train-load of $\frac{3}{4}$ ton per running foot over the whole bridge $= \frac{6 \times 450}{4250} = 0 \cdot 64$ tons tension, and $\frac{5 \times 450}{4250} = 0 \cdot 53$ tons compression.

524. Great economy from high unit-strains in very large girders.—The economy due to the adoption of high unit-strains in girders of great size, whose permanent weight forms by far the larger portion of the total load, is very conspicuous in this example. Compared with the preceding example, the saving amounts to $9086 - 3683 \cdot 4 = 5402 \cdot 6$ tons, or nearly 147 per cent. of the lighter bridge (**516**).

Example 10.

525. Single-line lattice bridge, 600 feet long, as in Ex. 8, but with increased depth.—Let us now examine the effect of a slightly increased proportion of depth to span. In Ex. 8 the depth is $\frac{1}{16}$th of the length; let the proportion now be $\frac{1}{15}$th, and retaining all the other dimensions and unit-strains as before, we have

$$l = 600 \text{ feet,}$$
$$d = \frac{l}{15} = 40 \text{ feet,}$$
$$\theta = 45°,$$
$$f = 5 \text{ tons tensile inch-strain of net section,}$$
$$f = 4 \text{ tons compressive inch-strain of gross section.}$$

Let the passing load equal $\frac{3}{4}$ ton per running foot, and assuming the permanent bridge-load to equal 6800 tons, we have the total distributed load,

$$W = 450 + 6800 = 7250 \text{ tons.}$$

The quantities are as follows (eqs. 202, 204).

	Tons.	Tons.
Theoretic tension bracing $= \frac{7250 \times 600}{4 \times 5 \times 144} =$		
1510·4 cubic feet, @ 4·6 feet per ton,	328·4	410·5
Rivet holes, say $\frac{1}{4}$th of net section, - -	82·1	

	Tons.	Tons.
Theoretic compression bracing ($\frac{5}{4}$ths of theoretic tension bracing), - - -	410·5	951·1
Add for stiffening the same as in ex. 8,* -	540·6	
Theoretic tension flange $= \frac{7250 \times 600 \times 15}{12 \times 5 \times 144} =$		
7552·1 cubic feet, @ 4·6 feet per ton,	1641·8	1970·2
Rivet holes, say $\frac{1}{5}$th of net section, - -	328·4	
Covers, say $\frac{1}{8}$th of the flange, - - - -		246·3
Theoretic compression flange ($\frac{5}{4}$ths of theoretic tension flange), - - - - - -		2052·2
Covers, say $\frac{1}{8}$th of flange, - - - - -		256·5
		5886·8
Rivet heads, packings, waste, say 10 per cent., -		588·7
Iron in main girders, - - - - -		**6475·5**
Road girders, as in Ex. 8, - - - - -		108·0
Weight of iron between end pillars, - - -		**6583·5**
Platform, rails, sleepers, ballast, as in Ex. 8, - -		216·0
Permanent bridge road between end pillars, -		**6799·5**

being 1·5 tons less than that assumed.

If the four pillars and cross girders at the ends weigh 160 tons, the **total weight of wrought-iron** in the bridge will equal $160 + 6583{\cdot}5 =$ **6743·5 tons.**

The main girders, weighing 6475·5 tons support themselves and 774·5 tons uniformly distributed. Consequently each ton of useful load uniformly distributed requires for its support $\frac{6475{\cdot}5}{774{\cdot}5} = 8{\cdot}36$ tons in the main girders.

526. Permanent Strains — Strains due to train-load. — The inch-strains due to the permanent bridge-load of 6800 tons between end pillars $= \frac{5 \times 6800}{7250} = 4{\cdot}69$ tons tension, and $\frac{4 \times 6800}{7250} = 3{\cdot}75$ tons compression. The inch-strains due to the main

* See note to Ex. 4, p. 423.

girders weighing 6475·5 tons are $\frac{5 \times 6475 \cdot 5}{7250} = 4 \cdot 47$ tons tension, and $\frac{4 \times 6475 \cdot 5}{7250} = 3 \cdot 57$ tons compression. The inch-strains due to a uniformly distributed train-load of $\frac{3}{4}$ ton per running foot over the whole bridge are $\frac{5 \times 450}{7250} = 0 \cdot 31$ tons tension, and $\frac{4 \times 450}{2250} = 0 \cdot 248$ tons compression.

527. Weights of large girders do not vary inversely as any definite power of their depth.—From this example we see that very considerable economy is effected in girders of great size, whose permanent weight forms the larger portion of the total load, by increasing the ratio of depth to length, even in a slight degree. Compared with Example 8, the saving in the main girders = 8778 — 6475·5 = 2302·5 tons, and the weight of these girders are inversely as the 4·7 power of their depths.

528. Extra bracing required for passing loads cannot be neglected in small bridges.—The examples given in the preceding pages are those of large bridges, exceeding 250 feet in span, in which the permanent bridge-load forms such a large portion of the total load that I have neglected the extra material required for counterbracing the web so as to enable it to meet the maximum strains produced by the passing load when in motion. This is allowable, since the empirical additions for stiffening the compression bracing are probably in excess of those actually required in large girders. In short girders, however, it is necessary to make some allowance in the bracing for the load being in motion in place of being uniformly distributed, and there is, moreover, a greater proportion of waste both in the flanges near the ends and in the web near the centre, than in large girders (**428, 427**). Hence the allowance for waste, &c., will be more than 10 per cent.

The following example will illustrate this matter:—

EXAMPLE 11.

529. Single-line lattice bridge 108 feet long.—A wrought-iron lattice bridge for a single line of railway, 108 feet long, 9 feet deep, and 14 feet wide between main girders. Using the same symbols as in Ex. 1, we have

$l = 108$ feet,

$d = \frac{l}{12} = 9$ feet,

$\theta = 45°$,

$f = 5$ tons tensile inch-strain of net section,

$f' = 4$ tons compressive inch-strain of gross section in the flanges, and 3 tons in the bracing (**481**).

Let the maximum passing load = 1·32 tons per running foot (**492**), and assuming that the permanent bridge-load = 105 tons, we have the total distributed load,

$$W = 143 + 105 = 248 \text{ tons.}$$

The quantities are as follows (eqs. 102, 104).

	Tons.	Tons.
Theoretic tension bracing $= \frac{248 \times 108}{4 \times 5 \times 144} =$		
9·3 cubic feet, @ 4·6 feet per ton, - -	2·02	2·69
Rivet holes, say ⅓rd of net section, - -	·67	
Theoretic compression bracing, (⅔rds of the theoretic tension bracing), - - -	3·37	10·11
Add twice as much for stiffening and counter-bracing (**528**), - - - - - - -	6·74	
Theoretic tension flange $= \frac{248 \times 108 \times 12}{12 \times 5 \times 144} =$		
37·2 cubic feet, @ 4·6 feet per ton, - -	8·09	10·11
Rivet holes, say ¼th of net section, - -	2·02	
Covers, say ⅙th of the flange,* - - - - -		1·68
Theoretic compression flange (⅘ths of the theoretic tension flange), - - - - -		10·11
Covers, say ⅙th of the flange, - - - -		1·68
		36·38
Rivet heads, packings, waste, say 25 per cent. (**528**),		9·09
Iron in main girders, - - - - - -		**45·47**

* In large girders it is important to diminish the dead load as much as possible, and it is therefore worth paying extra for large plates so as to diminish the percentage for covers (**466**). This, however, is not the case with small girders; hence the percentage of covers is larger in this than in the preceding examples.

	Tons.	Tons.
Iron in main girders, - - - - - -		**45·47**
Road girders = 108 × 0·18 tons (**450**), - - -		19·44
Cross-bracing, say, - - - - - - -		1·00
Iron between end pillars, - - - - - -		**65·91**
Platform, rails, sleepers, ballast = 108 × 0·36 tons (**450**), - - - - - - - -		38·88
Permanent bridge-load between end pillars, -		**104·79**

being 0·21 ton less than that assumed. If the four end pillars weigh $1\frac{1}{2}$ ton the **total weight of wrought-iron** in the bridge will equal 65·91 + 1·5 = **67·41 tons.** In this example the main girders, weighing 45·47 tons, support themselves and an additional load of 202·53 tons uniformly distributed over the bridge. Consequently each ton of useful load uniformly distributed requires for its support $\frac{45{\cdot}47}{202{\cdot}53} = 0{\cdot}2245$ tons of iron in the main girders.

580. Permanent strains—Strains due to train-load.—The inch-strains in the flanges, due to the permanent bridge-load of 105 tons are $\frac{5 \times 105}{248} = 2{\cdot}12$ tons, tension and $\frac{4 \times 105}{248} = 1{\cdot}7$ tons compression. The inch-strains due to the main girders alone weighing 45·47 tons are $\frac{5 \times 45{\cdot}47}{248} = 0{\cdot}92$ tons tension, and $\frac{4 \times 45{\cdot}47}{248} = 0{\cdot}73$ tons compression. The inch-strains in the flanges due to a uniformly distributed train-load of 1·32 tons per running foot over the whole bridge are $\frac{5 \times 143}{248} = 2{\cdot}88$ tons tension, and $\frac{4 \times 143}{248} = 2{\cdot}3$ tons compression.

581. Error in assuming the permanent load uniformly distributed in large girders.—In the foregoing examples it has been tacitly assumed that the weight of the main girders is uniformly distributed. This is erroneous, because there is a preponderance of material in the flanges at the centre. It is true that the amount of bracing, both in the web and in the horizontal bracing, increases towards the ends and thus to a great degree compensates for the variation of section in the flanges. Still the difficulty remains in the case of very large girders whose own weight forms the greater portion of the total load, and this preponderance of flange weight near the centre is the chief reason why single girders are less economical than continuous ones when the span is very great (**258**).

582. Empirical percentages open to improvement.—The empirical percentages adopted in the foregoing examples may perhaps be objected to, and it must be confessed that they are liable both to criticism and to correction from future experience. I have, however, made the most of the few recorded facts on which dependence can be placed, and would here suggest to my brother engineers that they should, as opportunity occurs, place on record in a tabular form the detailed weights of wrought-iron and steel girders, in order that this branch of our practice may attain that amount of precision that such statistical information alone can supply. In furtherance of this desirable object I have added in the Appendix the detailed weights of the Boyne Tubular Lattice Bridge, which I collected when resident there, and also the details of the Conway Tubular Plate Bridge, and the principal features of a few others.

The examples in the present chapter indicate the direction in which improvements in constructive detail may be sought with most prospect of success. In very large girders this is a matter of great importance, for even a very slight diminution of any of the empirical percentages may effect a large amount of economy.

583. Fatigue of the material greater in long than in short bridges.—Though the maximum unit-strains may be the same in two bridges, one long and the other short, the permanent unit-strains, that is, the *fatigue* of the material from the permanent load (**474**), will be much higher in the bridge of great span. Thus, comparing **504** and **580**, we find that the fatigue or permanent inch-strains of a railway bridge 400 feet long are 4·06 tons tension and 3·25 tons compression, while the corresponding inch-strains of a bridge 108 feet long are 2·12 tons tension and 1·7 tons compression. If iron possessed unlimited viscidity, that is, the property of slowly and continuously changing shape like pitch under prolonged strains of moderate extent, it seems reasonable to suppose that the longer bridge would fail sooner than the short one in consequence of its progressive deflection increasing more rapidly (405). Experience does not favour this hypothesis, for though experiments render it probable that all ductile metals will change shape to an unlimited extent under enormous pressure, in this respect resembling plastic clay, it seems equally certain that no continuous deformation takes place in structures whose unit-strains are kept well within the limits of elasticity (412.) Again, it is conceivable, nay probable, that severe fatigue (especially if aided by vibration) may so alter the constitution of iron as to weaken parts in tension, either by rendering them brittle or by actually diminishing their tensile strength (410). If this were the case within the limits of strain which occur in practice the longer bridge should still fail first. If, on the other hand, large fluctuations in the amount of strain affect the molecular condition of iron injuriously, and produce a tendency to rupture, then the short bridge should fail sooner. The experiments recorded in Chap. XXVI. will prevent anxiety in either case when the working strains do not exceed those in usual practice.

GIRDERS UNDER 200 FEET IN LENGTH.

584. Flanges nearly equal in weight to each other, and web nearly equal in weight to one flange.—When an iron lattice girder of the ordinary proportions of length to depth does not exceed 200 feet in span, the flanges are very nearly equal in weight to each other (481), and the web is very nearly equal in weight to one flange. Moreover, the quantity of material in the compression flange is nearly equal to its theoretic central area multiplied by its length; for though in correct practice the section of the flange is reduced towards the ends, it so happens that the empirical allowance for covers, rivet heads, packings and waste, that is, the difference between the actual and the theoretic flange, is compensated for by assuming that the flange has its theoretic central area carried uniform throughout the whole length. Hence we have the following empirical formula for the quantity of material in the main girders which will be found convenient in practice.

$$\mathrm{G} = \frac{3al}{4\cdot 6} = \frac{2}{3}al \text{ nearly,} \tag{230}$$

where G = the weight of the main girders and end pillars in tons,
a = the theoretic area of the compression flange at the centre in square feet,
l = the length in feet,
$4{\cdot}6$ = the number of cubic feet of wrought-iron in one ton.

For girders loaded uniformly we have (eq. 26), $a = \frac{Wl}{8fd}$, whence, by substitution in eq. 230,

$$G = \frac{Wl^2}{12fd} \tag{231}$$

where W = the total distributed load in tons, including the weight of the girder,
l = the length in feet,
d = the depth in feet,
f = the working-strain in tons per *square foot* of gross section.*

Ex. In Ex. 11, for instance, $G = \frac{248 \times (108 \times 108)}{12 \times (4 \times 144) \times 9} = 46{\cdot}5$ tons, which is but very slightly less than the former result.

535. Anderson's rule—Table of weights of railway girders under 200 feet in length.—I am indebted to William Anderson, Esq., for the following simple rule, derivable from eq. 231, for approximate estimates of railway bridges under 200 feet in length, whose depth is $\frac{1}{12}$th of their length, and whose working inch-strains are 5 tons tension and 4 tons compression. *Multiply the distributed load in tons by* 4, *and the product is the weight of the main girders, end pillars, and cross-bracing, in lbs. per running foot.*

Ex. 1. Thus, in Ex. 11, the total distributed load equals 248 tons; then $4 \times 248 = 992$ lbs. = the weight of main girders, end pillars, and cross bracing per running foot, and their total weight $= \frac{992 \times 108}{2240} = 47{\cdot}8$ tons, which agrees very closely with the former result.

