inner edge of the section b will probably be equal to the stress in B, or nearly so.

The compressive stress upon the bearing area of the pin will of course be equal to the gross load, but in calculating its intensity it is not quite certain how the area of the bearing surface should be reckoned. Sir C. Fox takes the whole semi-cylindrical area, so that when the diameter of pin is $0{\cdot}66\ B$, the average intensity of compressive stress on this area would be nearly equal to the tensile stress in the shank of the bar, but would be still greater if the effective bearing area is taken as equal to the diameter of pin multiplied by the thickness of the bar. It may seem surprising that the metal should resist so great a pressure without bulging or upsetting, but this may perhaps be explained by the fact that the compressed particles receive so much extraneous support on all sides that they are not permitted to bulge; and it is well known that even a very weak material, when supported or confined in this way, will exhibit an almost unlimited weight-bearing capacity.

158. Steel Eye-Bars.—It does not by any means follow that the proportions determined by experiment for wrought-iron eyes will apply also to steel, and it has been remarked that some difficulty has been experienced in the formation of steel eyes, which seems to indicate that a different set of proportions may yet have to be found to suit this material.

In the steel cantilever bridge recently erected across the St. John's River in Canada, with a central span of 477 feet, the upper member of each cantilever has been formed of steel eye-bars, each link having a length of about 26 feet, and being composed of either two or four bars, whose section varies from $7'' \times 1\frac{3}{16}''$ to $10'' \times 1\frac{5}{8}''$ in the different panels of the bridge. The author is not aware what proportions of head were adopted in this case, but the diameter of pin is from $0{\cdot}7$ to $0{\cdot}75\ B$.

It has recently been stated[1] that the Edgemoor Iron Company of Wilmington, Delaware, are now manufacturing steel eye-bars "by a new method of upsetting, without buckling or welding," by which the full strength of the bar is developed with proportions of head which differ considerably from those of the wrought-iron eye-bars above mentioned. When D is less than B, the width $b+b$ is made equal to $1{\cdot}50\ B$; but this width across the eye is reduced to $1{\cdot}40\ B$ when D is greater than B—thus reversing the rule observed by Mr. Shaler Smith in hammered and hydraulic forged eyes. At the same time it is stated that these values of $b+b$ (which were provisionally adopted) may be reduced to $1{\cdot}40$ and $1{\cdot}30$ respectively, while still maintaining the full strength of the eye-bar; and that in some experiments the width across the eye has been reduced to $1{\cdot}20\ B$, and the bar has still broken through the shank. The experiments here quoted were made in 1885; the material of these bars was a very mild and ductile quality of Bessemer steel, having an ultimate strength of 60,000 to 67,000 lbs., with an elongation of 31 to

[1] *Vide* Mr. Wilson's paper on "Specifications for Iron Bridges," in the *Transactions of the American Society of Civil Engineers*, June 1886.

40 per cent. in 8 inches; while the pins were of large diameter, varying from $1{\cdot}11\ B$ to $1{\cdot}30\ B$, the largest pin being used along with the smallest excess of width across the eye.

159. Rivetted Joints.—In plate-built members with rivetted joints it is not practicable to obtain the full strength of the solid section of plate; the loss of strength may be minimised by a special arrangement of the rivets, but in the ordinary practice of construction a considerable percentage of the gross section is lost at the rivet-holes. In boiler work the pitch of the rivetting is necessarily very close, and when single rivetting is employed (as sketched in Fig. 180), the strength of the joint is generally not much more than half the strength of the plate, *i.e.*, the "efficiency" of the joint is about equal to $0{\cdot}50$; while the efficiency is only increased to $0{\cdot}66$ or $0{\cdot}70$ when the rivets are arranged in two rows, as in Fig. 181. But in bridge work a greater efficiency is easily obtained by a judicious arrangement of the rivets; and in all cases the object aimed at must be to secure an equal strength in every part of the joint, and to make that strength approximate as nearly to the full strength of the plate as may be practicable with ordinary methods of workmanship.

