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A TREATISE

ON

IRON BRIDGES, GIRDERS, ROOFS

ETC.

A TREATISE

On the Application of Iron to the Construction of

BRIDGES, GIRDERS, ROOFS

AND OTHER WORKS

SHOWING THE PRINCIPLES UPON WHICH SUCH STRUCTURES ARE DESIGNED, AND THEIR PRACTICAL APPLICATION

ESPECIALLY ARRANGED FOR THE USE OF STUDENTS AND PRACTICAL MECHANICS, ALL MATHEMATICAL FORMULÆ AND SYMBOLS BEING EXCLUDED

By FRANCIS CAMPIN, C.E.
PAST PRESIDENT OF THE CIVIL AND MECHANICAL ENGINEERS' SOCIETY, LONDON,
AUTHOR OF A "PRACTICAL TREATISE ON MECHANICAL ENGINEERING,"
ETC. ETC.

With numerous Illustrations

FOURTH EDITION, REVISED AND CORRECTED

Capio Lumen

LONDON
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1888

PREFACE.

In introducing the present work to the Public, a few prefatory words appear called for to explain the objects with which the Author prepared it. The great proficiency in mathematics requisite for the comprehension of more elaborate treatises on girders, roofs, &c., has rendered such works unsatisfactory to the great bulk of engineering students, and useless to those artizans who, having spent their early years in labour, have not had leisure for the more abstruse branches of education. For these two classes the present work has been especially written, after several years' consideration of the subject, and throughout care has been taken to preserve the pure principles of structures in their exactitude, without using any mathematical processes beyond arithmetic, except in the simple algebraical demonstrations of the rules which are inserted for the use of such as may desire to study them.

FRANCIS CAMPIN.

ADVERTISEMENT TO THE SECOND EDITION.

A SECOND edition of this Treatise having been called for, advantage has been taken of the opportunity thus offered to correct certain typographical and clerical errors that found their way into the first edition.

The Author has also carefully revised the work throughout, and, where it appeared desirable, re-written portions, but in no case has he altered the *opinions* expressed in the first edition, the practice of the intervening period of five years having in all cases tended to confirm them.

CONTENTS.

INTRODUCTION.

The vast progress made during the last fifty years in metallurgical art has caused the metals to assume a very forward place amongst the materials used for structures of all descriptions, such as bridges, roofs. lighthouses, and public and private buildings of all kinds, and it may specially be remarked that at the present time much more attention has been drawn to the adaptation of iron to the purposes of the builder than that subject had hitherto attracted.

Iron being, from its physical qualities, suitable for a great variety of uses, extending up to the construction of the greatest works, it is easy to account for the fact that almost only works of great magnificence have received that general consideration which smaller undertakings equally deserve. The engineer may point with justifiable pride to the bridges which in safety carry our heavy traffic over the widest rivers, and contemplate with satisfaction the colossal roofs which afford a covering to our spacious railway stations; but equally with these the student should examine the requirements and principles of those works which, though less pretentious, acquire equal importance from their greater frequency. It cannot be denied, even by the most ardent admirer of stupendous works, that the due proportioning and arrangement of iron structures, which may perhaps form the nucleus of a future colony, and afford comfort and security to its founders, are not inferior in importance to the more elaborate designing of the most expensive structure required by the economic and æsthetic exigencies of a civilised community; hence, the engineer who would perform the duty which devolves upon him of extending his art through the widest sphere of usefulness must study the application of the materials with which he has to deal, even to works of apparent insignificance, and, moreover, by so doing he will acquire a knowledge which will subsequently be of great value to him in setting out the subordinate details of any larger structures with the execution of which he may be entrusted.

In treating of wrought and cast iron structures, we shall endeavour to set forth in the simplest possible form the fundamental principles which rule the application of the metals referred to, to structural purposes, whether as arches, pillars, girders, or trussing, and subsequently to explain the mode in which works having been designed in accordance with such principles are executed, in the iron-yard or foundry, as the case may be.

It is not sufficient that the engineer should merely be able to calculate the strength of every part of his work, which he may generally do by the aid of books of rules, &c., but he requires a keen perception of *possible contingencies* which may arise during the process of manufacture, and a knowledge of æsthetics, so that by duly considering the characters of his materials, he may produce a work combining in the highest degree utility with a pleasing appearance. Of course no one can expect to possess these qualifications without previously having extensive practical experience; but careful unremitting study of the works of predecessors will tend very materially to assist in educating the eye and maturing the judgment of the student, provided that he reflects carefully upon the good and bad features—both constructive and artistic—observed in such structures as come under his notice; and speaking of bad features, let it be remembered that much useful information is to be gained from studying *errors*, for from an examination of their causes, their future occurrence may be avoided, and from their results the importance of obviating them may be estimated.

In respect to the production of pleasing effects in iron structures we have always held a very decided opinion that such a result can seldom be attained by ornamentation, having no other duty than that of hiding unsightly work; it must be due to a consistency of design, the forms introduced should be such as are suggested by the nature of the material wrought, otherwise they are not in accordance with common sense, hence are opposed to good taste. If we consider some of our stone structures, there is but little of actual ornament about them, such, for instance, as London Bridge and Waterloo, but yet from the consistency of design both these noble works produce a pleasing effect upon the eye; the same results may doubtless be secured with other materials, but it must not be forgotten that each typical material has a style of architecture specially suited to it, and if that style be not followed in its application, discordant effects will be produced, and in point of fact it is better to have a work absolutely plain than loaded with ornamentation inconsistent with its general character.

In regard to the strength of structures, the modes of ascertaining it are sufficiently clear and reliable, and once comprehended cannot give rise to failure if correct data are used as to those matters which are determined by experiment, such as the resistance of the materials to the various forces to which they are subjected when fitted into their respective places in the structures of which they form the details. It is, however, very remarkable that engineers in this country have not availed themselves of the improvements in the iron trade to so great an extent as they might in reducing the weight of wrought-iron structures; in fact, these works are made stronger, and therefore more costly, than is necessary. If metal will safely carry five tons in tension per sectional square inch, surely it is a waste to make bridges of such proportions that the heaviest strain cannot exceed two and a half or three tons—it may be instructive to examine the amount of loss thus caused.

If we merely take the area of the metal required to carry the live or moving load, the waste caused by using too low a co-efficient of strength is at once observable; but by increasing the areas of the parts of a structure, the dead load or weight of such structure is also increased, and all the strength taken up in the support of the work itself (although a necessary absorption of resistance), must be regarded as prejudicial. One reason to which may be assigned the excessive sectional area found in the works of many engineers, is that habitually in the first calculation of strains the dead load is much over estimated, sometimes as much as fifteen or twenty per cent.

Now, although undoubtedly it is always best to have some excess of strength, yet it is both unwise and unscientific, as well as ruinous, to have too great an excess;

and moreover it shows a deficiency or mistrust somewhere; the engineer loses faith either in his own design or in the quality of materials or workmanship supplied by the manufacturer, and he can assure himself of the two latter points by exercising personally, or by a competent representative, proper supervision of the work during its construction.

Turning to the works of nature, which should ever guide the engineer, there is seen ample strength provided, but no waste of material; each object is nicely adapted to meet the exigencies of its own case, and the works of certain insects exhibit such examples as are well worthy of the careful study of human constructors, for we have to arrive by our intellectual efforts at a stage of perfection comparable to that evolved from the instincts of the lowest orders of the animal world.

In making allowances for possible weakness of material and for other uncontrollable emergencies, these should be treated *en masse*, and the requisite excess of strength above that required by theory at once determined, instead of pursuing the common system of assuming the loads and strains to be greater than they ever can be, and after calculating from the data of these unattainable loads to throw in an excess of sectional area over that ascertained. It is not as if *maximum* loads were difficult to be determined, for, on the contrary, the means of finding their amounts are always available. If a bridge is required to carry general traffic, the weight of that traffic and the nature of the concussions or vibrations produced by it may be found. If the work forms a portion of a line of railway the weight of the stock on such line is known to the engineer. Similarly with girders and columns used in the erection of warehouses and other buildings, the total superimposed load can always be ascertained. The actual maximum load having been arrived at, there need be no further allowance made, provided the metal be duly tested and the workmanship good, then a sound but light work will be obtained, one combining the two requisites of ample strength and minimum weight.

Sometimes it may, of two plans, be more economical to adopt the heavier; and this will be the case when in the lighter design all that is saved in material is lost by increased cost of manufacture or material; hence, that the engineer may be thoroughly competent to judge in such instances, he must have a knowledge of the mechanical manipulations to which his materials are subjected in the process of being manufactured into the required forms, otherwise he will be at the mercy of the contractor to whom he applies for advice, and whose business it is to turn out the work in such a manner that, whilst it does him no discredit, he can make the greatest profit out of it. In fact, the civil engineer is the buyer, or rather is entrusted virtually with the buying, of certain things of which the contractor is the seller; hence, considering the great trust reposed in him and the heavy responsibilities attaching to his position, he should, by every means in his power, endeavour to gain a thorough knowledge of his profession, theoretically and practically, and thus qualify himself for the sphere of usefulness he has selected.

CHAPTER I.

CAST AND WROUGHT IRON AND STEEL.

The mechanical properties of cast-iron differ widely from those of the wrought metal, hence the modes of manipulation, and the purposes to which these materials are especially suitable, also vary considerably.

Cast-iron is rigid, inflexible, and brittle, incapable of being forged, welded, or drawn, although it may readily be cast into any required form. Wrought-iron is remarkable for its toughness; it may be beaten into various shapes at a red heat, and welded at a white heat; these welded joints being, when properly executed, equal in strength to the solid metal; in fact, a good weld *is* solid metal. Wrought-iron may be drawn cold into fine wire, hence is very ductile.

Of cast-iron there are several qualities and kinds, varying from that which has a silvery-white, crystalline fracture, to the tougher kinds, exhibiting a grey fracture. By certain modes of treatment castings, when made, may be rendered malleable; but, in fact, the nature of the material becomes more similar in the process used to that of wrought-iron; but malleable castings have not nearly the strength of ordinary wrought-iron, although they lose the brittleness of common cast-iron, and thus become less liable to fracture; hence this metal is valuable for articles of common use, which, although subject to rough handling and falls, are not called upon to bear strains of any magnitude.

The quality of malleable iron depends upon the foreign bodies associated with it, hence in great measure upon the quality of cast-iron from which it is made, being prepared from that metal by processes of purification; but this is also influenced by the amount of hammering or working the wrought metal has undergone while in a state approaching semi-fusion, the continued working rendering the metal more homogeneous or uniform in its structure, and, therefore, tougher and of more equal strength throughout.

Steel, which consists of iron combined with a portion of carbon, possesses most of the properties of wrought-iron in a higher degree; it must, however, be forged at a lower temperature; it admits also of being cast, and various degrees of hardness may be imparted to it by a process of hardening and tempering, the greatest hardness being obtained by plunging the red-hot metal into cold water. In this state, however, it is exceedingly brittle, hence for most purposes it is necessary to temper it lower, which is effected by gradually heating the metal to a certain temperature, regulated according to the degree of hardness and elasticity required in the work under manipulation.

The resistance of wrought-iron to a tensile or stretching strain is far greater than that of cast-iron, but the strength of the latter as opposed to compressive or crushing strain is superior to that of the former.

The tensile resistances of the materials per square inch of sectional area are:—

Swedish bar-iron . .	65,000 lbs.	=	29·01 tons.
Russian „ . . .	59,470 „	=	26·54 „
English „ . . .	56,000 „	=	25·00 „
Wire rope	90,000 „	=	40·17 „
Cast-iron	17,628 „	=	7·87 „

The compressive resistances are:—

Cast-iron	120,000 lbs.	= 53·57 tons.
Wrought-iron . . .	36,000 ,,	= 16·07 ,,

Generally in practice no strain is allowed upon metal which shall exceed *one-sixth* part of its ultimate strength or breaking weight, although sound metal is not supposed to be injured until one-third of its breaking weight is reached, injury being shown by permanent alteration of form, for although a body may be bent, extended, or compressed, no injury can be assumed to have been caused so long as upon the removal of the strain such body resumes its former figure and dimensions. The permanent deflection of structures due to imperfections in the joints is not to be confounded with that due to physical injury of the material, and is, in fact, quite immaterial to the strength of the work, provided it be kept within reasonable limits.

In calculating bridges and other works specified to be of *good* (*i.e.* above the average) iron, it is usual to allow as safe strains per square inch,—

Wrought-iron	in tension 5 tons	—in compression	4 tons.
Cast-iron	,, 1·5 ,,	— ,,	7 tons.

These mechanical differences in the properties of the various kinds of commercial irons are due to variations in chemical composition, hence it is desirable briefly to consider the constitution of the metal and the changes brought about in it by processes of manufacture. In nature but little iron, comparatively speaking, occurs in the metallic state, the largest deposit being probably found in the beds of magnetic iron-sand at Taranaki. This sand may be regarded as almost pure iron, containing as it does about 95 per cent. of that metal; the remainder consists of small proportions of various foreign substances, such as manganese, titanium, &c. This material is not largely used, on account of its condition rendering it awkward to manipulate.

The ores or iron-stones, from which the iron of commerce is mainly derived, contain the metal in the condition of oxide or rust, associated with various metallic and earthy matters, which must be got rid of by metallurgical operations. The ores contain the iron in proportions varying from 70 per cent. down to 28 per cent.; those poorer than the latter are not worked to extract their metal, though they are used as fluxes in the treatment of the richer minerals.

The iron being found combined with oxygen, it is necessary to remove the latter in order to release the metal, and this is effected by the smelter in a process of reduction. The iron-stone is put into a blast-furnace in contact with carbon (as fuel), and a high temperature being maintained, the carbon combines with the oxygen of the oxide of iron, and the reduced metal melts and flows through the fuel to the bottom of the furnace. Besides the fuel and iron-stone, it is also necessary to put into the furnace certain materials to act as fluxes, which, becoming partially liquified, dissolve some of the foreign matters associated with the iron, retaining them in the form of *slags*. The theory of the action set up is sufficiently simple, but in practice the processes are somewhat more complex than might be imagined, hence it is advisable to explain the operations by which the iron is eliminated from its ores.

As a general rule, a mixture of several different kinds of ore is used for smelting, because experience has shown that this process is then more easily and completely carried out than if one kind of ore be employed alone. Such ores as contain carbonic acid, water, or sulphur must, previous to undergoing the smelting process, be roasted in suitable furnaces to expel those bodies which will pass off in the gaseous state under the influence of heat. The ores always contain a certain amount of impurities termed *gangues*, such as silica, clay, lime, phosphorus, manganese, &c. *Silica* especially forms a principal ingredient in iron ores, and this will not melt, even when exposed to the greatest furnace heat, alone; but it must be melted in order that the iron may flow out from its ores and be obtained as a coherent mass. This is effected by the addition of a flux, commonly lime, which, being a base, combines with the silicic acid; a lime-glass is thus formed, and if loam and clay be also present, an alumina-glass, both of which, when combined, melt more easily than each separately, and flow off as slag. The combination of ores and fluxes ready for the smelter is called the mixture. Alternate layers of this mixture and of wood-charcoal or coke are thrown into the blast-furnace in suitable proportions, according to the quality of the mineral used.

The top or mouth of the furnace serves both for charging the materials and for the escape of smoke; it is thus at once a door and a chimney. In the upper part of the shaft the mixture is heated to redness, hence a roasting effect is produced; during this process the carbonic acid of the limestone also escapes. Further down the carbon abstracts from the iron ore its oxygen, and escapes with it as carbonic oxide, which at the mouth, on coming into contact with the atmospheric air, is consumed, exhibiting a bright flame.

In the boshes or lower part of the furnace, where the heat is most intense, the reduced iron melts and falls in drops upon the hearth, together with the silica, lime and clay; these form a slag which floats on the melted iron, and is drawn off from time to time, as occasion may require. The molten iron is allowed at intervals to flow off through a hole in the side of the hearth. After having heated to 200° or more the air requisite for the combustion of the charcoal or coke, it is forced into the blast-furnace by a blowing-engine or other suitable apparatus, and a temperature of probably 2000°, or 2600° Fahr., is obtained.

In proportion as the melted iron and slag are removed from beneath, fresh charges of ore, lime and fuel are introduced at the top, and in this manner the smelting is often continued for five or six years, according as the furnace holds out. The following table shows the materials used and the resulting products:—

Materials.	Products.
Iron ore—	Carburetted iron (cast-iron).
Flux—	Carbonic oxide and carbonic acid (gases of combustion).
Fuel—	Silicates of lime and alumina (slag).

The siliceous slags from the blast-furnace usually have a green or blue colour, which is due to the oxides of iron and manganese dissolved in it whilst in a state of fusion; it is frequently formed into square blocks, and used for building stones.

The metal obtained by the above process is termed crude cast-iron; it is by no means pure, but is chemically combined with carbon, and also contains small proportions of other foreign bodies, such as silica, alumina, manganese, &c. A hundredweight of iron will take up at the hottest white heat from about four to five pounds of carbon, likewise some silicon from the silicic acid, and aluminum from the clay. Traces of sulphur, phosphorus, and arsenic are also sometimes present.

As the molten crude iron flows from the hearth of the furnace, it is directed into a trough or channel formed in a bed of sand, from which other channels branch off on either side; the iron cast in the main channel is termed the *sow*, and that in the smaller ones the *pigs*, whence the term *pig-iron*.

Of the two kinds of iron commonly known in commerce, *grey iron* has a granular texture, and admits of being filed and bored with facility; hence it is suitable for castings.

White iron is of a silvery whiteness, and too hard to be worked with steel instruments, and is most suitable for the manufacture of malleable iron and steel. Crude white iron, by remelting and very slow cooling, is changed to grey, and, on the other hand, grey iron is changed to white by heating and then suddenly cooling it. Thus, by pouring molten metal into a cold mould it acquires a very hard surface, and presents what is termed a *chilled* casting.

Castings may be locally chilled by forming the mould with means for cooling that part of the surface which corresponds with the portion of the casting required to be hard.

Malleable iron is obtained from crude iron by depriving it of its carbon, which is done by various processes of oxidation, such as the following:—

1. The carbon is oxidised by the action of atmospheric air on the molten iron, which is kept stirred to expose new surfaces to its action whilst in a refinery

or a puddling furnace. (The old method of puddling.)

2. Air is forced *through* the iron while in a state of fusion in a vessel termed a converter, through the bottom of which the air is blown. (Bessemer's method.)

3. Super-heated steam is forced through the molten metal, thus oxidising the carbon, and also removing sulphur and phosphorus as sulphuretted and phosphoretted hydrogen. (Galy-Cazalat's method.)

4. The melted metal is acted on with certain salts, such as nitrate of soda, &c., by which the carbon is oxidised out. (Heaton's process.)

In all these processes the carbon escapes as carbonic oxide or carbonic acid. The process invented by Galy-Cazalat has not yet been applied on a large scale in England, but in France it has been found to yield results eminently satisfactory, and it certainly is a very elegant process, both the constituent gases of the water which are evolved by its decomposition being rendered subservient to some useful purpose.

The quality of malleable iron is improved by being well hammered and cut up, and again worked up in the forge, as by these means its quality is rendered more uniform and its texture more homogeneous; thus scrap iron, that is old iron re-worked, is much esteemed for certain manufactures, such as for gun-barrels, &c., requiring great strength and soundness.

Steel is iron containing a certain quantity of carbon, but not so much as is found in cast-iron; it may be prepared in either of the two following ways:—

1. By keeping bars of wrought-iron at a temperature close upon the melting point in contact with powdered

charcoal, access of air being prevented for a length of time, dependent on the size of the bars. This process is called cementation, and evidently the bars will be more carbonised on the exteriors than the centres; hence, to obtain the steel uniform, the cemented bars must be cut up and re-wrought into bars or plates, as may be required.

2. By carrying the refining of crude iron to such a point that there is sufficient carbon left in it to form steel, and then arresting the process. This method gives better results at a much reduced cost of production.

Malleable iron, for general commercial purposes, is manufactured in the following forms:—

Bars.—Round, square, flat, elliptical.

Do.—Angle, tee, and flanged, having sections L T and H, also half-H iron or channel iron bars.

Plates.—Of various sizes.

Special forms frequently used, as railway-bars, sash-bars, deck-beams, rolled-girders, &c.

The rolls in which the bars and plates are formed are adjustable, so that any required thickness may be obtained.

In ordering iron of a manufacturer for any considerable work, it is usual, after the working drawings have been finally settled, to go carefully through them, and note the sizes of all the plates and bars required, then from these data the order list for the rolling-mills can be made out.

Medium-sized bars will run up to 25 feet in length, and similar angle-iron bars up to 30 feet; but when bars or plates exceed certain gross weights per plate or per bar, the price per ton is increased, otherwise the longer the bars the better, as reducing the number of joints in a structure.

Some years back a method of plate-welding was introduced, to supersede rivetting, by Mr. Bertram, but it has not come largely into use, although it was found that joints thus made were equally strong with the rest of the plate when experimentally tried. Probably the practical difficulties in manipulation have militated against its adoption.

CHAPTER II.

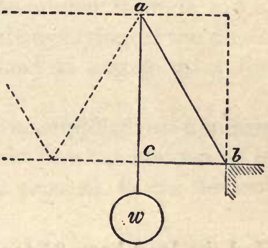
STRAINS ON STRUCTURES.

THE strains which are brought to bear upon the different elements of structures are five in number, namely, tension, compression, transverse or bending strain, shearing, and torsion or twisting strain. The two first are direct, and the third may be resolved into them. Shearing force tends to cut or shear off some portion of material, such for instance as the head of a rivet. Twisting strain does not often occur in the elements of structures, being more common in machinery,—it may, however, be resolved into shearing force.

A strain of a direct character may act upon an element lying in the same direction—such is the stress produced by a load on a column or a weight hanging at the extremity of a chain or vertical suspending rod; but, on the other hand, the strain may not act in the direction of the sustaining framework, being borne by two or more inclined bars. In such a case the intensity of the strain on the element will be different from that of the force to which it is due, and, moreover, in each element different from the proportion or share of force resisted thereby.

Let *a b* in Fig. 1 represent an inclined bar forming part of a truss, or lattice girder, its lower end resting on the point of support or pier, *b*, and let this inclined bar sustain a weight or part of a weight, transmitting the load thus imposed upon it to the pier. Let this load, which passes through *a b*, be represented by the ball *w*. From the foot *b* of the inclined bar draw the line *b c*, *horizontal*, then it will be at right angles to the *vertical* line *a c*. It is required to find the strain on the bar *a b*, due to the load *w*. Taking *b* as a starting point, the position of the extremity *a* will be described by giving the two measurements *b c* and *c a*. *a* is the point at which the load is applied, and evidently *b c* is its *horizontal distance* from the point *b*; that is to say, its distance from *b*, measured only in a horizontal direction; and, in like manner, *a c* is the *perpendicular distance*, or height of the point *a* above the point *b*; that is, *c a* is the distance from *b*, measured perpendicularly. The rule to find the strain on the inclined bar will be as follows :—

Fig. 1.



Rule (1). To find the strain, multiply the load by the length of the bar in feet, and divide the product by the "perpendicular distance" in feet. The quotient will be the strain, which will be in pounds if the load is taken in pounds, or in tons if the load is taken in tons.

Example. Let the weight be 2,500 lbs., the length of the bar 20 feet, and the height or "perpendicular distance" 16 feet, then the rule should be thus worked :—

2,500 lbs. weight or load,
20 feet length of bar,

"Perpen. distance," 16)50,000(3,125 lbs.' strain on bar.
48
20
16
40
32
80
80

Had the load been 25 tons, the working would be—

25 tons' load,
20 feet length of bar,

"Perpen. distance," 16)50,000(31·25 tons, or $31\frac{1}{4}$ tons' strain on bar.
48
20
16
40
32
80
80

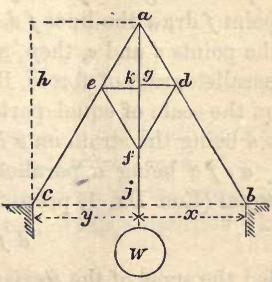
This practical rule will serve in every case to determine the strain on a bar due to the force or load transmitted by it, no matter in what position such bar is placed, when the following quantities are given, viz., the amount of weight or force acting upon the bar, the length and position of the bar, and the direction of the force.

To make the diagram, the bar is drawn in any convenient position, as *a b*, Fig. 1; *a c* is then drawn from the point of application, *a*, of the force, and *b c* is drawn from the end *b* of the bar at right-angles to *a c*, so as to complete the right-angled triangle *a b c*, the "perpendicular distance" always lying in the direction in which the force or load is acting upon the inclined bar.

Having given the rule and exemplified its application, it is now necessary to show in what manner it is obtained; we will, therefore, proceed to its demonstration.

In Fig. 2, let *W* be a weight at the extremity of a cord or rod, *a W*, which weight is supported by the two inclined bars, *a b*, *a c*, the load being suspended from apex *a*, and the lower ends resting on suitable abutments at *b* and *c*; of course, *a W* will be vertical, and the supporting elements are assumed to be straight. It is required to determine the proportion of the load borne by each point of support and the strain upon each of the bars *a b*, *a c*. Let *x* and *y* represent the respective distances of the points of support *b* and *c* from the weight *W*, then, accord-

Fig. 2.



ing to the principles of the lever, the load on b will be

$$= W \times \frac{y}{x + y}$$

Here c is the fulcrum and b the point at which the pressure is given off, the former having a leverage y, the latter a leverage $x + y$. For simplicity, let $x + y = l$, as l will be generally put for the span or clear distance between the points of support in all kinds of structures, then the load on b

$$= W \times \frac{y}{l}$$

and by a similar mode of reasoning the load on c is found to be

$$= W \times \frac{x}{l}$$

hence the loads on the points of support are inversely as the distances of the supports from the weight W.

Next let the strains on $a\ b$, $a\ c$ be determined.

On the vertical line $a\ W$, and with any convenient scale of equal parts, mark off from a a distance $a\ f$, representing the intensity of the weight W, and from the point f draw the lines $f\ d$, $f\ e$, intersecting $a\ b$ and $a\ c$ in the points d and e, then, according to the doctrine of the parallelogram of forces, the strains will be represented on the scale of equal parts by the lengths $a\ d$ and $a\ e$, $a\ d$ being the strain on $a\ b$, and $a\ e$ the strain on $a\ c$.

$a\ e\ f\ d$ being a parallelogram the opposite sides are equal (Euc. Bk. 1, prop. 34), hence

$$d\ f = a\ e$$

but the sums of the *strains* caused by the weight W

$$= a\ d + a\ e = a\ d + d\ f$$

Thus it is seen that the sum of the strains produced by the weight W will be represented by the two sides $a\ d$, $d\ f$ of the triangle $d\ f\ a$, but, because any two sides of a triangle are, together, greater than the third side (Euc. Bk. 1, prop. 20), $a\ d$, $d\ f$ are, together, greater than $a\ f$. But $a\ f$ is equal to W, hence the sum of the strains due to the weight W are, together, greater than the intensity of such weight.

Let, in the present case, the points of support b and c be in the same horizontal plane, and let h = the height of the point a above such plane, the same scale of equal parts being used for measurement throughout.

From the points d and e let fall the perpendiculars $d\ g$, $e\ k$, upon $a\ f$, then $a\ g$ and $a\ k$ will represent the proportions of the *weight* borne by the points of support b and c respectively.