The following table contains the weights of wrought-iron lattice girders for railway bridges up to 200 feet in length, calculated by

* The reader will recollect that the usual tensile working-strain of iron, namely, 5 tons per square inch of *net* section, practically requires the same gross area as the usual compressive working-strain of 4 tons per square inch of *gross* section (481).

the foregoing rule for the three different standard working loads described in **493**. In making use of this table the reader will bear in mind the following conditions.

a. The working strains in the flanges are 5 tons per square inch of net section for tension, and 4 tons per square inch of gross section for compression.

b. The proportion of depth to length = $\frac{1}{12}$.

c. The dead weight of cross-girders, platform, ballast, sleepers, and rails = 0·54 tons per running foot of single line (**450**).

d. The weight of main girders for a double-line bridge is twice that given in the table for a single-line bridge.

e. It is probable that the weights in the table for the longer bridges, say above 140 feet, are rather in excess of truth.

TABLE I.—WEIGHTS OF MAIN GIRDERS, END PILLARS, AND CROSS-BRACING OF SINGLE-LINE LATTICE RAILWAY BRIDGES OF VARIOUS LENGTHS UP TO 200 FEET, THE DEPTH BEING $\frac{1}{12}$TH OF THE LENGTH.

Length of Bridge from centre to centre of bearings.	Weight of Main Girders, End Pillars, and Cross Bracing.		
	Standard load on a 100 foot Bridge = 1 ton per foot.	Standard load on a 100 foot Bridge = 1¼ ton per foot.	Standard load on a 100 foot Bridge = 1½ ton per foot.
Feet.	Tons.	Tons.	Tons
12	0·7	0·8	0·84
16	1·14	1·36	1·44
24	2·19	2·59	2·73
32	3·4	4·0	4·2
40	4·9	5·8	6·2
60	11·3	13·4	14·0
80	20·8	24·3	25·5
100	33·5	39·0	40·7
120	49·7	57·6	60·2
140	70·5	80·3	84·0
160	95·4	108·2	112·6
180	125·4	141·6	146·7
200	162·2	180·0	186·7

Ex. 2. What is the weight of iron required for a single-line lattice girder bridge 140 feet long between bearings, whose depth = 11 feet 8 inches, and whose working inch-strains are the ordinary ones of 5 and 4 tons tension and compression respectively, the standard load being $1\frac{1}{2}$ tons per foot on a 100 foot bridge? From the table we find that the weight of the main girders, the end pillars and cross-bracing equals 80·3 tons, adding to this the weight of the cross-girders supposed 8 feet apart, namely, 140 × ·18 = 25·2 tons (450), we have the total weight of iron = 105·5 tons.

536. No definite ratio exists between the lengths and weights of girder bridges.—An analysis of the table shows that the ratio of the weights of similar railway girders from 40 to 200 feet in length vary between the square and the 2·3 power of their lengths. In Example 2, the main girders, 400 feet long, weigh 1047 tons, and in Example 5, a similar pair of main girders, 480 feet long, weigh 2449·6 tons. These weights are nearly as the 5th power of the lengths. Again, comparing Examples 3 and 6, which differ from the two former merely in having higher unit-strains, we find the weights of the main girders, which are 713 tons and 1405·9 tons respectively, are nearly as the 4th power of the lengths. These comparisons show that no definite ratio exists between the lengths and weights of girders, and any argument based on such an assumption must be altogether fallacious.

CHAPTER XXVIII.

LIMITS OF LENGTH OF GIRDERS.

587. Single cast-iron girders rarely exceed 50 feet in length—Compound girders advisable for greater spans if cast-iron is used.—Cast-iron girders in one piece rarely exceed 50 feet in length, though this is by no means the possible limit of length of single castings, for Mr. Hawkshaw has employed cast-iron in single girders of 86 feet span,* and Mr. Fairbairn mentions a girder bridge with beams 76 feet span, all in one casting, that were made in England and erected on the Haarlem Railway in Holland.†

When cast-iron girders are required of greater length than 40 or 50 feet it is advisable to truss them with wrought-iron, as cast-iron is ill-suited for resisting tension (**248**). Disastrous results have sometimes attended the use of these compound girders, and they consequently acquired a very bad reputation at one time, but the fault lay not so much in the combination of the two materials as in the mode of combination, which sometimes betrayed sad ignorance of the elementary principles on which girders should be constructed, the depth of the trussed girder having been in some instances considerably less at the centre than at the ends.

588. Practical limit of length of wrought-iron girders with horizontal flanges does not exceed 700 feet.—Vested interest and local peculiarities generally determine the spans of large girders, and it may therefore seem useless to attempt solving the question, "What practical limit is there to the length of a girder?" Curiosity on this subject is, however, natural, and I may therefore claim indulgence for devoting a short space to

* *Proc. Inst. C. E.*, Vol. xiii., p. 474.

† *On the Application of Iron to Building Purposes*, p. 27.

investigating a question which indeed is not altogether devoid of practical utility.

When the dimensions, weight and unit-strains, of any given girder are known, we can find the length of a similar girder which will barely support itself; for it has been already shown in **65** that, if the weight of a given girder equals $\frac{1}{n}$th of its breaking weight, a similar girder n times longer will just break with its own weight. Thus, in Ex. 1 (**502**), a pair of girders whose depth equals $\frac{1}{12}$th of their length, 267 feet long and weighing 335·44 tons, sustain from their own weight 1·64 tons tension and 1·31 tons compression per square inch; taking the tensile and compressive strength of plate iron at 20 tons and 16 tons per square inch respectively, these working strains are equal to the breaking strains divided by 12·2. Hence a similar girder 12·2 times longer, or 3257 feet in length, will just break down from its own weight. Now, the length of a similar girder whose working strains are only one-fourth of its ultimate strength will be $\frac{3257}{4} = 814$ feet nearly, which therefore is the extreme possible limit of an iron lattice girder whose depth equals $\frac{1}{12}$th of its length, whose inch-strains are 5 tons tension and 4 tons compression, and whose empirical percentages are similar to those in the first example of the preceding chapter. The practical limit is of course far short of this, and probably does not exceed 650 feet.

Again, in Ex. 4 (**509**), the main girders, 400 feet long, whose depth equals $\frac{1}{15}$th of their length and which weigh 937·4 tons, sustain 3·14 tons tension and 2·52 tons compression per square inch from their own weight. As these strains are equal to the ultimate strength of plate iron divided by 6·35, a similar girder 6·35 times longer, or 2540 feet in length, will just break down from its own weight. Hence the length of a similar girder whose working-strains from its own weight are $\frac{1}{4}$th of its ultimate strength will be $\frac{2540}{4} = 635$ feet, which therefore is the limiting length of an iron lattice girder whose length equals 15 times its depth, whose inch-strains are 5 tons tension and 4 tons compression, and whose empirical percentages are similar to those adopted in the fourth example of the preceding chapter. The practical limit probably does not exceed 500 feet.

Again, in Ex. 9 (528), the main girders, 600 feet long, whose depth equals $\frac{1}{16}$th of their length and which weigh 3396·6 tons, sustain 4·8 tons tension per square inch from their own weight. This equals the ultimate tensile strength of plate iron divided by 4·16; hence a similar girder 4·16 times longer, or 2496 feet in length, will just break down from its own weight, and the length of a similar girder whose working tensile inch-strain from its own weight is 6 tons, or $\frac{1}{3{\cdot}333}$ of its ultimate strength, will be $\frac{2496}{3{\cdot}333} =$ 749 feet. This therefore is the limiting length of an iron lattice girder whose tensile inch-strain is 6 tons, whose depth equals $\frac{1}{16}$th of the length and whose empirical percentages are the same as those adopted in Ex. 9 of the preceding chapter. The practical limit is, doubtless, below 600 feet.

From these few examples we may reasonably infer that, even with the most careful attention to proportion and economy, the practical limit of length of wrought-iron girders with horizontal flanges does not exceed 700 feet. For girders of greater span steel must be employed.

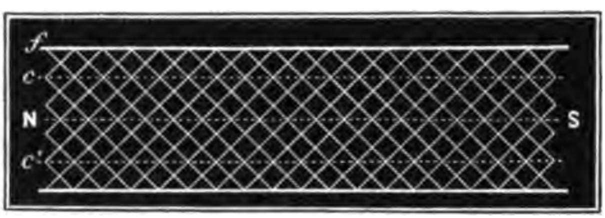
CHAPTER XXIX.

CONCLUDING OBSERVATIONS.

539. A force cannot change its direction without being combined with another force—Hypothesis to explain the nature of strains in continuous webs.—The reader who has perused the foregoing pages with even slight attention has probably arrived at the conclusion that diagonal strains are not confined to braced girders, but are also developed in every structure which is subject to transverse strain. This follows at once from the mechanical law, that a force cannot change its direction unless combined with another force whose direction is inclined to that of the former. Thus a vertical pressure cannot produce horizontal strains in the flanges without developing diagonal ones at the same time in the web.

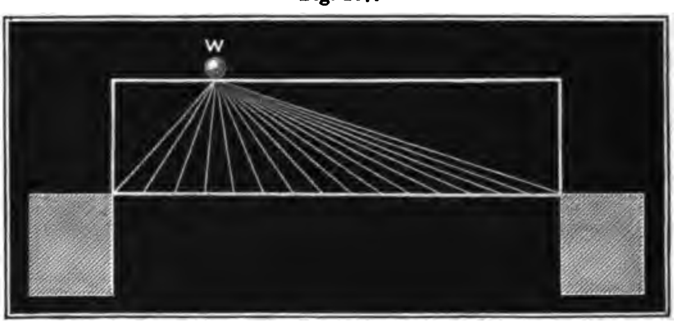
The following hypothesis will perhaps give a clearer conception of the nature of the strains in continuous webs. It is offered, however, merely as a conceivable condition of these strains.

Fig. 106.



Let Fig. 106 represent part of a very closely latticed girder whose neutral surface, or surface of unaltered length, is N S (**55**). The strain in each diagonal of a lattice girder is uniform throughout its entire length (**142**). Now suppose that horizontal stringers are attached to the lattice bars at their first intersections next the flanges, and let us confine our attention to the upper one marked c. As soon as the girder deflects under a load this stringer will become compressed, and consequently it will relieve the upper flange of a certain portion of the horizontal strain which the flange would sustain were the stringer absent. The unit-strain in the stringer will be to that in the flange as $\frac{Nc}{Nf}$ (7, 82). The part of each diagonal above the stringer will also be relieved of a certain portion of its strain depending on the horizontal component it yields to the stringer. Now conceive similar stringers attached at each line of intersections of the latticing above and below the neutral surface. The result will be that each stringer will sustain horizontal unit-strains directly proportional to its distance from the neutral surface where they are cipher, while on the other hand the strains in the diagonals will diminish as they approach the flanges, their *decrements* of strain being cipher at the neutral surface and increasing towards the flanges in the direct ratio of their distance from the neutral surface, provided the stringers are all of equal area. We thus see that the diagonal strains in solid girders act with greatest intensity in the neighbourhood of the neutral surface where the horizontal strains are nil, while they act with least intensity at the upper and lower edges where the horizontal strains are most intense.

Fig. 107.

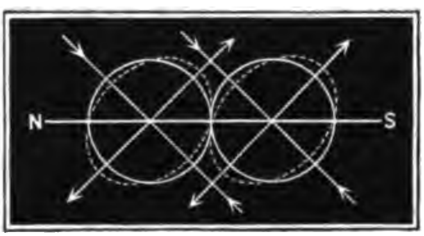


When a single weight rests upon a girder with a continuous web it sends off strains radiating out from the weight in all directions, as represented in Fig. 107, and we may conceive that this first series of diagonal strains are resolved at every point along their length into diagonal and horizontal strains as in the lattice girder, this second series of diagonal strains being again resolved in a similar manner and so on, and thus we have horizontal and diagonal strains interlacing at various angles in all girders except those in which they are forced to take definite directions by means of the bracing, but there will probably exist certain lines of maximum strain, either straight or curved, whose directions will vary according to the position and amount of the weight as well as the flexibility of the material.

The student may make some instructive experiments on this subject by the aid of a model girder formed by stretching a web of drawing paper over a light rectangular frame of timber, which will represent the flanges and end pillars. By the aid of little movable wooden struts, to represent verticals, he can vary the directions of the lines of strain to a very considerable extent.

It is not at first sight easy to see how strains are transmitted through the neutral surface, for the particles there are apparently undisturbed in form. It is conceivable, however, that particles which are spherical when free from strain may become elongated by tension in one direction and shortened by compression at right angles to it, so as to assume an oval shape, while horizontal lines parallel to the neutral surface, N S, retain their original length as represented in Fig. 108.

Fig. 108.



540. A ship resembles a tubular girder—Top flange of the ship frequently deficient in strength—Deck-bracing and bulkheads stiffen a ship horizontally and transversely.— An iron ship is a large tubular structure, more or less rectangular in section, underneath which the points of support are continually moving, so that when the waves are high and far apart the deck and bottom of the vessel are alternately extended and compressed in the same way that the flanges of a continuous girder are near the points of inflexion when traversed by a passing train. The sides of a ship are formed of continuous plating with vertical frames at short intervals and form very efficient webs. The bottom also is, from its large area, fully adequate to its duty as a flange. The sides and bottom flange of the girder are therefore fully developed, but the upper iron flange is sometimes altogether wanting, or else sadly out of proportion to the remainder of the structure. This deficiency is properly remedied either by attaching what are technically called stringers to the topsides, or better still, by making the upper deck entirely of iron with a thin sheeting of planks resting on the iron. Deck stringers are horizontal plates which run continuously fore and aft beneath the planking of the deck. They are seldom more than 3 or 4 feet in width, but in some few cases extend as far as the hatchways. Similar stringers are occasionally riveted to the sides underneath each of the lower decks, and when stringers in the same plane on opposite sides of the ship are connected by diagonal tension braces, the latter in conjunction with the deck beams form very efficient cross-bracing, and greatly increase the strength and stiffness of the ship when labouring in a heavy sea.

Bulkheads act as gussets or diaphragms, and stiffen the tube transversely by preventing any racking motion from taking place in the direction of their diagonals.

541. Iron and timber combined form a cheap girder—Timber should be used in large pieces, not cut up into planks—Simplicity of design most desirable in girder-work.—Within certain limits of length the cheapest form of girder is one made of timber in compression with wrought-iron in tension (**194**, **228**). The earlier types of wooden lattice bridges had little or no iron in their composition, and were characterized by the small scantlings of the parts, the closeness of the latticing, and in many cases a want of stiffness under passing loads. This defect was, no doubt, often due to insufficient flange area, but may also be attributed to the small size of the scantlings and consequent multiplicity of joints. The remedy is obvious. Timber in compression should be used in bulk and not cut up into thin planks. Laminated arches, it is true, are an apparent exception to this rule, but in reality a laminated beam possesses the aggregate section of its component parts which are bound together so that they act as one solid piece. Even when used in tension it may be doubtful economy to use several thin planks where one of larger section would suffice. The liability to decay from moisture lodging in the numerous joints is another serious objection to close timber latticing, though this is sometimes diminished by the protection of a roof extending over the whole bridge (**487**).