For this purpose it is chiefly necessary to consider two points, viz.—

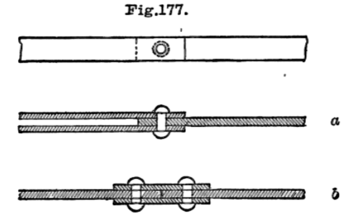
1. The strength of the rivets in resisting the shearing stress; and
2. The strength of the plate across the line of easiest fracture through the rivet-holes.

And in connection with these it may also be necessary to consider the bearing area of the rivets, and the tendency to upset the plate under the direct pressure of the rivet.

160. Shearing Stress on the Rivets.—Disregarding the frictional resistance of the plates, which are forcibly pressed together by the grip of the rivet, and assuming that every rivet in the joint does its fair share of work, the shearing stress on each rivet will be equal to the total pull of the tie divided by the number of rivet sections that must be sheared in order to pull the bars asunder. The number of rivet sections will either be equal to the number of rivets or to twice that number, according as the rivets are in "single-shear" or in "double-shear."

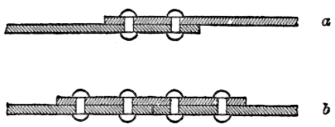
Thus the rivet shown in Fig. $177a$ must be sheared through at two sections before the tie can be pulled asunder, and the same with regard to either of the two rivets shown in Fig. $177b$. These rivets being in "double-shear," the pull of the tie is, in each case, divided equally between the two ends of the rivet, and the shearing stress at each section is therefore equal to half the pull of the tie.

Fig.177.



But when the rivets are in "single-shear," as illustrated in Figs. $178a$ and $178b$, each rivet has only to be sheared through at one section; and if the number of rivets were the same as before, the shearing stress on each would be twice as great. In the Figs., *a* represents a lap-joint, and *b* a butt-joint with cover plates, and the Figs. show the comparative number of rivets required in each case, the shearing stress per rivet section being the same in all the Figs. Apart from the theoretic advantage of placing the rivet in double-shear, it is evident that this arrangement puts the rivet in a much better position to do its work, and avoids the one-sided pull which takes effect in single-shear, and which would doubtless be attended with an injurious twist or bending of the rivet, if that effect were not in great measure resisted by the firm grip which is produced by the contraction of the rivet in cooling.

Fig.178.



161. Shearing Strength of Iron Rivets.—The resistance of wrought iron to a shearing stress, is nearly equivalent (per square inch of sheared section) to the ultimate strength of the same material under direct tensile stress, or at least equal to $\frac{9}{10}$ths of that quantity; so that if the iron were of the same quality in rivets and in plates, a working-stress of $4\frac{1}{2}$ tons per square inch (at least) might be taken as equivalent to a working tensile stress of 5 tons per square inch in the tie. But rivet iron is generally stronger than plate, and frequently as much as 10 per cent. stronger; and some engineers adopt the rule of making the aggregate sectional area of the rivets equal to the net sectional area of the plate taken through the rivet holes. On the other hand there is the contingency that the stress may be more severe on some of the rivets than on others, and this may arise from several causes—the holes may not be punched quite accurately, and may have to be rimered out to bring them fair, and from this or other causes the rivet may not entirely fill the hole; and even if this is avoided by careful workmanship or by machine-drilling, the stretch of the plates, or the unsymmetrical arrangement of the rivets, may easily produce an unequal distribution of the stress.

To provide for these contingencies, the aggregate sectional area of rivets is generally made greater than the net section of the plate by about 10 per cent., and some engineers would increase it by 15 or 20 per cent., and would apply this percentage to the sectional area of rivets as calculated from their *nominal* diameter, notwithstanding the fact that the holes are generally punched $\frac{1}{8}$th inch larger, and are supposed to be filled by the rivet when the latter is properly upset in the process of rivetting.