But the triangles $a\ d\ g$, $a\ k\ e$ are similar to the triangles $a\ j\ b$, $a\ j\ c$, therefore, by the principles of similar triangles,

$$\frac{a\ d}{a\ g} = \frac{a\ b}{a\ j} = \frac{a\ b}{h}$$

and

$$\frac{a\ k}{a\ e} = \frac{a\ c}{a\ j} = \frac{a\ c}{h}$$

But $a\ d$ has been shown to be equal to the strain on $a\ b$, due to the weight W, and $a\ g$ = to the proportion of such weight carried through the bar $a\ b$ on to the point of support b, and by previous reasoning this proportion was shown to be

$$= \frac{W\ y}{l}$$

hence, if $S =$ the strain on $a\,b$, we have

$$S = a\,d\,,$$

and

$$\frac{W\,y}{l} = a\,g$$

therefore, replacing the letters by these values, we find,

$$\frac{a\,d}{a\,g} = \frac{S}{\frac{W\,y}{l}} = \frac{a\,b}{h}$$

Let $a\,b = L$, the length of the bar under consideration, then

$$\frac{S\,.\,l}{W\,y} = \frac{L}{h}$$

wherefore

$$S = \frac{W\,.\,y\,.\,L}{l\,h}$$

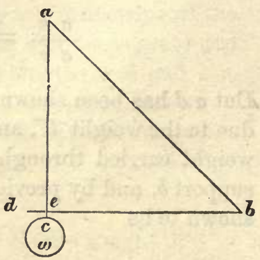
This is one of the fundamental formulæ for inclined bars of every kind under direct strain; hence it is necessary to set it very clearly forth, so that it may always be recalled to mind by a diagram, which is very easy to be remembered.

In Fig. 3 let $a\,b$ represent an element of framework inclined to the horizon, the position of its extremities being determined by the form of such framework, and let it be required to support a load or proportionate part of some load),

$$= w$$

it is required to find the strain on $a\,b$ due to the load w. From the point a draw the *vertical* line $a\,c$, and from the point b draw the

Fig. 3.



horizontal line $b\ d$, intersecting the line $a\ c$ in the point e, then

$$w = \frac{W\,y}{l}$$

$$a\,b = L$$

$$a\,e = h$$

and the strain will therefore be

$$S = \frac{w\,.\,L}{h}$$

from which is stated the rule for determining the strain on trussing due to a weight.

The strain on the bar is equal to the load carried by it, multiplied by the length of the bar and divided by the vertical height of its summit above its base.

It does not, however, always happen that the load or force producing the strain acts in a vertical direction, hence it is necessary to have a general law applicable to all cases, and having regard to the direction of the force relatively to that of the bar upon which it brings a strain.

Let the force w act upon $a\ b$ in any direction $a\ c$, and from the point b let fall the perpendicular $b\ e$ upon $a\ c$, then $a\ e$ is the perpendicular distance between the point at which the force acts (a) and the abutment or point to which the force has to be transmitted (b). The formula remains the same as before, but the general rule will be, calling $a\ e$ for brevity the "perpendicular distance" to the bar $a\ b$, as follows:—

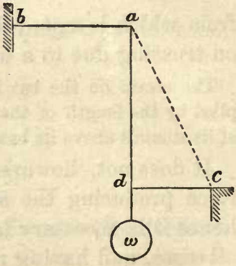
The strain on the bar is equal to the force acting on it, multiplied by the length of the bar and divided by its "perpendicular distance."

This may be accepted as the universal rule by means of which the strain on any bar produced by a force acting at an angle to it may be determined, when the intensity of the force and the directions of the force and bar are known.

There is, however, *one* special case which requires separate consideration; it is that in which the force is acting at *right angles* to the element, the strain upon which is required to be determined.

In Fig. 4 let *a b* be a horizontal bar forming part of a frame or truss, its duty being to fix the position of the upper end *a* of an inclined bar, shown by the dotted lines *a c*, and which inclined bar has to transmit a load or force, *w*, from the point, *a*, of its action to the pier *c*. It is required to determine the amount of strain upon the bar *a b*. This case will always apply where the bar receiving the strain is at right angles to the *direction of the strain;* and it may be here observed that the bar does not carry any *load*, but merely preserves the angular position of *a c*, hence the strain on *a b* will be ruled by the position of *a c* in relation to it and to the direction of the strain. Draw *a d* in the direction of the strain and draw from *c* the line *c d* parallel to *a b*, and therefore at right angles to *a d*, then *c d* will be the "horizontal distance," referred to in the description of Fig. 1. To find the strain on *a b* the rule is as follows:—

Fig. 4.



Rule (2). To find the strain on the horizontal bar (or bar at right angles to the direction of the force), multiply the weight or load by the "horizontal distance" in feet, and divide the product by the "perpendicular distance" in feet, the quotient will be the strain (in pounds or tons as the weight is taken in pounds or tons).

The perpendicular distance (as in Fig. 1) is the measurement of *a d.*

Example :—Let the weight or load be 4,000 lbs., the "horizontal distance" 4 feet, and the "perpendicular distance" 8 feet :—

4000 lbs.' weight or load,
4 ft. "horizontal distance,"

Perpendicular distance - 8) 16000

2000 lbs., strain on the horizontal bar.

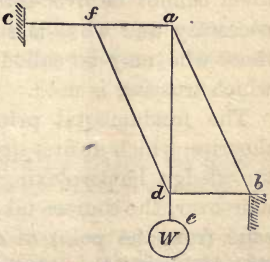
Let the weight or load be 15 tons, the "horizontal distance" 5 feet, and the "perpendicular distance" 12·5 feet:

15 tons weight or load,
5 feet "horizontal distance,"

Perpendicular distance - 12·5) 75·0 (6 tons' strain on
75·0 horizontal bar.

We will now proceed with the demonstration of this second or special rule.

Fig. 5.



In Fig. 5 let a weight or force, *W*, be sustained by an inclined bar, *a b*, the extremity being retained in position by another element *a c*, which is at *right angles* to the direction *a e*, of the force. Proceeding as in the general case for *a b*, from *b* draw *b d* at right angles to *a e*, and from the point *d* draw *d f* parallel to *a b*, then in the parallelogram of forces *a b d f*, the side *a f* or *b d* will represent the strain on the bar $a\ c$; if the diagonal $a\ d$ is taken as equal to the load or force, W, put

$$a\ d = h = \text{"perpendicular distance"} = P$$
$$b\ d = \text{base of triangle} = B$$

then it is evident that the strain is found from the formula,

$$S = \frac{W \cdot B}{P}$$

Thus the special or *supplementary* rule to find the strain produced by any given force upon the element at right angles to its direction will be as follows:—

The strain on the bar is equal to the force multiplied by the length of the perpendicular drawn from the foot of the inclined bar to the direction of the force, and divided by the "perpendicular distance" of the inclined bar.

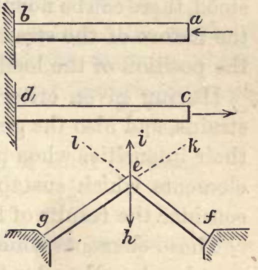
These two general rules, or rather, we would say, the *general law* and its *supplement*, furnish all that is required for the resolution of the strains on elements of structures of every description when the intensities and directions of the forces producing such strains are known; hence the importance of becoming thoroughly conversant with them cannot be over-estimated in considering what is necessary and what is superfluous to the education of those who may be called upon to design structures in which trussing is used.

The fundamental principles being seen, elaborate theories, which at first sight would, without this primary knowledge, be perplexing, become simple and easy; and, moreover, the courses taken by the strains in their passage from the point of application of the load to the foundations of the structure are more readily perceived.

The direct force acting on any bar will necessarily produce either tension or compression; in the former case the bar acts as a tie, in the latter as a strut; we must, therefore, proceed to consider the circumstances which determine the nature of a strain—that is, whether it will be tensile or compressive.

In Fig. 6 let *a b* represent a bar upon which a force is acting *towards* the *abutment* or point of support *b*, then the strain on *a b* will be a compressive strain, and *a b* will act as a *strut*.

Fig. 6.



Let *c d* represent a bar on which a force acts in the direction of the arrow at *c*, then the strain on *c d* will be in tension, and *c d* will act as a *tie*.

Let *e f*, *e g* be two rafters supported on abutments *f* and *g*, and meeting at the point *e*. If a force act at *e* downwards, as shown by the arrow *h*, the rafters *e f*, *e g* will be in compression, but if the force act upwards, as shown by the arrow *i*, they will be in *tension*; in the former case they will do duty as *struts*, in the latter as *ties*.

For general guidance the following five rules may be laid down in order to prevent any confusion of ideas on this very important subject:—

RULES.

1. If a force act on a bar in a direction towards the point of support of the bar, the strain on the bar will be compressive, and the bar will be a strut.

2. If a force act on a bar in a direction away from the point of support of the bar, the strain on the bar will be tensile, and the bar will be a tie.

3. If a force is sustained by two bars, and acts between them

and towards their points of support, those bars act as struts and are in compression.

4. If a force is sustained by two bars, and its direction is from the points of support, but between the lines formed by producing the bars (as *e k* and *f l*, Fig. 6), the strains are tensile, and the bars are ties.

5. If a force is sustained by two bars, and its direction lies between one bar and the prolongation of the other (as between *f e* and *e k*, Fig. 6), one bar will be in tension, the other in compression, that bar being in compression towards the point of support of which the force is acting, and the other bar being in tension.

If these rules be once thoroughly considered and understood, there can be no subsequent difficulty in determining the nature of the strain on any part of a structure when the position of the load or force is known.

Having given criteria for determining the nature of strains, and also the general and special rules for finding their intensities when produced by forces inclined to the elements which sustain them, we will now pass on to consider the results of loads producing transverse strain.

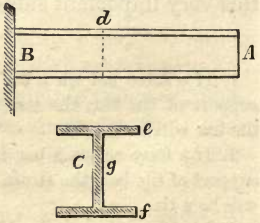
Those elements which are subject to transverse strain may be classed under two heads :—

1st. Cantilevers, which are supported at one end.

2nd. Beams or girders, which are supported at both ends.

Let *A B*, Fig. 7, represent a side view of a cantilever fixed at *B* to a wall or pier, the end view or cross section of this cantilever being shown at *C*. It consists of two flanges, *e* and *f*, united by a central web, *g*, this being the ordinary section of flanged girders and canti-

Fig. 7.



levers. Let it be required to determine the strain on either flange at any point, *d*, the distance of which from the end *A* being known, as also the depth of the section. The strain on the top flange will be tensile, that on the bottom compressive.

First find the strain due to a weight suspended from the end *A* of the cantilever.

Rule (3). The strain is equal to the weight at the end of the cantilever multiplied by the distance of the point at which the strain is required, from the end of the cantilever in feet, and divided by the depth of cantilever in feet. The strain will be in terms of the same name as the weight.

Example:—Let the weight be 5 tons, the depth of the cantilever 1 foot 6 inches, and the distance of the point at which the strain is required from the free end 11 feet:—

11 feet distance of point of strain,
5 tons' weight,

Depth of cantilever } - 1·5)55·0(36·66 tons' strain on either flange.
45
100
90
10

This will be tensile on the top flange and compressive on the bottom.

The geatest strain upon the flanges will be at the end *B* of the cantilever, which is at a distance from the free end equal to the length of the cantilever. Let the length of the cantilever be 18 feet, the other quantities remaining as before, then the maximum strains on the flanges at the point of support will be thus found:—

18 feet distance of point of strain,
5 tons' weight

Depth of cantilever } - 1·5)90·0(60 tons' strain on either flange.
90
·0

Next, let the strain be due to a weight or load *uniformly distributed* along the whole length of the cantilever, then we have

RULE (4). The strain is equal to the weight per foot multiplied by the square of the distance of the point of strain in feet from the free end of the cantilever, and divided by twice the depth of cantilever in feet. The strain will be in terms of the same name as the weight.

Let the distance of the point at which the strain is required from the free end of the cantilever be 7 feet, the depth of the cantilever 1 foot 3 inches, and the load 1,500 lbs. per lineal foot of length of cantilever:—

7 feet distance of point of strain,
7 ditto ditto

49 square of distance of point of strain,
1500 lbs.' weight per foot.

Depth of cantilever - 1·25   24500
2   49

2·5)73500·0(29400 lbs. strain on either flange.
50
235
225
100
100
0·0

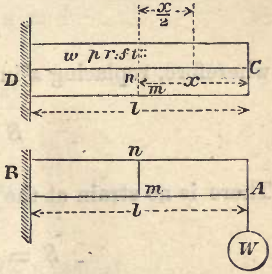
As in the last case, the maximum strain occurs at that point where the distance is equal to the whole length of the girder, which occurs at the point of support.

It sometimes happens that a cantilever has to carry both descriptions of load at once—that is, both a load at the free extremity and a load uniformly distributed over its entire length. In this case, calculate the strains produced by each load separately, and add them together for the total strain on the flange.

The demonstration of the rules given above is as follows:—

Let $A\ B$, Fig. 8, represent a cantilever fixed at the end B in a *wall*, and free at the other end, and supporting at its free end a weight W. Let l = total length of cantilever from its free end to its point of fixture, and d = depth of cantilever. It is required to find the strain at any point, n, in (say) the top flange, distant x from the point A.

Fig. 8.



We may regard the part of the cantilever $A\ m\ n$ as a bent lever *in equilibrio*, m being the fulcrum, the weight acting at the extremity A of the arm $A\ m$, and the resistance of the flange acting at the end n of the arm $n\ m$. That equilibrium may be maintained the resistance of the flange must be equal to the strain to which it is subjected; hence let S = this strain, then, by the principles of the lever,

$$S \times m\,n = W \times A\,m$$

but,

$$m\,n = d \text{ and } A\,m = x$$

wherefore,

$$S \cdot d = Wx$$

and

$$S = \frac{W\,x}{d}$$

By putting which equation into words is obtained the practical rule (as above) following:—

RULE. The strain is equal to the weight at the end of the cantilever multiplied by the distance of the point at which the strain is required from the end of the cantilever in feet, and divided by the depth of the cantilever in feet.

To ascertain the formula for the maximum strain, make the distance of the point of strain equal to the length of the cantilever, then

$$x = l$$

wherefore, replacing x in the above equation,

$$S = \frac{W\,l}{d}$$

There is no strain at the free end A, for if $x = o$,

$$S = \frac{W}{l} \times o = o$$

Next, let $C\,D$ (Fig. 8) illustrate a cantilever loaded with a weight uniformly distributed over its length. It is required to find the strain on (say) the top flange at the point n, distant x from the free end C.

Let $l =$ length of cantilever
$d =$ depth of ,,
$w =$ load per lineal unit.

The weight acting to produce a strain at n will be that lying between n and C, the total weight of which evidently is

$$= w\,x$$

but this may be considered as collected at its centre of gravity, which is situated midway between m and C; hence, regarding $C\,m\,n$ as a bent lever, we have the weight $w\,x$ acting in the centre of the arm $C\,m$, producing a strain at the end of the arm $m\,n$ on the flange at n, hence, by the laws of the lever, calling $S =$ strain on flange,

$$w\,x \times \frac{C\,m}{2} = S \times m\,n$$

but,

$$C\,m = x \text{ and } m\,n = d$$

wherefore,

$$S \cdot d = \frac{w\,x^2}{2}$$

and

$$S = \frac{w\,x^2}{2\,d}$$

from which formula is obtained the practical rule:—

RULE. The strain is equal to the weight per foot multiplied by the square of the distance of the point of strain in feet from the free end of the cantilever, and divided by twice the depth of cantilever in feet.

For the maximum strain at the point of support, putting $x = l$, we have

$$S = \frac{w\,l^2}{2\,d}$$

The next cases of transverse strain to be dealt with are those referring to beams supported at both ends and loaded between the points of support.

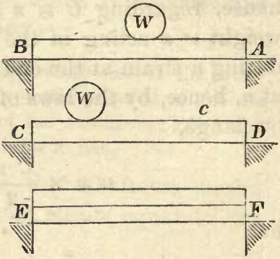
There will be three cases for consideration, as follows:—

1st. A beam loaded in the centre of its span, as at *A B*.

2nd. A beam loaded at a point not central, as at *C D*.

3rd. A beam having the load uniformly distributed over its length, as at *E F*.

Fig. 9.



In the first and third cases the maximum strain on either flange is at the centre of the span, but in the second it is immediately under the load, wherever that may be. We shall deal with the first case first.

Rule (5). To find the strain on either flange at the centre multiply the weight by the span (or distance between points of support), and divide the product by four times the depth of the beam. The strain will be in terms of the same name as the weight.

Example:—Let the weight be 7·5 tons, the span of the beam 12 feet, and its depth 1 foot 3 inches.

Depth of beam - 1·25	7·5 tons weight,
4	12 feet span,
5	) 90·
	18 tons' strain on either flange at centre.

To determine the strain at any other point than the centre on either flange, we must proceed as follows:—

Rule (6). To find the strain on either flange at any point, due to a central load, multiply the load by the distance of such point from the nearest pier or point of support, and divide by twice the depth of the beam. The strain will be in terms of the same name as the weight.

Retaining the notations of the last case, let it be required to determine the strain at a distance of 4 feet from one of the piers.

Depth of beam - 1·25   7·5 tons,
2   4 ft. distance of point,
2·5) 30·0 (12 tons' strain on either flange.
25
50
50

Beams supported at each end and loaded between the points of support, have their top flanges in *compression* and their lower flanges in *tension*, this being just the reverse of what is observed in the case of a cantilever fixed at one end and free at the other.

Proceeding to the second case, it is required to determine the strain on either flange at a point immediately under the weight or load W.

Rule (7). To find the strain on either flange under the load, multiply the load by its distance in feet from the farthest pier (C), multiply the product by its distance in feet from the nearest pier, and divide this product by the span of the beam in feet and by its depth in feet. The strain will be in terms of the same name as the load.

Example:—Let the span of the beam be 10 feet, its depth 1 foot 6 inches, the load 4 tons, and let the load be placed 3 feet from the nearest point of support, then will it be 7 feet from the farthest pier.

```
                          4 tons' load
                          7 feet distance from pier C,
                         ----
Span of beam -  10        28
Depth of beam   1·5        3 feet distance from D,
               ----      ----
                 15      )84( 5 9/15 tons' strain on either
                          75                 flange.
                         ----
                           9
```

It is next required to find in this second case the strain at any point not immediately under the load, such as at *c*.

Rule (8). To find the strain at any point on either flange, such point lying between the weight and one pier, multiply the weight by its distance from the other pier in feet, multiply the product by the distance of the point of strain in feet from that pier which is on the opposite side of it to that occupied by the weight, and divide this product by the span of the beam in feet and by its depth in feet. The strain will be in terms of the same name as the load.

Example:—Retaining the notations above, let the strain be required at a point distant 4 feet from the pier farthest from the weight.

```
                            4 tons' weight,
                            3 feet distance from pier,
                           ----
Span of beam - - 10         12                [opposite pier,
Depth of beam -  1·5        4 feet distance of point from
                ----       ----
                  15       )48( 3 1/5 tons' strain on either
                            45               flange.
                           ----
                             3
```

In the third case the load is uniformly distributed over the girder or beam $E F$. To find the strain in the centre we have

Rule (9). To find the strain on either flange at the centre, multiply the total load on the beam by the span of beam in feet, and divide the product by eight times the depth of the beam in feet. The strain will be in terms of the same name as the load.

Example:—Let the span of the beam be 50 feet, the total distributed load 75 tons, and the depth of the beam 4 feet.

$$\begin{array}{lrrl}
\text{Depth of beam -} & 4 & 75 & \text{tons total load,} \\
 & 8 & 50 & \text{feet span of beam,} \\
\hline
 & 32 &)\,3750\,(& 117\tfrac{3}{16} \text{ tons' strain on} \\
 & & 32 & \text{either flange at centre.} \\
\cline{3-3}
 & & 55 & \\
 & & 32 & \\
\cline{3-3}
 & & 230 & \\
 & & 224 & \\
\cline{3-3}
 & & 6 & \\
\end{array}$$

Now let it be required to determine the strain under an uniformly distributed load at any other point.

Rule (10). To find the strain on either flange at any point, multiply the span of the girder in feet by the distance in feet of the nearest point of support; from the product subtract the square of the distance in feet of the nearest support, and multiply the remainder by the load per foot, and divide by twice the depth of the beam in feet. The strain will be in terms of the same name as the load.

With the previous notations, let it be required to determine the strain on either flange at a point distant

12 feet from the nearest pier. As the total load is 75 tons, and this load is uniformly distributed over a distance of 50 feet span, it follows that the load per lineal foot will be 1·5 tons; from this we can obtain the required strain.

12 ft. distance of nearest pier,		50 span,
12 ft. ,, ,, ,,		12 feet distance of nearest pier,
144 square of dist. ,, ,,		600
		144 square of dist. of nearest pier,
		456
		1·5 tons' load per ft.,
Depth of beam -	4	2280
	2	456
	8	)684·0
		85·5 tons' strain on either flange.

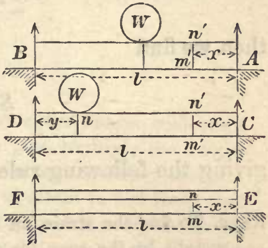
If a girder supported at both ends, subject to two or more loads differently arranged, let the effect of each load on any point at which the strain is required be separately determined, and then those effects added together to find the total strain on the flange; then the foregoing rules will serve to solve any case that may arise. These practical rules, however, we must now proceed to demonstrate.

In the first case we have a beam loaded in the centre with a weight W. Let, in all three cases, $l =$ span and $d =$ depth of beam, and $S =$ the strain on the flange.

Because W, the weight, is situated at equal distances between the points of support A and B, each pier will sustain one-half of the weight; but as action and reaction must be equal and opposite, in order to satisfy the condition of equilibrium, each pier must react upwards in the direction of the arrow, with a force equal to the pressure upon it, hence the reaction of the pier will be

$$= \frac{W}{2}$$

Fig. 10.



Let the strain be required on the flange at the point n, distant x from the pier A, then, regarding $A\ m'\ n'$ as a bent lever, we find the reaction at the end of the arm $A\ m'$ exerting a strain at the end n of the arm $m\ n$ hence

$$S \times m'\,n' = \frac{W}{2} \times A\,m'$$

but,

$$m'\,n' = d \text{ and } A\,m' = x$$

wherefore,

$$S\,.\,d = \frac{W\,x}{2}$$

and

$$S = \frac{W\,x}{2\,d}$$

whence is derived the practical rule, thus :—

Rule. To find the strain on either flange at any point due to the central load, multiply the load by the distance of such point from the nearest pier or point of support, and divide by twice the depth of the beam.

In order to ascertain the strain at the centre of the span on either flange from the above formula, make

$$x = \frac{l}{2}$$

then we find

$$S = \frac{W l}{4 d}$$

giving the following rule:—

RULE. To find the strain on either flange at the centre, multiply weight by the span (or distance between points of support), and divide the product by four times the depth of the beam.

In treating analytically the second case $C\ D$, let it be required to determine the strain on either flange at any point n' distant x from the pier C, the weight W being distant y from the pier D. The reaction of the pier C must first be determined; it will be equal to the weight upon it. Regarding $C\ D$ as a lever on which the force is applied at W, the fulcrum being at D, the weight on C will be

$$= \frac{W \cdot y}{l}$$

and this will represent the amount of reaction which, acting at the extremity C of the imaginary bent lever $C\ m'\ n'$, produces a strain at n on the flange; we have, therefore,

$$S \times m'\ n' = \frac{W y}{l} \times C\ m$$

but,

$$m'\ n' = d \text{ and } C\ m' = x$$

wherefore,

$$S \cdot d = \frac{W \cdot y \cdot x}{l}$$

and

$$S = \frac{W \cdot y \cdot x}{l\, d}$$

Whence is derived the rule:—

Rule. To find the strain at any point on either flange, such point lying between the weight and one pier, multiply the weight by its distance from the other pier in feet, multiply the product by the distance of the point of strain in feet from that pier which is on the opposite side of it to that occupied by the weight, and divide this product by the span of the beam in feet and by its depth in feet.

To determine the rule for the strain directly under the weight, let

$$x = l - y$$

then, from the former equation,

$$S = \frac{W\, y}{l\, d} \left\{ l - y \right\} \quad . \quad . \quad . \quad (a)$$

$$= \frac{W\, y}{d} - \frac{W\, y^2}{l\, d}$$

From equation (a) is found the rule:—

Rule. To find the strain on either flange under the load, multiply the load by its distance in feet from the farthest pier, multiply the product by its distance in feet from the nearest pier, and divide this product by the span of the beam in feet and by its depth in feet.

The third case, shown at $E\ F$ (Fig. 10), is that of a beam uniformly loaded throughout its length. Let the load be w per lineal foot, and let the strain on either flange be required at a point distant x from one of the piers E. The whole weight on the beam is evidently equal to the weight per foot multiplied by the span of the beam, or

$$= w \, . \, l$$

and, as this is symmetrically disposed, it will be borne equally by the two piers E and F; hence the reaction of either pier will be

$$= \frac{w \, . \, l \, .}{2}$$

The load between n and the end E of the beam is

$$= w \times E\,m = w \, . \, x$$

and this may be regarded as collected at its centre of gravity, that is, mid-way between E and n; hence this weight acts *downwards* (notice the direction) at the centre of the arm $C\,m$, producing a strain at n,

$$= w\,x \times \frac{\frac{C\,m}{2}}{m \, . \, n}$$

but

$$C\,m = x \text{ and } m \, . \, n = d$$

hence the above

$$= \frac{w\,x^2}{2\,d}$$

Again, the reaction of the pier acts *upwards* at the end E of the arm $E\,m$, producing a strain at n,

$$= \frac{w\,l}{2} \times \frac{C\,m}{m \, . \, n}$$

$$= \frac{w\,l\,x}{2\,d}$$

but as these two forces act in opposite directions, one must be deducted from the other to find the actual strain at n, then

$$S = \frac{w\,l\,x}{2\,d} - \frac{w\,x^2}{2\,d}$$

$$= \frac{w}{2\,d}\left\{l\,x - x^2\right\}$$

Putting this equation into words, we have

RULE. To find the strain on either flange at any point, multiply the span of the girder in feet by the distance in feet of the nearest point of support; from the product subtract the square of the distance in feet of the nearest support, and multiply the remainder by the load per foot, and divide by twice the depth of the beam in feet.

For the formula for the strain at the centre, make $x = \frac{l}{2}$, then

$$S = \frac{w\,l^2}{4\,d} - \frac{w\,l^2}{8\,d} = \frac{w\,l^2}{8\,d}$$

but $w\,l =$ the total load distributed over the girder; let this be called $= W$, then

$$S = \frac{W\,l}{8\,d}$$

From this formula is derived the practical rule for determining the strain at the centre of either flange, which is as follows:—

RULE. To find the strain on either flange at the centre, multiply the total load on the beam by the span of beam in feet, and divide the product by eight times the depth of the beam in feet.

This completes the demonstration of the rules for determining the strains upon the flanges of flanged cantilevers and beams of the section generally shown at *C* in Fig. 7; but hitherto we have not taken into consideration the duty which devolves upon the central part or *web* connecting the flanges.

Generally, it may be assumed in flanged beams that the *web* sustains the weight of the load, the flanges serving to keep the web extended, for which reason they have by some been termed "booms." The strain on the web will be simply a shearing strain acting vertically, and the amount of this strain will in the foregoing cases be evidently found by the following rules:—

Rule (11). The shearing strain on the web of a cantilever supporting a weight at its extremity will throughout be equal to the amount of weight so supported.