In conclusion, it may not be amiss to say a few words on designing girders. Simplicity and consequent facility of construction should never be lost sight of. Complicated arrangements are to be deprecated, whether designed to effect some saving more apparent than real, or, as one is sometimes tempted to conjecture, from a craving after novelty. The various parts of girder-work should, as much as possible, be repetitions of the same pattern, easily put together and accessible for preservation or repair. Hence, as a rule, closed cells, difficult forgings, or curved forms where straight ones would effect the object equally well, are to be carefully avoided.

APPENDIX.

BOYNE LATTICE TUBULAR BRIDGE.

542. General description — Detailed weights of girder-work. — The Boyne Viaduct carries the Dublin and Belfast Junction Railway across the valley of the River Boyne near Drogheda, and consists of several lofty semi-circular stone arches on the land, and a wrought-iron lattice tubular bridge in three spans over the water, the surface of which is about 90 feet below the girders, so that vessels of considerable tonnage can sail beneath. The girder-work is formed of two lattice tubular main-girders, having their top flanges connected by cross-bracing and the lower flanges connected by cross-bracing and road-girders, so as collectively to form a tubular bridge for a double line of railway, as shown in cross-section in Plate IV. Each main-girder is a continuous girder, 3 feet wide and 550 feet 4 inches long, in three spans. The centre span is 267 feet from centre to centre of bearings, and 264 feet long between bearings. Each side span is 140 feet 11 inches long from centre to centre of bearings, and 138 feet 8 inches long between bearings. The flanges are horizontal, and the depth of girder, measured from root to root of angle-irons, is 22 feet 3 inches, or $\frac{1}{12}$th of the centre span and $\frac{1}{6 \cdot 34}$ of a side span. Each of the terminal pillars is 18 inches broad in elevation, and has a bearing surface of 4·5 square feet; each of the pillars at the ends of the centre span is 3 feet broad in elevation, and has a bearing surface of 9 square feet. The road-girders are 7 feet 5 inches apart from centre to centre, and correspond with the intersections of the lattice bars, which are placed at an angle of 45°, and form squares of 5 feet 3 inches on the side. For further description the reader is referred to a paper by J. Barton, Esq., on "Wrought-iron Beams," in the fourteenth vol. of *The Proceedings of the Institution of Civil Engineers*. The quantities of material in the girder-work are as follows:—

TABLE I.—WEIGHT OF WROUGHT-IRON IN ONE SIDE SPAN, 140 FEET 11 INCHES BETWEEN CENTRE OF BEARINGS, AND 30 FEET WIDE FROM OUT TO OUT.

	Tons.	Tons.
TWO TOP FLANGES.		
Plates and angle irons,	27·45	
Covers,	3·57	39·84
Packings,	6·38	
Rivet heads	2·44	
TWO BOTTOM FLANGES.		
Plates and angle irons,	27·10	
Covers,	3·84	39·59
Packings,	6·40	
Rivet heads,	2·25	
TWO DOUBLE LATTICED WEBS.		
Tension diagonals,	10·96	
Compression do.,	27·70	38·79
Rivet heads at intersections,	0·13	
CROSS-BRACING.		
6 lattice cross-beams connecting top flanges,	3·70	
Diagonal tension bars (top and bottom) and a longitudinal angle iron stiffener along the centre at top,	5·36	9·16
Rivet heads,	0·10	
ROAD GIRDERS.		
18 road girders, including end gussets,		23·72
Iron between end pillars,		**151·10**
Platform planking,	29·40	
Longitudinal sleepers (double line),	2·45	40·41
Rails and joint plates (Barlow's),	8·56	
Permanent load on one side span, equal to 1·36 tons per running foot for the double line.		**191·51**

TABLE II.—WEIGHT OF WROUGHT-IRON IN THE CENTRE SPAN, 267 FEET BETWEEN CENTRES OF BEARINGS AND 30 FEET WIDE FROM OUT TO OUT.

	Tons.	Tons.
TWO TOP FLANGES.		
Plates and angle iron, ...	79·09	105·48
Covers, ...	9·38	
Packings, ...	11·83	
Rivet heads, ...	5·18	
TWO BOTTOM FLANGES.		
Plates and angle iron, ...	82·19	109·12
Covers, ...	9·85	
Packings, ...	11·90	
Rivet heads, ...	5·18	
TWO DOUBLE-LATTICED WEBS.		
Tension diagonals, ...	30·80	82·81
Compression do., ...	51·76	
Rivet heads at intersections, ...	·25	
CROSS-BRACING.		
11 lattice cross-beams connecting top flanges, . .	6·77	17·66
Diagonal tension bars (top and bottom) and a longitudinal angle-iron stiffener along the centre at top, . .	10·69	
Rivet heads, ...	·20	
ROAD GIRDERS.		
35 road girders, including end gussets, ...		46·13
Iron between end pillars, ...		**361·20**
Platform planking, ...	55·57	76·39
Longitudinal sleepers (double line), ...	4·62	
Rails and joint plates (Barlow's), ...	16·20	
Permanent load on centre span, . . . equal to 1·64 tons per running foot for the double line.		**437·59**

TABLE III.—WEIGHT OF WROUGHT-IRON IN THE PILLARS AND CROSS-GIRDERS OVER SUPPORTS.

	Tons.	Tons.
PILLARS, &c., OVER ONE LAND ABUTMENT.		
2 terminal pillars at end of one side span, . . .	6·38	
1 lattice cross-beam connecting heads of pillars, . .	3·40	13·23
1 road girder and gussets connecting feet of pillars, . .	3·45	
PILLARS, &c., OVER SOUTH RIVER PIER.*		
2 pillars at south end of centre span, ...	15·30	
1 lattice cross-beam connecting heads of pillars, . .	5·24	
1 road girder connecting feet of pillars, . . .	1·09	24·06
2 gussets between pillars and pier, ...	2·43	
PILLARS, &c., OVER NORTH RIVER PIER.		
2 pillars at north end of centre span, ...	15·30	
1 lattice cross-beam connecting heads of pillars, . .	5·24	25·56
1 road girder connecting feet of pillars, . . .	5·02	

* The pillars are firmly secured to this pier; rollers are used on the north pier and on both abutments.

TABLE IV.—SUMMARY OF WROUGHT-IRON.

	Tons.
One side span, ...	151·10
One do., ...	151·10
Centre span, ...	361·20
Pillars, &c., over one land abutment, ...	13·23
Do. do.	13·23
Do. south river pier, ...	24·06
Do. north river pier, ...	25·56
Total weight of wrought-iron, in the Boyne Lattice Bridge, . 3 spans, 550 feet 4 inches in length, equal to 1·344 tons per running foot for the double line of railway.	**739·48**

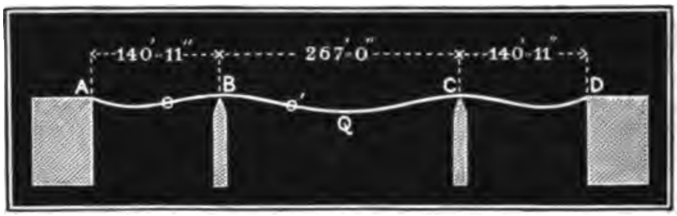
TABLE V.—WEIGHT OF SOLE-PLATES, ROLLERS AND WALL-PLATES.

	Tons.	cwts.	qrs.	lbs.
OVER TWO ABUTMENTS.				
4 planed cast-iron sole-plates riveted to feet of pillars,		17	2	16
4 planed cast-iron wall-plates resting on the masonry,	2	11	0	0
2 sets of 4-inch wrought-iron rollers and frames over the north abutment, ...		10	2	0
2 sets of 4½-inch wrought-iron rollers and frames over the south abutment, ...		12	2	26
OVER SOUTH PIER.				
2 cast-iron sole-plates riveted to feet of pillars, .		19	0	12
2 cast-iron wall-plates resting on the masonry, .	5	4	0	0
OVER NORTH PIER.				
2 planed cast-iron sole-plates riveted to feet of pillars,		19	0	12
2 planed cast-iron wall-plates resting on the masonry,	4	13	0	16
2 sets of 5-inch chilled cast-iron rollers and wrought-iron frames, ...	1	15	0	16
Total weight of sole-plates, rollers and wall-plates, ...	**18**	**2**	**1**	**14**

543. Working strains—Area of flanges.—The strains produced by the permanent bridge-load plus one ton of train-load per running foot on each line of way do not exceed 5 tons tension per square inch of net section, *i.e.*, after deducting the rivet holes, and 4½ tons compression per square inch of gross section. The gross sectional area of the top flange of each main girder in the centre of the centre span = 113·5 square inches; the gross area of the bottom flange at the same place = 127 square inches and its net area = 99 square inches; over the piers, between the centre and side spans, the gross area of the top flange = 132·6 square inches and its net area = 103·4 square inches; the gross area of the bottom flange at the same place = 127 square inches. At a point 40 feet from the piers, measured towards the centre of the centre span, the gross area of each flange = 68·5 square inches.

544. Points of inflexion—Pressures on points of support.— The points of inflexion may be obtained by the method explained in **253**, as follows.

Fig. 109.



Let Q be the centre of the centre span, and o and o' the points of inflexion.

Let l = AB = CD = 141 feet nearly,

$$AQ = nl, \text{ whence } n = \frac{274 \cdot 5}{141} = 1 \cdot 95 \text{ nearly,}$$

w = the load per running foot on either side span,

w' = the load per running foot on the centre span,

R_1 = the reaction of either abutment, A or D,

R_2 = the reaction of either pier, B or C.

When the bridge supports its own weight only,

$$w = 1 \cdot 36 \text{ tons and } w' = 1 \cdot 64 \text{ tons.}$$

CASE 1.

545. Maximum strains in the flanges of the side spans.— These occur when the passing load covers both side spans and the centre span is unloaded (**255**); in which case, assuming that the maximum train-load is equivalent to one ton per running foot on each line of way, we have

$$w = 3 \cdot 36 \text{ tons and } w' = 1 \cdot 64 \text{ tons.}$$

From equations 181 and 182 the pressures on the points of support are as follows.

$$R_1 = 170 \text{ tons and } R_2 = 523 \text{ tons.}$$

The positions of the points of inflexion, obtained from equations 183 and 184, are as follows.

$$Ao = 101 \cdot 2 \text{ feet and } Bo' = 53 \cdot 2 \text{ feet.}$$

The strain in *each* of the four flanges midway between A and o, *i.e.*, in the centre of the first segment, is $96\frac{1}{2}$ tons (eq. 26).

CASE 2.

546. Maximum strains in the flanges of the centre span.—These occur when the passing load covers the centre span alone, in which case

$$w = 1 \cdot 36 \text{ tons and } w' = 3 \cdot 64 \text{ tons.}$$

The pressures on the points of support are as follows.

$$R_1 = -24 \cdot 6 \text{ tons and } R_2 = 704 \text{ tons.}$$

R_1 being negative signifies that a load of 24·6 tons is required at each end to prevent the girder from rising off the abutments (**254**), and this was actually the case when the bridge was proved with one ton per running foot on each line of the centre span, the side spans being unloaded. The girder was temporarily tied down to the abutments by bolts secured to the masonry, but the bolts drew out and the ends of the girder rose more than an inch above their normal position on the rollers. The weight of a locomotive at each end, however, soon brought them down again. With the lighter working loads which occur in practice this rising off the abutments never occurs. The position of the points of inflexion in the central span is as follows.

$$Bo' = 40 \cdot 3 \text{ feet,}$$

and the strain in *each* of the four flanges in the centre at Q = 355 tons (eq. 26).

CASE 3.

547. Maximum strains in the flanges over the piers.—The maximum strains over a pier occur when the centre span and the adjacent side span are loaded and the remote side span is unloaded. We have, however, no formula for this condition of load, but we have a close approximation to it when the passing load covers all three spans (**255**), in which case

$$w = 3{\cdot}36 \text{ tons and } w' = 3{\cdot}64 \text{ tons.}$$

The pressures on the points of support are as follows.

$$R_1 = 107 \text{ tons and } R_2 = 853 \text{ tons.}$$

The positions of the points of inflexion are as follows (eqs. 183, 184).

$$Ao = 63{\cdot}4 \text{ feet and } Bo' = 44{\cdot}7 \text{ feet.}$$

The strain in *each* of the four flanges over the piers = 406·4 tons (eq. 13).

548. Points of inflexion fixed practically — Deflection— Camber.—The points of contrary flexure in the centre span were practically fixed in the manner described in **250**. Two joints in the upper flange, 170 feet apart and equi-distant from the piers, were selected for section. The rivets were cut out and drifts temporarily inserted in their place. These drifts were then cautiously struck out with a light hammer, and a slight closing of the joints proved that a certain amount of compression had previously existed in place of perfect freedom from strain. The extreme ends of the side spans were then lowered, one an inch, the other half an inch, which caused the joints to open slightly, about $\frac{1}{64}$th of an inch. In this condition it was obvious that no strain was being transmitted through the joints, and they were then finally riveted up, the altered levels of the extreme ends of the side spans being maintained by rollers of the proper diameter placed beneath the terminal pillars. Tables VI. and VII. contain the deflections produced by various conditions of load during the first, or Engineer's, testing and the second, or official, testing of the bridge by the Government Inspector (**410**).

TABLE VI.—DEFLECTIONS IN INCHES MEASURED DURING THE FIRST OR PRIVATE TESTING OF THE BRIDGE, MARCH 28TH, 1855.

NORTH SIDE SPAN.		CENTRE SPAN.					SOUTH SIDE SPAN.			
Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.		
89	44·5	44·5	89	133·5	89	44·5	44·5	89	Distances measured in feet from centres of River Piers.	
Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.		
0·675	0·475	+0·175	+0·2	+0·175	+0·2	+0·112	0·5	0·4	West girder,	138 tons on each line of both side spans, *i.e.*, *maximum* load on both side spans together.*
0·8	0 55	+0·2	+0·25	+0·275	+0·25	+0·2	0·6	0·6	East girder,	
+0·45	+0·375	1·0	1·7	2·15	1·85	1·025	0·225	0·3	West, .	271 tons on each line of centre span and 138 tons on each line of south side span, *i.e.*, *maximum* load on centre span and on south side span.
+0·3	+0·233	1·075	1·75	2·075	1·675	0·8	0·15	0·5	East, .	
+0·95	+0·65	2·3	2·4	2·925	2·7	1·6	0·0	0·0	West, .	271 tons on each line of centre span, *i.e.*, *maximum* load on centre span alone.†
+0·525	0·0	1·375	2·4	2·8	2·45	2·025	+1·1	+0·6	East, .	
0·1	0·0	0·3	0·55	0·65	0·65	0·4	0·05	0·1	West, .	Residual set on the 29th, after all the load had been removed (**457**).
0·175	0·1	0·3	0·5	0·6	0·5	0·25	0·05	0·025	East, .	

NOTE—The sign + prefixed to a deflection denotes that the girder cambered, or rose, above its original position.

* With this maximum load on both side spans together the north-west abutment pillar sank 0·11 inches; the north-east do., 0·25 inches; the south-west do., 0·2 inches; the south-east do., 0·25 inches, in consequence of the compression of some timber packing temporarily placed beneath the pillars.