In view of these several considerations, the shearing stress per square inch of rivet section, which will be equivalent to a working tensile stress of 5 tons per square inch in the plate, may be taken as either $4\frac{1}{2}$ tons or 4 tons per square inch, according to the character of the joint and of the workmanship. For ready calculation the following table gives

the value of the pull T (in tons) which can be taken up by the shearing strength of one rivet section, and also the decimal coefficient N, representing the number of rivet sections required to resist a pull of 1 ton, so that the required number of rivet sections for any joint may be found by multiplying the pull of the tie (in tons) by the coefficient N.

TABLE 3.—*Strength of Iron Rivets.*

Nominal Size of Rivet.		Shearing Stress taken at $4\frac{1}{2}$ Tons per Square Inch.		Shearing Stress taken at 4 Tons per Square Inch.	
Diameter.	Sectional Area.	*T.*	*N.*	*T.*	*N.*
Inch.					
$\frac{5}{8}$	0·3068	1·38	0·725	1·227	0·817
$\frac{3}{4}$	0·4417	1·987	0·503	1·767	0·566
$\frac{7}{8}$	0·6013	2·706	0·370	2·405	0·416
1	0·7854	3·534	0·283	3·142	0·318

These values are very often used without paying any regard to the intensity of pressure that may take effect upon the bearing area of the rivet; *i.e.*, the diameter of rivet × thickness of plate; although some experiments appear to show that the shearing strength of the rivet is really affected by this question, or at all events that it is reduced if the intensity of bearing pressure exceeds a certain maximum.

162. Shearing Strength of Steel Rivets.—For Landore-Siemens steel having a tensile strength of 26 to 30 tons, the shearing strength is found to be 80 per cent. of the tensile strength; but in stronger steel having a tenacity of 36 to 52 tons per square inch the percentage is less, and falls to 72 and to 63 per cent. respectively.

Rivet steel has commonly exhibited a shearing strength of 20 to 25 tons per square inch of rivet section, and in the numerous experiments that have been recently made by Professor Kennedy and by Mr. Moberly, the shearing strength of the steel rivets was very uniformly equal to nearly $24\frac{1}{2}$ tons per square inch, the tensile strength being about 29 tons.

It would appear therefore that when the calculation of strength is made with a constant factor of safety = 4, the working-stress for rivet steel of this quality may be taken at about 6 tons per square inch; but making the same allowances as before for unequal distribution of stress, the figure may be reduced either to $5\frac{1}{2}$ or to 5 tons per square inch; and the resulting values of T and N will then be as follows:—

TABLE 4.—*Strength of Steel Rivets.*

Nominal Size of Rivet.		Shearing Stress $5\frac{1}{2}$ Tons per Square Inch.		Shearing Stress 5 Tons per Square Inch.	
Diameter.	Area.	*T.*	*N.*	*T.*	*N.*
Inch.					
$\frac{5}{8}$	0·3068	1·687	0·593	1·534	0·652
$\frac{3}{4}$	0·4417	2·429	0·411	2·208	0·453
$\frac{7}{8}$	0·6013	3·307	0·302	3·006	0·333
1	0·7854	4·320	0·232	3·927	0·255
$1\frac{1}{8}$	0·9940	5·467	0·183	4·970	0·201

It must be remarked that the above-mentioned experiments were made with especial reference to boiler work, and the rivets were intended to be used along with steel plates having a tensile strength of about 30 tons. Thus the shearing strength of the rivets (per square inch) was equal to three-fourths of the direct strength of the plates, whereas in wrought-iron work the rivet strength is generally quite equal to the plate strength. It follows that in using steel plates of this quality the joints will require a larger rivet section than in iron plates of the same dimensions; and if a stronger quality of steel plate is used, the rivet section must be still further increased.