Rule (12). The shearing strain on the web of a cantilever carrying a uniformly distributed weight will at any point be found by multiplying the weight per foot lineal by the distance of such point from the free end of the cantilever in feet.

Example:—The shearing strain is required to be found at a point distant 7 feet from the free end of a cantilever carrying 1,200 lbs. per lineal foot:—

1200 lbs.' load per lineal foot,
7 feet distance of point of strain,
8400 lbs.' shearing force required.

Rule (13). In the web of a beam supported at both ends and loaded in the centre, the shearing force is throughout equal to one-half of the central load.

Rule (14). In the web of a beam supported at both ends and loaded uniformly, the shearing strain at any point is found by multiplying the load per foot by the distance of such point in feet from the centre of the span.

Example :—Required the shearing strain of the web of a beam carrying 1·5 tons per foot at a distance of 8 feet from the centre of the span.—

1·5 tons' load per lineal foot,
8 feet distance of point of strain,

12·0 tons' shearing force required.

RULE (15). In the web of a beam carrying a concentrated load not in the centre of span, the shearing force throughout on *one* side of the load will be found by multiplying the load by its distance in feet from the point of support on the *other* side of the load, and dividing the product by the span of the beam in feet.

Example :—Let a beam of 20 feet span support a weight of 4 tons at a distance of 16 feet from one point of support, then the shearing strain throughout the 16 feet of web will be thus found :—

4 tons' load,
4 feet distance.

Span of beam - 20)16·0(0·8 tons' shearing strain.
160

and the strain on the remaining 4 feet of web thus :—

4 tons' load,
16 feet distance,

Span of beam - 20) 64 (3·2 tons' shearing force.
60

40
40

The following rules will serve to find the resistance of *solid* rectangular bars to transverse strain :—

RULE (16). To find the resistance in pounds of a rectangular bar fixed at one end and loaded at the other, multiply the breadth in inches by the square of the depth in inches by a CONSTANT MULTIPLIER (found by experiment), and divide by the length of bar in inches.

Example:—Let the resistance of a bar 20 inches long, 1·5 inches wide, and 2 inches deep, be required, the *multiplier* for the material of which it is composed being 8000:—

	1·5	inches' breadth,
	4	square of depth,
	6·0	
	8000	multiplier,
Length of bar - 2,0	) 4800,0·0	
	2400	lbs.' resistance of bar.

RULES.

1. The dimensions being the same, a cantilever with the weight distributed uniformly over it will bear twice the weight.

2. If the bar be supported at both ends, and have a central load, it will bear four times the weight.

3. If the bar be supported at both ends and uniformly loaded, it will bear eight times the weight.

(*Note*).—The above constant would give the breaking strain of the beam if made of cast-iron.

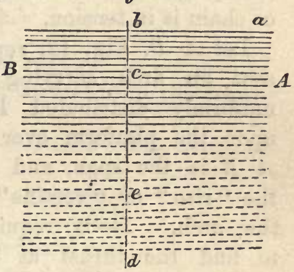
From these rules it will be seen that the law of the resistance of *solid* bars to transverse strain is as follows:—

LAW. The resistance of a solid beam to transverse strain, the span and conditions of supports and load being unaltered, varies directly as the breadth of the beam and as the square of the depth of the beam.

If two beams of equal size be put side by side, it is evident they will support twice the load that one of them alone would; but using two bars is equivalent to doubling the width of one, hence it is clear that *the strength of the bar varies as its breadth.*

Let *A B* (Fig. 11) represent a solid bar supposed to consist of a number of layers, and *a b* one arm of the assumed bent lever, referred to in previous demonstrations, in the arm *b d* may be found a point *c* at which the resisting efforts of all the fibres may be supposed to be centred, and this will always bear a certain relation to the whole depth of the beam; hence, if the bar is increased by layers, shown by the dotted lines (but the whole being *solid*, not in *separate* layers), the length to this centre of resistances will increase from *b c* to *b e*, varying as the depth of the bar. Again, by increasing the depth of the bar, we also, in like proportion, increase the number of layers of resisting fibres; hence thus also increase the strength. Thus, by increasing the depth of the bar, we increase the *number* of resisting fibres as the *depth*, and increase their *mean leverage* as the *depth;* wherefore it is found that *the strength of the bar varies as the square of its depth,* and thus the accuracy of the law is proved. There is another law to be mentioned in connection with rectangular bars; it is this:—

Fig 11.



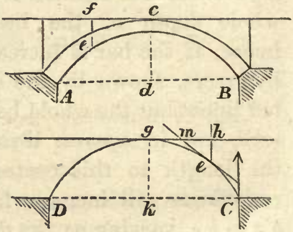
LAW. All other things being unaltered, the strength of the bar varies inversely as its length.

This is evident, for as the length of the bar varies so does the length of the arm *a b* of the assumed lever upon which the load acts vary, and the greater the leverage of the load the less will be the amount of load which the bar is capable of sustaining.

Having thus disposed of the strains upon straight beams, we must now proceed to treat of the effects of distributed load upon arches and chains. The intensity of the load will in both cases be determined by the same rules, the difference between the two being that the upright arch is in compression, but the inverted arch or chain is in tension.

Let *A B*, Fig. 12, represent an arch carrying an uniformly distributed load upon the platform over it. *A B* is the span, and *d c* the "rise" or "versine" of the arch. It is required to find the thrust at the centre or crown of the arch.

Fig. 12.



RULE (17). Multiply the load per lineal foot by the square of the span of the arch in feet, and divide the product by eight times the versine of the arch in feet.

Example :—Let the arch be 240 feet span, its versine 30 feet, and the load per lineal foot 5 tons :—

```
                        240  feet span,
                        240
                       9600
                       480
                      -----
Versine of arch 30    57600  square of span,
                 8        5  tons' load per lineal foot,
               240  )288000(  1200 tons' thrust at crown
                      240                          of arch.
                      ---
                      480
                      480
                      ---
                       00
```

It is now requisite to determine the thrust at any other point, *e*, lying vertically under the point *f*.

RULE (18). To find the thrust at any other point than the crown, multiply the load per lineal foot by the distance of such point from the crown, square the product, and add to it the square of the thrust at the crown, then the square root of the sum will be the thrust at the point referred.

(*Note.*—This rule will of course give the thrust at the abutment by taking the point at a distance of half the span from the crown of the arch.)

Example:—Let the thrust be required in the case of the above arch at a distance of 40 feet from the crown or centre:—

```
                         1200   tons' thrust at
                         1200   [crown.
                       -------
                       1440000
                       =======

         5   tons' load per lineal foot,
        40   feet distance of point,
       ---
       200
       200
     -----
     40000
   1440000   square of thrust at crown,
  --------
1 )1480000(1216 tons' thrust at point (nearly).
   1
  --
22 ) 48
     44
    ---
241 ) 400
      241
     ----
2426 )15900
      14556
      -----
       1344
       ====
```

After what has already been shown in previous pages, the demonstration of these rules will be comparatively simple.

Let $C D$ (Fig. 12) represent the outline of an arch having a span $C D = l$, and a versine $k g = v$; let the load per lineal unit $= w$, the thrust at centre $= T$, and the thrust at any point distant x from the centre $= S$:—As the arch is uniformly loaded throughout its length, the actual weight or vertical reaction on each pier will be half the total load, or

$$= \frac{w l}{2}$$

and regarding $C k g$ as an imaginary bent lever, this force will act with the leverage $C k$, producing a thrust at the end g of the arm $k g$; hence the thrust on the crown of the arch will be the same as that at the centre of the top flange of a flanged beam having a span equal to that of the arch and a depth equal to its versine, therefore

$$T = \frac{w l^2}{8 v} \quad . \quad . \quad . \quad . \quad . \quad (b)$$

The thrust at any point e is the resultant of two forces acting at right angles to each other, the one being the horizontal thrust above given, the other the vertical weight of the load between h and g. Let $h m$ and $h e$ represent these two forces, then, according to the parallelogram of forces, $m e$ will be the resultant strain. But because $e h m$ is a right-angled triangle (Euc., Bk. 1, prop. 47)—

$$\overline{e m}^2 = \overline{e h}^2 + \overline{h m}^2$$

but,

$$e h = w x$$

and

$$h m = \frac{w l^2}{8 v}, \; e m = S$$

hence

$$S^2 = (w\,x)^2 + \left(\frac{w\,l^2}{8\,v}\right)^2$$

$$S = \sqrt{(w\,x)^2 + \left(\frac{w\,l^2}{8\,v}\right)^2} \quad . \quad . \quad . \quad (c)$$

From equation (b) we have the rule for the strain at the centre:—

RULE. Multiply the load per lineal foot by the square of the span of the arch in feet, and divide the product by eight times the versine of the arch in feet.

From equation (c) the rule is found for the strain at any other point.

RULE. To find the thrust at any other point than the crown, multiply the load per lineal foot by the distance of such point from the crown, square the product, and add to it the square of the thrust at the crown, then the square root of the sum will be the thrust at the point referred to.

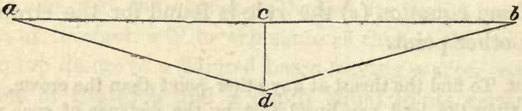
CHAPTER III.

COMBINATIONS OF ELEMENTS AND DISTRIBUTION OF LOADS.

IN the preceding chapter we have shown how to calculate the *strains* on various kinds of elements when the *loads* or *forces* by which such strains are caused are known; in the present we intend treating of the distribution of loads due to certain combinations of elements, as preliminary to explaining the modes of proportioning complex structures according to the weights they are designed to sustain. In order to make clear the mode of determining the distribution of loads, a number of generic cases will be taken, and dealt with *seriatim.* The rules referred to for ascertaining the strains being given in Chapter II., they will here be indicated by numbers corresponding to those in brackets following the word rule in the previous chapter, thus: "Rule (2) To find, &c."

Fig. 13 represents the simplest form of truss, consisting of a horizontal member *a b* trussed by two tension-

Fig. 13.



rods *a d*, *b d*, attached at their lower ends to a strut *c d* placed in the centre of the truss.

If the load be placed at the centre, its whole weight will be on *c d*, and as half its weight will be transmitted to *a* and half to *b*, the load for each tie-rod will be one half the central weight.

If the load be uniformly distributed over the distance *a b*, one half will be on *a c* and one half on *c b*, and of that portion on *a c* one half (one quarter of the whole load) will be sustained at the point *a*, and the other at *c*; and in like manner one half of the load on *c b* will be supported at *b*, and the other at *c*; hence on the strut *c d* we have half of the loads on *a c*, *c b*, that is, half the whole load on the truss, and this being divided between the tie-bars *a d*, *d b*, gives one quarter the whole load to be carried by each tie-rod.

Let the lengths be, *a b* = 32 feet, *c d* = 4 feet, and *b d* = 16·5 feet, the load 600 lbs., then *c d* will represent the "perpendicular distance" of the bar *b d*, hence the strain on the latter will be thus found by Rule (1):—

The *load* on tie-rod is

```
                         4 ) 600
                             ———
                               150 lbs.' load,
                               16·5 ft. length of bar.
                             ———
                               750
                              900
                             150
                            ——————
Perpendicular distance - 4 ) 2475·0
                            ——————
                               618·75 lbs.' tension on
                            ======      [each tie-bar.
```

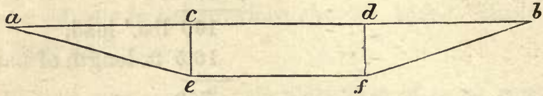
The parts *a c* and *c b* act as horizontal members to fix the positions of the ends *a* and *b* of the tie-rods, hence the load on *b d* must be resolved for the direction of the element *c b*, the length of which (half 32 feet) is 16 feet, which is also equal to the horizontal distance of the bar *b d*. The thrust on *c b* is found by Rule (2):—

```
                              150 lbs.' load,
                               16 feet " horizontal
                             ———
                              900        distance."
                             150
                            —————
Perpendicular distance - 4 ) 2400
                            —————
                              600 lbs.' thrust on hori-
                            =====  zontal member a b.
```

If this truss were inverted so as to represent the main principal of a roof, and loaded to the same extent as here supposed, the intensity of the strains would remain the same as given herein, but the bars *a d*, *d b*, would be in compression and *a b* in tension; and if the truss were loaded on its straight member, the rod *c d* would be in tension.

The truss shown in Fig. 14 is somewhat similar to the above. Let it be supposed to carry a distributed load,

Fig. 14.



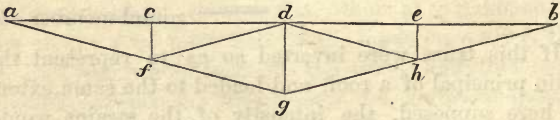
then the load upon the strut *d f* will be equal to half the load between *d* and *b*, plus half the load between *c* and *d*, or equal to one half the load between *c* and *b*, and this load has to be sustained by the inclined bar *f b* (Rule 1), kept in its position by the two horizontal elements *d b* and *e f*, both of which bear an equal strain (Rule 2), but the top member is in compression and the bottom one in tension.

This truss is sometimes used in an inverted position for roof principals, and also for bridging narrow streams.

It is evident, from what has already been shown, that when a structure is of symmetrical form about its centre, and is also similarly loaded on each side of its centre, that the strains are similar; hence, in many cases, it is only necessary to calculate the strains for one half of the work.

Fig. 15 illustrates a form of compound truss, of which *a g b* is called the primary truss, and *a f d*, *d h b*,

Fig. 15.



secondary trusses. The horizontal member *a b* is divided into four equal parts by the struts at *c*, *d*, and *e*, hence, if the load be uniformly distributed over it, there will be one quarter of the load sustained at each of the three struts, and one-eighth of the load direct at each of the points a and b.

We will now proceed to calculate the strains.

Let the length $a\,b = 64$ feet, and the various bars be of the following lengths:—

$$d\,g = 8 \text{ feet}$$
$$c\,f\text{, or } e\,h = 4 \text{ feet}$$
$$a\,g\text{, or } g\,b = 33 \text{ feet}$$
$$\left.\begin{matrix} a\,f\text{, or } f\,d \\ h\,d\text{, or } h\,b \end{matrix}\right\} = 16{\cdot}5 \text{ feet}$$
$$\left.\begin{matrix} a\,c\text{, or } c\,d \\ d\,e\text{, or } e\,b \end{matrix}\right\} = 16 \text{ feet}$$
$$a\,d \text{ or } d\,b = 32 \text{ feet}$$

We must commence with the secondary truss, as a part of the weight carried by it is transmitted through the primary truss to the points of support a and b. Let the total load on the whole truss be 24,000 lbs., then the load on the strut $c\,f$ or $e\,h$, will be 6,000, and that for each tie-bar

2) 6000
3000 lbs.' load on $a\,f$ or $f\,d$.

The strain *due to this load* on the bars $a\,f$, $f\,d$, $d\,h$, $h\,b$, must now be found by Rule (1):—

3000 lbs.' load,
16·5 feet length of bar.

Perpendicular distance, 4) 49500·0
12375 lbs.' tension on each tie-bar.

Proceeding to the primary truss, the load on the strut *d g* is composed of three quantities, the load acting simply at *d*, and the portions of the loads transmitted from *c* and *e*, through the bars *f d* and *d h*, being half of each of these loads, hence on *d g* we have

6000	lbs.' load at *d*,
3000	lbs' ,, from *c*,
3000	lbs.' ,, ,, *e*,
12000	total load.

From this the strains on *g a*, *g b*, due to the load at *d g*, are found, half the load, or 6,000 lbs., being on each.

6000 lbs.' load,
33 ft. length of bar,

Perpendicular distance - 8)198000

24750 lbs.' strain on bars *g a* and *g b*.

This will be the amount of strain on the parts *g f* and *g h* of the bars *g a* and *g b*, but the parts *f a*, *h b* will, in addition, have the strains due to the loads on *c* and *e*, hence the total strain will be

12375	lbs. from load *c*,
24750	lbs. on ,, *d g*,
37125	total tension on *f a* or *h b*.

It now remains to determine the thrust on the horizontal member *a b*; it will be the sum of the thrusts brought upon it by the load on *c f* acting through *a f*, and on *d g* acting through *g a*.

The load transmitted through *a f* is 3000 lbs. ; hence, by Rule (2),

3000 lbs.' load,
16 feet "horizontal [distance."

Perpendicular distance - - 4) 48000

12000 lbs.' thrust due to load on *a f*.

The load transmitted through *g a* is 6000 lbs., hence

6000 lbs.' load,
Perpendicular 32 feet "horizontal distance."

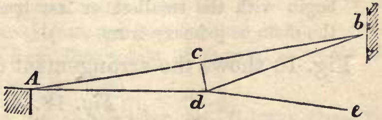
distance - - - 8) 192000

24000 lbs.' thrust due to load on *g f*,
12000 lbs.' " " " *a f*.

36000 lbs.' total thrust on *a b*.

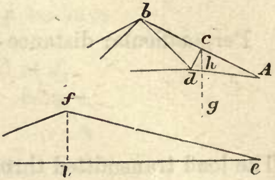
Frequently two inclined trusses are put together to form a main principal for a roof, a half principal thus formed is shown in Fig. 16. *A b d* is a simple truss placed at an angle to the horizon, and at the point *b*, meeting another similar truss (not shown), which completes the roof principal, the two trusses are prevented from separating under the load by the tie-bar *d e*, which takes the thrust at the abutments.

Fig. 16.



From the point *c*, Fig. 17, draw the vertical line *c g*, and, from the point *d*, the horizontal line *d h*, meeting *c g* at *h*, then *c h* is the "perpendicular distance" of the inclined strut *c d* the strain upon the strut can now be calculated from Rule (1), and that done, the strains on the tie-bars can be calculated precisely as in the case of the simple horizontal truss; *but* the *strain* on the strut must be substituted for the *load* on the same.

Fig. 17.



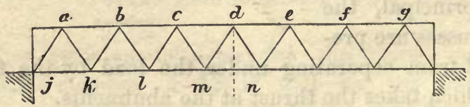
There yet remains the tension on the rod *d e*, Fig. 16, or *e i*, Fig. 17, to be considered. In this case the whole weight on the rafter *e f* is to be taken as load for the thrust at *e*, and this load resolved into strain on *e i*, as the horizontal member fixing the position of the inclined member at the point *e* by Rule (2). The length *e i* is the "horizontal distance," and *f i* is the perpendicular distance.

The foregoing cases will serve to explain the method of ascertaining the distribution of loads in all cases of trusses of the types referred to, for it matters not how complicated a truss may be, the method pursued in determining the loads and strains is the same, and may be generally stated in the following rule.

RULE. In determining the strains on a compound truss, always begin with the smallest or last truss, and work from that tc the main or primary truss.

Fig. 18 shows the arrangement of bars in a warren

Fig. 18.



girder, in which they form a series of equilateral or equal-sided triangles.

In this case, the load being equally distributed over the girder, it may be considered as divided into as many equal parts as there are triangles in the girder; in the case illustrated these will be seven in number. The dotted line *d n* indicates the centre of the span of the girder; of the load at *d* one half passes to each pier, but the whole of the loads on *c b* and *a* pass to the pier *i*, the other point of support bearing the corresponding weights on the summits of the triangles on that side of the central line *d n*.

In this arrangement the length of the inclined bars to their perpendicular distances does not vary, because the bars are all of one length, and the perpendicular distance is always equal to the depth of the girder, which is the same throughout; hence the ratio of the strain to the load will be the same throughout the girder—that is, the strains of the inclined bars will vary in the same ratio as the loads by which they are produced. The load at *d* will (one half of it) produce a compressive strain at *d m*, and that will produce a tensile strain on *m c*, whence the strain on *c l* will be compressive, and so forth; hence the following rule will determine the nature of the strains on the bars of lattice or triangular girders.

Rule. All bars inclining downwards from the loads *towards* the points of support are struts, and bars inclined in the opposite direction, or *away* from the points of support towards the centre or point of load, are ties.

Let the total load on the girder be 70 tons, then the load on the apex of each triangle at the points *a*, *b*, *c*, &c., will be

7) 70 tons' total load.

10 tons' load on *a*, *b*, *c*, &c.

The load on the bar *d m* will be 5 tons, that on *c l* will be 5 tons transmitted from *d m* through *m c*, and in addition 10 tons, the load on *c* making 15 tons, and in like manner the loads on *b* and *a* will be 25 tons and 35 tons.

The load on the bars being determined, it remains therefrom to deduce the strain caused by such load. We will take a case where the proportions are such as are found in the Warren girder.

Let the length of the bars be 10 feet and the depth of the girder or perpendicular distance be 8 feet 8 inches, then the strain on the bar *c l* will be thus found by Rule (1):—

```
                          15 tons' load.
Depth of girder, or       10 feet length of bar.
                          ———
Perpen. distance, 8·66 )150·00( 17·3 tons' strain (nearly).
                          866
                          ———
                          6340
                          6062
                          ————
                           2780
                           2598
                           ————
                            182
                           ════
```

And in a similar manner the strains of the other bars may be determined. The general rule will be as follows:—

Rule (19). To find the strain on any lattice bar, multiply the load upon it by the length of such bar in feet, and divide by the depth of the girder in feet.

And if the total load on the girder be equally distributed, and the load on the summit of each triangle be given:

RULE (20). To find the strain on any strut and its following tie, multiply the load on one summit by the number of triangles included between the strut and the centre of the girder, and by the length in feet of the bar, and divide by the depth of the girder.

Example:—Let it be required to determine the strain on the strut $c\,l$. There are one-and-a-half triangles between it and the central line, $l\,e\,m$ being one and $m\,d\,n$ the half, hence the load per summit being 10 tons:—

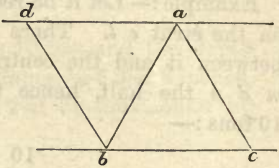
```
                          10   tons' load per summit,
                          1·5  number of triangles,
                         ----
                          15
                          10 feet length of bar,
                         ------
Depth of girder - 8·66 ) 150·00 ( 17·3 tons' strain (nearly).
                          866
                         ------
                          6340
                          6062
                         ------
                           2780
                           2598
                          ------
                            182
```

The strain on each strut acts as a force, producing a tensile strain on the bottom flange, and that on each tie causes compression on the top flange; the forces thus produce strains on the horizontal members, increasing in intensity from the points of support to the centre of the span. For instance, a certain tension is thrown from the strut $a\,j$ on $j\,n$, which at k is augmented by the strain caused by the force on *b k*, and, again, it is similarly increased at the points *l* and *m*.

The force being known (that is, the strain) on any strut or tie, it is necessary to ascertain the corresponding strain on the horizontal member.

The strain on the tie *a b*, Fig. 19, being known, it is required to find that which it causes on *a d*. The strain on *a b* being taken up by the elements *a c*, *a d*, *a c b d*, is the paralellogram of strains, hence the rule for this case will be as follows :—

Fig. 19.



RULE (21). To find the strain on the horizontal member produced by the tie or strut, multiply the strain on the tie or strut by the length of the horizontal member *a d* in feet, and divide by the length of the tie or strut in feet.

In the present case, as the triangles are equal sided, the bars are of equal length; therefore, the strain produced by the tie or strut is equal to that which it sustains itself, except in the case of the end strut :

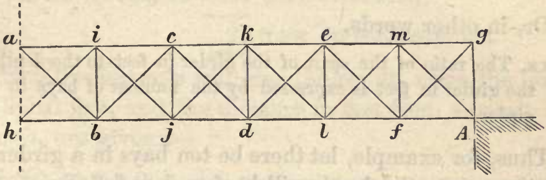
The strain on the end strut is resolved vertically on to the point of support, and horizontally on to the member *j k*, Fig. 18. Hence *j k* assumes the position of a horizontal element proper, and the strain upon it may be determined from Rule (2), which, for the present case, may be worded as follows :—

RULE (22). To find the strain thrown on to the horizontal member by the last strut, multiply the *load* on the last strut by half the length of the base of the end triangle, and divide by the depth of the girder.

These rules will solve all questions having reference to girders the ties and struts of which are of equal length, the relative length of the bases of the triangles not impairing their accuracy; but they will not apply to cases in which the lengths of the struts differ from those of the ties.

At Fig. 20 is shown half of a lattice, or triangular girder, of a different form from that in the last case. It will be observed that the inclined bars in the present

Fig. 20.



instance are placed at an angle of 45 degrees to the horizontal flanges of the girder, and therefore at right angles to each other.

A represents one pier or point of support, *a h* being the centre line of the whole girder. *A h* is the bottom flange, and *g a* the top flange of the girder; *m f*, *e l*, *k d*, *c j*, *i b*, and *a h*, are uprights, serving, as will hereafter be shown, to distribute the loads upon the girder so that it may fall equally upon the two systems of triangulation which form the web of the girder, one of which systems comprises the bars *a b*, *b c*, *c d*, *d e*, *e f*, *f g*, whilst the other consists of the bars *h i*, *i j*, *j k*, *k l*, *h n*, *m A*.

The half girder is, as shown, divided by the vertical bars into six *square* bays, *A g m f*, *f m e l*, *l e k d*, *d k e j*, *j c i b*, *b i a h*, hence the whole girder would contain twelve bays, and, as the bays are square, it follows that in this instance the length or span of the girder will be twelve times its depth, the measurements being taken from the points of intersection of the centre lines of the struts and ties, which centre lines alone are shown in the Figure.

Therefore when this system is adopted we have the following rule:—

RULE. To find the number of bays in a triangular girder of the form shown in Fig. 20, divide the span by the depth of the girder, both in feet.

Or, in other words,

RULE. The ratio of the span of the girder in feet to the depth of the girder in feet is expressed by the number of bays in the girder.

Thus, for example, let there be ten bays in a girder of 150 feet span, the depth will be found as follows:—

Number of bays, 10) 150 feet span of girder,

15 feet depth of girder.

Again, if the span be 200 feet and the number of bays twelve:—

Number of bays, 12) 200 feet span of girder.

$16\frac{8}{12}$, or 16 feet 8 inches depth of girder.

So much for the proportions entailed by this system of construction. Let us now proceed to the distribution of loads, and the strains thereby produced. The load being, as usual, supposed to be uniformly distributed along the entire length of the girder, the load acting at each point of junction of ties and struts. as at *m*, *e*, *k*, &c., if the load be on the top of the girder, or at *f*, *l*, *d*, &c., if it be on the bottom flange will evidently be equal to the total load on the whole girder divided by the number of bays in the girder; hence:—

RULE (23). To find the load in tons carried at one summit of a triangle, divide the total load in tons upon the girder by the number of bays of which the girder consists, the quotient will be the required load in tons.

Example:—Let a girder of twelve bays be designed to carry a total load of 300 tons:—

Number of bays, 12) 300 tons' total load

25 tons on each summit.

The following rule will give the result, when the load per lineal foot, and the distance in feet from summit to summit, are given:—

RULE (24). To find the load in tons carried at one summit, multiply its distance in feet from the next summit by the load in tons per lineal foot of the girder, the product will be the required load in tons.

Example.—Let the load per lineal foot on a girder be 2·5 tons, and the distance between two contiguous summits be 14 feet 3 inches:—

14·25 feet distance between summits,
2·5 tons' load per lineal foot,

7125
2850

35·625 tons' load on each summit.

One half of this load will be taken up at once by one series of triangles, and the other half will pass through the upright to the other series of bars.