† With this maximum load on the centre span the north-west abutment pillar rose 1·25 inches; the north-east do., 1·06 inches; the south-west do., 2·0 inches; the south east do., 1·62 inches, thus confirming previous calculations (**546**).

TABLE VII.—DEFLECTIONS IN INCHES, MEASURED DURING THE TESTING OF THE BRIDGE BY THE GOVERNMENT INSPECTOR, MARCH 30TH, 1855.

NORTH SIDE SPAN.		CENTRE SPAN.					SOUTH SIDE SPAN.			
Feet. 89	Feet. 44·5	Feet. 44·5	Feet. 89	Feet. 133·5	Feet. 89	Feet. 44·5	Feet. 44·5	Feet. 89	Distances measured in feet from centres of river piers.	
Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.		
0·65	0·5	+0·25	+0·475	+0·5	+0·5	+0·325	0·5	0·7	West girder,	One ton per running foot on each line of both side spans, *i.e., maximum* load on both side spans together.
0·625	0·45	+0·275	+0·425	+0·5	+0·45	+0·35	0·5	0·7	East girder,	
0·3	0·15	0·7	1·3	1·65	0·0	0·0	+0·3	+0·2	West, .	One ton per running foot on each line of north side span and centre span, *i.e., maximum* load on north side span and on centre span.
0·25	0·05	0·75	1·35	1·65	1·425	0·75	+0·3	+0·3	East, .	
+0·25	+0·225	0·85	1·6	1·9	1·6	0·9	+0·3	+0·275	West, .	One ton per running foot on each line of centre span, *i.e., maximum* load on centre span alone.*
+0·25	+0·3	0·85	1·55	1·9	1·575	0·8	+0·4	0·35	East, .	
0·3	0·175	0·675	1·25	1·525	1·25	0·625	0·1	0·3	West, .	One ton per running foot on each line of the three spans, *i.e., maximum* load all over the bridge = 1,100 tons.
0·325	0·1	0·65	1·175	1·45	1·175	0·55	0·1	0·35	East, .	
0·1	0·075	0·05	0·1	0·1	0·0	0·0	0·0	0·05	West, .	Load all removed.
0·1	0·025	0·05	0·075	0·1	0·05	0·0	0·0	0·1	East, .	

NOTE—The sign + prefixed to a deflection signifies that the girder cambered, or rose, above its original level.

* When this maximum load rested on the centre span alone, the extreme ends of the girder were held down by an engine or waggon placed immediately over the first and last road girders (**546**).

Each span was built on the platform with a camber, in order that the sky-line might be nearly horizontal when the bridge was finished (**457**). The camber at the centre of the centre span at different periods was as follows.

TABLE VIII.—CAMBER AT CENTRE OF CENTRE SPAN AT DIFFERENT PERIODS.

	Inches.
During construction on the platform, ...	3·48
After wedges were struck and bridge was self-supporting, . . .	1·56
After fixing points of inflexion and lowering extreme ends of side spans, .	1·80
After second, or official, testing of bridge, ...	0·84
After four months' traffic, ...	0·90

549. Experiments on the strength of braced pillars.— The following experiments were made at the Boyne Viaduct in 1854 to determine the strength of one of the compression diagonals of the web which were made of flat bar iron similar to the tension diagonals, but with the addition of internal angle irons and cross-bracing riveted between them as already described in **387**. The theory of braced pillars was then imperfectly understood, and Mr. Barton wisely determined to test by direct experiment whether this arrangement of internal cross-bracing would enable a bar, thin in proportion to its length, to sustain an endlong pressure like a pillar, such as the compression diagonals should sustain in the bridge. Accordingly the following experiments were made on one of the smaller compression diagonals which occur near the centre of the centre span, the author being present and recording the results.*

EXPERIMENT No. 1.

The first experimental pillar resembled Fig. 1, Plate V., in every respect, except the lower portion, which was formed as shown in Fig. 4. This pillar, which was 31′ 6″ in length with 4″ × ⅝″ side bars, was erected in the midst of some timber scaffolding which had been used for a stone hoist. The testing weight was suspended

* See a description by the author of "Experiments made at the Boyne Viaduct in 1854."—*Proc. Inst. C.E. of Ireland*, Vol. v.

below the wooden framing on which the pillar stood by long suspender rods which were attached to cross pieces of timber resting on the top of the pillar (see Figs. 2 and 3). By this arrangement the pressure was made to pass more accurately through the axis of the pillar than if the testing weight had been heaped up on top. It was also more convenient to load at the lower level. Cross bars *f*, *f*, *f*, were attached to the sides at the same intervals as the latticing in the main girders, and were connected at their ends to the scaffolding, so as to represent the tension diagonals in the bridge; and here I may again remind the reader that the chief advantage of a multiple over a single system of triangulation consists in the more frequent support given by the tension bars to those in compression, as well as by both to the flanges. The parts in compression are in fact subdivided into short pillars, and thus prevented from deflecting in the plane of the girder (**155**). A cord was stretched vertically, in order to get the lateral deflections during the experiment. These were taken at three points, A, B, C (Fig. 1); and the symbols + or — placed before a deflection in the table signifies that it was in the direction of the same sign engraved at the sides of the figure.

TABLE IX.—LATERAL DEFLECTIONS OF A BRACED PILLAR.

Date.	Tons.	A.	B.	C.	REMARKS.
1854.		Inches.	Inches.	Inches.	
Nov. 16	5	+ 0·03	+ 0·05	+ 0·03	
„	10	0·0	0·0	+ 0·05	
„	15	— 0·03	+ 0·08	+ 0·05	
„	20	— 0·05	0·0	0·0	
„	25	— 0·05	0·0	0·0	
„	30	— 0·05	— 0·03	— 0·04	
„	37½	— 0·06	— 0·07	— 0·06	
„	40	— 0·05	— 0·01	— 0·05	With 40 tons the side bar at *a* (Fig. 4), bent slightly at right angles to the plane of figure. The deflection at B seems anomalous; probably a mistake for 0·10?
„	42½	— 0·10	— 0·10	— 0·13	With 42½ tons the lower part of pillar at *b*, *b*, became slightly curved, with the convex side towards the — side.
Nov. 17	42½	— 0·10	— 0·10	— 0·13	Left on all night; no change in the morning.
„	45	— 0·10	— 0·15	— 0·16	
„	47½	...	...	...	The side bars gave way, as shown in Fig. 5.

Looking at Fig. 4, it will be seen that about 8 inches in length of each side bar near the ends of the pillar were left without internal angle iron, and when the weight amounted to 47½ tons this part yielded sideways, as shown in Fig. 5. The area of the two side-bars at the part which failed amounts to 5 square inches; the compressive strain which passed through them at the moment of yielding equalled therefore 9½ tons per square inch.

Experiment No. 2.

The pillar in the first experiment failed, as indeed had been anticipated, by the upper part moving sideways past the lower, as if connected to it by hinges. The pillar was taken down, the injured part removed, and the length thus reduced to 28′ 6″. The repaired pillar, Fig. 1, was then replaced within the scaffolding, and the following table contains the observations recorded, which include the contraction in length of each side under compressive strain. These latter observations were made by the aid of wooden rods suspended at each side from near the top of the pillar. Each rod was 24′ 8½″ in length from the point of suspension to the index at the lower end, and it will be observed that the contraction of one side exceeds that of the other in a very anomalous manner, which can only be explained by supposing that the timber framing yielded more beneath one side than the other, and thus caused a greater strain of compression to pass through that side which contracted most.

TABLE X.—Lateral Deflections and Vertical Contraction of a Braced Pillar.

Date.	Tons.	A.	B.	C.	Rod on + side.	Rod on − side.	Observations.
		Inches.	Inches.	Inches.	Inches.	Inches.	
1854. Nov. 25	30	+0·03	+0·04	+0·01	0·05	0·25	At 30 tons the side bar at *c* bulged outwards slightly, with a tendency to increase; also a slight hollow was produced at *d*; to remedy this bulging (which seemed to be caused by the unequal compression of the timber packing, that on the + side yielding more than the opposite), a strut was placed against *c*, and the weight was blocked up until the 27th; wedges also were driven between the wooden packings underneath, in order to tighten them up.

Date.	Tons.	A.	B.	C.	Rod on + side.	Rod on — side.	Observations.
		Inches.	Inches.	Inches.	Inches.	Inches.	
Nov. 27	0	...	...	...	0·0	0·20	Load removed, bulging at *c* removed as nearly as possible by means of a screw-jack which was left in position; opposite side similarly blocked out from staging, and blocks were placed at similar positions at top of pillar, as there appeared a tendency of top to move over to — side.
"	30	...	...	...	0·05	0·25	
"	35	...	...	...	0·06	0·27	
"	40	0·0	—0·01	—0·03	0·08	0·31	Left hanging on all night, wind so strong as to make deflections uncertain.
Nov. 28	40	0·0	—0·37	—0·07	0·09	0·31	The hollow at *d* still well marked, and a tendency to deflect towards + side, at the centre of pillar.
"	45	0·0	—0·01	—0·07	·09	·34	Wind in gusts; 45 tons left hanging on one hour.
"	50	+0·07	+0·06	—0·03	·10	·40	Wind much abated; no visible change.
"	55	...	...	...	·10	·44	Wind so strong as to prevent deflections being taken. No visible change.
"	60	...	...	...	·10	·49	No change visible.
"	62½	...	...	...	·105	·50	Left hanging on all night.
Nov. 29	...	...	...	...	·11	·50	No visible change this morning.
"	65	...	...	...	·12	·53	The buckle at centre strongly marked.
"	70	+·15	+·14	+·03	·12	·56	Wind much abated.
"	72½	...	...	...	·13	·60	
"	75	...	...	...	·14	·65	No visible change or upsetting of any part.
"	77½	...	...	...	·13	·69	
"	80	...	...	...	...	...	Left hanging on all night.
Nov. 30	...	...	...	...	·14	·78	In morning.
"	82½	...	...	...	·15	·795	
"	83½	...	...	...	...	...	Broke down as the additional ton was being laid on, parts *b* and *e* (Fig. 1), giving way. At *e* both sides of the pillar bent, and the internal lattice was completely distorted, the L iron being broken away from side bar (see Fig. 2).

The sectional area of the pillar subject to compression, and including the angle irons, is 7·5 inches. The compression per square inch therefore equalled 11 tons at the period of failure. For a very short portion at *c* where the bracing ended, the angle irons of the lower cell and that to which the internal lattice bars were connected were not in one continued piece, and the whole weight passed through the unsupported side bars, which were, however, a little thicker here than elsewhere from a weld having been made at that point, so that the area of both side bars together equalled 6 inches. This short length was therefore subject to a compression of nearly 14 tons per square inch. If we wish to compare the economy of this form of pillar with a tubular one, we must add the cross area of the lattice bars to that of the side bars, in order to obtain the strain per sectional inch of material in the whole pillar. The cross area of the lattice bars = 2 inches nearly; adding this to the area of the side bars and angle irons, we have the total sectional area of the braced pillar = $9\frac{1}{2}$ inches, and the strain per square inch of material employed = 8·7 tons. This is a favourable result when compared with those arrived at by Mr. Hodgkinson in his experiments on tubes subject to compression, for if the same amount of iron were thrown into the form of a tube it would have such thin sides that the ultimate crushing inch-strain would probably fall very far short of 8·7 tons (**220**). We may regard the lattice pillar as one side of a tube, in the corners of which the chief part of the material is collected, and the sides of which are formed of bracing, connecting and holding the corner pillars in the line of thrust (**221**).

550. Experiments on the effect of slow and quick trains on deflection.—The following experiments were made at the Boyne Viaduct to try the effect of slow and quick trains on vibration and deflection.

April 5th, 1855.—The lateral vibration at the centre of the centre span from an engine and tender going at the rate of from 30 to 50 miles an hour equalled 0·05 inch on each side, *i.e.*, the total oscillation equalled ·1 inch. The lateral vibration from a slow engine was scarcely perceptible.

The deflection at the centre of the centre span, measured on the same side as the line on which the engine and tender travelled, both for quick and slow speeds equalled ·25″. The same deflection was produced when the engine was brought to a stand at the centre of the centre span. If any difference of deflection with different speeds was perceptible, those deflections which were produced by rapid travelling exceeded the others by a very small amount, perhaps the width of a fine pencil stroke, but for all practical purposes they were identical. On starting the engine from rest at the centre of the bridge the deflection was momentarily increased to a very slight extent. There were about five quick trains, of which one travelled at 48 and the others 50 miles an hour, and about as many slow ones (**458**).

NEWARK DYKE BRIDGE, WARREN'S GIRDER.*

551.—This bridge carries the Great Northern Railway across the Newark Dyke, a navigable branch of the river Trent. It is a skew girder bridge, formed of a single system of equilateral triangles on Warren's principle. Each girder consists of a hollow cast-iron top flange, and a bottom flange, or tie, of wrought-iron bar links, connected together by diagonal struts and ties, alternately of cast and wrought-iron, which divide the whole length into a series of equilateral triangles 18 feet 6 inches long on each side. There are two main girders to each line between which the train travels on a platform attached to the lower flanges. The length from centre to centre of points of supports is 259 feet, and the clear span between the abutments is 240 feet 6 inches. The depth from centre to centre of flanges is 16 feet, or nearly $\frac{1}{16}$th of the length. The permanent weight of bridge for a single line of railway, consisting of two main girders, top and bottom cross-bracing, platform, &c., is as follows:—

	Tons.	Cwts.
Wrought-iron, ...	106	5
Cast-iron, ...	138	5
	244	10
Platform, handrail and moulded cornice, .	50	0
Total permanent weight, . .	**294**	**10**

* See *A Description of the Newark Dyke Bridge on the Great Northern Railway.* By Joseph Cubitt, M. Inst. C.E. Vol. xii., Proc. Inst. of C.E.

With a load of one ton per running foot the central deflection amounted to 2¾ inches. The strain due to this load, whether tensile or compressive, is said not to exceed 5 tons per square inch on any part.

CHEPSTOW BRIDGE, GIGANTIC TRUSS.*

552.—This bridge was erected by Mr. I. K. Brunel to carry the South Wales Railway across the river Wye near Chepstow. It consists of two gigantic trusses, one for each line of way, 305 feet long and about 50 feet deep, and resembling Fig. 63, p. 115, with this exception, that the roadway is attached to the lower flange. The compression flange of each truss is a circular plate-iron tube, 9 feet in diameter and ⅝th inch thick, supported by cast-iron arched standards or end pillars which rest on the piers. The main girders are plate girders which are suspended from the truss at two intermediate points with their ends resting on the piers, and are thus divided into three spans. The weight of iron in one bridge for a single line of railway is as follows:—

	Tons.
298 feet run of tube and butt plates, . .	127½
Hoops of ditto over piers, . . .	7¾
Side and bottom plates for attachment of main chains, ...	15
Side plates for attachment of diagonal chains, .	2¼
Stiffening diaphragms, 26 feet apart, . .	4¼
Rivet heads, &c., ...	4¾
Total weight of one tube, . . .	161½
Main chains, eyes, pins, &c., . . .	105
Diagonal chains, eyes, pins, &c., . . .	23
Vertical trusses, ...	18½
Total weight of side-bracing, . .	146½

* See *Encyc. Brit., Art. Iron Bridges*, and *Clark on Tubular Bridges*, p. 101.