It must also be remarked that the shearing strength above given cannot be relied upon, unless the bearing pressure is kept within a certain limited intensity per square inch of bearing area. The relation between the two quantities is not yet made manifest; but Professor Kennedy found that when the bearing pressure amounted to 53 tons per square inch, the shearing strength of the steel rivets was reduced from 24 to $16\frac{1}{2}$ tons per square inch of rivet section; and in another case the rivets sheared at 18 tons per square inch when the bearing pressure was 46 tons per square inch. Of course this reduction of shearing strength may have been partly due to an unequal distribution of stress among the rivets, which was found commonly to reduce the *mean* strength of the rivet section (for the whole joint) from 24 to 22 tons or even to 21 tons; but beyond this ascertained loss, the reduction of strength appears to be due to excessive bearing pressure; and Professor Kennedy recommends that this pressure should not exceed 42 or 43 tons per square inch, for the quality of rivets and plates used in these experiments.

163. Strength of the Plate across Rivet Holes.—In the ordinary practice of iron girder construction, the strength of the plate per square inch of net section between the rivet holes is commonly assumed to be independent of the proportions of the joint; or, at all events, the working stress for wrought-iron ties, whether 5 tons per square inch or what not, is understood as being applicable to that net sectional area. This practice

is to a certain extent justified by the average results of experience, and by such experiments as that mentioned in Article 153; but in the case of steel it is certainly necessary to proceed with greater caution for reasons which were sufficiently exemplified in the same Article.

The most complete experimental results that have been obtained for steel are those summarised in the Report of the Research Committee on Rivetted Joints made to the Institution of Mechanical Engineers,[1] and having reference chiefly to the single or double rivetted joints of boiler work. So far as can be gathered from these and other experiments, the strength of the net section appears to be the algebraical sum of three of four separate gains and losses, due to causes which are quite distinct in themselves, but whose separate effects are not easily to be distinguished in any one experiment. These are—

1st. A *loss* due to the injury caused by punching; the amount of this loss depends on the thickness of the plate; it may be more or less perfectly removed by remedial measures, and is absent in the case of drilled plates.

2nd. A *gain* due to the perforation of the plate, by whatever means; and caused apparently by the consequent equalisation of the stress between the remaining lands, or bars of metal between the holes.

The net result so far is a gain in the case of drilled plates, and may be either a loss or a slight gain in the case of plates which are perforated by punching.

3rd. A further loss, due to the unequal distribution of stress in the width of each remaining land of plate, and its concentration at the edges of the pulling rivets; this becomes sensible when the bearing pressure is too great, just as in the case of eye-bars, but is no doubt modified by the hold of the rivet-heads.

4th. The last-named effect may evidently be intensified if the stress is unequally divided between the several rivets, and if a tear is thus commenced at the side of the most heavily pulled rivet, the effect would seem to be particularly injurious in the case of steel; for it is known that a slight nick inflicted on one side of a steel tie-bar is sometimes enough to effect a very great reduction in its strength.

Summarising the results of the above-named experiments, and some others which have been made at a recent date, the several items of gain and loss appear to have the values quoted in the next ensuing Articles.

164. Injury caused by Punching.—In punching a hard and thick plate, the metal is subjected to intense stresses which produce a molecular change in its structure, rendering it harder and more brittle or less capable of stretching. This effect appears to be confined to a narrow zone of metal surrounding each hole, but it reduces the average strength of the section between the holes to an extent depending on the thickness and quality of the plate, or on the intensity of the pressure required to punch it.

[1] Vide *Proceedings of the Institution of Mechanical Engineers*, 1885.

There is no doubt that the same injury is caused in punching iron plates, but to a less extent than in the case of steel.

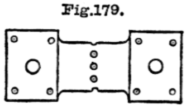
From certain experiments made in 1878 by Mr. W. Parker, and reported to Lloyd's Committee,[1] it was found that thin steel plates lose comparatively little by punching, the loss of strength being only 8 per cent. for $\frac{1}{4}$-inch or $\frac{3}{8}$-inch plates, while for $\frac{1}{2}$-inch plates it amounted to 26 per cent., and rose in some cases to 33 per cent. in the case of plates having a thickness of $\frac{5}{8}$ inch or $\frac{3}{4}$ inch.