(*Note.*—It is desirable here to observe that the term "summit" of a triangle is here used as applying to its apex, whether at the top or bottom of the girder: thus *e* is the summit of triangle *f e d;* also *l* is the summit of triangle *m l k.*)

For the sake of clearness it will be advisable to take two distinct cases, according to the general methods of loading, and a third special or supplementary case.

1*st Case.*—When the load, being on the top of the girder, comes first upon it at the points *a*, *i*, *c*, *k*, *e*, *m*.

2*nd Case.*—When the load, being at the bottom of the girder, comes first upon it at the points *h*, *b*, *j*, *d*, *l*, *f*.

3*rd Case.*—When the load, being between the top and bottom flanges, comes first upon certain intermediate points in the uprights *a h*, *i b*, *c j*, *k d*, *e l*, *m f*.

In each case, however, let it be borne in mind that all loads on the half-girder *A h* are being transmitted through the inclined bars towards and ultimately on to the pier *A*, except the central load on the line *A h*, one half of which goes to the pier *A*, the other passing to the other point of support, not shown in the diagram.

1*st Case.*—Let the girder consist of twelve bays, and be 200 feet span, the total load upon such girder being 250 tons, carried on the top flange, then the load on each summit *m*, *e*, *k*, &c., is found by Rule (23).

Number of bays, 12)250 tons' total load on girder,

20·83 tons' load on each summit

As the terminal decimal is current, we will call the total load on each summit 20·84 tons, except the end *g*, where the girder terminates, and on which the load is one half of this, or 10·42 tons. Before proceeding farther, it is desirable to test our calculations, by showing that the loads thus distributed will in the aggregate make up the total load on the girder with which we started at the commencement. On the centre point *a* we have the load 20·84 tons, and a like load on each of the five points *m*, *e*, *k*, *c*, *i*, and on the five corresponding points in the other half of the girder—thus we have the load 20·84 tons on eleven points situated over the summits *between* the piers. Then over the pier *A* there is a load of 10·42 tons, and a similar load over the pier on which the other end of the girder rests. The sum of the weights on the eleven intermediate summits will be—

20·84	tons per summit,
11	summits,
229·24	tons on eleven intermediate summits

The sum of the loads over the two piers—

10·42	tons over each pier,
2	piers.
20·84	tons on standards on piers.

Making the gross load on the girder—

229·24	tons on intermediate summits,
20·84	tons on end standards,
250·08	total load on girder.

The amount given was 250 tons; the ·08 tons' excess is due to replacing the current decimal ·3 by the final decimal ·4; the error here is practically nothing, amounting as it does to only 179·2 lbs. in 250 tons.

The uprights *m f*, *e l*, &c., it must be remembered, serve only to distribute the load; they are not designed to take up any strain from the inclined bars; in the present case the load on each intermediate summit *m*, *e*, *k*, &c., is 20·84 tons, of which one half is immediately taken up by the struts at those points, whilst the other half is transmitted through the uprights to the lower summits *f*, *l*, *d*, &c., to be taken up by the ties at these points, thus exerting a compressive strain upon the uprights equal to half the summit-load, or

2) 20·84 tons' summit-load,
10·42 tons' strain on each upright.

The end uprights have, however, also to carry a strain thrown upon them by the end ties, but this is a matter of after consideration.

To ascertain the actual load producing strain on a strut or tie at any point, it is evident we must take one half of the summit-load at that point, as one half is thus shown to be taken up by each series of triangles, but at the centre only one half of the summit-load is taken in the first instance, as only one half of it passes to the pier *A*, hence at that point one quarter of the summit-load must be taken as exerting strain upon either *a b* or *h i*.

It is well to make clear why the load is taken at the top by the struts and at the bottom by the ties, in case any mistake might arise on this point.

Wherever a load is taken up by an inclined bar to be transmitted to the pier *A*, it is evident that the bar must incline from the point at which the load is applied towards that at which it is to be given off; but the bars *i j*, &c., incline from the top flange towards the pier, and the bars *b c* incline from the bottom towards the pier, and from our rules already given the bars *a b*, *i j*, &c., must be struts, whilst the bars *h i*, *b c*, must be ties.

The foregoing remarks being thoroughly understood and carefully borne in mind, the summation of the loads to arrive at the load on any particular bar will be exceedingly simple, being merely a matter of diligence and observation; but one or two examples may perhaps be with advantage here introduced, therefore we will select two,—one having reference to a strut, the other to a tie.

In the first place let it be required to determine the load to be carried by the strut *k l*. The load on *k l* consists of that due to its proportion of the summit-load at *k*, added to the load brought upon it by the tie *j k*, but the load on *j k* is due to the proportion of summit-load thrown upon it from *c* through *c j* and the load brought to it by strut *i j*, and the load on *i j* is due to its share of the summit-load at *i* added to the load brought to it by the tie *h i*, the load on *h i* being its share of the summit load at *a* transmitted through the upright. Now each proportion or "*share*" of summit-load taken up has been shown to be equal to one half the summit-load, except for the centre bars, where the share is one-quarter of the summit-load, hence in the present case the strut *k l* has three half summit-loads (at *k*, *c* and *i*) to carry and one quarter summit-load (at *a*), hence the total load carried by *k l* will be—

10·42 tons' summit-load,
3
_____
31·26
5·21 tons' quarter summit-load,
_____
36·47 tons' total load on strut *k l*.

Let it now be required to determine the load carried by the last tie *f g*. Proceeding in the same manner we find it has to carry five half summit-loads and one quarter summit-load, hence the total load carried by this bar will be

10·42 tons' half summit-load,
5
———
52·10
5·21 tons' quarter summit-load,
———
57·31 tons' total load on tie *f g*.

But as a like load is borne by the strut *m A*, and the two together bring half the total load, minus twice the load carried at the summit *g* (which has been shown to be half a summit-load), on the whole girder to the pier *A*, it follows that four times the total load, transmitted through the bar *f g*, added to twice the load at *g*, should be equal to the total load on the whole girder; let us ascertain if such be the case:—

57·31 tons' total load on tie *f g*,
4
———
229·24
20·84 tons' twice load at *g*,
———
250·08 tons' total load,

which shows our calculations to be accurate (the ·08 excess having been already accounted for in replacing ·3 by ·4 terminal).

The load carried by any tie or strut having been ascertained, the intensity of the strain produced by it is determined in the usual way by Rule (1), which, worded to suit the particular case, becomes as follows;—

Rule. To find the strain in tons on any tie or strut, multiply the load on such tie or strut by its length in feet, and divide the product by the depth of the girder in feet; the quotient will be the required strain.

Any tie or strut is, however, equal to the diagonal of a square, of which the depth of the girder is equal to the side, therefore the length of a tie or strut in feet divided by the depth of the girder in feet, is equal to 1·414, the rule may, therefore, be simplified thus :—

Rule. To find the strain in tons on any tie or strut multiply the load in tons by 1·414.

It has been shown above that the load on the strut *k l* is equal to 36·47 tons, let it be required to find the strain on the same :—

```
    36·47 tons' load on strut k l.
    1·414
    -----
    14588
   3647
 14588
 3647
 --------
 51·56858 tons' strain (compression)
                on strut k l.
```

In order to ascertain the strain brought upon the top or bottom flange at any point by any particular tie or strut, the strain on such tie or strut is taken as the force acting on the flange, and Rule (1) applied.

Let it be required to find the strain produced upon the bottom flange by the strut *k l*. It is evident that *l j* will be the virtual length of the bar under strain, to which *k l* will be the perpendicular distance, but the relation of *l j* to its perpendicular distance is the same as that of $k\ l$ to its perpendicular distance, hence we have the rule.

Rule. To find the strain thrown upon either flange by any strut or tie in tons, multiply the *strain* on such strut or tie by 1·414.

But if the strain on the bar $l\ j$ is required, the *load* on the strut or tie being given, the following rule will suffice :—

Rule. To find the strain thrown upon either flange by any particular strut or tie in tons, multiply the load in tons on such strut or tie by 2.

This rule, however, does not apply to the end struts and ties, where the members $g\ m$ and $A\ f$ act as true horizontal bars; the question is, therefore, solved by Rule (2), whence we derive the following:—

Rule. The strain thrown upon the top or bottom flange of the girder by any end strut or tie is equal to the load upon such strut or tie.

We will now proceed to the second and third cases.

2nd Case.—When the load is at the bottom of the girder. In this instance the intensities of the strains are precisely the same as in the *1st Case*, but the uprights $m\ f$, $e\ l$, &c., are in tension instead of compression.

3rd Case.—When the load is between the top and bottom flanges. In this case the only alteration from the foregoing is that the portions of the uprights *above the load* are in tension, and those portions *below the load* are in compression. The intensity of the strains remains the same throughout.

Before passing from the class of combinations of elements known as lattice girders, it is desirable to give some *general rules* applicable to all descriptions of such structures, first giving them for a central load, and then for an uniformly distributed load. The rules (*a*, *b*, *c*), however, will be given for a single series of triangles, hence if there be more than one series the resulting strains must be divided by the number of series of triangles, for it is evident that the number of series of triangles may practically be regarded as derived from a primary series by the splitting up, as it were, of the large lattice-bars into a number of smaller ones.

Fig. 21 will serve to elucidate the idea here intended to be expressed; it represents a portion of the end of a lattice girder. Suppose, in the first instance, it is proposed to have only one series of triangles, then that series will be represented by the broad lines *A*, *B*, *C*, *D*, but if it be determined to have six series of triangulations, then may the large bar *A B* be regarded as split into *six* smaller ones, occupying the positions *A B*, *a f*, *b g*, *c h*, *d i*, *e j*, the six series of triangles commencing respectively at the six points, *A*, *a*, *b*, *c*, *d*, *e*.

Fig. 21.

We must, therefore, insert the following:—

Preliminary Rule. If in any case, in Rules *a*, *b*, &c., there be more than one series of triangles in the girder, the results obtained from the following rules must be divided by the number of the series of triangles in the girder to obtain the true result applicable to the case.

We will now proceed to give the general rules, and some examples of their application.

Rule (*a*). To find the strain on any lattice-bar under a central load, multiply the load in tons by the length in feet of one lattice bar, and divide the product by twice the depth in feet of the girder, the quotient will be the strain on the bar in tons.

Example.—Let the load in the centre of a lattice girder be 30 tons, the length of a lattice bar 4 feet 6 inches, and the depth of the girder 4 feet :—

30 tons' central load,
4·5 feet length of lattice-bar,

Depth of girder - 4   150
2   120
8)135·0
16·875 tons' strain on any lattice-bar.

If there be two series of triangles we shall have

2)16·875
8·4375 tons' strain on any lattice-bar.

Rule (*b*). To find the strain on the flange at any point under a central load, multiply the load in tons by the base (*A C*, Fig. 21) in feet of one triangle, and by the number of bars (ties and struts) between the point at which the area is required and the nearest point of support, and divide the product by four times the depth of the girder, the quotient will be the strain on the flange in tons.

Example.—Let the central load, as above, be 30 tons, the base of one triangle 4 feet, and the number of bars between the given point and the nearest point of support 6, the depth of the girder being 4 feet :—

30 tons' central load,
4 feet base of triangle,

Depth of girder - 4   120
4   6 bars,
16)720 (45 tons' strain on flange at given point.
64
80
80

We will now give the rules for girders uniformly loaded throughout their length.

Rule (*c*). To find the strain on any lattice-bar under an uniformly —distributed load, multiply the load in tons at each summit (or junction of tie, strut and flange), by the number of summits between the centre of the girder and the bar on which the strain is to be found, and by the length of a lattice-bar in feet, and divide the product by the depth of the girder in feet, the quotient will be the strain on the bar in tons.

Example.—Let the weight per summit be 5 tons, the number of summits between the centre of the girder and the bar 3, the length of a lattice-bar 7 feet, and the depth of the girder 5 feet:—

5 tons per summit,
3 summits,
15
7 feet length of lattice-bar,
Depth of girder 5) 105
21 tons' strain on lattice-bar.

If there be three series of triangulations we have

3) 21
7 tons' strain on lattice-bar.

Rule (*d*). To find the strain on the top flange at any point, multiply the sum of the strains on all the ties between that point and the nearest end of girder by the base of one triangle in feet, and divide the product by the length of a lattice-bar in feet,—the quotient will be the strain on the flange in tons, for any number of series of triangles.

(*Note.*—If the last bar is a tie, take only half the strain on it in the sum).

Example—Let the sum of the strains be 160 tons, the base of one triangle 2 feet, and the length of a lattice-bar 2 feet 6 inches :—

```
                        160 tons' sum of strains on ties,
                          2 feet base of triangle,
                        ------
Length of lattice-bar 2·5)320·0(128 tons' strain on
                          25                  [flange.
                         ----
                          70
                          50
                         ----
                          200
                          200
                         ====
```

As a matter of course, this rule cannot be used until the strains on all the ties have been determined by means of Rule (*c*).

Rule (*e*). To find the strain on the bottom flange at any point, multiply the sum of the strains on all the struts between that point and the nearest end of girder by the base of one triangle in feet, and divide the roduct by the length of a lattice-bar in feet,—the quotient will be the strain on the flange in tons, for any number of series of triangles.

(*Note.*—If the last bar is a strut, take only half the strain on it in the sum.)

Example.—Let the sum of all the strains be 185 tons, the base of one triangle 4 feet, and the length of a lattice-bar 5 feet :—

```
                    185 tons' sum of strains on struts,
                      4 feet base of triangle,
                  -----
Length of lattice-bar 5)740
                    -----
                    148 tons' strain on flange
                    =====
```

With a perfect knowledge of these last five rules (*a*), (*b*), (*c*), (*d*), and (*e*), the strains may be calculated on any part of any lattice girder loaded centrally, or uniformly and freely supported at each end; but the following numbers may be useful in expediting such calculations.

Two angles only are almost exclusively adopted in the construction of lattice girders. They are:—

The angle of 60 degrees between the lattice-bar and the flange, which gives equilateral triangles; and

The angle of 45 degrees between the lattice-bar and the flange, which gives triangles having their vertices right-angled.

In the first case,

The length of lattice-bar divided by depth of girder is equal to—1·154.

Base of triangle divided by length of lattice-bar is equal to—1.

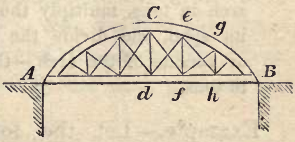
In the second case,

The length of lattice-bar divided by depth of girder is equal to 1·414.

Base of triangle divided by length of lattice-bar is equal to 1·414.

These numbers form convenient multipliers for reducing the loads to strains on lattice-bars, and the latter to strains on flanges.

Fig. 22.



In Fig. 22 is shown a combination called a bowstring girder, supported upon two piers A and B. $A\ C\ B$ is an arch on which the strains may be calculated in the ordinary way by means of

Rules (17) and (18). Instead, however, of the thrust of the arch being borne upon the piers, it is taken up by the bowstring or chord *A d B*, to which the roadway is usually attached, the whole being connected with the arch by the vertical suspension-rods *C d*, *e f*, &c. The diagonal bracing is interposed to give increased steadiness, and hinder any slight distortion of the arch which might occur.

It is important thoroughly to understand the duties of the different elements of the bowstring girder, so that it may in no way be confounded with a trussed or lattice girder. In point of fact it is not, strictly speaking, a girder at all, but a *tied arch;* that is, an arch of which the abutments are united by a tie, instead of being otherwise enabled to resist the thrust thrown upon them by the arch.

The load comes upon the arch through the suspension-rods, at the lower ends of which, *f*, *h*, *d*, the weights are suspended, and these rods have no other duty to perform. Their lower ends are kept in position by attachments connecting them with the chord. The arch may be regarded and treated as an arch uniformly loaded. The chord or tie *A B* is in tension, and the amount of strain upon it is equal to that at the crown of the arch, and the tension on the chord is the same throughout. Hence it may be found thus:—

RULE. To find the tension on the chord of a bowstring or tied arch in tons, multiply the total load on the arch in tons by its span, and divide the product by eight times the rise or versine of the arch,—the quotient is the required strain in tons.

Example.—Let the total load be 140 tons, the span of the arch 83 feet, and its rise or versine 7 feet:—

```
                      140  tons' total load,
                        83 feet span of arch,
                     ------
Rise of arch, 7        420
               8      1120
             ------------
              56 )11620( 207·5 tons' tension on chord.
                   112
                  -----
                     420
                     392
                    -----
                      280
                      280
                     =====
```

In two or three instances arches and chains have been combined together in the general form shown in Fig. 23, and in one case for so large a clear span as 445 feet. We allude to the late Mr. Brunel's magnificent bridge at Saltash, of which the two main spans are designed on this principle:—*A c B* is the arch, and *A d B* the chain, which are connected together at each extremity in such a way that the thrust of the arch equipoises and is equipoised by the pull of the chain.

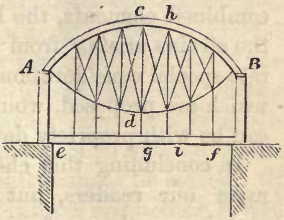


Fig. 23.

e f is the roadway, which is suspended from the arch and chain by the rods *c g*, *h i*, &c., so that the chain carries one half of the load and the arch the other, under which circumstances the thrust and pull at the piers *A B* will be equal.

The suspension-rods have no duty to perform beyond transmitting the load on the roadway to the sustaining members, and the diagonal bracing between the suspension-rods tends to check vibration.

In calculating the strains on these combined elements it is only necessary to assign one half of the load to the arch and the other half to the chain, and proceed as usual by means of the RULES (17) and (18). Between the suspension-rods the roadway is carried by any convenient description of light, straight girder. This mode of construction may at first sight appear heavy and complicated, but in reality it is not so, but, on the contrary, forms a light and elegant structure.

Amongst other buildings in metal which the engineer is called upon to design are included lighthouses. These may be conveniently formed of columns of iron braced together laterally and diagonally to enable them to resist the force of the winds and waves; but in the present chapter they need no notice, as sufficient information has already been given to show how, in any case of combined elements, the loads may be apportioned and the strains arising from them calculated, and to enter into special consideration of the great variety of systems which are proposed, would occupy far more space than can be with propriety devoted to this subject.

In concluding this chapter we would strongly urge upon our readers, but more especially upon those entering on the profession, to thoroughly master those rules which have been set forth, and upon which our future calculations are based, for in the comprehension of the principles from which they are derived consists the knowledge of the fundamental doctrines upon which that branch of engineering to which the present volume is devoted depends, and these doctrines, once understood, will not readily be forgotten

CHAPTER IV.

JOINTS AND CONNECTIONS.

UNDER the head of "Joints and Connections" are included an almost innumerable variety of methods of uniting the various elements of which structures are built up; and it not unfrequently occurs that the greatest amount of study and care devoted to the designing of any work must be expended in determining the best arrangement of the joints, and from the mode adopted the practical ability of the designer may generally be estimated.

It is comparatively easy to learn to calculate the right dimensions of the various elements of any combination; but to unite them in such a manner that, whilst the strength is not impaired, the appearance shall not be injured, requires a certain amount of that kind of knowledge known as "tact," and without it one can scarcely expect to become a neat designer. Many things may be copied from previous examples, or modified from them in such a manner as to suit an emergency; but joints most frequently cannot be copied. Of course we are not speaking of simple rivetted or bolted joints, by means of which two or three thicknesses of plates or bars are held together; we refer to such more complex joints as occur in some kinds of lattice girders, and also in the connection of a number of girders with one column, and others of a similar nature.

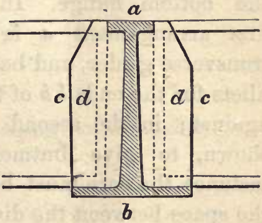
We cannot well illustrate the importance of correctly arranging the joints of girders used in sustaining buildings more strikingly than by reference to a catastrophe which occurred on the 6th of December, 1869, in the dining-hall of King's College, London. The dining-hall was 70 feet long by 26 feet wide, and its floor and roof were supported by cast-iron girders put down about thirty-five years since. Lengthwise of the apartment three pairs of piers, 3 feet deep and 1 foot 9 inches wide, supported three cast-iron cross girders. Longitudinal girders rested upon these and upon the end walls, so that the roof was divided into sixteen bays of brick arches, carrying superstrata of tiles, concrete, and earth covered with turf, the whole being about 2 feet in thickness. Each cross girder had a top flange 3½ inches wide by 2 inches deep (a not only useless but positively destructive addition, but of the general design we shall not speak, as here we are treating of joints only), an upright web, and a bottom flange; the longitudinal girders had bottom flanges and upright webs. Where, however, the longitudinal girders rested on the transverse girders—where the greatest amount of strength was requisite—the top flange was omitted, evidently for the purpose of more conveniently dropping in the ends of the longitudinal girders; hence, whatever strength would have been given by the upper flange otherwise was by this arrangement lost, so that the whole weight of that top flange was an additional load upon the web and lower flange of the girder. A pair of vertical snugs were also in each case cast at this point, inclining inwards so as to form a recess to receive the butt-end of the longitudinal girder. The effect of such snugs on the castings would be to weaken them still further by affecting them in the cooling in the weakest part. It may be argued in reply to these remarks that the girders would not have stood thirty-five years had they not had sufficient strength for the end to which they were designed; but, be it remarked, so soon as an extra strain was put upon these girders by the settlement of some of the masonry, they immediately gave way, breaking at the joint in each case,—that is, fracturing across the recess or pocket,—thus showing that throughout the girders the joints were the weakest places; and the maxim to be followed and made a rule, never to be swerved from, is:—

In any structure the joints should be at least as strong as the solid parts of the work, and if a little stronger so much the better, as they are liable to special defects, such as arise from unequal bearing, &c.

It now devolves upon us to show how these joints should be formed; but in the first instance we would make a few remarks as to the proper shape of the girders themselves.

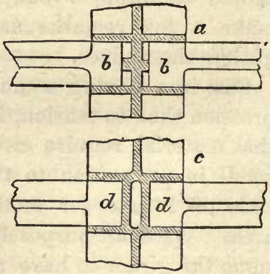
Cast-iron having a much greater resistance to compression than to tension, it follows that girders made of that material require either no top flange or else one small in proportion to the bottom flange, say in the ratio of 1 to 5 for sectional area. We shall purposely assume the girder to have a top flange, for otherwise the matter would be simple enough, as abolishing the whole top flange would leave the joints in the same state as in the above case, excepting as far as the recesses are concerned; hence, the cross girders will be assumed of the section shown in Fig. 24, though we do not in this place enter upon details as to the section required to sustain any particular load. *a* shows the

Fig. 24.



top and *b* the bottom flange, the web being between them and connecting them together; *c*, *c*, are the ends of the longitudinal girders as seen *in situ*. The sides *d*, laterally holding the longitudinal girders, should be of the same thickness as the web, so as to cool in the same time and so avoid straining from unequal contraction in cooling when cast. It is, however, necessary for steadiness that there should be something for the ends of the longitudinal girders to butt against, and it would *not* be proper to make the thickness of the web between the cheeks *d* equal to the breadth of the top flange, as then the web at that place would, when cast, be slower in cooling; but the matter might be arranged in either of the ways shown in Fig. 25. Both views show the transverse girder in horizontal section and the longitudinal girders in plan. It will be observed that in the longitudinal girders the top web is expanded at the ends so as to be equal there in width to the bottom flange. In the first arrangement *a* is the transverse girder, and between the cheeks are cast small fillets for the ends *b b* of the longitudinal girders to butt against; in the second the web divides into two, as shown, to give butments for *d d*. In the latter instance the core must be left in the cavity formed, as the space between the divisions of the web must not be carried through either flange. In both instances it will be observed the integrity of the top flange is not interfered with, hence the transverse girder maintains its strength unimpaired all through its length.

Fig. 25.



Having mentioned the above disaster and pointed out its cause as an evidence of the importance of care being taken in the proportioning of joints, we will return to the more systematic treatment of our subject. We will, in the first place, consider the question as relates to rivets and bolts.

The strains on bolts or rivets are of two sorts, tensile and shearing, sometimes shearing strain only being upon them, and sometimes both strains. If there is a tensile pull in the direction of the length of the bolt or rivet, its sectional area must be proportioned to bear such pull according to the quality of the metal of which it is made (commonly five tons per square inch is allowed as a safe load), and if the tendency of the strain is to shear the bolts or rivets asunder the areas must be similary proportioned (four tons per square inch may be allowed as safe for shearing strain).

Rule. To find the sectional area of a rivet or bolt in square inches, multiply the square of its diameter by 0·785.

Example.—What is the area of a rivet ¾ inch in diameter?

0·75 inch in diameter,
0·75 " "
375
525
5·625 square of diameter,
0·785
28125
45000
39375
·441·5625 square inch area of rivet or bolt (say 0·44).

Let it be required to determine the number of ¾-inch bolts required to sustain a force of twenty-seven tons, safe load being five tons per inch tension :—

```
0·44  square inch area of bolt,
   5  tons per inch,
────
2·20  strength of one bolt.
════
   Tons' load.
2·2) 27·0 (12·2, that is, 13 bolts
     22
     ──
      50
      44
      ──
       60
       44
       ──
       16
       ══
```

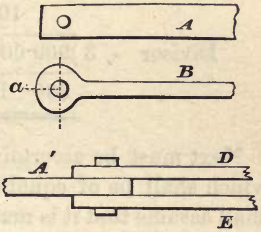
How many ¾ rivets will be required to resist a shearing strain of fifty tons at a strain of four tons per square inch area?

```
0·44  square inch area of rivets,
   4
────
1·76  strength of one rivet.
════
   Tons' load.
1·76 )50·00( 28·4, that is, 29 rivets.
      352
      ────
      1480
      1408
      ────
        720
        704
        ───
         16
         ══
```

The heads of bolts should never be of a thickness less than 5-16 of their diameters, nor should the nuts be less than double this, or $\frac{5}{8}$ of their diameters.

Let *A*, Fig. 26, represent one end of a bar forming part of the chain of a suspension bridge, or of the bottom member of a lattice girder, and in the end let there be a hole through which to pass a bolt, in order to connect it with the other bars. It is very evident that the bolt-hole weakens this bar by as much as it reduces its sectional area, which, supposing it to be of uniform thickness, will be reduced in the same ratio as the width of the bar.

Fig 26.



We will take an example, in order to make the matter clear, the bar being supposed to be, when in position between two others, as shown in plan *A′*, being the bar with which we are dealing, which is held between two other bars, *D* and *E*. Let the bar *A* be 12 feet long from end to end (when not otherwise stated the lengths are taken from centre to centre of the bolt-holes), 10 inches in width, and $\frac{3}{4}$-inch in thickness First,—let us ascertain the weight of this bar. A piece of wrought-iron of 1 inch sectional area, 1 foot in length, weighs $3\frac{1}{3}$ lbs., hence to find the weight of a bar of iron we have the following rule :—

RULE. To find the weight of a rectangular bar of iron, multiply its length in feet by its breadth in inches, and by its thickness in inches, and by 10, and divide the product by 3. The quotient will be the weight of the bar in pounds.

The weight of the above bar is thus found:—

12 feet length of bar,
10 inches breadth of bar,

120
·75 inch thickness of bar,

600
840

90·00
10 multiplier,

Divisor - 3)900·00

300· lbs. weight of bar.

Next must be ascertained the size of the pin or bolt which shall be of equal strength with the bar, and we shall assume that it is made of material which will safely carry a shearing strain of 5 tons to the square inch of sectional area, hence as the iron of the bar is assumed to be capable of sustaining the same strain per square inch in tension, the area to be sheared in fracture must not be less than the sectional area of the bar itself; the latter is

10 inches breadth of bar,
·75 inch thickness of bar,

7·50 square inches sectional area.