	Tons.
Main roadway girders, cross girders, &c., . .	130
Saddles, collars, &c., at points of suspension, .	22
Total weight of iron, for one line of railway,	**460**

CRUMLIN VIADUCT, WARREN'S GIRDER.*

558.—The Crumlin Viaduct is situated on the Newport section of the West-Midland Railway about five miles from Pontypool. The structure is divided by a short embankment into two distinct viaducts of exactly similar construction. The larger viaduct has seven, the smaller three, openings of 150 feet from centre to centre of piers. The girders are "Warren's Patent" of 148 feet clear span, but not connected together as in continuous girders. The compression flange is a rectangular plate-iron box or tube, and the tension flange is formed of flat wrought-iron bars; both flanges increase in sectional area from the ends towards the centre. The diagonals form a series of equilateral triangles of angle and bar iron, the section of those in compression being in the form of a cross. The length of each side of the triangle is 16 feet 4 inches.

The maximum tensile strain in the diagonals from the permanent load and a train-load of one ton per running foot is 6·65 tons per square inch of net section. The maximum tensile strain in the lower flange from the same load is 5·75 tons per square inch of net section. In no part does the maximum compression strain from the same load exceed 4·31 tons per square inch of gross section.

The viaduct has four girders, two to each line of railway. The road is above the upper flanges. The weights for a single line 150 feet long were as follows when the bridge was first made. Additional material appears to have been added subsequently for the purpose of strengthening it.†

* See *Trans. Inst. C.E. of Ireland*, Vol. vii., p. 97 ; and *Humber on Bridges*.
† *Engineer*. 1866. Vol. xxii., p. 384.

	Tons.	Cwts.
A pair of main girders,	37	18
Cross-bracing of do.,	3	3
Platform,	18	1
Permanent way,	15	3
Hand-railing,	9	0
Total permanent weight,	83	5

The tension flange of one girder weighs 119·4 cwt., of which 30 cwt., or one-fourth, is required to make the connexions of the flange.

LATTICE BRIDGES ON THE VICTOR EMMANUEL RAILWAY.*

554.—The first of these bridges was erected by Mr. Edwin Clark to carry the Victor Emmanuel Railway (single line) over the river Isere, in Savoy. The railway is supported by two single-webbed lattice girders which are continuous throughout, forming a total length of 558 feet, divided into four uniform openings of 130 feet $4\frac{1}{2}$ inches each. The depth is uniform throughout and equals $\frac{1}{11}$th of each span, or 11 feet 9 inches from out to out. The weight of iron work in each pair of main girders for each span of 130 feet $4\frac{1}{2}$ inches is 50 tons, and the weight of its proportion of roadway is 21 tons. The total weight of iron in the whole structure is 322 tons wrought-iron and 15 tons cast-iron.

On the same railway are two other bridges, similar in every respect, but of only one span each. The weight of these girders, not having the advantage of continuity, is increased; that of a pair of main girders in each bridge being 72 tons, and of the whole bridge with roadway 96 tons.

BOWSTRING BRIDGE ON THE CALEDONIAN RAILWAY.†

555.—This bridge was erected by Mr. E. Clark, to carry the Caledonian Railway over the Monkland Canal. The arch is partly

* *Encyclopædia Britannica*, Art. "Iron Bridges," p. 598.

† *Idem*, p. 605.

wrought-iron, and partly cast-iron, and the tie or lower flange consists of wrought-iron plates. The total length of the girders is 148 feet, and the depth is 15 feet or about $\frac{1}{10}$th of the length. The whole weight of the girders for a double line is 128 tons.

CHARING-CROSS LATTICE BRIDGE.*

556.—This bridge was erected by Mr. Hawkshaw to carry the Charing-Cross Railway across the Thames on the site of the Hungerford Suspension Bridge, the chains of which were removed to Clifton. It comprises nine independent spans, six of 154 feet and three of 100 feet. The leading particulars of one of the 154 feet spans are as follows. The main girders are wrought-iron lattice tubular girders, the web consisting of two systems of nearly right-angled triangles. The tension diagonals are Howard's patent rolled suspension links, and the compression diagonals are forged bars, varying in thickness from $2\frac{1}{2}$ to 3 inches, and united in pairs by zigzag internal cross-bracing (**337**). The flanges are formed of horizontal plates in piles with four vertical ribs attached by angle irons to the horizontal plates, the two outer ribs being 2 feet deep, and the two inner ones 21 inches deep. The flanges therefore resemble the usual trough section (**441**), but with 4, in place of 2, vertical ribs. The diagonals have swelled ends with eyes, and are attached to the flanges by turned pins of puddled steel passing through the vertical ribs. In addition to the diagonals already mentioned there are vertical bars 1 inch thick connecting each pin in the upper flange with that in the flange directly beneath; these vertical bars form diagonals to the squares made by the diagonal bracing and are superfluous (**198**). The extreme depth of the main girders is 14 feet, and the depth from centre to centre of pins is 11 feet 6 inches, but the distance between the centres of gravity of the flanges is 12 feet 9 inches, or nearly $\frac{1}{12}$th of the span, and this seems to have been assumed to be the correct depth for calculating the working strains, which with $1\frac{1}{4}$ ton per foot on each line are 5 tons tension per square inch of net section, and 4 tons compression

* See *Proc. Inst. C.E.*, Vol. xxii.; and *Trans. Soc. Eng. for* 1864.

per square inch of gross section. The cross girders are attached to the under sides of the lower flanges, and have cantilever ends which extend outside the main girders and support footpaths 7 feet wide. These cross-girders are 11 feet apart, and agree with the apices of the diagonals in the lower flanges. There are four lines of railway, and the width in the clear between main girders is 46 feet 4 inches. The weight of one main girder (supporting two lines) is as follows:—

	Tons.	Cwts.	Qrs.
Top flange, . . .	70	4	2
Bottom do., . . .	67	15	2
Web, ...	46	0	0
End pillars, . . .	6	0	0
Total weight of one main girder, .	190	0	0

Taking the rolling load at 1¼ tons per foot of single line, the total distributed load on each main girder is as follows:—

	Tons.
Rolling load 156 × 2½ tons, ...	390
One main girder, deducting end pillars, . .	184
One-half the cross-girders and cantilevers (11 feet apart),	67
Rails for 2 lines, ...	7
Timber in half the platform and longitudinals under rails,	41
Load of people on one footpath at 100 lbs. per square foot,	48⅓
Total distributed load on one girder, . .	737⅓

The above load is exclusive of cornice, hand-rail, fish plates, bolts, spikes, chairs for rails, hoop-iron tongue and bolts for planking, (and ballast?).

CONWAY PLATE TUBULAR BRIDGE.*

557.—The Conway bridge was erected by Mr. Robert Stephenson to carry the Chester and Holyhead Railway over the river Conway,

* *Britannia and Conway Tubular Bridges.* By E. Clark.

in North Wales. It consists of two wrought-iron plate tubular bridges placed side by side, with one line of railway in each tube.

The entire length of each tube is 424 feet, and the clear span 400 feet, which allows a bearing of 12 feet at each end. The external depth at the centre is 25 feet 6 inches, or nearly $\frac{1}{16}$th of the length, thence it diminishes gradually towards the ends where it is 22 feet 6 inches. The external width is 14 feet 9 inches. The clear width inside is about 12 feet 6 inches. The tubes are placed 9 feet apart and are not connected in any way.

TABLE XI.—TABULAR SUMMARY OF WROUGHT-IRON WORK IN THE CONWAY BRIDGE.—ONE TUBE, SINGLE LINE, LENGTH 424 FEET.

	Upper Flange.	Sides.	Lower Flange.	Summary.
	Tons.	Tons.	Tons.	Tons.
Plates,	239	201	242	682
Angle and T-iron,	115	146	59	320
Covers,	15	21	78	113
Rivet-heads,	23	24	17	64
Total,	392	393	395	1180

Plates, 58 per cent.; angle and T-iron, 27 per cent.; covers, 10 per cent.; rivet-heads, 5 per cent.; total, 100.

The following is an analysis of the wrought-iron in one tube 412 feet long, *i.e.*, 6 feet longer at each end than the clear span. This was the length of the tube when floated into its place between the abutments; 6 feet were afterwards added to each end.

TOP FLANGE.

	Tons.	Cwts.	Per cent.
Plates and angle-iron in compression,	336	0	87·5
Plates and angle-iron acting as covers,	17	8	4·5
Transverse keelsons,	7	0	2·0
Rivet-heads,	22	7	6·0
	382	15	100·0

SIDES.

	Tons.	Cwts.	Per cent.
Plates acting as sides, . . .	163	0	43·0
Covers and proportion of T-iron acting as covers, ...	90	10	24·0
Gussets, stiffeners, and projecting rib of T-iron engaged in stiffening the sides,	101	16	27·0
Rivet-heads, ...	23	15	6·0
	379	1	100·0

LOWER FLANGE

	Tons.	Cwts.	Per cent.
Plates and angle-iron in tension, .	279	9	72·5
Plates and angle-iron acting as covers, .	76	6	20·0
Transverse keelsons, . . .	14	0	3·5
Rivet-heads, ...	15	17	4·0
	385	12	100·0

This makes the total weight of wrought-iron in 412 feet of one tube = 1147·4 tons, or 2·78 tons per running foot for each line.

Summary of cast-iron work in the Conway Bridge for both lines:—

	Tons.
Castings fixed in the ends of tubes, . .	201
Bed-plates, rollers, &c., . . .	108
Fixed in the masonry, .	325
Total weight of castings for both tubes,	634

The weight of wrought-iron in each tube, 400 feet long in the clear, is 1112 tons, and the working inch-strains, as already given in Table V. (**483**), are 6·85 tons tension and 5 tons compression with a train-load of one ton per foot uniformly distributed.

The mean deflection of the two tubes, immediately after the removal of the platform on which they were built, was 8·04 inches inches, and 8·98 inches after taking their permanent set due to the strain (**412**). The deflection, from additional weight placed at the centre, is ·01104 inch per ton. The difference of deflection due to change of temperature, between noon and midnight on the 5th July, 1848, was 1·56 inches (**419**).

TORKSEY BRIDGE, PLATE TUBULAR GIRDERS.*

558.—The Torksey bridge is a continuous girder bridge in two equal spans, and was erected by Mr. Fowler to carry the Manchester, Sheffield, and Lincolnshire Railway over the river Trent. Each span is 130 feet long in the clear, with a double line of railway between two plate tubular main girders with cellular top flanges. The main girders are 25 feet apart, with single-webbed plate cross-girders, 14 inches in depth and 2 feet apart, attached to the lower flanges. The extreme depth of each main girder is 10 feet. The depth from centre to centre of flanges is 9 feet $4\frac{5}{8}$ inches, or $\frac{1}{14}$th of each span. The gross sectional area of each top flange at the centre of each opening is 51 inches, the net area of each lower flange, is about 55 inches. The thickness of each plate of the web at the same place is $\frac{1}{2}$ inch, increasing to $\frac{5}{8}$ inch over the abutments and central pier.

The load on each span of 130 feet was estimated as follows:—

	Tons.	Tons.
Rails and chairs, . . .	8	
Timber platform, . . .	15	
Cross-girders, . . .	27	177
Ballast, 4 inches thick, . .	35	
Two main girders, . . .	92	
Rolling load, as agreed upon by Mr. Fowler and Capt. Simmons (Government Inspector),		195
Total distributed load, .		372 tons.

The strength of the Torksey bridge as a continuous girder was calculated by Mr. Pole from the following data (**251**):—

The length of each span = 130 feet = 1,560 inches.

* *Proc. Inst. C.E.*, Vol. ix.

The total distributed load on the first span = 400 tons, or for each girder 200 tons.

The distributed load on the second span = 164 tons, or for each girder 82 tons.

The coefficient of elasticity is taken equal to 10,000 tons for a bar one inch square.

By eqs. 166, 167, and 168, the pressures of one girder on the points of support are as follows:—

$$R_1 = 82{\cdot}375 \text{ tons.}$$
$$R_2 = 176{\cdot}250 \text{ tons.}$$
$$R_3 = 23{\cdot}375 \text{ tons.}$$

By eq. 169, the distance of the point of inflection in the loaded span is 22 feet 11 inches from the centre pier. The moment of inertia = 372,500 by Mr. Pole's calculation. The distance of the top plates from the neutral axis = 64 inches; that of the bottom plates from the same axis = 56 inches, and the maximum strains in the flanges of the loaded segment, 107 feet long, are 4·55 tons compression per square inch of gross area in the top flange, and 4 tons tension per square inch of net area in the bottom flange. The deflection, with 222 tons distributed over one span, was 1·26 inches.

BROTHERTON PLATE TUBULAR BRIDGE.*

559.—The Brotherton bridge, on the York and North Midland Railway is a plate tubular bridge with one line of railway in each tube. The span is 225 feet, the depth 20 feet or $\frac{1}{11}$th of the length nearly, and the width of each tube between the side plates is 11 feet.

The weight of one tube is as follows:—

Wrought-iron between the bearings, .	198 tons.
Wrought-iron on the bearings, . .	13 ,,
Cast-iron on the bearings, . .	14½ ,,
Cast-iron in rollers and plates, . .	9½ ,,
Total weight, . .	235 tons.

* *Encycl. Brit.*, Art. "Iron Bridges," p. 609.

The top flange is composed of a single plate, and no cells whatever have been used either in the top or bottom.

PLATE GIRDER BRIDGE OVER THE ROCHDALE CANAL.*

560.—This bridge was erected by Mr. Fowler to carry the Manchester, Sheffield, and Lincolnshire Railway over the Rochdale canal. The main girders are single-webbed plate girders, 7 feet 5 inches deep, and 24 feet $\frac{3}{8}$ inch apart from flange to flange, with a double line of railway resting on cross-girders, which are 15 inches in depth, 2 feet from centre to centre and attached to the lower flanges. Each main girder is 168 feet long, but divided by an intermediate pair into two unequal spans of 96 and 60 feet respectively.

The weight of each span is as follows:—

Over the large span 96 feet in length—

	Tons.
Two main girders,	44
Cross-girders,	29·5
Timber platform,	11
Rails, &c.,	4
Maximum rolling load, say 2 tons per running foot,	192
Total distributed load on large span,	280·5

Over the small span 60 feet in length—

	Tons.
Two main girders,	20
Cross-girders,	18·5
Wooden platform,	7
Rails, &c.,	2·5
Maximum load 2 tons per running foot,	120
Total distributed load on small span,	168·0

* *Humber on Bridges and Girders.*

Regarding each span as an independent bridge the strains in the flanges are as follows:—

	Large span.	Small span.
Compression flange, .	3·95 tons per square inch.	4·73 tons per square inch.
Tension flange, .	4·24 do. do.	5·31 do. do.