The injury occasioned by punching may of course be avoided by drilling the holes instead of punching them; and in large or important girder-work it has frequently been the practice of English engineers to have all the rivet-holes drilled at uniform pitch throughout, by multiple machine-drills—a method which not only secures the full strength of the plate, but also produces an accuracy of workmanship which is quite unattainable by any process of punching.

But when plates have been punched, the injury above mentioned may be remedied to a great extent, either by annealing, which appears to undo the molecular change produced by the punching; or by rimering out the hole and thus cutting away the zone of affected metal. In the latter case the hole is of course punched smaller than the intended diameter, and the hole is then rimered out about $\frac{1}{8}$-inch all round. One or the other of these methods is generally employed in steel work, but in wrought-iron girder-work the holes are very generally punched without adopting either of these remedies.

165. Effect of Perforation.—Apart from the structural injury inflicted by punching, the perforation of the plate has been shown by Professor Kennedy to produce a distinct increase in the tensile strength of the remaining section between the holes. These experiments were made with test pieces of the form shown in Fig. 179, and it was believed that the pull was applied in such a way as to distribute the stress evenly over the whole width in all cases; but in testing the solid plate before perforation, it was found by minute examination that in the vicinity of the line of ultimate fracture, the stretch was much greater in the centre than at the sides of the plate. This unequal distribution of the strain cannot take place when the plate is perforated, as in Fig. 179; because the strain, or elongation of the fibres, is then confined to the very short length of the bars between the holes, and must be nearly equal in each bar. At all events this is the cause to which Professor Kennedy attributes the observed increase of strength, about which there can be no doubt whatever.

Fig.179.



Moreover, it is evident that the narrower we make these intervening spaces the greater will be the difference between the stress intensity at these points and at the broad untouched spaces behind the rivets; consequently the less will be the strain in these broad spaces, and the more

[1] *Transactions of the Institution of Civil Engineers*, vol. lxix.

evenly will it be divided between the several bars or lands. This view agrees, at any rate, with the results of the experiments,[1] which are summarised as follows.

For $\frac{3}{8}$-inch plates of 30-ton steel, holes *drilled* to the diameter d :—

		Excess of Tenacity.
Pitch of rivetting	$p=1{\cdot}9d$	20 per cent.
,,	$p=2{\cdot}0d$	15 ,,
,,	$p=3{\cdot}6d$	10 ,,
,,	$p=3{\cdot}9d$	6·6 ,,

For $\frac{3}{4}$-inch plates of 30-ton steel, drilled holes :—

Pitch of rivetting	$p=1{\cdot}9d$	20 per cent.
,,	$p=2{\cdot}8d$	7·8 ,,

When the holes are punched, these percentages of gain are, of course, subject to a deduction on account of the injury done in the punching, which varies with the thickness of the plate, and will leave a net loss if the plate exceed a certain thickness. Thus Mr. Moberly deduces from his own experiments that if the solid plate is good for 30 tons, the strength of the net section between punched holes will be as follows : [2]—

Thickness of plate	$\frac{1}{4}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$
Ultimate strength of net section	32	31	30	29	28	27

In all cases the plates were of mild and extremely ductile quality, and it is not to be assumed that the results would apply to any stronger quality of steel. It is also evident that the percentage of gain becomes very small when the pitch of the rivetting amounts to four times the diameter; and in bridge work the pitch is seldom less than this, and is generally greater; so that this gain of strength is really inconsiderable so far as our purpose is concerned, but the results are nevertheless important as showing the great effect of any unequal distribution of stress in a steel plate.

166. Effect of Unequal Distribution of Stress.—It has already been mentioned that in the case of eye-bars, the stress is *not* evenly distributed over the sections bb (Fig. 172), and that the strength of these sections is consequently less than that of the solid bar in the proportion $\frac{B}{b+b}$.

Comparing the eye-bar with the rivetted joint shown in Fig. 177, it seems probable that a similar concentration of stress, and drawing out of the metal would take place at the edges of the rivet hole, if the bearing area of the rivet were so small as to produce any elongation of the hole. But this tendency would be moderated by the grip of the rivet head, which must certainly distribute the pull over a certain area of metal

[1] *Vide* Professor Kennedy's experiments on rivetted joints, published in *Engineering*.
[2] Vide *Proceedings of the Institution of Civil Engineers*.

around the hole, and must also diminish the actual bearing pressure upon the rivet shank.