A glance at the plan *A′ D E* shows that for the bar *A′* to be pulled away from the two bars *D* and *E* the pin or bolt connecting them must be sheared through in two sections, hence the sectional area of the pin must be not less than one half the sectional area of the bar, and as the latter is 7·5 square inches the former will be 3·75 square inches. To find the diameter corresponding to this area we have this rule :—

RULE. To find the diameter in inches of a bolt to give any area in square inches, divide such area by 0.785, and extract the square root of the quotient.

```
·785 )3,750( 4·7770, &c.
      3140
      ----
       6100
       5495
       ----
        6050
        5495
        ----
         5550
         5495
         ----
          550
```

We have now to extract the square root of the quotient, 4·7770, which is carried to a sufficient number of decimal points for our present purpose.

```
   2 )4·7770( 2·18 inches, (say) 2¼ inches
      4                       [diameter of bolt.
     ----
  41) 77
      41
     ----
 428 )3670
      3424
      ----
       246
```

The bolt-hole then would be $2\frac{1}{4}$ inches in diameter, hence the width of the bar *A* would require to be increased by $2\frac{1}{4}$ inches in order that its full strength may be retained; the weight of the bar will of course be increased in proportion, that is, in the ratio of $2\frac{1}{4}$ to 10; let us see how much per cent. this will add to the weight of the chain:—

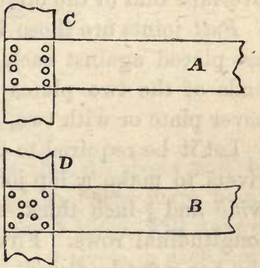
$$\begin{array}{r} 10 : 2{\cdot}25 :: 100 \\ 100 \\ \hline 10\,)\,225{\cdot}00 \\ \hline 22{\cdot}5 \text{ per cent.} \end{array}$$

This loss of metal in the chains is not, however, the only one which occurs through this form of bar, for the chains being 22·5 per cent. heavier than they need be, necessitates the giving of increased strength to some other parts of the structure which have to sustain the weight of the chains.

In order to avoid this loss, a form of link swelled at the end, as shown at *B*, has been adopted, and should in every case be applied when the weight of the links is at all considerable. The breadth of the link, measured across the eye in the direction of the dotted line, should be slightly in excess of the breadth of the body of the bar, added to the diameter of the bolt-hole. The amount of metal beyond the bolt-hole at *a* must be sufficient to afford area enough to resist the shearing strain caused by the tendency of the bolt to push out the piece of metal beyond it; therefore, measured from the edge of the hole along the dotted line to *a*, the distance should not be less than one and a half times the diameter of the bolt. Wherever it is necessary to weld links the greatest care must be taken that the weld is sound, as an internal flaw does not admit of easy detection when the work is finished, for although a method has recently been discovered of detecting defects by means of a magnetic needle, yet, as far as we can ascertain, it has not yet found its way into the workshop.

Fig. 27 represents two methods of attaching an upright bar or standard to a horizontal flange. *A* and *B* are the upright bars, or rather the extremities of them, and *C*, *D*, represent the horizontal flanges. Both the upright and horizontal bars are supposed to be in tension, hence it is important not to weaken the section of either by rivet holes more than is absolutely necessary. Let the number of rivets required be eight, then in the arrangement shown by *A* and *C* it will be observed that no section of *C* has more than two rivet holes in it, but two sections of *A*, taken on the line of rivets, have each four rivet holes in them. If, however, the arrangement shown at *B* and *D* be adopted, neither of the bars will have more than two rivet holes in any one section. But care must be taken that any section taken zig-zag across the bar from centre to centre of rivet holes, less the rivet holes through which it passes, shall not be smaller than the cross section of the bar less two rivet holes, because as a matter of course the bar will break in its weakest part; and if the effective zig-zag section be less than the effective transverse section, the former will be the measure of the strength of the bar.

Fig. 27.



It is a common practice in setting out girder work to make the total sectional area of the rivets equal to the sectional area of the plates when the strain actually comes upon the rivets, as it does in all joints under tension, and in some under compression, we shall therefore take a few examples by way of illustration.

As rivets $\frac{3}{4}$ inch in diameter are very generally used for light and moderately heavy girders, we shall in each case assume that as the adopted size.

Lap joints are those in which one end of one plate overlaps that of the other simply.

Butt joints are those in which the ends of the plates are placed against each other (or butted together), the ends of the two plates being covered either with one cover plate or with two, one being on each side.

Let it be required to determine the number of $\frac{3}{4}$-inch rivets to make a lap joint between two bars 11 inches wide and $\frac{3}{4}$-inch thick, the rivets being placed in two longitudinal rows. First, the *effective* sectional area of the bar must be found.

RULE. To find the effective sectional area of a bar on any line of rivets under tension, multiply the diameter of the rivets in inches by the number of rivets in the section; deduct the product from the width of the bar in inches, multiply the remainder by the thickness of the bar in inches, and the product will be the effective area of the bar in square inches.

Applying this to the above case we have—

·75 inch diameter of rivet,
2 number of rivet,
1·50 . . . . (*a*)

11·00 inches width of bar,
1.50 . . . . (*a*)
9·50
·75 inches thickness of bar,
4750
6650
7·1250 square inches effective area of bar.

And to this effective area of the bar the sum of the rivets' areas must be made equal, the number of rivets will be found by dividing the above area by the area of one rivet. The sectional area of a $\frac{3}{4}$-inch rivet is ·44 square inches

$$
\begin{array}{r}
\cdot 44\,)\,7\cdot 125\,(\,16 \\
44 \\
\hline
272 \\
264 \\
\hline
85
\end{array}
$$

Thus we see somewhat more than 16 rivets will be required; hence, as the rivets go in pairs, there being two rows of them, not less than 18 rivets will answer the required purpose, that will be 9 rivets in each row; so that if the rivets have a "pitch" or distance apart from centre to centre of 3 inches, and at each end of the joint a lap or distance from the last rivet centre to the end of the plate of $1\frac{1}{2}$ inches be allowed, the total length of the joint will be—

9 rivets in length,
3 inches' pitch,

27 inches length of joint.

For another example of a lap joint let the plates be 30 inches wide and $1\frac{1}{8}$ inch thick (in one or two thicknesses), let the number of rows of rivets be six, and the pitch of rivets 4 inches, then we have—

·75 inch diameter of rivet,
6 number of rivets,

4·50 . . . (*a*)

$$
\begin{array}{rl}
30\cdot0 & \text{inches width of bar,} \\
4\cdot5 & \quad . \quad . \quad . \quad (a) \\
\hline
25\cdot5 & \\
1\cdot128 & \\
\hline
2040 & \\
510 & \\
255 & \\
255 & \\
\hline
28\ 7640 & \text{square inches effective sectional} \\
\hline\hline
 & \quad \text{area of plate.}
\end{array}
$$

To find the number of rivets requisite we have—

$$
\begin{array}{r}
\cdot44\ )\ 28\cdot764\ (\ 65 \\
264 \\
\hline
236 \\
220 \\
\hline
164 \\
\hline
\end{array}
$$

Thus rather more than 65 rivets, that is to say, 66 rivets, will be required to make the joint, and as there are six rows of rivets, this will give 11 rivets in each row. The pitch of the rivets being 4 inches, the length of the joint (allowing at each end half a pitch for lap) will be 44 inches, or 3 feet 8 inches.

If, instead of being a lap joint, the plates were butted with *one* cover plate, then it is evident that the strain has to pass first from one main plate on to the cover plate, and then from the cover plate on to the other main plate, thus virtually making two joints of it, hence, in this case, twice the number of rivets will be required. If, however, two cover plates be used, one on each side of the main plates, then each rivet presents two sections to be sheared before failure can take place;

hence, a butt joint, in tension with two cover plates, requires the same number of rivets to hold it as a single lap joint, the dimensions and other particulars being the same.

In butt joints in compression, if truly made, there should be no shearing strain on the rivets, as the pressure passes from the end of one plate to that of the one against which it is butted; hence, the only duty the rivets have to perform is to hold the ends of the plates opposite to each other. In this case it is evident that very short cover plates will suffice for butt joints in compression, nor is any deduction to be made for the loss by rivet holes, for the rivets filling the holes the thrust or compressive strain will be transmitted through the bodies of the rivets the same as through the solid plate.

In determing the length of rivets required for any joint two matters have to be taken into consideration, the thickness of metal through which the rivets have to pass, and the amount of length of rivet necessary to make that head which is completed in the act of rivetting up the joint. The first is the sum of all the thicknesses of metal through which the rivet has to pass, the second is generally provided for by allowing an extra length, equal to one and a half times the diameter of the rivet. The length of the rivet is measured from under the head to the point.

If it is required, therefore, to determine the length of $\frac{3}{4}$-inch rivets necessary to unite three thicknesses of metal—being respectively $\frac{1}{2}$-inch, $\frac{5}{8}$-inch, and $\frac{3}{4}$-inch, the sum of these three thicknesses will be $1\frac{7}{8}$ inches, and one and a half times the diameter, $\frac{3}{4}$-inch, will be $1\frac{1}{8}$ inch; hence, the total length of the rivet will be $1\frac{7}{8}$ inches added to $1\frac{1}{8}$ inches, making altogether a length of 3 inches.

This is applicable when the number of plates to be joined together is but small, but when a larger number are to be connected an allowance must be made for their not lying perfectly flat and close together, the following will then be a safe rule:—

RULE. To find the length of rivets in inches, measured from under the head to the point, to hold together a number of plates of iron, add together the thicknesses in inches of all the plates, to this add 1-32 part of the number of the plates and one-and-a-half times the diameter of the rivet in inches.

Example.—Let it be required to find the length of rivets 1 inch in diameter necessary to hold together six wrought-iron plates, of which three are $\frac{3}{4}$-inch and three $\frac{5}{8}$-inch in thickness :—

$$\begin{array}{rl} \cdot 75 & \text{inch thickness,} \\ 3 & \\ \hline 2 \cdot 25 & \text{inch thickness of three } \tfrac{3}{4}\text{-inch plates,} \end{array}$$

$$\begin{array}{rl} \cdot 625 & \text{inch thickness,} \\ 3 & \\ \hline 1 \cdot 875 & \text{inch thickness of three } \tfrac{5}{8}\text{-in. plates.} \end{array}$$

The number of plates is 6, hence we must find 1-32 part of 6.

$$\begin{array}{l} 32\ )\ 6 \cdot 00\ (\ 0 \cdot 1875 \quad . \quad . \quad (a) \\ \quad\ \ 32 \\ \quad\ \ \overline{280} \\ \quad\ \ 256 \\ \quad\ \ \overline{\ 240} \\ \quad\ \ \ 224 \\ \quad\ \ \ \overline{\ 160} \\ \quad\ \ \ \ 160 \end{array}$$

And, finally, one and a half times the diameter of rivet (1 inch is 1·5) inch. Adding all these quantities together we arrive at the required length of the rivets :—

2·25 inches' thickness of three $\frac{3}{4}$-inch plates,
1·875 inches' thickness of three $\frac{5}{8}$-inch plates,
·1875 . . . . . (*a*)
1·5000 inches' length of $1\frac{1}{2}$ diameters,

5·8125 inches' total length of rivets.

Practically, this length would be taken as $5\frac{7}{8}$ inches, decimal measurement only being used in calculation.

Where cover plates are used to joints they must, of course, be equally strong with the main plates, as they have to convey the whole strain from one plate to another; hence, assuming the widths of the cover plates to be the same as that of the main plates—

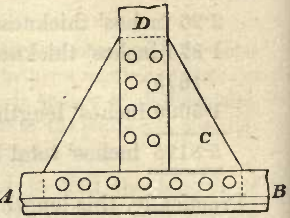
For a single cover the thickness must equal thickness of main plate.

For two covers the thickness of each must equal half the thickness of main plate.

We must now pass on to speak of *joint plates.* These are plates introduced to unite elements where they cannot conveniently be united directly—that is, one to the other; in fact, joint plates serve as cover plates, but are very frequently of irregular forms, to suit the nature of the work for which they are designed.

In Fig. 28 is shown a joint plate for enabling a standard or upright to be joined to the flange of a girder. *A B* is the horizontal member or flange, *D* the standard, and *C* the joint plate. The joint plate, which is wide at the bottom, is rivetted on to the angle iron, or rather it should be between the two angle irons of the flange, and *D* the upright is rivetted, as shown, on to the upper part of the joint plate, which tapers down in width until it is at its top the same width as the piece *D*; if the latter consists of two bars one will be placed on each side of the joint plate, and all three will then be rivetted together in the manner shown.

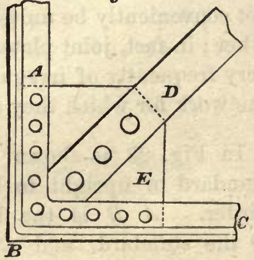
Fig. 28.



In this case, if the element *D* had been rivetted direct on to the angle iron without the intervention of the joint plate, it would, in order to get in the necessary number of rivets, have required them to be placed so close together as very seriously to weaken both the upright and the flange; but as the joint plate may be made of any height and width desirable, a sufficient number of rivets may be brought into action without the slightest amount of crowding.

In Fig. 29 is shown a joint plate, which at the same time acts as a gusset plate. (A gusset is a plate inserted at an angle in order to maintain the proper relative inclination of any two parts of a structure.) *A B C* shows a part of the corner of girder and *D* a diagonal bar to be attached to it; *E* is the joint plate. The height and width of this plate allow of a sufficient number of rivets being brought into play to hold it to the bottom flange and end

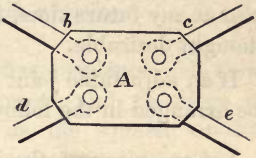
Fig. 29.



of the girder, and its diagonal measurement affords room for the insertion of the number of rivets requisite to the proper attachment of the diagonal bar *D*. In this case the plate, in its capacity as a gusset plate, keeps the end *A B* of the girder to which it is supposed to belong at right angles to the bottom flange *B C*, but of course plates of this description may be cut to any angle according to the requirements of the case.

Fig. 30 shows a form of joint plate not unfrequently seen in roof-work and in the larger class of lattice girders used in buildings of magnitude. *A* is the joint plate, and its duty is to connect the four diagonal ties, *b*, *c*, *d*, *e*, which are assumed to come from the four corners of some bay which requires bracing. The plates are two in number, the swelled ends of the tie bars being placed between them and there secured by bolts or rivets.

Fig. 30.



Pairs of plates of this description are made in a great variety of forms to meet the requirements of light trussed work, where the ties and struts are so small as not to allow of rivet-holes being punched in them conveniently, and also where a number of ties lying in the same plane meet.

It may also here be observed that where plates of this description have to be employed, by care in designing them they may be caused to add very materially to the general appearance of the work.

In some instances wrought-iron tie-rods have been joined in the centre or some other part of the length, as the case might be, by means of coupling boxes. The two extremities of the bars to be joined have screws cut upon them, but the threads are cut in opposite directions, that is to say, on one end a right-hand screw is cut, but on the other a left-hand screw, in the same manner as the railway couplings are made. A box, or long nut, is made to fit these screws, having an internal right-hand thread at one end, and an internal left-hand thread at the other; when this box or nut is adjusted to the two ends to be brought together, it is evident that by turning it in one direction or the other the ends of the tie-bars are caused to approach, or recede from, each other, and thus the required length of the whole tie, which is made up of the two bars, admits of being adjusted with the greatest nicety, not only at the time of its erection, but also at any future time, if from any cause it should be thought desirable.

If an adjustable joint be required for a strut, it can be arranged in the following manner:—

Let one piece of the strut have an ordinary screw-thread cut upon its extremity, being that extremity which meets the other piece of the strut. The latter part is to be tubular at its end, so as to admit the former, and the tubular part must be sufficiently long to give a steady good hold, and of such a diameter that the screwed portion will just slide in or out, but without play. The strut being put together, it can be screwed out to the required length when in place, by turning the nut which bears upon the end of the tubular part, and thus affords a means of adjustment whenever it may be required.

In connecting different cast-iron elements generally, rivets are out of the question, as, from the very unyielding nature of that metal, it would be constantly being fractured, either in the rivetting or by the subsequent contraction of the rivets in cooling, hence bolts and nuts are invariably used.

It is held by many mechanical engineers that a well-made bolt, one inch in diameter, should safely carry a load of five tons, and this may be safe where no violent jars or shocks are likely to come upon it; but in girder work the safe tensile strength of iron is assumed to be five tons per sectional square inch, and the sectional area of a bolt, one inch in diameter, is 0·785 square inch. Hence the safe strength of such a bolt is thus found:—

·785 square inch sectional area,
5 tons per square inch,
3·925 tons strength of bolt.

We may, however, with perfect prudence, call this 4 tons; then, as the sectional areas and therefore the strengths of the bolts vary as the squares of their diameters, we have the following simple rules:—

Rule (25). To find the safe load on any given wrought-iron bolt, multiply the square of the diameter of the bolt by 4: the product is the load in tons.

Example.—What is the safe load on a bolt $\frac{5}{8}$ inch in diameter?

·625 inch diameter,
·625 " "
3125
1250
3750
·390625 square of diameter,
4
1·562500 tons working strength of $\frac{5}{8}$ in. bolt

The next rule serves to find the number of bolts requisite to support a given load.

Rule (26). To find the number of bolts of a given diameter to support a given load in tons, divide the given load by four times the square of the diameter of the bolts in inches,—the quotient will be the number of bolts required.

Example.—Let it be required to be determined how many bolts $\frac{3}{4}$ inch in diameter will be necessary to carry safely a load of 43 tons:—

```
  ·75 inch diameter of bolts,
  ·75   "       "       "
  ---
  375
 525
 ----
 ·5625 square   "
     4
 ------
 2·2500
 ======

        Load in tons.
   2·25 )43·00( 19
          225
          ----
          2050
          2025
          ----
            25
```

Practically this will be 20 bolts required.

The following rule serves to determine the diameter of the bolts when the load and number of bolts are given.

Rule (27). To find the diameter of bolts in inches, the load and number of bolts being given, divide the load in tons by four times the number of bolts given, and the square root of the quotient will be the required diameter of bolts in inches.

Example.—Let the load to be sustained be 25 tons, and the number of bolts admissible 14, then applying the rule, we find,

```
Number of bolts, 14
                  4
                 ——    Tons.
                 56 )25·0( 0·446, &c.
                     224
                     ———
                      260
                      224
                      ———
                       360
                       336
                       ———
                        24
                        ══
```

We must extract the square root of 0·446.

```
          ·6 )0·4460( 0·66 diameter of bolts
               ·36
              ————
         126 ) 860
               756
               ———
               104
               ═══
```

Practically, these bolts would be 9-16 inch in diameter.

The different elements of cast-iron structures, which, from being at an angle, cannot be bolted directly together, are joined through the medium of brackets or dogs, either cast on to one of the pieces, or else bolted to both of them. Lugs, or ears, also cast on to the sides of certain elements, are used as a means for bolting them together. As a general rule, wherever a bolt passes through a piece of cast-iron, at that place the metal should be made thicker than in the general body, and that for a distance round the hole about as far as the nut or head of the bolt will extend.

In conclusion, it may be observed that although bad joints of all descriptions will sometimes make their presence in a structure evident the first time it is tested, yet such is not always the case, and, in fact, they must be bad indeed to yield at the first strain; and in general it is after the structure has for a long time undergone continuous strain and frequent vibration that the joints find their real bearings. Sometimes the actual resistance of rivets to shearing will not be called into play at all, because their contraction in cooling has pressed the plates together with so great a force that the friction of their surfaces of contact is alone sufficient to prevent them from sliding one upon the other.

CHAPTER V.

GIRDERS AND COLUMNS FOR BUILDINGS.

The application of iron columns and girders to the construction of warehouses and other buildings is now so rapidly extending that the circumstances of its adaptation to such purposes require the most careful consideration on the part of builders and engineers.

In ordinary warehouses and private buildings the loads on the girders will generally be of an uniformly distributed character, being due to the weight of walls, merchandise, and inhabitants. Of course in each case the weight must be taken at a maximum. A few examples of modes of determining the loads on warehouse girders, &c., will serve to illustrate this portion of our subject.

Let it be required to construct a wrought-iron flanged girder of the common 1 section, to carry a wall 25 feet high and 2 bricks thick across an opening or gateway 20 feet in the clear span or distance between the points of support.

To find the weight of the wall we have the following rule :—

Rule. To find the weight of a brick wall in pounds, multiply its height in feet by its length in feet, by its thickness in bricks (each brick is 9 inches) and by 75. The product will be the weight of the wall in pounds.

Applying this to the above case, we have

25 feet height of wall,
20 feet in length car. by girder

500
2 bricks thick,

1000
75

75,000 lbs. load on girder.

To reduce this to tons we must divide by 2240, being the number of pounds in a ton :—

2240)75000(33·48 tons.
6720

7800
6720

10800
8960

18400
17920

480

This will in practice be taken as 34 tons' load distributed over the whole length of the girder.

From this load the size of the girder may be determined when its general proportions have been arranged.

For flanged plate girders, about the most economical proportion of span to the depth is as 12 to 1, but we cannot always get even so much depth as this would indicate on account of limitations as to space or headway; hence, in many instances, girders for buildings have to be made with unusually heavy flanges; in the present example, however, we shall assume that the desired depth of girder may be attained; the span being 20 feet, the depth will be

$$\begin{array}{l} 12)20 \\ \hline \quad 1{\cdot}667 \text{ feet depth of girder.} \end{array}$$

Then, to find the strain on either flange, we have

RULE.—To find the strain on either flange of a girder supported at both ends and uniformly loaded, multiply the load in tons by the clear span in feet, and divide the product by eight times the depth of the girder in feet; the quotient will be the strain at the centre on either flange in tons.

$$\begin{array}{rl}
\text{Depth of girder - } 1{\cdot}667 & 34 \text{ tons' load,} \\
8 & 20 \text{ feet span,} \\
\hline
13{\cdot}336 &)680{\cdot}000(50{\cdot}9 \text{ tons' strain.} \\
& 66680 \\
\hline
& \quad 132000 \\
& \quad 120024 \\
\hline
& \quad\;\; 11976
\end{array}$$

Practically, the sectional area for such girders is so proportioned as to allow a safe strain of 4 tons per square inch both in tension and compression, no allowance being made for loss by rivet holes. This, in fact, amounts to the same as allowing 5 tons for tension per square inch of nett effective area and 4 tons per square inch in compression; for it will generally be found that the effective sectional area of the bottom flange is about four-fifths of the gross sectional area; hence if the top and bottom flanges be made of the same gross sectional area the proper proportions will be obtained. In the example selected the sectional area of either flange at the centre will be found thus:—

$$\begin{array}{l} 4\,\underline{)50{\cdot}9}\ \text{tons' strain,} \\ \underline{\underline{12{\cdot}725}}\ \text{square inches.} \end{array}$$

The thickness of the wall being 2 bricks, or 18 inches, the breadth of the flanges of the girder should not be less than 15 inches,—and the thickness may be taken at half an inch, the flanges to be attached to the web by angle-irons to be 3 inches by 3 inches and half an inch thick. To find the sectional area of an angle-iron, the following rule may be used:—

Rule. To find the sectional area in square inches of an angle-iron add together the lengths of its two limbs or sides in inches, from the sum subtract the thickness of metal in inches, and multiply the remainder by the thickness in inches; the product will be the sectional area in square inches.

The two sides of each angle-iron each measure three inches, the thickness of metal is half an inch, hence the area is formed thus:—

$$\begin{array}{ll} 3 & \text{inches on side,} \\ \underline{3} & \quad\text{do.}\quad\text{do.} \\ 6{\cdot}0 & \\ \underline{\ {\cdot}5} & \text{inch thick,} \\ 5{\cdot}5 & \\ \underline{\ {\cdot}5} & \text{inch thick,} \\ \underline{\underline{2{\cdot}75}} & \text{inch sectional area of each angle-iron.} \end{array}$$

There are two angle-irons to each flange, hence the gross sectional area of each flange will be—

Flange plate, 15″ by ½″ . . .	7·5	square inches,
Two angle-irons, 3″ by 3″ by ½″	{2·75	do.
	{2·75	do.
	13·00	square inches,

which gives a very slight excess over that required, 12·725 square inches.

This sectional area will be continued throughout the girder, as in those of small span it is not worth while varying the sections to accommodate them to the diminishing strain towards the points of support.

The rule for determining the sectional area of the flanges at the centre becomes much simplified if we assume that the ratio of depth to span of one-twelfth is adhered to, thus—

Rule. To find the sectional area of either flange at centre, multiply the total load on the girder in tons by three, and divide the product by eight. Or, multiply the total load on the girder by 0·375.

Thus, in the above case—

$$\begin{array}{r} 34 \text{ tons' load.} \\ \cdot 375 \\ \hline 170 \\ 238 \\ 102 \\ \hline 12\cdot 750 \end{array}$$

The slight deficiency in the area previously obtained is due to the omission of decimals in calculating the strain, which is in reality somewhat more than 50·9 tons on each flange.

If the flanges be made of plates 10 feet in length there will be one cover plate required on each flange; the length of it must be determined. Let the girder be put together with rivets three-quarters of an inch in diameter pitched three inches apart from centre to centre. The joints are to be butt joints, according to invariable practice. As we have previously shown, there is no necessity for having long cover plates over joints in the compression member, but yet in such small work as that under consideration it is usual to make the covers on both flanges alike; hence we shall take them to be so. Taking the nett area of the flange plate (there being two rows of rivets) we find—

7·50 sq. in. gross area
·75 do. loss by two rivet holes.

Area of ¾ rivet, 0·44) 6.75 (15·3
44

235
220

150
132

18

Hence we must use on each side of the joint 16 rivets, or 32 rivets altogether in the cover plate; that will be 16 rivets in each row; and as the pitch of the rivets is 3 inches, the length of cover plates is thus found:—

16 rivets
3-inch pitch,

48 inches.

Or, each cover plate must be 4 feet in length. Sometimes, in order to save metal in the cover plates, the rivets are pitched closer where they occur; thus in the present case, by pitching them 2 inches apart where the covers occur, the length saved on each cover plate would be 16 inches, but the arrangement would require 16 extra rivets in each flange, hence it becomes a question whether there be any saving or not. This is not an imaginary case, as in many instances when setting out girder work which has been taken at a very low price, we have been obliged to consider every trifling detail, and have at times had as many as four different pitches in one girder.

The amount of metal saved on each cover will be a piece 15 inches by 16 inches and half an inch thick, but one square foot of wrought-iron one quarter of an inch thick weighs 10 lbs., hence the weight of this piece will be—

1·25 feet,
1·33 feet,
375
375
125
1·6625
20 lbs. per sq. foot of $\frac{1}{2}''$ plate.
33·2500 lbs.

Taking iron at £9 10*s.* per ton the cost of 33 lbs. will be thus found :—

£ *s.*
9 10
20
190 shillings per ton,
33 lbs of iron,
570
570
lbs. per ton, 2240) 6270 (2·79
4480
17900
15680
22200
20160
2040

This shows a saving of 2/10 on each cover plate; the extra cost of rivets, taking them at 2*d*. each, will be,

$$\begin{array}{r} 10 \text{ rivets,} \\ 2d. \\ \hline 20 \text{ pence,} \\ \hline \end{array}$$

being an outlay of 1/8, hence the actual saving by putting the rivets closer together where the cover plates occur will be

$$\begin{array}{rr} s & d. \\ 2 & 10 \\ 1 & 8 \\ \hline 1 & 2 \\ \hline \end{array}$$

and as there are two cover plates to each girder, the saving per girder will be 3/2.