In computing the sectional area of the flanges, the angle irons by which they are united to the web have not been taken into consideration; this will perhaps compensate for the weakening of the plates of the tension flange by the rivet holes.

561. Mechanical properties of steel—Size and weights of various materials.—The first of the following tables contains the results of recent experiments by Mr. Fairbairn on the strength of steel, which were published too late to introduce into the body of the book. The remaining tables refer chiefly to the size and weights of various materials, and will be found useful for reference.

TABLE XII.—EXPERIMENTS ON THE MECHANICAL PROPERTIES OF STEEL BY W. FAIRBAIRN, LL.D., F.R.S., &c.

(*Report of the British Association for* 1867.)

MANUFACTURERS.	Coefficients of Rupture. S. (59).	Tearing strain per square inch.		Corresponding elongation, or ultimate set (350).	Greatest crushing weight laid on per square inch.		Corresponding contraction, or set due to compression.
	Tons.	lbs.	Tons.	per cent. of the original length.	lbs.	Tons.	per cent. of the original length.
MESSRS. BROWN & CO.							
Best cast-steel from Russian and Swedish iron, for turning-tools,	6·326	68,404	30·53	·25	225,568	100·7	25·3
Do. milder,	6·326	91,520	40·85	1·50	"	"	26·3
Cast-steel from Swedish iron, for tools,	6·958	106,714	47·64	1·00	"	"	18·3
Do. milder, for chisels,	6·134	116,183	51·86	3·62	"	"	29·3
Do. mild, for welding,	6·134	110,055	49·13	3·31	"	"	24·3
Bessemer steel,	5·297	91,972	41·05	19·62	"	"	40·3
Specimen of double shear steel from Swedish iron,	5·527	92,555	41·31	5·43	"	"	44·3
Do., foreign bar, tilted direct,	5·394	76,474	34·27	13·56	"	"	49·3
English tilted steel, made from English and foreign pigs,	5·170	59,538	26·57	21·06	"	"	55·3
C. CAMMELL & CO.							
Specimen of cast-steel, termed "Diamond Steel,"	7·504	110,055	49·13	1·53	"	"	23·3
Do. do. termed "Tool Steel,"	5·904	109,072	48·69	1·50	"	"	26·3
Do. do. termed "Chisel Steel,"	7·413	120,398	53·75	2·50	"	"	31·3
Do. do. termed "Double Shear Steel,"	5·132	96,665	43·15	2·37	"	"	30·3
Bar of hard Bessemer steel,	4·588	89,121	39·78	20·87	"	"	48·3
Do. soft do.	4·988	81,483	36·37	20·43	"	"	49·3
MESSRS. NAYLOR, VICKERS, & CO.							
Cast steel, called "Axle Steel,"	5·742	88,665	39·58	16·25	"	"	42·3
Do. do. "Tyre Steel,"	5·505	91,520	40·85	9·00	"	"	38·8
Do. do. "Vickers' Cast-steel, Special,"	7·856	134,145	59·87	1·00	"	"	15·3
Do. do. "Naylor & Vickers' Cast-steel,"	7·358	118,066	52·70	1·75	"	"	18·3
S. OSBORNE.							
Specimen of best tool, cast-steel,	5·432	98,942	44·17	0·93	"	"	20·3
Do. best chisel, do.	6·400	123,686	55·21	3·18	"	"	24·3
Sates-cup, shear-blades, and boiler-makers' steel,	4·691	115,849	51·71	2·12	"	"	25·3

Best cast-steel for taps and dies,	6·037	98,790	44·10	1·68	,,	,,	26·3
Toughened cast-steel for shafts, &c.,	5·559	103,116	46·03	5·25	,,	,,	32·3
Specimen of best double shear-steel,	4·329	87,931	39·25	2·43	,,	,,	32·3
Extra best tool, cast-steel,	6·860	85,724	38·26	0·43	,,	,,	19·3
Cast-steel for boiler plates,	5·671	111,676	49·85	13·50	,,	,,	33·3
H. Bessemer.							
Specimen of hard Bessemer steel,	6·882	103,085	46·02	1·87	,,	,,	22·3
Do. milder do.	5·317	88,175	39·36	20·00	,,	,,	44·3
Do. soft do.	4·778	78,606	35·09	19·12	,,	,,	47·3
Sanderson, Brothers,							
Bar of cast-steel from Russian iron suitable for welding,	5·539	83,484	37·26	2·25	,,	,,	39·8
Specimen of double-shear steel,	4·808	107,940	48·18	3·31	,,	,,	30·3
Do. single do.	6·780	107,182	47·84	2·81	,,	,,	28·3
Bar of faggot-steel, welded,	5·572	75,199	33·57	1·25	,,	,,	32·3
Specimen of drawn bar, not welded,	4·907	103,960	46·41	3·43	,,	,,	33·3
Messrs. Turton & Sons.							
Steel intended for the manufacture of cups,	5·392	100,155	44·71	2·75	,,	,,	28·3
Do. do. drills,	6·625	87,552	39.08	1·06	,,	,,	19·3
Do do. cutters,	6·718	95,372	42·57	1·37	,,	,,	24·3
Do. do. turning tools,	6·337	80,273	35·02	0·12	,,	,,	26·3
Do. do. machinery,	6·576	102,915	45·94	1·43	,,	,,	23·3
Do. do. punches,	5·440	102,567	45·79	1.62	,,	,,	25·3
Do. do. mint dies,	5·861	106,237	47·42	2·87	,,	,,	29·3
Do. do. dies,	5·570	87,471	39·04	0·87	,,	,,	27·3
Do. do. taps,	5·788	97,994	43·74	1·87	,,	,,	27·3
Specimen of double shear-steel,	4·561	73,266	32·70	0·81	,,	,,	29·3

The mean coefficient of elasticity, derived from experiments on deflection under moderate transverse strain = 31,000,000 lbs. per square inch. The coefficients of rupture, S (mean = 6 tons,) were derived from experiments on the transverse strength of bars 1 inch square, 4 feet 16 inches long between supports, and loaded at the centre. The tearing and the crushing experiments were subsequently made on portions of the same bars which had been previously strained by transverse pressure. The mean tearing strength per square inch = 47·7 tons. The elongations (ultimate sets) were taken on eight-inch lengths of bar. Thirty-two of the bars supported a compression of 100·7 tons per square inch without undergoing any sensible fracture, whilst twenty-three bars were more or less fractured with this pressure, but all of them bulged laterally under pressure, some, it will be perceived by the last column, to a very considerable extent.

TABLE XIII.—VALUES OF GAGES FOR WIRE AND SHEET METALS IN GENERAL USE, EXPRESSED IN DECIMAL PARTS OF THE INCH.*

Birmingham Gage for Iron Wire and for Sheet Iron and Steel.		Birmingham Gage for Sheet Metals, Brass, Gold, Silver, &c.		Lancashire gage for round Steel Wire, and also for Pinion Wire. The smaller sizes distinguished by numbers. The larger by letters, and called the Letter Gage.					
Mark.	Size.	Mark.	Size.	Mark.	Size.	Mark.	Size.	Mark.	Size.
0000	·454	1	·004	80	·013	40	·096	A	·234
000	·425	2	·005	79	·014	39	·098	B	·238
00	·380	3	·008	78	·015	38	·100	C	·242
0	·340	4	·010	77	·016	37	·102	D	·246
1	·300	5	·012	76	·018	36	·105	E	·250
2	·284	6	·013	75	·019	35	·107	F	·257
3	·259	7	·015	74	·022	34	·109	G	·261
4	·238	8	·016	73	·023	33	·111	H	·266
5	·220	9	·019	72	·024	32	·115	I	·272
6	·203	10	·024	71	·026	31	·118	J	·277
7	·180	11	·029	70	·027	30	·125	K	·281
8	·165	12	·034	69	·029	29	·134	L	·290
9	·148	13	·036	68	·030	28	·138	M	·295
10	·134	14	·041	67	·031	27	·141	N	·302
11	·120	15	·047	66	·032	26	·143	O	·316
12	·109	16	·051	65	·033	25	·146	P	·323
13	·095	17	·057	64	·034	24	·148	Q	·332
14	·083	18	·061	63	·035	23	·150	R	·339
15	·072	19	·064	62	·036	22	·152	S	·348
16	·065	20	·067	61	·038	21	·157	T	·358
17	·058	21	·072	60	·039	20	·160	U	·368
18	·049	22	·074	59	·040	19	·164	V	·377
19	·042	23	·077	58	·041	18	·167	W	·386

* From Holtzapffel's Mechanical Manipulation.

TABLE XIII.—VALUES OF GAGES FOR WIRE AND SHEET METALS IN GENERAL USE EXPRESSED IN DECIMAL PARTS OF THE INCH—*continued*.

Birmingham Gage for Iron Wire and for Sheet Iron and Steel.	Birmingham Gage for Sheet Metals, Brass, Gold, Silver, &c.	Lancashire gage for round Steel Wire, and also for Pinion Wire. The smaller size distinguished by numbers. The larger by letters, and called the Letter Gage.		
Mark. Size.	Mark. Size.	Mark. Size.	Mark. Size.	Mark. Size.
20 — ·035	24 — ·082	57 — ·042	17 — ·169	X — ·397
21 — ·032	25 — ·095	56 — ·044	16 — ·174	Y — ·404
22 — ·028	26 — ·103	55 — ·050	15 — ·175	Z — ·413
23 — ·025	27 — ·113	54 — ·055	14 — ·177	A 1 — ·420
24 — ·022	28 — ·120	53 — ·058	13 — ·180	B 1 — ·431
25 — ·020	29 — ·124	52 — ·060	12 — ·185	C 1 — ·443
26 — ·018	30 — ·126	51 — ·064	11 — ·189	D 1 — ·452
27 — ·016	31 — ·133	50 — ·067	10 — ·190	E 1 — ·462
28 — ·014	32 — ·143	49 — ·070	9 — ·191	F 1 — ·475
29 — ·013	33 — ·145	48 — ·073	8 — ·192	G 1 — ·484
30 — ·012	34 — ·148	47 — ·076	7 — ·195	H 1 — ·494
31 — ·010	35 — ·158	46 — ·078	6 — ·198	
32 — ·009	36 — ·167	45 — ·080	5 — ·201	
33 — ·008		44 — ·084	4 — ·204	
34 — ·007		43 — ·086	3 — ·209	
35 — ·005		42 — ·091	2 — ·219	
36 — ·004		41 — ·095	1 — ·227	

Column 1 refers to the gage commonly called the *Birmingham Wire Gage*, which is employed for iron, brass, and other wires, for black steel wire, for sheet iron, sheet steel, and various other materials.

The gage referred to in the second column is called the *Birmingham Metal Gage*, or the *Plate Gage*, and is employed for most of the sheet metals, excepting iron and steel.

TABLE XIV.—Weight of a superficial Foot of various Metals in lbs.

NAMES.	THICKNESS BY THE BIRMINGHAM WIRE GAGE.														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Wrought-Iron,	12·50	12·00	11·00	10·00	8·74	8·12	7·50	6·86	6·24	5·62	5·00	4·38	3·75	3·12	2·82
Copper, .	14·50	13·90	12·75	11·60	10·10	9·40	8·70	7·90	7·20	6·50	5·80	5·08	4·34	3·60	3·27
Brass, .	13·75	13·10	12·10	11·00	9·61	8·93	8·25	7·54	6·86	6·18	5·50	4·81	4·12	3·43	3·10
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
Wrought-Iron,	2·50	2·18	1·86	1·70	1·54	1·40	1·25	1·12	1·00	·90	·80	·72	·64	·56	·50
Copper, .	2·90	2·52	2·15	1·97	1·78	1·62	1·45	1·30	1·16	1·04	·92	·83	·74	·64	·58
Brass, .	2·75	2·40	2·04	1·87	1·69	1·54	1·37	1·23	1·10	·99	·88	·79	·70	·61	·55

NAMES.	THICKNESS IN PARTS OF AN INCH.											
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
Wrought-Iron,	2·5	5·0	7·5	10·0	12·5	15·	17·5	20·	25·	30·	35·	40·
Copper, .	2·9	5·8	8·7	11·6	14·5	17·2	20·0	23·2	28·9	34·3	40·4	46·2
Brass, .	2·7	5·5	8·2	11·0	13·7	16·4	19·0	21·8	27·4	32·5	37·9	43·3
Lead, .	3·7	7·4	11·1	14·8	18·5	22·2	25·9	29·6	37·0	44·4	51·8	59·2
Zinc, .	2·3	4·7	7·0	9·4	11·7	14·0	16·4	18·7	23·4	28·1	32·8	37·5

TABLE XV.—SPECIFIC GRAVITY AND WEIGHT OF A CUBIC FOOT OF DIFFERENT WOODS.*

Kind of Wood, and state.	Specific gravity.	Weight of a cubic foot in pounds.
Abele, dry	·511 T.	32·00
Acacia (false) green	·820 E.	51·25
Do., dry	·791 H.	49·43
Do., dry	·748 T.	46·75
Do., (three-thorned)	·676 H.	42·25
Alder	·800 M.	50·00
Do., dry	·555 E.	34·68
Almond tree	1·102 H.	68·87
Apple tree	·793 M.	49·56
Apricot tree	·789 H.	49·31
Arbor vitæ (Chinese)	·560 H.	35·00
Ash (heart-wood) dry	·845 P.	52·81
Do., dry	·832 W.	52·00
Do., young wood, dry	·811 T.	50·68
Do.	·800 J.	50·00
Do.	·760 B.	47·50
Do. (old tree) dry	·753 T.	47·06
Do., dry	·690 E.	43·12
Bay tree	·822 M.	51·37
Beech (meanly dry)	·854 P.	53·37
Do.	·852 M.	53·25
Do.	·720 H.	45·00
Do.	·696 B.	43·50
Do., dry	·690 E.	43·12
Birch, dry	·720 E.	45·00
Box (Dutch)	1·328 M.	83·00
Do., dry	1·030 J.	64·37
Do.	1·031 P.	64·43
Do. {from	1·024 B.	64·00
{to	·960 B.	60·00
Do., dry	·950 W.	59·37
Do., Turkey	·949 R.	59·31
Brazil wood (red)	1·031 M.	64·43
Canary wood	·723 R.	45·18
Cedar (Indian)	1·315 M.	82·18
Do. (Canadian)	·753 C.	47·06
Do. (Virginian red) dry	·650 T.	40·62
Do. (Palestine)	·596 M.	37·25
Do. (American)	·560 M.	35·00
Do. do. seasoned	·453 C.	28·31
Cedar of Libanus	·603 H.	37·68
Do. do. dry	·486 T.	30·37
Cherry tree	·741 H.	46·31
Do. do., dry	·672 T.	42·00
Chesnut (sweet) green	·875 E.	54·68
Do.	·685 H.	42·81
Chesnut (sweet) dry	·606 T.	37·95
Do., another specimen, dry	·535 T.	33·45
Do. (horse)	·657 H.	41·06
Chestnut (horse), dry	·596 T.	37·28
Do. do., another specimen, dry	·483 T.	30·18
Cocoa wood	1·040 M.	65·00
Cork	·240 M.	15·00
Cowrie	·579	36·20
Crab tree, meanly dry	·765 P.	47·81
Cypress	·655 H.	40·93
Do. (Spanish)	·644 M.	40·25
Deal, white. See fir		
Do., yellow. See pine		
Ebony (American)	1·331 M.	83·18
Do. (Indian)	1·209 M.	75·56
Do.	1·108 R.	69·25
Elder tree	·695 M.	43·43
Elm, green	·940 C.	58·75
Do.	·693 S.	44·41
Do., seasoned	·588 C.	36·75
Do.	·553 B.	34·56
Do. (common) dry	·544 E.	34·00
Do., wych, young tree, green	·763 E.	47·68
Do. do. dry	·684 T.	42·75
Filbert tree	·600 M.	37·50
Fir (Norway spruce)	·512 T.	32·00
Do., (white American spruce)	·465 T.	29·06
Do. (silver) green	·531 Wi.	33·20
Do., dry	·408 Wi.	25·22
Do. (Scotch.) See pine		
Fustic	·817 R.	51·06
Hazel	·606 M.	37·87
Hickery	·929 S.	58·06
Hornbeam	·760 H.	47·50
Jasamine (Spanish)	·770 M.	48·12
Juniper wood	·556	34·75
Laburnum	·843 T.	52·70
Lance wood	1·038 L.	64·87
Do. do., dry	·943 R.	58·93
Larch, green	·858 Wi.	53·63
Do. (red wood) seasoned	·640 T.	40·00
Do., dry	·612 Wi.	38·31
Do., dry	·496 T.	31·00
Do., (white wood) seasoned	·364 T.	22·75
Lemon tree	·703	43·93
Letter wood	1·286 C.	80·37
Lignum vitæ	1·333 M.	83·31
Do.	1·327 P.	82·93

* *Tredgold's Carpentry*, p. 298.