According to the rules already laid down the greatest shearing strain on the web is equal to half the total load, or 17 tons, and allowing 3 tons per sectional square inch as safe stress on the web, we find

$$\begin{array}{rl} 3\,)\,17 & \text{tons' load,} \\ \hline 5{\cdot}66 & \text{square inches.} \\ \hline \end{array}$$

The depth of the web is 20 inches less the thickness of the top and bottom flange plates, or

$$\begin{array}{rl} 20 & \text{inches total depth,} \\ 1 & \text{inch sum of thickness of flange plates.} \\ \hline 19 & \text{inches depth of web.} \\ \hline \end{array}$$

Let the web be $\frac{5}{16}$ of an inch in thickness, then

$$\begin{array}{rl} 19 & \text{inches deep.} \\ 5 & \\ \hline 16\,)\,95\,(& 5{\cdot}9 \text{ square inches.} \\ 80 & \\ \hline 150 & \\ 144 & \\ \hline 6 & \\ \hline \end{array}$$

Each end of the girder should be strengthened by an end plate of the same thickness as the flange plates, and there should also be three T irons on each side of the web to act as stiffeners; these should measure five inches on the back by three through the feather, the metal being half an inch thick. These T irons will also serve as covers to the joints in the webs. We can now proceed to calculate the weights of the girder under consideration :—

A bar of iron 1 square inch sectional area and 1 foot long weighs $3\frac{1}{3}$ lbs., and from this datum we can determine our quantities. In the first place we have plate 15 inches by $\frac{1}{2}$ inch; that is, 7·5 inches in area, the length used being

	Feet.	
Two flanges, each 20 feet .	40·0	
Two covers for each—ea. 4 feet	8·0	
Two end plates—ea. 1 ft. 8 inches	3·3	
	51·3	feet length,
	7·5	inch. area.
	2565	
	3591	
	384·75	
	3·34	lbs. per ft.
	153900	
	115425	
	115425	
	1285·0650	lbs. of plate 15″ by $\frac{1}{2}$″

The web plates amount to 20 feet of plate, having an area of 5·9 inches :—

```
    20    feet length,
     5·9  inches' area.
   ----
   180
  100
  -----
  118·0
    3·34  lbs. per foot.
  -----
  472
 354
354
-------
394·12  lbs. weight of web.
```

Of angle-iron running round the girder on both sides of the web, we have

	Feet
Top and bottom flanges	80·0
End plates . . .	6·6

```
  86·6  feet length,
   5·5  square inches' area
  ----      of angle-iron
  4330
 4330
 ------
 476·30
    3·34 lbs. per foot.
 ------
  19052
 14289
14289
--------
1590 842  lbs. weight of
              angle-iron.
```

The length of T-iron stiffeners (six, each 19 inches) will be 9·5 feet; the sectional area is thus found:—

5 inches' breadth,
3 „ depth,

8·0
·5 inches' thickness,

7·5
·5 inches' thickness,

3·75 square inches' area.

Hence the weight of the T irons will be—

9·5 feet length,
3·75 inches' area.

475
665
285

35·625
3·34 lbs. per foot.

142500
106875
106875

118·98750 lbs. weight of T iron

By adding all these weights together, and allowing 5 per cent. for rivet heads, we get the total weight of the girder, dropping the decimals—

	lbs.
Plates, 15 inch by $\frac{1}{2}$ inch . .	1285
Webs, 19 inches by $\frac{5}{16}$,, .	394
Angle-iron, 3 inch. by 3 inch. by $\frac{1}{2}$ inch	1591
Tee-irons, 5 inch. by 3 inch. by $\frac{1}{2}$ inch	119
	3389
5 per cent. for rivets .	169

```
2240 ) 3558 ( 1·59 tons
       2240      [nearly.
       13180
       11200
        19800
```

Assuming this work can be completed at £12 per ton, the cost per girder will be—

```
      1·59
        12
£    19·08
        20
s.    1·60
        12
d.     7·2
```

In all, £19 1/7. Let us now see what amount per cent. was saved on this by shortening the cover plates to four feet. The total cost is £19·08, the saving effected is, per girder, 2/4, which is equal to £0·116 nearly—

```
        ·116
         100
19·08 ) 11·600( 0·6 per cent.
        11448
          152
```

Hence this saving is hardly worth effecting.

Such girders as this, when put up, should be very carefully bedded, so as to avoid any lateral or twisting force upon them, and the rivets on the bottom flange at each end where the bearing is taken on the points of support, should be countersunk flush with the surface of the girder, so that the plate itself may take a bearing on the piers. Beams thus used to support walls are not usually liable to much vibration; but in some cases they are, as, for instance, when used in buildings which, being founded on sandy soil, are shaken by passing vehicles.

The method of designing the girder will be the same in all cases where the load is uniformly distributed, but the amount of such load must always, in the first instance, be very carefully determined.

If the load, or a portion of the load, consist of an assembly room, where there is liable to be a crowd, the greatest weight of people that the room can hold must be taken as the maximum load.

It may be safely assumed that six men will occupy one square yard of floor surface, each man on the average being supposed to weigh 160 pounds; hence, the live load which may come upon any floor per square yard is—

$$\begin{array}{r} 160 \\ 6 \\ \hline 960 \text{ lbs.} \end{array}$$

Hence, to find the total load on any given floor, we have the following rule:—

RULE. To find the maximum load that can come upon any floor due to persons standing on it, multiply the length in feet by the breadth in feet, and divide the product by 21, then the quotient will be the live load in tons.

Example.—Required the load in tons on a floor 50 feet by 75 feet:—

```
         75
         50
        ----
21 )3750( 179 tons (nearly),
    21
    ---
    165
    147
    ---
    180
```

Of course no general rule can be given for merchandise, the weight of which must be ascertained for each special case.

It does not always happen that the girders used in buildings are under a uniformly distributed load, as sometimes they are required to sustain a central load or some other concentrated load. The stress thus thrown upon the flanges may be determined by the rules already laid down in a previous chapter. The maximum strain in each case will be immediately under the load. Supposing the ratio of depth to span adopted to be as above taken, 1 to 12, then the rule for the area of flange of a wrought-iron girder for a central load will be:—

Rule. To find the sectional area of either flange in square inches, multiply the load in tons by 0·75 for a central load.

Example.—Required the sectional area of the flanges of a girder to support a load of 156 tons in the centre of its span.

```
  156 tons' load,
   ·75
  ----
   780
 1092
 ------
 117·00 sectional square inches.
```

If the load is not central, the following rule will serve :—

RULE. To find the sectional area of either flange under a concentrated load not in the centre of the girder, multiply the distance of one point of support in feet from the point of application of the load by the distance of the other support in feet from the load ; multiply the product by the amount of the load in tons and by 3, and divide by the square of the span in feet ;—the quotient will be the required sectional area in square inches.

Example.—Let it be required to find the sectional area of either flange of a girder 20 feet in span, which has to carry a load of 14 tons at a distance of 7 feet from one pier ; then the distance from the other pier will be 13 feet.

```
                           13  feet,
                            7   ,,
                           ---
                           91
                           14  tons' load,
                          ----
                          364
                          91
                          ----
                         1274
                            3
                         -----
Square of span, 4,00 )38,22
                         -----
                          9·555  square inches.
```

If, however, there be two equal loads placed equidistant from the centre of the span, then the maximum strains will exist at the two points under such loads, being equal ; and the sectional area of the flange may be found from this rule :—

RULE. To find the sectional area in square inches of either flange of a girder to carry two equal loads placed equidistant from the centre of the span, multiply one of the loads by its distance from the nearest point of support in feet, and divide the product by four times the depth of the girder in feet.

Example.—A girder is required to carry two loads of 11 tons each, so placed as to be each one 9 feet from the nearest pier or point of support, the depth of the girder being 2 feet :—

	Feet.		
Depth of girder,	2	11	tons' load,
	4	9	feet distance,
	8) 99		
		12·375	square inches.

It will here be noticed that so long as the distance of the loads from the piers remains unaltered, the span of the girder does not enter into the calculation of the area.

When the loads to be carried are very considerable, it is usual to apply girders having double webs, being like two I girders put together thus, II forming what is called a box girder. These also possess greater lateral rigidity than the single-webbed girders, and are specially suitable where wide flanges are required, in which case the flanges gain much in rigidity by the additional web.

In designing ordinary girders for building purposes, the stiffeners should be in proportion to the angle-irons, and placed on both sides of the web, if single, or one outside each web, if double, at distances of about five feet apart. If there is much vibration, the stiffeners must be put closer together, and where very heavy loads are to be sustained, one or two extra stiffeners should be introduced over the points of support. If the general angle-irons be 3 in. by 3 in. by ½ in., the tee-irons should be 5 in. by 3 in. by ½ in. ; if the angles are 4 in. by 4 in. by ⅝ in., the tee-iron should be 6 in. by 3½in. by ⅝ in.

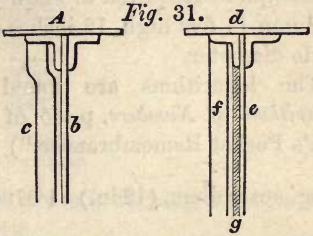
As a general rule, the metal of angle irons should never in thickness be less than $\frac{1}{8}$ of the length of the longest side.

Thus an angle-iron 4 inches by 3 inches should not be less than $\frac{1}{2}$-inch thick. A very serviceable-sized angle-iron for average work is that which measures $3\frac{1}{2}$ inch by $3\frac{1}{2}$ inch by $\frac{1}{2}$ inch thick. If angle-irons having unequal sides are used, the broadest side should be placed against the flange of the girder, as the more metal there is at the greatest distance from the centre of the girder the greater will be its strength. It is evidently desirable to have as few joints in a girder as possible; hence it is desirable to have the plates as long as may be without running to extra expense, for when plates get over a certain weight they require extra men to handle them, hence cost more per ton, both in production and in subsequent working. Ten feet is a very convenient general length for flange-plates, but if light they may sometimes be used 12 to 15 feet in length. Angle-irons up to 4 inch by 4 inch by $\frac{3}{4}$ inch may be easily obtained over 30 feet in length, hence in nearly all warehouse and similar girders the angle-irons can be run in one length from end to end of the girder, thus avoiding any joint in the angle-irons.

In applying stiffeners to a girder on the sides of the web, it is evident that their ends must be bent or joggled or else have a packing-piece under it, the latter plan being generally the most economical.

These two methods are shown in Fig. 31. In the first view the T-iron stiffener is bent. *A* is a plate of the main flange in section, *b* the web, and *c* the T-iron bent as shown at the top over one limb of the angle-iron, which unites the flange to the web-plates of the girder. In the second view *d* shows the flange-plate, *e* the web-plate, and *f* the T-iron stiffener, which is brought up to the level of the vertical limb of the angle-iron by having underneath it a packing-piece shown shaded at *g*.

Fig. 31.



The method to be adopted depends upon the length of the stiffeners principally, for which the cost of bending the ends is constant and independent of length; the weight and cost of packing-pieces vary directly as their lengths hence there is a point at which the cost of working the iron is equivalent to the cost of packing-pieces.

We must now pass from girders to treat of the columns by which they are frequently supported, and which are also commonly applied for other purposes where loads have to be sustained. Hollow cast-iron columns are most commonly used, and their strength was some years back determined experimentally by the late Professor Eaton Hodgkinson, who tried columns of Low moor iron and arrived at the following rule, which, however, can only be worked by the aid of a table of logarithms.

Rule. To find the breaking weight of a hollow cast-iron column in tons, multiply the *logarithm* of the outer diameter in inches by 3·6, and also multiply the *logarithm* of the inner diameter in inches by 3·6; find the natural numbers corresponding to the two *logarithms* thus obtained and subtract the latter from the former, multiply the remainder by 44. Find the *logarithm* of the length in feet, multiply it by 1·7, find the natural number corresponding to this logarithm, and by it divide the former product.

This rule applies to columns having a length of mor than thirty times the diameter.

Example.—Required to know the breaking weight of a column 14 feet high, 12 inches outside and 10·5 inches inside diameter.

(The logarithms are found from the Table of *Logarithms of Numbers*, p. 35 of the Author's "Engineer's Pocket Remembrancer.")

Log. outer diam. (12 in.)	1·079181	
	3·6	
	6475086	
	3237543	
	3·8850516	Number 7675
Log. inner diam. (10·5 in.)	1·021189	
	3·6	
	6127134	
	3063567	
	3·6762804	Number 4745
		2930
		44
		11720
		11720
		128920

This is the first product :—

Log. of length (14 feet)	1·146128	
	1·7	
	8022896	
	1146128	
	1·9484176	Number 88·8

88·8)128920·0(1452 tons nearly.
888
4012
3552
4600
4440
1600

Hence, allowing $\frac{1}{8}$ as the safe working strength of the column, we should have—

8)1452
181·5 tons' safe load.

CHAPTER VI.

IRON ROOFS.

FROM its strength, durability, and incombustible nature, iron is a material eminently suitable for the construction of roof principals and purlins for warehouses, railway-stations, factories and store-houses; and, moreover, it admits of being arranged in such forms as will occupy a minimum space. Roofs admit of being classified under four distinct heads:—

1st. Roofs supported by triangular trusses or principals.

2nd. Roofs supported by arched trusses or principals.

3rd. Roofs supported by straight girder trusses or principals.

4th. Roofs supported by dome-shaped framework.

The actual loads to which a roof may be subject consists of three elements, viz.:—

The weight of the main principals, &c.,
The weight of the covering used,
The weight of snow, ice, &c.

for any particular case the two first elements will be constant, but the last must be taken for maximum cases.

The load due to weather can of course only be determined from the nature of the climate in the locality for which the roof is required, by which also to a very great extent the material used for the covering will be determined, but conjointly with the purpose to which the structure is to be applied. Thus in some cases, such as for sheds in temperate climates, corrugated iron sheets, weighing 6 or 7 lbs. per square foot, may be used, whereas in other instances, as for exposed buildings in tropical climates, thick coverings of sand or concrete may be required to exclude the intense heat, and we have known of cases in which it has been stipulated that roof principals should be tested with a load of 100 lbs. per square foot of area. In such positions, as we are much exposed to wind, care must be taken to prevent the covering being blown off by the wind getting underneath.

Obviously in all cases of roofs the load may be taken as uniformly distributed over the whole area of such roof, hence if it be required to find the load on any one principal it may be done by means of the following rules:—

Rule. To find the total load on any main principal in pounds, multiply the total load per square foot by the span of the principal in feet and by the distance between two contiguous principals in feet, the product will be the total distributed load in pounds on each principal.

Example.—Let it be required to find the total load in pounds on a principal in a roof 50 feet span, the load per square foot being 30 lbs., and the distance between the principals 10 feet :—

30 lbs. load per foot,
50 feet span of principal,

1500
10 feet distance between principals,

15000 lbs. total load on principal.

This may, if required, be reduced to tons by dividing by 2240 thus :—

```
2240 ) 15000 (6·69 (say) 6·7 tons.
       13440
       -----
        15600
        13440
        -----
        21600
        20160
        -----
         1440
```

To find the load per foot run of the main principals, we have—

Rule. To find the load in pounds per foot run on a main principal, multiply the load in pounds per square foot of roof by the distance in feet between two principals—the product will be the required load.

Example.—Let the load per square foot be 25 lbs., and the distance between the principals 12 feet, then

25 lbs. load per square foot,
12 feet distance between principals,

300 lbs. load per foot run of principal.

Let us take, as a general example, a light shed-roof of which each principal consists of two rafters meeting at the centre of the span, and having their ends tied by a horizontal tie-bar supported in the centre by a vertical tie-rod passing up to the crown of the roof. The two rafters will sustain the load, their ends being prevented from spreading by the tie-rod connecting them; the vertical rod only serves to prevent the tie-rod from sagging. This will in effect be the truss shown in Fig. 13, but in an inverted position.

Let the span of the roof be 12 feet, its rise or height in the centre 2 feet 6 inches, the length of each rafter will be 6 feet 6 inches.

Let the maximum load be 20 lbs. per square foot, and the distance of the principals apart 10 feet, then, by the above rule, the load on each principal will be found,—

20 lbs. load per foot,
12 feet span,

240
10 feet distance between principals,

2400 lbs. load on principal.

But the rafters being equal, each of them will carry one half of this load, or 1200 lbs., and the maximum strain on each rafter will be found from the rule as follows :—

Rule. To find the maximum strain on each rafter, multiply the load in pounds on the rafter by the length of the rafter in feet, and divide the product by the height of the roof in feet; the quotient will be the strain on the rafter (thrust) in pounds.

This, the maximum strain on the rafter, occurs at its root, from which point the thrust diminishes towards the crown of the roof; but the practice is, in building large roofs, to make the rafters of the same sectional area throughout.

In the above case the maximum strain is thus found:—

1200 lbs. load on rafter,
6·5 feet length of rafter

6000
7200

Height of roof 2·5) 7800·0 (3200 lbs. thrust on rafter
75 (nearly).

30

Allowing 6000 lbs. per square inch of sectional as a safe strain on wrought-iron, the theoretical area of the rafter would be

6000) 3200 lbs. thrust,

0·533 square inches,

which would be furnished by a T iron of which the web was $1\frac{1}{2}$ inches, and the back or flange 1 inch wide, the metal being $\frac{1}{4}$-inch thick; but this would be insufficient to carry the transverse or bending strain due to the load on the rafter; hence it will be necessary to use stronger rafters, or to carry a strut from the foot of the central vertical tie or king-rod to the centre of the rafter.

We will consider the web part of the T iron as carrying the load in the manner of a beam resisting transverse strain, and determine what depth would be requisite, the thickness of metal being taken at half an inch, to sustain the load of 1200 lbs. which rests upon each rafter. The effective span of each rafter is represented by the horizontal distance between the foot of any rafter and the centre or crown of the principal; this will evidently be one half of the whole span of the principal, and therefore 6 feet, the load being uniformly distributed. The rule by which the depth of the bar will be determined is as follows:—

Rule. To find the depth in inches of a wrought-iron bar to carry a load producing transverse strain, multiply the load in pounds by the span of the bar in feet, divide the product by 600 times the breadth in inches, then the square root of the quotient will be the required depth in inches.

As resistance to transverse strain is a property requisite in roof rafters, the T irons used in their construction are usually made of a depth which is large in proportion to the width of the flange or back of the T iron.

Applying the foregoing rule to the case under consideration, we obtain the following result:—

$$\begin{array}{rr} \text{Breadth of bar } 0{\cdot}5 & 1200 \text{ lbs. load on bar,} \\ 600 & 6 \text{ feet span of bar.} \\ \hline 300{\cdot}0\ )\ 7200 & \\ 24 & \end{array}$$

The square root of 24 is nearly 5 inches; hence, in this case, it would be preferable to apply the strut referred to, when the depth of the T iron might be safely reduced to 2 inches, sufficient strength being given by a bar 2 inches by 1½ inches by ½ inch thick; but by adopting this arrangement it must be remembered that one half of the load on the whole principal will be transmitted through the king-rod or vertical tie, thus producing on it a strain of 1200 lbs.

If we allow 10,000 lbs. per sectional square inch as safe load upon wrought-iron suspension rods, the sectional area required for this tie-rod will be 0·12 square inch; hence we may apply a round bar 7-16 of an inch in diameter, which will give an area of 0·15 square inches.

The strain on the tie-bar joining the lower extremities of the rafters will be found from the following rule:—

RULE. To find the tension on the tie-bar in pounds, multiply the whole load in pounds on the principal by the span of the principal in feet, and divide the product by eight times the rise of the roof in feet; the quotient will be the required tension in pounds.

In the present case the whole load on each principal is 2400 lbs.

Rise of roof 2·5	2400 lbs.,
8	12 feet span,
2,0·0)	2880,0
	1440 lbs. tension on tie-rod.

Allowing, as before, 10,000 lbs. per square inch as safe strain, the area of the horizontal tie must not be less than 0·144 square inches; hence a half-inch round rod may be used, which will give an area of 0·196 square inch.

The joints of the rods and struts may be determined according to the general principles and methods shown in Chapter IV.

If the roof be pitched at such a slope that snow instead of lodging on it slides off, not being obstructed by parapet walls, then the load becomes so exceedingly slight in cases like that under consideration, that the proportioning of the rafters and ties becomes a mere matter of adopting a rigid form, and in many cases where corrugated iron forms the covering, it is not necessary to have any principals at all in the work—the corrugated iron being curved to a circular form, and so retained by light tie-rods.

We will now take an example of a roof of a larger description of which we shall assume the principals to be circular arches, having the section of an ordinary flanged plate girder.

Let the span of the roof be 120 feet, its rise being 15 feet, and the distance between the principals 20 feet, and the maximum load, per square foot, 40 lbs. The total load on each principal will be—

```
 40   lbs. per foot,
 120  feet span,
 ----
 4800
    20 ft. distance between principals,
 -----
 96000 lbs.
```

Or reducing this to tons, we have

```
2240 ) 96000 ( 42·85 tons nearly.
       8960
       ----
        6400
        4480
        -----
        19200
        17920
        -----
         12800
         11200
         -----
          1600
```

To find the thrust at centre we have the following rule :—

RULE. To find the strain in tons at the centre of an arch, multiply the total load in tons on the arch by the span of the arch in feet, and divide the product by 8 times the rise of the arch in feet. The quotient will be the required strain.

In the present case we have

Rise -	15	42·85 tons load,	
	8	120 feet span,	
	12,0	)514,2·00	
		42·85	tons strain at centre.

For this class of work we may allow 4 tons per sectional square inch as a safe strain, hence the sectional area required at the centre will be

$$4\,)\,42{\cdot}85$$
$$\overline{10{\cdot}7125} \text{ square inches.}$$

The total depth of the central section may be made 10 inches, and the width of the flanges 5 inches, the thickness of the flanges being $\frac{3}{8}$ths of an inch, and that of the web plates $\frac{1}{4}$-inch, these being connected together by angle-irons 2 inches by 2 inches by $\frac{3}{8}$-inch thick, then the sectional areas will be as follows—being determined by rules already given in a previous chapter :—

Sectional area of 2 flange plates, 5 in. by $\frac{3}{8}$-in.	3·75	sq. ins.
,, ,, 4 angle irons, 2 in. by 2 in. by $\frac{3}{8}$-in.	5·43	,,
,, ,, 1 web plate, 9$\frac{1}{4}$ in. by $\frac{1}{4}$-in. -	2·31	,,
Total sectional area -	11·49	,,

Thus we have a sectional area somewhat in excess of that required according to the calculation.

The width (or depth) of the web plate is found by deducting the sum of the thicknesses of the flange plates from the total depth of the girder; thus the thickness of each flange plate being $\frac{3}{8}$-inch the sum of the thicknesses of the two will be $\frac{3}{4}$-inch, which, deducted from

10 inches, the total depth of the rib, leaves 9¼ inches for the depth of the web plate.

We must, in the next place, ascertain the sectional area at the points of support, or abutments, the strain at which point will be found by the following rule :—

RULE. To find the thrust in tons at the abutment of an arch, square the thrust in tons at the crown of the arch and add to it the square of half the total load in tons on the arch, extract the square root of the sum, and the result will be the thrust in tons on the abutment of the arch.

As stated above, the strain at the centre or crown of the ribs is 42·85 tons, and the total load on the arch is also 42·85 tons, hence half the total load on the ribs will be

```
2 ) 42·85
    21·425 tons.
```

The thrust at the abutments will be found as follows :—

```
        42·85 thrust at centre,
        42·85   ,,      ,,
       ------
       21425
      34280
      8570
    17140
   ---------
   1836·1225 square of thrust at centre.

       21·425 tons half total load,
       21·425  ,,     ,,
      -------
      107125
      42850
     85700
    21425
   42850
  -----------
   459·030625 square of half total load,
  1836·1225   square of thrust at centre,
  -----------
  2295·153125 sum of squares.
```

The square root of this sum must now be extracted; it will be enough for practical purposes to take it to four places of decimals.

$$
\begin{array}{rl}
4\,) & 2\dot{2}9\dot{5}\cdot1\dot{5}3\dot{1}\,(\,47\cdot90 \text{ tons nearly.} \\
 & 16 \\ \hline
87\,) & 695 \\
 & 609 \\ \hline
949\,) & 8615 \\
 & 8541 \\ \hline
9580\,) & 7431
\end{array}
$$

Allowing, as before, 4 tons per inch as safe strain on the metal, the sectional area required at the abutments will be—

$$
\begin{array}{rl}
4\,) & 47\cdot9 \\ \hline
 & 11\cdot975 \text{ square inches.}
\end{array}
$$

By gradually increasing the depth of the rib from the centre towards the abutments, so that at the latter it is 13 inches deep, the additional area due to increased depth of web will be equal to 3 inches by $\frac{1}{4}$-inch, or 0·75 square inches; hence, adding this to the sectional area at the centre of the rib, the sectional area at the abutment will be—

$$
\begin{array}{rll}
11\cdot49 & \text{inches} & \text{at centre,} \\
\cdot75 & ,, & \text{additional,} \\ \hline
12\cdot24 & ,, & \text{at abutments,}
\end{array}
$$

which is slightly in excess of what is required by calculation.

A few remarks are now necessary as to the construction of the ribs or arched principals.

The correct form for an arch which is loaded with an uniformly distributed load is that of the parabola, but the true curve will, in the case of arches formed as circular segments, be found to be contained between the inner and outer flanges—that is, between the *intrados* and *extrados* of the arch—provided that the rise of the arch, or its versine, be not very great in proportion to the depth of the ribs, hence the circular form is commonly adopted. There is, however, no reason why the arch should not be made to a parabolic curve, as it is as easy to set out in marking the metal as is the circle.

A rib of the description to which we have been referring should have stiffening irons of angle or tee iron rivetted on to the web at intervals of four or five feet, or sometimes closer, as the web being made thin is not of itself sufficiently rigid to preserve the relative distances of the top and bottom flanges of the rib. These stiffeners are usually proportioned according to the size of the angle-irons used to connect the flange plates with the web plates; thus in the present instance, where these angle irons are 2 inches by 2 inches by $\frac{3}{8}$-inch, if tee-irons were used they should be made $3\frac{1}{2}$ inches by 2 inches by $\frac{3}{8}$-inch. The purlins, which are supported by the main principals, may consist of angle or tee irons, or where the load to be carried is great they may be flanged girders, either rolled or made of plate and angle iron and rivetted up, but in any case they are calculated according to the rules already laid down for beams subject to an uniformly distributed load. The amount of load upon a purlin may be ascertained from the following rule:—

Rule.—To find the load on any purlin of a roof, multiply the load per square foot on the roof by the distance in feet between the main principals, and by the distance in feet between the purlins. The product will be the load on one purlin

Example.—Let the distance between the main principals be 20 feet, that between the purlins 4 feet, and the weight of the covering of the roof 8 lbs. per square foot :—

8 lbs. per square foot,
20 feet distance between principals,

160
4 feet distance between purlins,

640 lbs. load on each purlin.

Let it be assumed that these purlins are to be rolled girders four inches in depth ; it is necessary to determine the sectional area of the flanges ; the rule for strain on either flange at centre is—

Rule. To find the strain in pounds on either flange of a girder at the centre, multiply the total load in pounds on such girder by the span of the girder in feet, and divide product by eight times the depth of the girder in feet.

In the present case we have—

Depth of purlin -	$\cdot 3\dot{3}$	640 lbs. load,
	8	20 ft. length of purlin
	$2\cdot\dot{6}6$)	12800·00 (4816 lbs. strain nearly.
		1064
		2160
		2128
		420
		266
		1540

From this it will be seen that in ordinary cases of light covering the strain on the purlins is almost nominal, so that what we have to attend to is to see that they are sufficiently deep to give the required rigidity. Let us, therefore, instead of considering them to be rolled girders, regard them as bars placed on edge and half an inch thick, and determine the required depth to sustain the load.

We find it as follows :—

$$\begin{array}{lrr}
\text{Breadth in inches -} & \cdot 5 & 640 \text{ lbs. load} \\
 & \underline{600} & \underline{20 \text{ ft. span of purlin.}} \\
 & 3{,}00 &)\,128{,}00 \\
 & & \underline{} \\
 & & 42 \cdot 6\dot{6} \\
\end{array}$$

The square root of $42 \cdot 6\dot{6}$ must now be found—

$$\begin{array}{rl}
6\,) & \dot{4}2 \cdot \dot{6}6\;(\;6 \cdot 5 \text{ inches nearly.} \\
 & \underline{36} \\
125\,) & 666 \\
 & \underline{625} \\
 & 41 \\
\end{array}$$

Of roofs supported by straight girders, having either lattice or plate webs, it is here only necessary to remark that they are treated according to the ordinary rules for such girders already set forth in a previous chapter.

The framework of a dome-shaped roof may be regarded as being formed by a number of arched principals intersecting each other at one common point; hence, the ribs may be treated as arches; but in determining the load upon each the mean distance must be taken between them, and multiplied by the load per square foot to which the roof will be subject.

We shall not dilate further upon roofs, our object being to show by one or two examples how the rules already fully set forth are in practice brought to bear, without encumbering our space with special cases of a lengthy description.

CHAPTER VII.

IRON FLOORS.

Iron floors are used for two reasons. In the first place, iron is the most suitable material to employ where very heavy weights have to be supported; and, in the next, it is fire-proof; and it may also be added that in many places iron girders act as ties to the walls of large buildings.

As to the calculations of the girders used in iron floors but little need be said in this place, as, the loads being given, these girders are, as a matter of course, calculated in precisely the same manner as those employed for any other purposes where loads have to be sustained.

On the other hand, as regards the application of iron to fire-proof floors, there is much to be remarked both *pro* and *con.* What is required in a fire-proof floor is:—

1. That it shall be incombustible.

2. That it shall not spread combustion by transmitting heat freely from one combustible body to another.

3. That it shall not lose its strength or rigidity under the action of the greatest heat likely to be evolved in accidental fires in buildings.

4. That the materials of which it is composed shall not injure the walls of the building of which it forms a part.

The first condition is perfectly complied with in iron, as, although this metal is chemically combustible, yet practically it is perfectly incombustible in large masses.

The second requirement demands careful consideration, for it is certain that iron is a good conductor of heat, and will, in fires of long duration, become red hot, or even white hot, in which case it may, when other circumstances favour such a result, cause a fire which has occurred in one apartment to be transmitted to one above or below it, hence iron girders used in the construction of floors should not be so arranged as to allow of their remaining in contact with combustible bodies.

To fulfil the third condition cast-iron is evidently better suited than is wrought, as the latter from its malleability will soften when exposed to a very high temperature, and is, therefore, more liable to bend and give way than cast-iron, though the latter will sometimes fail by cracking.

The last stipulation iron scarcely accords with, on account of the changes of size which occur under changes of temperature; thus the expansion and contraction of long girders under the influences of a fire and the subsequent cooling may seriously impair the stability of the walls of a building.

Considering all things, then, we observe that when iron is used in the construction of a fire-proof floor it should itself be protected from fire as much as possible, and that for several reasons: first, in order that it may not become red hot, and by conducting the heat from one place to another extend a conflagration; second, to counteract great and rapid changes of dimensions by excluding heat, or rendering its influence upon the metal more gradual in its action; and thirdly, to prevent the girders from being cracked or split through water falling upon them when in a highly heated condition.

Hence we come to the conclusion that to attain the most satisfactory results the floor must be of a compound character, consisting of iron in combination with some non-conducting material.

A method rather extensively used some years since consisted in laying brick floors on cast-iron girders; the girders were made of the ordinary section proposed by Eaton Hodgkinson,—that is, having a small topflange and a large bottom one,—and being placed parallel to one another, brick arches were turned between them. The spaces on the springings of the arches were filled in with concrete or some other similar composition, which is a bad conductor of heat, and therefore prevents the iron from conveying the heat from one apartment to another; but here the difficulty of expansion and contraction is not overcome, for the lower flanges of the girders are exposed to heating from the combustion of materials beneath them.

Another kind of fire-proof floor is made of bricks so formed as to fit together, somewhat after the style of a joggled joint, but in such a way as to form a very flat brick arch, having a very trifling rise. Of course in such an arrangement the thrust at the springing sides of the floor is enormous in proportion to the load supported, but this does not come upon the side walls, being taken up by a number of tie-rods, which may pass under or even through the bricks, which are made hollow. There is much ingenuity in this mode of construction, but there are two causes of failure to be apprehended. The tie-rods being of wrought-iron would be apt to become ductile in the temperature accompanying a fire of any magnitude, when, stretching even a little, so flat an arch as that tied by it would yield under its own weight, and by falling form a new source of danger. Again, there is the possibility of ordinary changes of temperature between summer and winter, causing a settlement of the arched floor, and a corresponding displacement of the walls of the fabric in which it is contained.

In making floors supported by straight girders the expansion and contraction of the metal may be allowed to proceed without producing any deleterious effects by inserting the ends of the girders into loose sockets or boxes, in which they can slide, so as to allow for alterations of length accompanying variations of temperature. These girders may be covered with flags of refractory stone, such as will not very readily split with heat, and in addition to this the lower flanges may carry light plates of cast-iron, forming a sub-floor, to intercept heat or flame rising towards the stone above and thus render them more safe from cracking, besides serving to prevent their falling through, and leaving a hole in the floor if they should from any cause give way. This is a somewhat heavy floor, but we are inclined to think it a sound one. The stones should only *rest* upon the girders, but the cast-iron plates beneath should be secured to them, as they will serve to brace them laterally, and will expand and contract in the same manner.

Where flat cast-iron floor-plates are used to support any weight, the following rule may be useful to determine the proper thickness of metal to use :—

Rule. To find the thickness of cast-iron floor-plates in inches, multiply the square root of the load in pounds per square foot on the plates by the length of the plates in inches, and divide the product by 380, the quotient will be the required thickness in inches.

Example.—Let the length of the plates be 30 inches, and the maximum load to which they will be subject 144 pounds per square foot, which would be a fair allowance.

The square root of 144 is 12, hence we have—

```
            12 square root of lead in lbs.' per foot.
            30 length of plates.
         ─────────
      380 )360·0( .97 thickness of plates
            3320
           ──────
            2800
            2660
           ──────
             140
```

or something in excess of nine-tenths of an inch, hence practically such plates would be made one inch in thickness.

Instead of using flat cast iron plates, curved or corrugated plates may, with much advantage, be substituted, inasmuch as the curving adds very materially to the strength of the flooring, and so allows of thinner material being used. In iron floors for bridges curved wrought iron plates, varying from three-sixteenths of an inch to three-eighths of an inch, are now coming largely into use, but for the reasons already stated they are not suitable for fire proof floors.

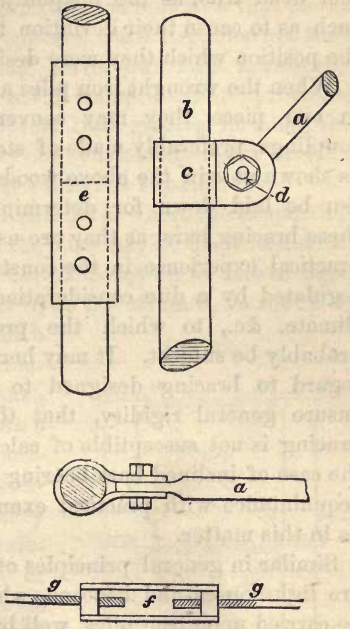
CHAPTER VIII.

MISCELLANEOUS IRON STRUCTURES.

In addition to those iron structures which may be specially classified as bridges, roofs, &c., there are numerous others, composed of girders, columns, &c., of a miscellaneous character, which we must here briefly notice.

Iron piers are now frequently formed of a number of iron piles, screwed or otherwise sunk into the ground, at moderate distances apart, and carrying on their summits the girders and flooring comprised in the superstructure. Formerly, cast iron piles were principally used in these works, but during the last few years wrought iron piles have come pretty generally into use, being made of various diameters, varying from three to seven inches in diameter. These piles, although having the disadvantage of being much more costly than those of cast iron, yet possess superior properties more than counterbalancing that evil, as the wrought iron is not so liable to rupture from collision of floating bodies as the cast, and if fairly protected is also more durable, and less trouble in erection. On the tops of the piles are fitted caps, to the flanges of which the girders carrying the platform may be bolted, and between the piles are ties or bracing bars to retain them in position, and maintain the general form and rigidity of the structure. The bracing bars are attached to the piles by means of clips encompassing the piles, and holding the ends of the bracing bars between their flanges, as shown in Fig. 32, where *a* represents a round bracing bar, flattened at its end, and formed into an eye, which is placed between the ends of the clip *c*, and secured there by the bolt *d*, which, in tightening up, also causes the clip firmly to embrace the pile *b*. It will be observed that in this arrangement the position of the clip may be determined to suit the length of the bracing bar by slipping it up or down the piles, as the circumstances of the case may require. Another mode of adjustment is shown at *f*, which represents a screw shackle, having a right-handed screw at one end and a left-handed screw at the other, the shackle receiving the end *g* of the halves of the tie-bar. By this means the practical length of the tie may be regulated by turning the shackle to the right or left, according as it may be desired to lengthen or shorten the tie.

Fig. 32.



That some means of adjusting the length of bracing bars is absolutely indispensable is evident, for it is impossible to be certain of putting down the piles of a pier dead true, as the inequalities of the soil may be such as to cause their deviation to a slight extent from the position which they were designed to occupy.

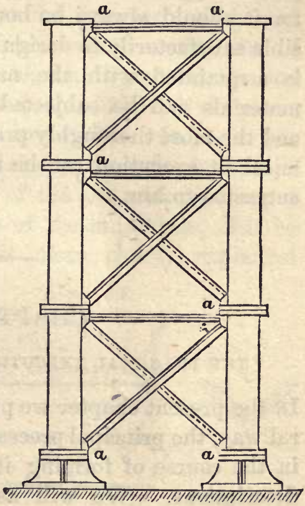
When the wrought iron piles are too long to be made in one piece they may conveniently be joined by couplings, preferably made of steel, and fixed by bolts, as shown at *e* in the above woodcut. No general rule can be laid down for determining the scantlings of these bracing bars, as they are usually proportioned by practical experience in the construction of such works, regulated by a due consideration of the exigencies of climate, &c., to which the proposed structure will probably be subject. It may here be observed, that in regard to bracing designed to resist vibration, and ensure general rigidity, that the strains upon such bracing is not susceptible of calculation the same as in the case of inclined bars bearing *loads*, hence a familiar acquaintance with practical examples alone can guide us in this matter.

Similar in general principles of construction to piers, are lighthouses and beacons, which may conveniently be carried upon iron piles, well braced together so as to resist the storms to which works are exposed, and, it will be seen, that the open braced work carrying the superstruction of a wrought iron light-house, by not opposing so large a surface to the action of the waves as does a mass of solid masonry, stands much less chance of being injured or swept away in a hurricane.

Moles and breakwaters have always occupied much attention amongst engineers who devote themselves to marine practice, and several methods for forming them of wrought and cast iron have been promulgated, and one we may especially allude to. It consists of a number of cast or wrought iron girders arranged in a step-like form, but at a certain distance apart, so that as the waves break upon the girders, the water does not rush back upon the following wave, but falls harmlessly through the girders into the tranquil waters, protected by the structure. The cast iron girders forming the breakwater proper may be sustained upon cast or wrought iron piles firmly set in the subsoil.

In all kinds of braced structures where the supporting columns are of cast iron the clips may be dispensed with, as the tie bars can be bolted direct on to ears or lugs, cast in suitable positions upon the columns, as shown at *a a a a* &c in Fig. 33 which represents a portion of framing for a water tower, hoist, or other lofty work. As a matter of course the supporting columns are proportioned according to the load they are designed to sustain. In the bracing round or flat, ties may be used or tie and angle iron bars according to the nature of the works, and the views of the designer. The latter sections of iron being better adapted for resisting vibration seem to us to be decidedly preferable, as when long round ties or flat bars are used, they offer little or no resistance to vibratory disturbances which act in a lateral direction and hence

Fig. 33.



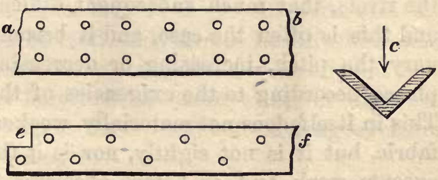
do not sufficiently resist any force which may tend to set the structure in a state if general tremor which may not unfrequently be caused by recurring guss of wind, or by the motion of machinery, connected with or contiguous to the structure. From what we have now said about miscellaneous structures it will be evident that the rules set forth in the earlier chapters of this treatise are sufficient to determine all calculable sections, for every class of structure composed of cast or wrought iron, no matter what may be its form or the duty it is intended to perform. We shall now conclude our discussion of the principles upon which structures are designed and proceed to the not less important consideration of the generalmethods in which such works when duly detailed are practically executed in the contractors, yards and shops, for it should always be borne in mind that it is impossible satisfactorily to design a work, unless the designer is acquainted with the manipulations to which the materials will be subjected in carrying out his plans, and the more thoroughly practical he is, the more economical in execution can he make those works which are entrusted to him.

CHAPTER IX.

THE PRACTICAL EXECUTION OF IRON STRUCTURES.

In the present chapter we purpose discussing, in a general way, the principal processes to which iron is subjected in the course of forming it into structures of various descriptions. We will assume that a wrought iron bridge, of the plate girder description, is to be constructed, and that the Engineer's plans and general drawings have been furnished. In the first place a few remarks as to *working drawings* will be necessary, as it happens most frequently that the drawings supplied by designers require revision before they can conveniently be worked to; and this is, in a great measure, due to the mode adopted by designers in determining the dimensions of the various parts of the work in hand. It is always advisable to get an even pitch for the rivets in the flanges throughout; and this might, in almost all cases, be done, if the matter were but duly considered in the first place. A very convenient and general pitch for bridge-work of ordinary dimensions is four inches; hence let a case be supposed in which that pitch is to be adopted—for the top and bottom flanges. It is evident that the lengths of the flange plates should be multiples of four inches, in order that the four inch pitch may work in, without any half holes in the ends, for of course, at each end of a plate, the distance of the end from the centre of the last hole should be half the pitch, that is to say, in this case, two inches. In the angle irons attaching the flange plates to the web plates, half holes will occur, on account of the alternate pitching of the rivets on the two limbs of the angle iron, if it be cut square across. This is more clearly explained by reference to Fig. 34, which represents an angle iron punched ready for rivetting. *a b* shows a plan of one end of a bar looking down upon it, in the direction of the arrow at the section *c*, and it will be noticed that in the

Fig. 34.



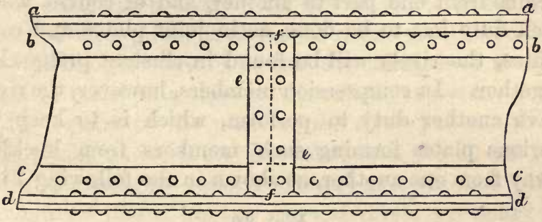
upper row of holes there is a distance of half a pitch from the end hole to the end of the bar, whereas in the lower row of holes the end of the bar coincides with the centre of a rivet-hole. By shaping the end of the bar as shown at *e*, in the view *e f* this point is obviated, but this method is scarcely ever adopted in practice, except for angle iron covers where the one limb is continued beyond the other, sufficiently far to get the room necessary for the rivet.

The actual importance of the results produced by the halved rivet hole, depends entirely upon the duty of that portion of the structure in which it occurs; thus, if it be in an element subject to *compressive* strain only, it is not of any consequence for the ends of the plates or bars pressing against each other, the rivet has no strain upon it if the work be true; but on the other hand, if the member should be in tension, the whole efficiency of the one rivet is lost, and it might as well be left out altogether.

The pitch and position of rivets must also be considered in respect to the locating of cross girders, stiffening pieces, gussets and other adjuncts to the main girders, hence it is very evident that if a bridge be designed, and the position of its various elements determined without due consideration of the position of the rivets, that much subsequent difficulty may arise; and this is often the case, and it becomes necessary to vary the pitch, increasing or decreasing it at certain places, according to the exigencies of the case in hand. This in itself does not materially weaken or injure the fabric, but it is not sightly, nor is it the *right way* to execute work, and, as before observed, it causes much trouble, and some very odd pitches are arrived at at times, such as 4·21 inches, pitch, &c.

In regard to the web plates, but little need be said about pitching the rivets where the joints occur, as these may be worked in to any convenient pitch, as shown in Fig. 35. *a a* shows a portion of the top flange plate, *b b*

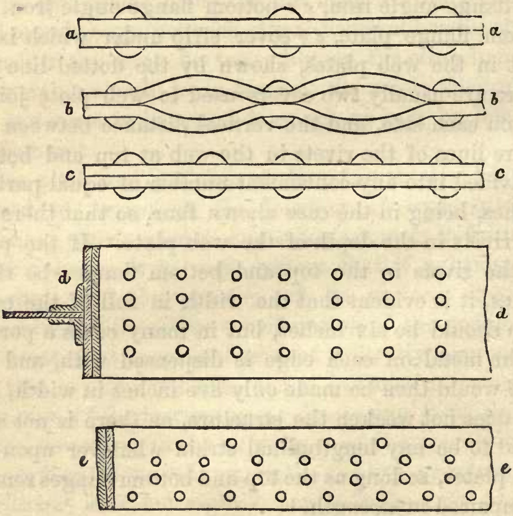
Fig. 35.



top flange angle iron, *c c* bottom flange angle iron, *d d* bottom flange plate, *e e* cover strip under which is the joint in the web plates, shown by the dotted line *f f*. There are usually two covers used to web plate joints, one on each side, and the vertical distance between the centre lines of the rivets in the web at top and bottom is divided into any convenient number of equal parts or pitches, being in the case shown four, so that there are five rivets in the depth of the web plate. If the pitch of the rivets in the top and bottom flanges be three inches, it is evident that the width in full of the cover strip should be six inches, but in many cases a portion of the metal on each edge is dispensed with, and the strip would then be made only five inches in width, and this does not weaken the structure, as there is not supposed to be any longitudinal strain whatever upon the web plates, so long as the top and bottom flanges remain unimpaired in strength.

Various opinions exist as to the relative values of different pitches used in bridge-work, so while on the subject of rivets we may express the views arising from our own experience in the matter. In bottom flanges in tension, and in other members under tensile strain only, it is evident that there can be no object in pitching the rivets closely, their sole use being to connect the different elements of the bridge together, and to transmit the strains from one part to another, and of course, where such duty has to be done, as in joint plates and cover plates, the rivets will be found in clusters pretty close together. In compression members, however, the rivets have another duty to perform, which is to keep the various plates forming such members from buckling away from one another, as shown in the following Fig.

Fig. 36.



In Fig. 36, *a a* represents the edges or side elevation of two plates forming a compression flange with the rivets pitched very widely apart. Now, as soon as a strain comes upon the member, the plates may have a tendency to buckle and shorten, as shown in an exaggerated form at *b b*; and should this occur, even in a very slight degree, it stands to reason that moisture and rain will get between the edges of the plates and cause their rapid decay. If, however, the rivets wer closer in their pitch, as shown at *c*, this result would be far less likely to ensure, in fact the strength of rivetted work to resist this class of distortion, varies inversely, as the square of the pitch of the rivets: thus a three-inch pitch, under these circumstances, would be four times as strong (to resist buckling) as a six-inch pitch. For narrow plates the two rows of rivets necessary for attaching the angle irons to the flanges, may be sufficient to hold the plates together, but for others four rows as shown at *d d*, or more if necessary, according to the width of the plates are used; and these rivets may be arranged in even rows as shown at *d d*, or may be alternated as shown at *e e*. The latter mode is preferable, as the plate is not so much weakened by the alternate as by the opposite rows of rivets.

In ordinary bridge-work it is not necessary in compression members to adopt any pitch less than three inches, below which we only need to go, in extreme cases where remarkably thin plates are used, which, however, is seldom done, as it is evidently more convenient in every way to use one-half-inch plate, than to have one-quarter-inch plate superposed on another; hence we may set down for small girders three-inch pitch as proper for the compression member, and six-inch for the tension member; this is convenient, as the three-inch and six-inch pitches will work into equal lengths: that is, a plate suited for six-inch pitch will always work with three-inch pitch, and the rivets thus being opposite each other in the top and bottom flanges, suit equally well for the attachment of stiffeners, &c. As the plates in the compression flanges are made thicker, so may the rivets be pitched wider apart; and it is very usual practice for bridges from eighty feet span to two-hundred feet, to adopt uniformly four-inch pitch for both top and bottom flanges, although a wider pitch may be used with advantage for the tension member. When four-inch pitch is used for the top, and six-inch pitch for the bottom flange, care must be taken in arranging the stiffeners, as the top and bottom rivets will only be opposite each other at the end of every even foot from the starting point, in setting out the rivets.

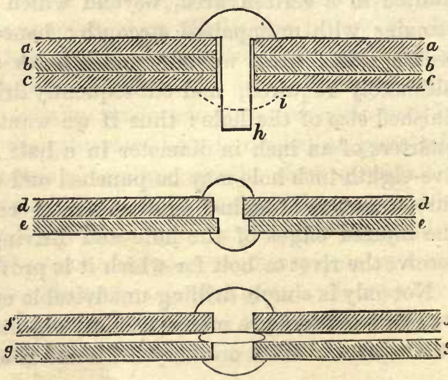
At certain times, ideas have been held that very close pitching of rivets in the flanges, under compression, gives a great increase of strength, but this is a in practical sense *utterly fallacious* and now is in all probability ignored by all *practical* engineers, for not only is there nothing gained by the very close pitching but it actually involves an element of danger, as the great number and close proximity of the rivet holes indicate the removal of a very great portion of the metal of the members accompanied, very likely, by injury to the strength of that metal which remains to resist those strains which the structure is destined to sustain.

Rivets are usually made one-sixteenth of an inch in diameter less than the rivet hole is drilled or punched, and in the process of heading up, the rivet spreads out and fills the rivet hole, and the rivets, as they contract in cooling, draw together the various plates through which they pass, thus rendering the various plates as rigid almost as if they formed one thick plate. It may here be observed, that in determining the diameter of the rivets it should be remembered never to make it less than the thickness of the plate through which it passes: thus, two-inch plates require rivets one-inch in diameter to join them together, therefore the rivet holes would be punched or drilled one-inch in diameter, and the rivets made of round iron fifteen-sixteenths of an inch in diameter. In speaking of rivet holes, punching and drilling are mentioned, and this raises a point of great importance, both to designers and manufacturers of bridges. Amongst civil engineers there prevails a very great objection to punched holes on account of the supposed straining of the plates and bars due to the action of the punch, therefore, let us in the first place consider how far this objection holds good in a practical sense. It must be admitted that in punching a hole, the metal immediately surrounding that hole is somewhat strained; one might almost say that the edge of the hole is in all probability slightly starred for a certain distance from the edge of the hole, this distance depending on the diameter of the whole, and the thickness of the plate punched. Allowing this to be the case it will be observed that the injury to the metal in punching is limited to a certain area, beyond which the material remains with unimpaired strength: hence in order to secure sound work we may punch holes smaller than ultimately required, and subsequently drill out to the finished size of the hole: thus if we want a hole three quarters of an inch in diameter in a half inch plate, a five-eighth inch hole may be punched and drilled out to three quarters of an inch in diameter, thereby removing the injured edges of the hole and leaving it clean to receive the rivet or bolt for which it is provided.

Not only is simple drilling unadvisable on account of its cost but there are many practical objections to it on other scores. With ordinary drills and machines it may be said to be practically impossible to drill a *perfectly circular* hole, as the drill will run according to the direction of the fibre of the metal and the greater the diameter of the hole in proportion to the thickness of the plate, the worse will be these irregularities, and to properly secure the plates and drills, so that perfectly circular holes are obtained, would be too tedious and costly to be adopted for bridge work, especially considering that the hole first punched and then drilled is equal, if not superior, to the hole which has been drilled throughout. As a general maxum holes having a diameter less than the thickness of the plate must be drilled, while those having a diameter greater than the plate may be punched, and very thin plates such as three-eighth, quarter, five-sixteenth, &c. should never be drilled for girder work.

It is very evident that rivetted work will not turn out perfectly satisfactorily, except the holes be opposite one another, so that the heated rivets may pass fairly through them as illustrated in Fig 37.

Fig. 37.

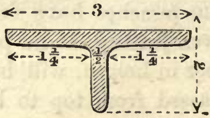


In Fig, *a a b b*, and *c c* show three plates, properly punched, and in position with the rivet inserted ready to be headed up, the head being made from the metal in the end *h* of the rivet which is hammered up into a head as shown by the dotted lines at *i*. If however the holes be not in line, but as shown at *d d e e*, then the rivet is in fact partly sheared through in the process of manufacture, and is of course weakened in a proportionate degree. Care should also be taken, that the plates to be rivetted are in close contact, hence under the blows of the rivetting hammer, the soft body of the heated rivet may form a collar as shown between the plates *f f*, and *g g*. It is impossible when the rivet holes are marked on the plates by hand one by one from templates, that they can be got *dead true* throughout— but for all ordinary work, they may be got sufficiently near for any purpose, a little rhymering out being sometimes necessary, but when a great number of plates are to be rivetted together, more accurate means of pitching the rivet holes may be requisite; in cases of this sort the multifarious punching machine designed by the late Richard Roberts is very useful, as by its means the plates way be punched one after another with perfect truth; so true indeed is the action of this machine, that we have after punching a large plate throughout, again passed it through the punching apparatus, and the punches have fallen into the same holes without even jarring, and a pile of plates thus punched, several feet in height, will be so true that rods may easily be passsd from top to bottom of the plates through the rivet holes.

Multifarious drilling machines are much used where very exact pitching of the rivets is required, and if well made they give satisfactory results. In these machines the plates may be drilled separately or laid together in piles and drilled throughout at once, the latter process is undoubtedly the best, but requires care in its execution. The lengths of rivets will depend upon the thicknesses of plate, through which they have to pass, and may be arrived at in the following way, add together the thicknesses of all the plates or bars through which the rivet has to pass, to that add one-thirty-second of an inch for each plate, and to the whole add one and a half times the diameter of the rivet to allow sufficient material to form the head. The one-thirty-second of an inch per plate is allowed for irregularities in the surfaces of the plates, preventing them from lying perfectly close together over their entire surfaces.