TABLE XV.—SPECIFIC GRAVITY AND WEIGHT OF A CUBIC FOOT OF DIFFERENT WOODS—*continued.*

Kind of Wood, and state.	Specific gravity.	Weight of a cubic foot in pounds.
Lime tree	·604 M.	37·75
Do.	·564 H.	35·25
Do.	·480 T.	30·00
Logwood	·913 P.	57·06
Mahogany (Spanish) dry	·852 T.	53·30
Do. dry	·816 W.	51·00
Do. (Honduras) dry	·560 T.	35·00
Maple (Norway)	·795 L.	49·68
Do. dry	·755 P.	47·18
Do. (common) dry	·624 T.	32·75
Medlar tree	·944 M.	59·00
Mulberry tree (Spanish)	·897 M.	56·06
Oak (live) half seasoned	1·216 Ch.	76·03
Do. (English green)	1·113 C.	69·56
Do. (French green)	1·063 Bu.	66·43
Do. (Irish bog)	1·046 C.	65·37
Do. (evergreen)	·994 H.	62·25
Do. (Adriatic)	·993 B.	62·06
Do. (black bog) dry	·965 R.	60·31
Do. (white American) half seasoned	·908 Ch.	56·75
Do. (*Quercus sessiliflora*)	·879 T.	54·97
Do. (American white)	·840 H.	52·50
Do. (Provence) seasoned	·828 D.	51·75
Do. (*Quercus robur*) dry	·807 T.	50·47
Do. (English) seasoned	·777 C.	48·56
Do. (Dantzic) seasoned	·755 T.	47·24
Do. (American) red	·752 L.	47·00
Do. (Riga) dry	·688 T.	43·00
Do. (English) from an old tree, dry	·625 T.	39·06
Olive tree	·927 M.	57·93
Orange tree	·705 M.	44·06
Pear tree, dry	·708 T.	44·25
Do.	·646 B.	40·37
Pine (American pitch) dry	·936 T.	58·5
Do. (do.) seasoned	·741 C.	46·31
Do. (pinaster) green	·837 Wi.	52·35
Do. (Scotch) green	·816 Wi.	51·08
Do. (Mar Forest)	·696 B.	43·50
Pine (planted Scotch) dry	·529 T.	33·06
Do. (Scotch) dry	·429 Wi.	26·81
Do. (Memel) dry from	·553	34·56
to	·544 T.	34·00
Do. (Riga) dry from	·480	30·00
to	·466 T.	29·12
Do. (Weymouth) dry	·460 T.	28·75
Do. (American) dry	·368 T.	23·00
Plane (occidental) dry	·648 E.	40·50
Do. (oriental)	·538 H.	33·62
Plane tree (common). See sycamore		
Plum tree	·785 M.	49·06
Do.	·663 P.	41·43
Poona (seasoned)	·635 C.	39·95
Poplar (Spanish, white)	·529 M.	33·06
Do. (black) dry	·421 T.	26·31
Do. (Lombardy) dry	·374 E.	24·37
Quince tree	·705 M.	44·00
Sassafras	·482 P.	30·12
Satin wood	·952 R.	59·50
Saul (Bengal) seasoned	·994 L.	62·12
Service tree	·742 H.	46·37
Sissoo (Bengal) seasoned	·889 L.	55·52
Stinkwood (seasoned)	·681 C.	42·56
Sycamore	·645 H.	40·31
Do. dry	·590 E.	36·87
Teak, dry	·832 Ch.	52·00
Do.	·745 B.	46·56
Do. seasoned	·657 C.	41·06
Tulip tree	·477 H.	29·81
Vine	1·237 M.	77·31
Walnut tree, green	·920 E.	57·50
Do. (American)	·735 H.	45·93
Do. (French)	·671 M.	41·93
Do. dry	·616 T.	38·50
Willow, green	·619 E.	38·68
Do. dry from	·568	35·50
to	·404 T.	25·25
Yellow wood (seasoned)	·657 C.	41·06
Yew (Spanish)	·807 M.	50·43
Do. (Dutch)	·788 M.	49·25
Do.	·788 H.	48·62

The letters following the specific gravities refer to the authorities—B., Barlow; Bu., Buffon; C., Couche; Ch., from *Chapman on Preservation of Timber;* E., Ebbels; H., from Rondelet's table; J., Jurin; L., Layman; M. Muschenbroek; P., *Philosophical Transactions*, Vol. i., *Lowthorp's Abridgement;* R. Ralph Tredgold; S. Scoresby; T., Tredgold; W., Watson (Bishop); Wi., Wiebeking.

TABLE XVI.—SPECIFIC GRAVITY AND WEIGHT OF A CUBIC FOOT OF VARIOUS MATERIALS.*

Name of the Substance.	Specific gravity.	Weight of a cubic foot in pounds.	Name of the Substance.	Specific gravity.	Weight of a cubic foot in pounds.
Air (atmospheric),	·0012	·075	Copper (British sheet),	8·785 Ha.	549·06
Alabaster. See gypsum.			(Do. (British cast),	8·607 Ha.	537·93
Basalt, from	3·00	187·50	Earth (common), from	1·520	95·00
to	2·478	154·87	to	1·984	124·00
Do. (Fairhead),	2·95 K.	184·37	Do. (loamy or strong),	2·016	126·00
Do. (Derbyshire),	2·921 W.	182·56	Do. (rammed),	1·584 Pa.	99·00
Do. (Giant's Causeway),	2·90 K.	181·25	Do. (loose or sandy),	1·520	95·00
			Firestone,	1·800	112·50
Do. do.	2·864 Br.	179·00	Flint, from	2·580	161·25
Do. (Rowley rag),	2·478 K.	154·87	to	2·630 Th.	164·37
Bees' wax (yellow),	·965	60·31	Do. (black Cambridge)	2·592 W.	162·00
Bismuth (cast),	9·822	613·87	Freestone. See stone.		
Bitumen, of Judea,	1·104	69·00	Glass, white flint,	3·000	187·50
Brass (wire drawn),	8·544	534·00	Do., plate,	2·760	172·50
Do. (plate),	8·441 W.	527·56	Do., crown,	2·520	157·50
Do. (cast),	8·100 P.	506·25	Gold, pure, cast,	19·361 Br	1210·06
Brick (common), from	1·557	97·31	Do., standard,	17·724 Th.	1107·75
to	2·000	125·00	Granite, from	2·999	187·47
Do. (red),	2·168 Re.	135·50	to	2·538 K.	158·62
Do. (pale red),	2·085 Re.	130·31	Do. (Guernsey),	2·999 W.	187·47
Do.,	1·857 Be.	116·06	Do. (Aberdeen gray),	2·664 R.	166·5
Do. (common London stock),	1·841 T.	115·06	Do. (Cornish),	2·662 Re.	166·37
			Do. (do.),	2·653 R.	165·81
Do. paving (English clinker),	1·653 R.	103·31	Do. (Aberdeen red),	2·643 R.	165·18
			Do. (Cornish),	2·624 T.	164·00
Do. (Dutch clinker),	1·432 R.	92·62	Gravel,	1·749 P.	109·32
Do. (Welsh fire),	2·408 T.	150·50	Gunpowder (solid),	1·745	109·06
Brickwork, about		95·00	Do. (shaken),	·922	57·62
Cement (Roman) and sand in equal parts,	1·817 T.	113·56	Gypsum (plaster stone),	2·286 W.	142·87
			Iron (bar), from	7·600	475·00
Do., alone (cast),	1·600 R.	100·00	to	7·800 K.	487·50
Chalk, from	2·315	144·68	Do., hammered,	7·763 M.	485·18
to	2·657 Th.	166·06	Do., not hammered,	7·600 M.	475·00
Do. (Cambridge clunch)	2·657 W.	166·06	Do. (cast), from	7·600	475·00
Do. (Dorking),	1·169 R.	116·81	to	7·200 Th.	450·00
Charcoal from birch,	·542 K.	33·87	Do. (horizontal ditto),	7·113 Re.	444·56
Do. from fir,	·441 K.	27·56	Do. (vertical castings),	7 074 Re.	442·12
Do. from oak,	·332 K.	20·75	Ivory,	1·826 P.	114·12
Do. from pine,	·280 K.	17·50	Lead (milled),	11·407 Th.	712·93
Clay (potter's), from	1·800	112·50	Do. (cast),	11·352 Br.	709·50
to	2·085 K.	130·31	Do., black. See Plumbago.		
Do. (common),	1·919 Be.	119·93			
Do., with gravel,	2·560	160·00	Lime, quick,	·843 Be.	52·68
Do., slate. See slate.			Limestone. See stone and marble.		
Coke,	·744 K.	46·50			
Coal (Kilkenny),	1·526 K.	95·37	Loam. See earth,		
Do. (Glasgow splint),	1·290 Th.	80·62	Marble, from	2·840	177·50
Do. (Cannel),	1·272 Th.	79·50	to	2·580	161·25
Do. (Newcastle caking),	1·269 Th.	79·31	Do., Parian white,	2·837 K.	177·31
			Do., veined white,	2·726 Re.	170·37

* *Tredgold's Carpentry*, p. 300.

TABLE XVI.—SPECIFIC GRAVITY AND WEIGHT OF A CUBIC FOOT OF VARIOUS MATERIALS—*continued.*

Name of the Substance.	Specific gravity.	Weight of a cubic foot in pounds.	Name of the Substance.	Specific gravity.	Weight of a cubic foot in pounds.
Marble, Carrara white,	2·717 K.	169·81	Plumbago, or black lead,	2·267	141·68
Do., do. blue, -	2·713 K.	169·56	Porphyry (green), -	2·875	179·68
Do., Italian black, -	2·712 K.	169·50	Do. (red), - -	2·793	174·56
Do., Derbyshire entrochal, - -	2·709 R.	169·31	Potstone, from	3·000	187·50
Do., Saxon gray, -	2·700 K.	168·75	Potstone, to	2·768 K.	173·00
Do., Brabant black, -	2·697 Re.	168·56	Puzzolana, from	2·570	160·62
Do., Derbyshire black,	2·690 W.	168·12	Puzzolana, to	2·850 K.	178·12
Do., Namur black, -	2·682 R.	167·62	Quartz (crystallized), -	2·655	165·93
Do., Sienna yellow, -	2·677 K.	167·31	Roe-stone. See stone.		
Do., Pallion brown figured,	2·586 R.	161·62	Road-grit. See sand.		
Marl, from	1·600	100·00	Sand (pure quartz), -	2·750	171·87
Marl, to	2·870 Th.	179·37	Do., river, - -	1·886 Be.	117·87
Mercury (fluid), -	13·568 Br.	848·00	Do. River Thames (best), - -	1·638 T.	102·37
Mortar, - -	1·715 Be.	107·18	Do., pit (clean but coarse),	1·610 T.	100·62
Do., of river sand three parts, of lime in paste two parts,	1·615 Ro.	100·93	Do., pit (fine-grained and clean),	1·523 T.	95·18
Do., do., do., well beat together,	1·893 Ro.	118·31	Do., scraped from London roads (road-grit),	1·494 T.	93·37
Do., of pit sand three parts, of lime in paste two parts,	1·588 Ro.	99·25	Do., pit (very fine grained),	1·480 T.	92·50
Do., do., do., well beat together,	1·903 Ro.	118·93	Do., River Thames (inferior),	1·454 T.	90·87
Do., of pounded tile three parts, of quick-lime two parts,	1·457 Ro.	91·06	Sandstone. See stone.		
Do., do., do., well beat together,	1·663 Ro.	103·93	Serpentine, Anglesey green,	2·683 R.	167·68
Do., common, of chalk lime, and sand, dry,	1·550 R.	96·87	Do., blackish green, -	2·574 K.	160·87
Do., the lining of an antique reservoir near Rome,	1·549 Ro.	96·81	Do., dark reddish brown	2·561 K.	160·06
Do., from the interior of an old wall, Rome,	1·414 Ro.	88·37	Silver, pure cast, -	10·474 Br.	654·62
Do., lime, sand, and hair, used for plastering, dry,	1·384 R.	86·50	Do., standard, -	10·312 Th.	644·50
Oolite. See stone, roe.			Slate, Welsh, - -	2·883 K.	180·50
Peat, hard,	1·329	83·06	Do., Anglesey, -	2·876 K.	179·75
Pebble (English), -	2·609	163·06	Do., Westmoreland, pale blue, -	2·791 W.	174 43
Pewter, - -	7·248	453·00	Do., do., dark blue, -	2·781 W.	173·81
Pitch, - - -	1·150 P.	71·87	Do., do., pale greenish blue,	2·768 W.	173·00
Plaster (cast), - -	1·286 Be.	80·37	Do., do., blackish blue, used for floors,	2·758 W.	172·37
Platina pure, - -	21·531 Th.	1345·68	Do., Welsh rag, -	2·752 K.	172·00
			Do., Westmoreland, fine grained pale blue,	2·732 W.	170·75
			Do., Cornwall, grayish blue,	2·512 K.	157·00
			Stone, Bath (roe-stone),	2·494 K.	155·87
			Do., do.	1·975 R.	123·43