It may here, while speaking of rivets, be well to mention a mistake often made by designers through mere thoughtlessness in arranging the size of angle irons, and the iron stiffeners for girder work, which is that they frequently determine the size of such elements without considering whether there is room in their flanges to hold the rivets necessary for their attachment to the web and flanges of the girder.—Let us suppose that the iron stiffeners, three inches by two inches by half an inch thick, are specified, and at the same time three-quarter inch rivets to be used, the dimensions will then be as shown in Fig. 38. It will be seen here that on each side of the web of the tee iron there is but one and a quarter inches to receive the three-quarter inch rivet, so that outside the rivet there would be but a quarter inch of metal, which would very likely burst out in the process of rivetting up—and this case we have selected is by no means one of the worst that has come under our notice.

Fig. 38.



We will now pass from this special point of rivetting, and proceed to consider the general manipulations to which the iron is submitted in the iron yards, where bridges are constructed, commencing with the delivery of working drawings into the hands of the makers.

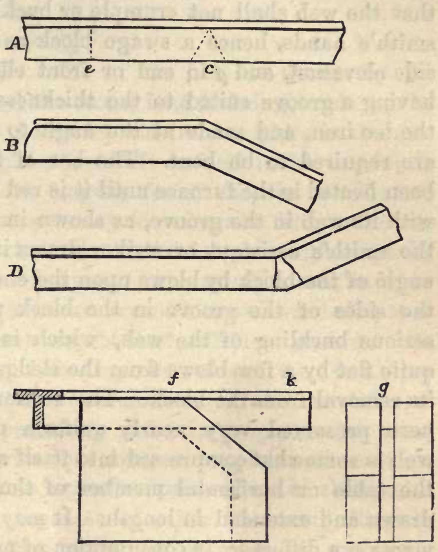
The first step to be taken is to prepare the specification (or list) of iron, bolts, &c., which is taken from the drawings, either by measuring the various dimensions to scale, or working from dimensions figured on the drawings, which latter is of course best, as there is less chance of error than exists when the scale is relied upon. This done, the drawings are handed over to the template makers, and copies of any cast iron work to the pattern makers, so that both the wrought and cast iron work may be proceeded with without delay The business of the template maker is to form frames or skeleton models of the plates, so as to comprise all parts where rivet holes occur, and on such templates (or battens, as they are termed in some yards) mark accurately out the position of the holes to be drilled in the various plates and bars, and drill them in the wood; from these templates the iron parts of the bridge are marked, a stump of suitable size being dipped in white paint and struck through the holes in the templates which have previously been securely clamped on the iron plates to be marked. By these white marks on the iron the punchers are guided in handling the plates under the punching machines, when those of the ordinary description are used, making one hole at a time, and without self-acting motions for working the plates along under the punches. When, however, self-acting punching machines are used, the templates may, for the greater part of the work, be dispensed with. If the pitch of the holes is even, as in such cases, the plate to be punched is fastened to the carriage or traverse table of the punching machine, and when set in motion that table is automatically moved between the strokes of the punch through a distance corresponding to the pitch of the rivets.

The templates being completed, and sent into the plating shop, the marking is soon effected, and the iron is then ready to pass into the hands of the various workmen who carry out the subsequent operations. With ordinary flat plates and bars, punching and *truing* the edges is all that is required before putting the work together. The former is simple enough, requiring merely care on the part of the workmen to follow the marks as closely as possible.

For *truing* the edges, in most cases shearing, in an ordinary shearing machine, is all that the works needs, but, where exact butt joints are to be made, the ends of the plates must be planed, which may be done either with a rectilineal planing machine, or a rotating cutter. The latter is most convenient, and if carefully adjusted will turn out sufficiently accurate work. These cutters consists of discs, similar to the face plates of lathes, but furnished with short cutters, which act in turn as the face plate revolves. Usually, a number of the plates are fastened together, and the ends of them all planed at one operation, which both saves time and makes a better job than would result if they were planed one by one. Joints of this description are requisite for members in compression, where equal bearing is required throughout, but in tension members it is a matter of no consequence. Plates requiring to be curved must be passed through ordinary boiler makers bending rolls until the required form, gauged by a template, is obtained, but knees and joggled stiffeners and strips must be made into the proper forms by smiths before being punched or drilled.

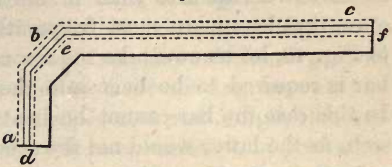
In Fig. 39 *A* represents an ordinary angle, or tee iron, which is required to be bent at an angle as shown at *B* or *D*. It will be noticed that in the first case the horizontal part or as it is commonly called, the table of the bar is at the back, or on the outside of the angle, but in the second case it is on the inner side or front.

Fig. 39.



There are two ways of making the joint; in the first case, a piece is cut out as shown by the dotted lines at *c*, and the angle iron is then bent, as shown at *B*, and the severed edges of the web welded or shut together, and in making this shut, it is of the greatest importance to secure soundness of work; for if it be weak, the knee is altogether useless, for as such knees are used to preserve the angular positions of other elements, they must possess full strength at the angle. From the liability of such shuts as these to be defective, it is always well to avoid them, if possible, and to make the bend without severing any part of the bar. In this process it is evident that the web of the bar is pressed into itself, being thickened or upset at the part where the angle occurs, and this is not in any way objectionable, as the thickening of the metal makes it stronger in the angle, but the operation should be so managed that the web shall not crumple or buckle up under the smith's hands, hence a swage block as shown at *f*, in side elevation, and *g* in end or front elevation, is used, having a groove suited to the thickness of the web of the tee iron, and made at the angle to which the bars are required to be bent. The bar of tee iron having been heated in the furnace until it is red hot, it is placed with its web in the groove, as shown in the figure, and the smith's assistant or striker draws it down over the angle of the block by blows upon the end, *k*, of the bar, the sides of the groove in the block preventing any serious buckling of the web, which is afterwards set quite flat by a few blows from the sledge, subsequent to its removal from the block. The section of the bar is here preserved very nearly uniform throughout, the web is somewhat compressed into itself as it were, while the table or horizontal member of the bar is slightly drawn and extended in length. It may here be well to mention a difference in consumption of material between the two modes of making these knees, in order to aid in determining the lengths of bars requisite for making such elements without waste. In either case the knee should be accurately drawn out, either full size, or to as large a scale as possible, so that its finished length may be accurately measured; let this be represented in Fig. 40. Then, if the bar is to be bent, by cutting a piece out in accordance with the first method above described, the length of bar required will be equal to the measurement of the knee or stiffener, along the back of

Fig. 40.



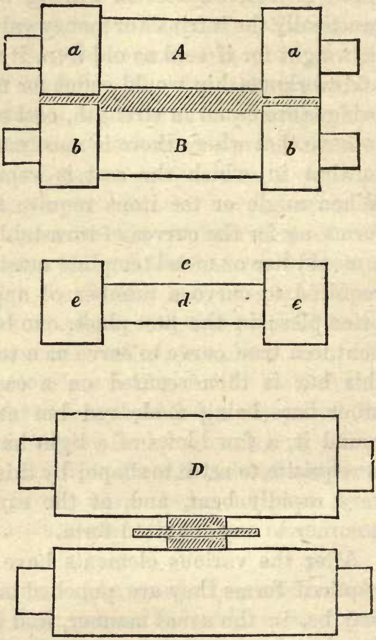
the iron as at $a\ b\ c$, but if the bars are to be drawn over a block, as described secondly, the length required will be that measured on the inside of the table of the bar, as shown by the dotted line d, e, f, the extra length being made up by the drawing of the table, which takes place in the process of bending the bar. It is well to be careful in determining these lengths, for if they be a little too short it becomes necessary to order new bars, or else to weld on pieces to make them long enough, thereby causing increase of expense unnecessarily, and, in addition to this, tee irons are somewhat awkward shapes to weld together, so as to obtain a satisfactory result. On the other hand, if the iron be ordered too long, that which has to be cut off becomes waste, and, as stiffeners are usually comparatively short in proportion to other elements, it follows that even short pieces of waste cut off them, run up to a greater per-centage of their total weight than in the case of other elements; thus, on stiffeners three feet in length, the waste of one and a half inches at each end would amount to $8\frac{1}{3}$ per cent. on the total weight of iron used in making the knees, and this on a large work, would soon mount up, for the knees, though small, are frequently so numerous as to form a considerable item in the weight of a bridge, and moreover, as a general rule, the amount of waste in any structure, or part of a structure, must not be judged of by its actual weight, but by the relation of that weight to that of the material in which it occurs, and by carefully watching such points as those, very sensible increments may be ensured, for it is rather in the number of small extra expenses than in the magnitude of great ones, that losses are most frequently made. To return to Fig. 40, let us now take the second case, in which the bar is required to be bent into the form shown at *D*. In this case the bar cannot be bent without cutting the web, as the latter would not draw out without contracting so much in section as to be practically useless, (unless, indeed, the angle at which the bar is bent is so obtuse as nearly to approach the straight form of the original bar), hence the web of the bar must be cut through, as shown by the dotted line at *e*, and then heated and bent, after which, an angular piece of iron, or V piece, as it is commonly called, is welded in to fill up the deficiency previously existing in the web of the tee iron. In any case, the length of the iron required for making these knees is determined by measuring along the back of the bar, but this form is not of common occurrence in girder work, never being adopted from choice, but only inserted where other forms could not be used.

It is very evident that such changes in form of elements as those above described, cannot be made without, at least the risk of straining the fibres of the metal operated upon, therefore, such work should be avoided in all cases if possible, so that the girders may be made of the iron as it comes out from the rolling mill, without any further subjecting to processes requiring the application of heat until the work is rivetted together, and this should be remembered by those who are engaged in designing bridges, or roofs, for though it may be argued that by the introduction of smith work in such structures, they may be made lighter, it may be clearly shown that in the majority of cases they are not made any cheaper, and, as a plain business point in such circumstances, there is a direct loss in having the lighter work, for practically the intrinsic or money value of a bridge lies in its weight for if sold as old iron, it would be at per ton, and workmanship would count for nothing, hence if two bridges are equal in strength, cost and design, it is best to have that where there is most material, in preference to that in which the cost is represented by labour. When angle or tee irons require to be bent in curved forms as for the curves of turn-tables, gas-holders, &c., a mould bar or metal template must be used. Let it be required to curve a number of angle irons for roofing principles; in the first place, one bar must be carefully bent to a true curve to serve as a template for the rest; this bar is then secured on a cast iron bed and the other bars being made red hot are successively bent round it, a few blows of a light hammer being all that is requisite to set it to shape; by this means bars may be very rapidly bent, and, at the same time with great accuracy to any required form.

After the various elements have been shaped to the required forms they are punched or drilled, as the case may be, in the usual manner, and then are ready to be attached to the girders of which they are destined to form parts.

Where the rods or bars with swelled ends are used these must be forged, except they are made by rolls especially adapted for their formation, and, which are formed as shown in Fig. 41 *A* and *B* represent the top and bottom rolls of a mill for rolling links with swelled ends. The ends of these rolls are made with collars *a a*. *b b*, that if a bar equal in thickness to the space between the central part of the rolls be passed *broadwise* between these rolls its centre portion will remain unaltered but its ends will be widened out or swelled as shown at at *c* where *d* is the centre part of the bar and *e e* shows the widened ends of the same after passing through the the mill. If the bar be now passed *lengthwise* between plain rolls as shown at *D* the centre part *d* of the bar becomes *elongated* and the ends *e e* remain unaltered so when the bar has been rolled down to an uniform thickness we have a long link with swelled ends and *no weld in it*, but of solid section from end to end. In this way all such bars when of any magnitude should be made as it is preferable to and far safer than that in which the ends of the bars are made, separately and subsequently shut or welded on to the body, or central part of the bar

Fig. 41



but for round bars the latter course must be adopted, or some other equivalent to it for it is plain that the flat end of a link could not be passed through rolls grooved in a suitable way to form the cylindrical body of the rod. It used to be a common custom to make eyes to the ends of round bars by bending the extremities round on to the body of the bar and then welding them up, but this is not satisfactory as by this method the hole in the eye is not cylindrical throughout, hence affords insufficient bearing for a bolt passing through it.

In making the rods, with eyes at their ends, it is necessary to forge the ends separately, and then shut them on to the round iron of which the ties are to be made, care being taken to keep the centres at the right distance apart, so that the ties may be of the proper length. The holes in the ends may be made in the forging, and subsequently drilled out true, or, if preferred, the holes may be drilled out of the solid metal, which is, perhaps, the best where considerable accuracy of fit is indispensable. Of course great care must be taken in shutting on the ends of such links, in order that the joint or scarf may be equally strong with the body of the bar. Screwed ends of ties and ends for cotters, &c., are in like manner made separately, and subsequently welded on to the body of the bar.

In fitting bolts to eyes in links, and holes in castings, they may be made true fits, the holes being carefully bored out and the bolts as truly turned; or on the other hand both the hole and bolt may be left rough, and a certain play allowed, in order to get the bolt into the holes in the elements it is intended to join together; the amount usually allowed is one-eighth of an inch, so that for a bolt three-quarters of an inch in diameter the bolt hole would be made seven-eighths of an inch diameter, and for a bolt one-and-a-half inches in diameter the bolt hole would be made one inch and five-eighths in diameter, and so forth.

All the different elements of the structure having been duly prepared, they are ready to be put together in the erecting shop, and rivetted up so far as is consistent with the requirements of subsequent conveyance from the manufacturer's yard.

The girders are erected on a series of bars or blocks, supported on iron brackets firmly set in the ground; these brackets may carry cross bars, which are so arranged as to admit of adjustment, so that their upper surfaces may be placed at the same level, or disposed to give the girder any required amount of camber. On these bars the bottom plates of the girders are duly laid, and so piece by piece they are built up until the full height is reached, the various parts being held together by bolts or cotters passed through some of the rivet holes, only a sufficient number being used to hold the plates and bars together, while they are permanently rivetted up. When the work is being thus put together, any discrepancy in length of plates, &c., may be ascertained and rectified; and it may then also be seen if the rivet-holes are truly in line through the various layers of material, and if they be not so they may be trued-up by rhymering out the holes, which will do no harm if the work be tolerably accurate, although some engineers are so strict as to prohibit in their specifications the use of the rhymer; we consider this injudicious, as a slight clearing of the hole smooths the edges of the holes at the contact of the plates, and renders their bearing upon the rivets more uniform than it would otherwise be, unless the plates were perforated *dead true*, which, in ordinary bridge-work, is a thing not to be expected.

The work being sufficiently held together, and found accurate, is now rivetted up, either completely or not, according to circumstances. If it is not intended to test the bridge on the contractor's premises, it is sufficient to rivet it together at all parts, except those where it breaks for shipment or carriage, where for the time being it may be held by temporary bolts, but if the bridge is to be submitted to a test before leaving the yard in which it is made, it should be completely rivetted up; even though it will afterwards be necessary to cut out some of the rivets, in order to pull the structure down for shipment, for although bolts may be amply strong to sustain the load, yet they do not (unless turned, which is not done in such cases), hold so closely as the rivets intended ultimately to connect the various parts of the structure; hence the deflection will, in all probability, be much greater if the rivets be left out. and so justice would not be done to the manufacturer, and very possibly the inspecting engineer would not be satisfied.

In ordinary girder-work there is not usually much to be done in the way of fitting, and what there is is not of a delicate nature; hence highly skilled artizans are not required for this purpose, all that needs to be done, consisting in planing bed plates, &c., and boring bolt-holes and turning up bolts and cast-iron friction rollers, and in some cases the bolts are not turned but merely ground bright on a revolving lap or stone, which, however, naturally makes them but little truer than when in their rough state.

In rivetting girders together it is usual to strike up the head of the rivets first with the flogging hammers, and finally to reduce them to a uniform shape and size by placing over them a die with a recess sunk on its face (and which is termed a snap), and striking it with a hammer. In some instances, if too much length of rivet has been allowed for making the head, it will form a slight collar under the edge of the snap and around the head of the rivet; in order to preserve appearances this collar is sometimes cut off, but it is best to let it remain, for in cutting it off the chisel used for that purpose may injure the plate underneath the rivet head.

The work being so far completed, it only remains to oil and paint the work, and to mark it previous to taking it apart, so that, by the aid of key plans, the erector may be enabled readily to re-construct the work on the site for which it is destined.

CHAPTER X.

INSPECTION AND TESTING OF MATERIALS.

The most carefully prepared designs cannot, in execution, give satisfactory results unless the quality of the material used is up to the standard of strength assumed by the engineer in calculating the dimensions of the various elements of the proposed structure, hence it is necessary to be assured that the material used shall be sufficiently good, and this is invariably stipulated in specifications. It is not unusual to specify that iron shall be procured from a certain maker, or of a certain brand, or, in event of that iron not being procurable, that such material as is used shall be equal to it in strength and elasticity. Now the quality of iron, even of the same brand, will vary slightly from year to year, hence, instead of specifying a given brand, it would be more consistent merely to stipulate for a certain amount of strength and elasticity in the material to be used. The first tests in respect to wrought iron consist chiefly in determining its resistance to tensile force, which force it is best qualified to withstand. Plates, flat bars, angle irons, &c., may be considered of excellent quality for girder-work if they do not break under a strain of twenty-three tons per sectional square inch, or stretch before breaking more than one inch per foot of length, or less than half an inch. If the bars stretch over much, the structures made from them will be liable to take a considerable permanent set, but if it will not stretch fairly there is a danger of its snapping under sudden loads, or violent vibration or concussion. The iron should not stretch permanently under a strain of ten tons per sectional square inch, and its maximum load should never in working exceed five tons per square inch, or one half of the load at which permanent set commences, thus, if the material is such as to reach twelve tons, tensile strain per square inch before permanent set commences, it may safely be worked ordinarily at a strain of six tons per square inch.

Bars that have been used for testing should never be used in making a structure, as testing them over their working weight may possibly cause some slight injury to the fibre, which will gradually be augmented by other, even smaller, strains, until ultimately failure ensues, perhaps under a comparatively small load, whereas, had no such excessive test been applied, the work would have stood the weight of the maximum ordinary load for an indefinite period of time.

In compression wrought iron will most generally give way by crippling, hence the strength of members in compression must be secured by making them of a form suited to resist bending tendencies, but the resistance of wrought iron to crushing force should be equal to not less than sixteen tons per sectional square inch, and it should not show signs of failure under a smaller load than eight tons per square inch, then it may safely be loaded in ordinary working up to as much as four tons per sectional square inch.

Cast iron is admirably adapted to withstand compressive strains, both on account of its rigidity, and also by reason of its superior resistance to crushing stress. Good cast iron will not yield to compressive force until it reaches about forty-five tons per square inch, hence seven-and-a-half tons per square inch is perfectly safe as a working strain in compression upon cast iron of good quality, but in tension cast iron breaks at about seven tons per square inch, hence should not be loaded ordinarily with any tensile strain exceeding one-and-a-half tons per sectional square inch.

For shearing strain on wrought iron, four tons is the working strain usually allowed on bolts or rivets if of iron, or six tons if of steel.

When the preliminary tests are made and found satisfactory by the inspecting Engineer, it remains for him to see that the material used in the work under his charge is made of uniform quality throughout, and well and cleanly rolled. In order to be thoroughly competent to undertake this duty, it is necessary he should be well acquainted with the manufacture of iron in all its branches, for there are many apparent blemishes on rolled iron which are not of the slightest consequence as regards the actual strength, whereas defects of a more serious character which do not so obviously appear, often would escape the notice of any one not practically acquainted with the manufacture of wrought iron.

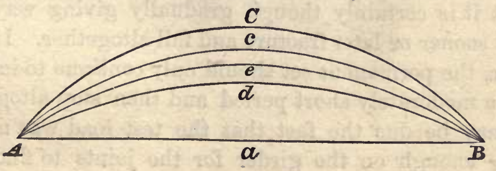
We will now proceed to briefly refer to the testing of entire structures, such as girders, bridges, &c., in their complete form. In these tests there is not only the strength of the structure to be verified, but also the quality of workmanship put into it, especially in the joints and articulated parts of the work, which will be found to yield more or less uniformly, according to the care which has been taken in the manufacture.

The curvature, or camber given to a bridge, which is usually one inch rise at centre for every forty feet of span, is intended to secure that it shall not deflect below a straight line when its greatest load comes upon it, and if the work is well done this is generally found sufficient to ensure that result.

When the bridge is ready to be tested, a diagram should be carefully drawn to show the exact curvature of the bridge as it stands, then the supports are gradually removed from beneath, so that it has then to sustain its own weight; the diagram of deflection under this load may now be taken, and the work then by degrees loaded up to the full test load which it is required to sustain, when the diagram of the curve due to this maximum load is to be noted.

The test load having been allowed to remain on the structure, for such time as may be thought sufficient, in order that all the joints may take their proper bearings, it may be removed and another diagram drawn showing

Fig. 42



to what curve the bridge has returned, these diagrams may be blotted on a sheet of paper, in the form shown at Fig. 42. This diagram should be for clearness drawn to a distorted scale, that is to say, the horizontal scale may be half-inch to one-foot and the vertical scale natural size, then the curves of deflection can be more accurately judged. *A a B* is a Horizontal line drawn between the piers upon which the bridge or girder to be tested, is supported, and *A C B* the curve showing the position of the of the bottom flange of the girder as constructed and before the supports upon which it has been built, are removed.

On removal of the blocks, the girder assumes the position indicated by the curved line *A c B* and when the total load is added *A d B* shows the curve of ultimate deflection, on removal of the test load the girder will return to some position more or less nearly approaching the curve *A c B*. This curve shows the permanent deflection or permanent set of the girder, and of this and the causes whence it arises, it is necessary to make a few remarks here.

If the bridge is properly tested, the load being allowed to remain on it for a sufficient length of time, the deflection curve should never afterwards descend below *A d B* so long as the test load is not exceeded, nor should the permanent set of the girder increase, and if the permanent set be found to increase continually, this will indicate that the work is not sufficiently strong and that it is certainly though gradually giving way, and will sooner or later fracture and fail altogether. If however, the permanent set should only continue to increase for a moderately short period and then stop altogether, it may be due the fact that the test load was not left lnog enough on the girder for the joints to find their bearing, but it does not indicate any danger of future failure. The cause of the deflection from the curve *A C B* to *A e B* is duplex in its nature being partly found in the weight of the structure itself and partly in the settlement due to the various joints which occur throughout the work.

Rivetted joints will generally give a little before all the rivets take a fair bearing, and to this is due the permanent set, over and above the deflection due to the weight of the structure itself. If, in taking permanent set, the girder assumes the form of a tolerably true curve, it may be concluded that the workmanship is good, and the joints uniformly made throughout, but if an irregular line of permanent set occurs, it indicates slovenly work, and rivets which do not nearly fill the rivet holes. An irregular deflection curve under a maximum load uniformly distributed, indicates a want of uniformity in the strength of the girder, showing that the various sections have not been all proportioned in the same ratio to the strains to which they are subject. If the permanent set continually increases, it is certain that either the iron itself is overstrained, and is gradually stretching to its breaking point, or that the joints themselves are giving way by degrees. In the former case there is no remedy but to reconstruct the work, but in the latter, the joints may be renewed.

CONCLUSION.

In concluding the present treatise, a few general remarks upon the subject considered in the previous pages, are necessary in order to state the comparative merits of various systems of construction and their adaptability to certain specific requirements.

To commence with bridges, it is to be observed that there are two main purposes, for which bridges are required, first to carry railways, and secondly to carry miscellaneous traffic, such as occurs on ordinary carriage roads. In the first case, the load in its full intensity, passes from one end of the bridge to the other, being a concentrated rolling load, whereas on a road bridge, the load is more uniformly distributed as it usually consists of a number of comparatively trifling loads passing in both directions, so as to nearly maintain the symmetry of the load, and correspondingly that of the strains. In order that the strains on an arched girder may be only thrust, it is necessary that the load should be uniformly distributed over the whole length of the arch, otherwise if one haunch be loaded more than the other, there will be a tendency to distort the arched ribs, and of course this will produce a bending strain on the arch, a strain for which it is not especially designed, hence it is evident that an ordinary arch though very suitable to carry a roadway bridge, if by no means so well adapted for a railway bridge. For a roadway bridge of any dimensions, where taste has to be regarded, it may be indeed said that we have but two forms of structure to chose from, viz., the arched bridge and the suspension chain, either of which well afford a good basis for an elegant structure, but those of the latter description are not sufficiently rigid to give results as satisfactory as are shown by arches.

For railway bridges, however, tied arches, or bowstring girders, may be used, more conveniently as the bracing bars between the arch and the tie, uniting the extremities of the former in a very great degree, resist the tendency to distort the bow, hence, when large spans are required, this form is not unfrequently adopted. In very large girders, with parallel flanges, there appears to be a considerable element of unsteadiness, inasmuch as a great proportion of the load on the piers may be regarded as passing through the end uprights or pillars of the girders, and being, as it were, resting on the tops of them; if, however, the bowstring form be used the weight is delivered from the arched member directly on the pier or foundation plate, upon which it rests, and thus obviates the objection found to the loftier girders.

Lattice girders present many advantages for railway bridges of large spans. In the first place, they both look light and really are light if properly designed. They are easily erected, and convenient for carriage and shipment, consisting as they do chiefly of elements of small section in proportion to their length, whereas, in plate web girders, the web plates, being of a square or oblong form, are awkward to pack or to move about. For foreign work, Warren girders, being composed of bars which form equilateral triangles, have been used to an exceedingly great extent, not only on account of the convenience of packing them, but also because of the readiness with which they may be erected *in sitû* in countries where it is impossible to obtain skilled labour, in such places, in fact, whatever style is adopted, it frequently becomes necessary to build up the work without doing any rivetting whatever on it, and then it becomes imperative to join the different elements of the bridge together by means of bolts, which should be turned, and accurately fitted to drilled holes, so as to hold as tight as rivets.

The flooring of bridges of course depends upon the purpose for which they are required, and the convenience of getting materials in the locality where it is to be erected. For railway bridges, corrugated iron sheets are frequently used, or timber planking, if timber is plentiful.

For road-way bridges, buckle plates have been, perhaps, more extensively used than any other kind of covering, but they are now being superseded by simple curved iron plates, tied at the edges, so as to prevent their springing. Cast-iron plates which were formerly used, are scarcely ever now applied.

Roofs may be divided into two classes, large and small roofs. For the former it is necessary to adopt strong ribs to form the principals, and for this the arch or trussed arch is extremely useful. The trussed arch is in fact a bow-string girder, of which the tie has a very considerable rise in the centre of the span, being held up by vertical or queen rods, which attach it to the arch above. These principals are often placed at considerable distances apart, in which case it is necessary to have very rigid purlins, and not unfrequently lattice girders are employed to do this duty.

For small spans, the various kinds of triangular or trussed principals, simple and compound are used, but they are not suitable for spans of great width, not having sufficient rigidity.

The coverings of roofs are very various, according to the purposes to which they are applied; thus for storehouses and sheds, corrugated iron is sufficient for the purpose. For dwelling-houses, gas-houses, &c., slated roofs are very generally used; in this case the purlins have to be put very close together if the roof is not boarded previously to having the slates fastened on, in fact the distance between them is only ten and a-half inches generally, and the purlins consist of small angle irons one and a-half inches by one and a-half inches, by a quarter of an inch thick; for roofs of this description the principals should not be placed farther apart than six feet or else, the purlins will not be sufficiently rigid to sustain the weight of the super-imposed slates.

Glass, though heavy, is very frequently used, partially or entirely, for the roof coverings of public buildings, railway stations, and other places where much light is required; and in some tropical climates the roofs are, in some instances, covered with layers of sand or concrete three or four inches thick, in order to exclude the heat, which would otherwise be unbearable.

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