TABLE XVI.—SPECIFIC GRAVITY AND WEIGHT OF A CUBIC FOOT OF VARIOUS MATERIALS—*continued.*

Name of the Substance.	Specific gravity.	Weight of a cubic foot in pounds.	Name of the Substance.	Specific gravity.	Weight of a cubic foot in pounds.
Stone, blue lias (limestone),	2·467 R.	154·18	Stone, Portland (roe-stone),	2·461 W.	153·81
Do., Bromley-fall (sand-stone),	2·506 Re.	156·62	Do., Portland (roe-stone)	2·423 Re.	151·43
Do., do.,	2·261 R.	141·31	Do., do., do.,	2·113 R.	132·06
Do., Bristol stone,	2·510	156·87	Do., pumice,	·629 R.	39·31
Do., Burford (dry piece),	2·049 P.	128·06	Do., Purbeck,	2·680 W.	167·50
Do., Caen (calcareous sandstone),	2·108 R.	131·75	Do., do.,	2·599 Re.	162·43
Do., Clitheroe limestone,	2·686 W.	167·87	Do., Roach Abbey (magnesian limestone),	1·893 R.	118·31
Do., Collalo, white (sandstone),	2·423 Re.	151·43	Do. (Tottenhoe calcareous sandstone),	1·800 T.	112·50
Do., do.,	2·040 R.	127·50	Do., Woodstock flag-stone,	2·614 K.	163·37
Do., Craigleith, sandstone,	2·452 Re.	153·25	Do., Yorkshire paving,	2·507 Re.	156·68
Do., do,	2·360 R.	147·50	Do., do., do.,	2·356 R.	147·25
Do., Derbyshire (red friable sandstone),	2·346 Re.	146·62	Stonework, mean weight according to Belidor, about		107·00
Do., Dundee,	2·530 Re.	158·12	Shingle,	1·424 Pa.	89·00
Do., do.,	2·517 T.	157·31	Steel, {from	7·780	486·25
Do., (grindstone)	2·143	133·93	Steel, {to	7·840 Th.	490·00
Do., Hedding-stone, lax kind,	2·029 P.	126·81	Syenite (Mount Sorrel),	2·621	163·81
			Tile (common plain),	1·853 R.	116·15
Do., Hilton (sandstone),	2·177 R.	136·06	Do.,	1·815 Be.	113·43
			Tin, hammered,	7·299 Br.	456·18
Do., Kentish rag,	2·675 R.	167·18	Do., pure cast,	7·291 Br.	455·68
Do., Ketton (roe-stone)	2·494 K.	155·87	Toadstone (Derbyshire),	2·921 W.	182·56
Do., do.,	2·058 R.	128·62	Tufa (Roman),	1·217 Ro.	76·06
Do., Kincardine (sandstone),	2·448 T.	153·00	Water, sea,	1·027 Th.	64·18
			Do., rain,	1·000	62·50
Do., Limerick (black compact limestone),	2·598 Re.	162·37	Whinstone (Scotch),	2·760 W.	172·50
			Wood ashes,	·933 P.	58·32
Do., Pennarth (limestone),	2·653 W.	165·81	Wood petrified,	2·341 P.	146·31
			Zinc,	7·028 W.	439·25

Part of the letters of reference are explained in a note to the preceding table. The rest are as follows:—Be., Belidor; Br., Brisson; Ha., Hatchet; K., from *Kirwan's Mineralogy;* Re., Rennie, *Phil. Magazine*, Vol. liii.; Ro., Rondelet; Th., from *Dr. Thomson's System of Chemistry*, 5th edition; Pa., Pasley, Course of Military Instruction.

TABLE XVII.—For Converting Tons into Lbs. Avoirdupois.

Tons.	Cwts.	Lbs.	Tons.	Cwts.	Lbs.
0·05	1	112	0·55	11	1,232
0·10	2	224	0·60	12	1,344
0·15	3	336	0·65	13	1,456
0·20	4	448	0·70	14	1,568
0·25	5	560	0·75	15	1,680
0·30	6	672	0·80	16	1,792
0·35	7	784	0·85	17	1,904
0·40	8	896	0·90	18	2,016
0·45	9	1,008	0·95	19	2,128
0·50	10	1,120	1·00	20	2,240

TABLE XVII.—FOR CONVERTING TONS INTO LBS. AVOIRDUPOIS—*continued.*

Tons.	Lbs.	Tons.	Lbs.	Tons.	Lbs.	Tons.	Lbs.
1	2,240	26	58,240	51	114,240	76	170,240
2	4,480	27	60,480	52	116,480	77	172,480
3	6,720	28	62,720	53	118,720	78	174,720
4	8,960	29	64,960	54	120,960	79	176,960
5	11,200	30	67,200	55	123,200	80	179,200
6	13,440	31	69,440	56	125,440	81	181,440
7	15,680	32	71,680	57	127,680	82	183,680
8	17,920	33	73,920	58	129,920	83	185,920
9	20,160	34	76,160	59	132,160	84	188,160
10	22,400	35	78,400	60	134,400	85	190,400
11	24,640	36	80,640	61	136,640	86	192,640
12	26,880	37	82,880	62	138,880	87	194,880
13	29,120	38	85,120	63	141,120	88	197,120
14	31,360	39	87,360	64	143,360	89	199,360
15	33,600	40	89,600	65	145,600	90	201,600
16	35,840	41	91,840	66	147,840	91	203,840
17	38,080	42	94,080	67	150,080	92	206,080
18	40,320	43	96,320	68	152,320	93	208,320
19	42,560	44	98,560	69	154,560	94	210,560
20	44,800	45	100,800	70	156,800	95	212,800
21	47,040	46	103,040	71	159,040	96	215,040
22	49,280	47	105,280	72	161,280	97	217,280
23	51,520	48	107,520	73	163,520	98	219,520
24	53,760	49	109,760	74	165,760	99	221,760
25	56,000	50	112,000	75	168,000	100	224,000

ATE IV.

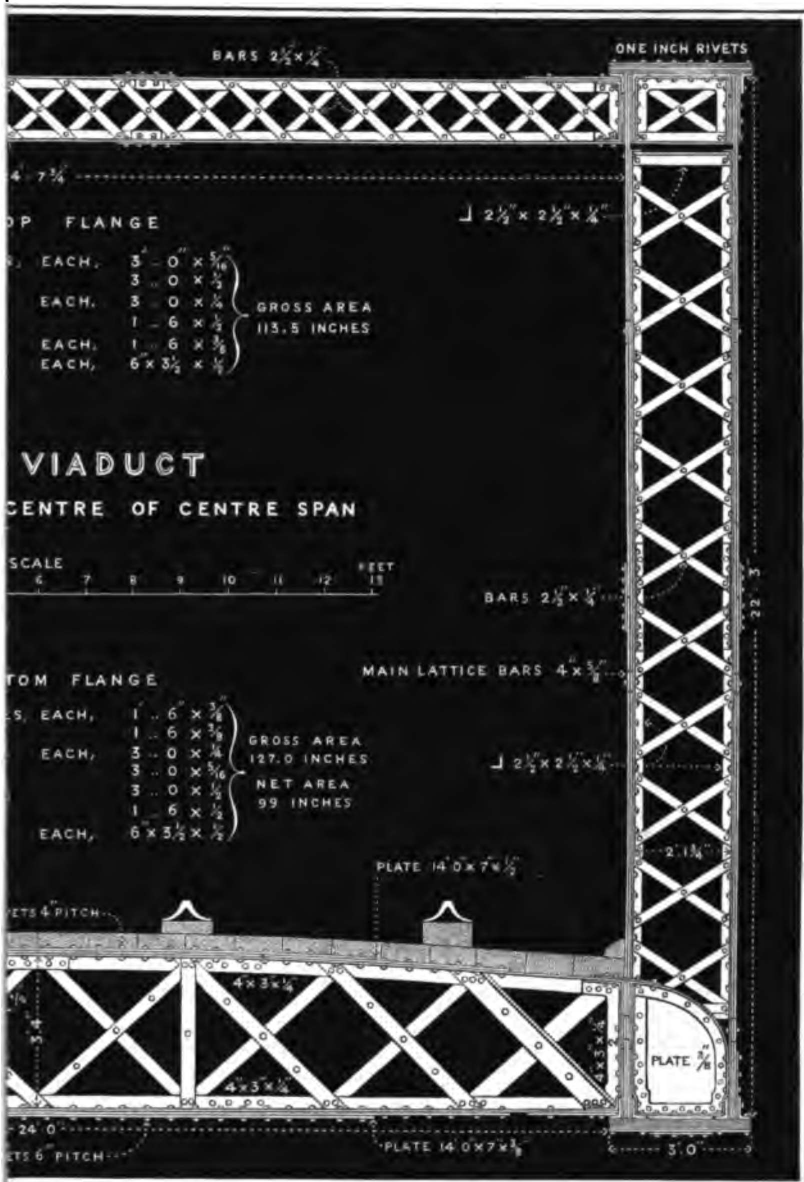
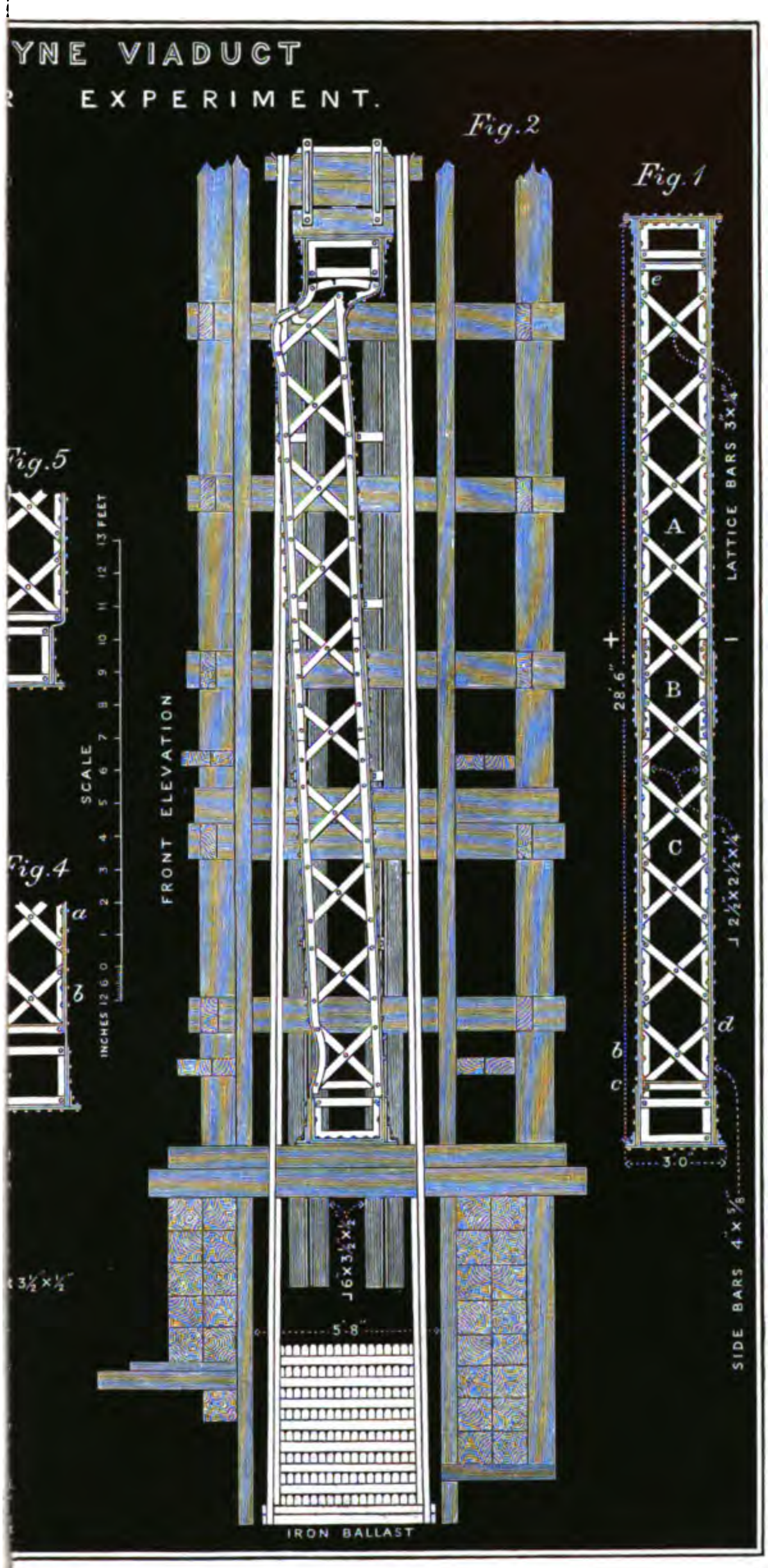

BARS 2½"×¼"
ONE INCH RIVETS
4' 7¾"
OP FLANGE
, EACH, 3' – 0" × 5/16"
3 .. 0 × ½
EACH, 3 .. 0 × ¼
1 .. 6 × ½
EACH, 1 .. 6 × ⅜
EACH, 6"× 3½ × ½
GROSS AREA
113.5 INCHES
2½"× 2½"× ⅜"
VIADUCT
CENTRE OF CENTRE SPAN
SCALE
6 7 8 9 10 11 12 13
FEET
BARS 2½"× ⅜"
22' 3"
MAIN LATTICE BARS 4"× ⅝"
TOM FLANGE
S, EACH, 1' .. 6" × ⅜"
1 .. 6 × ⅜
EACH, 3 .. 0 × ¼
3 .. 0 × 5/16
3 .. 0 × ½
1 .. 6 × ½
EACH, 6"× 3½ × ½
GROSS AREA
127.0 INCHES
NET AREA
99 INCHES
2½"× 2½"× ⅜"
2' 1¾"
PLATE 14' 0"× 7"× ½"
ETS 4" PITCH
4"× 3"× ¼"
3' 4"
4"× 3"× ¼"
4"× 3"× ⅜"
PLATE ⅜"
24' 0"
ETS 6" PITCH
PLATE 14' 0"× 7"× ⅜"
3' 0"

PLATE V.



YNE VIADUCT
EXPERIMENT.
Fig. 2
Fig. 1
Fig. 5
Fig. 4
SCALE
INCHES 12 6 0 1 2 3 4 5 6 7 8 9 10 11 12 13 FEET
FRONT ELEVATION
LATTICE BARS 3"×¼"
A
B
C
28'.6"
┘2½"×2½"×¼
a
b
c
d
e
3'.0"
SIDE BARS 4"×5/8
┘6"×3½"×½
3½"×½"
5'.8"
IRON BALLAST

INDEX.

2 L

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