Any loss of strength that may be due to this cause would evidently be missed in experiments made in any such manner as that indicated in Fig. 179; for in these test pieces there were no rivets in the holes at all, and the tensile stress was imposed in a manner quite different from that which occurs in a rivetted joint.

In the case of experiments made with the complete rivetted joint (in mild plates of 30 ton steel) it was found by Professor Kennedy that the plates exhibited a distinct loss of strength amounting to 10 or 12 per cent. when the bearing pressure was 46 tons per square inch, but in most cases the actual loss could not be ascertained, because when the bearing area was too small the joint generally gave way by shearing the rivets.

It was also found that by increasing the size of the rivet head, the strength of the plate per square inch of net section might in such cases be increased by $8\frac{1}{2}$ per cent.

These results were obtained with a closer pitch of rivetting than is generally adopted in bridge-work, and it is probable that any such concentration of stress at the edge of the hole would be proportionally more intense in the case of wider rivetting.

It must also be remarked that these results apply to joints containing a very small number of rivets fitted with great care in accurately drilled holes; and it is evident that when the strength of the joint depends upon the distribution of the pull between a large number of rivets, some of which may fit badly, it may happen that an intense bearing pressure is brought upon one or two rivets, and if the result should be to draw out the edges of these holes and to start a crack in the plate, it would be difficult to estimate the extent to which the strength of the plate might be reduced.

In the case of wrought iron, experience has proved that these contingencies are not more serious than can be covered by the allowances already mentioned; but experiments are still needed to ascertain their effect upon the shearing strength and the plate strength of large joints in steel work.

167. Working-Stress on Oblique Line of Fracture.—When the rivets are arranged in crow's-foot fashion as in Fig. 181, some engineers have assumed that the resistance of the plate along the zigzag line of section is practically the same as along the direct line shown in Fig. 180. At this rate the plate ought never to break along the zigzag unless the dimension s is less than $\frac{b}{2}$; but it is well known that in

Fig.180.

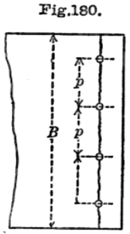
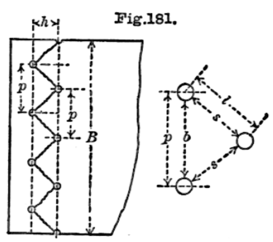


Fig.181.



practice the fracture will generally follow the zigzag unless this line of fracture is considerably *longer* than the direct line.

Experiments are still needed to determine the strength of a section inclined at any variable angle with the line of direct stress, and to ascertain the best value of the ratio $\frac{h}{p}$. In Mr. Moberly's experiments[1] with $\frac{9}{16}$-inch steel plates, the pitch was $p = 3{\cdot}775$, and h was at first made equal to 1·5 inch. The line of zigzag fracture $s+s$ was then equal to $1{\cdot}04b$ to $1{\cdot}10b$ (according to the size of the rivet), but the plate always broke on the zigzag line; and the same thing occurred when the distance h was increased to $1\frac{5}{8}$-inch, making the zigzag equal to $1{\cdot}13b$. But in the $\frac{5}{16}$-inch plates the line of zigzag fracture was increased to $1{\cdot}33b$, and the plates then broke indifferently on either line,—some on the straight and others on the zigzag line.

This result agrees exactly with the practice of some engineers, who make the working-stress per unit of area measured on the zigzag line equal to three-fourths of the working-stress on the direct line. The value of the ratio $\frac{h}{p}$ which would yield this desired proportion depends on the diameter of the rivets, and it is obvious that the rule cannot possibly be true for all varying angles of obliquity; but for ordinary proportions of rivetting the rule is probably near enough to the truth for all practical purposes, and may be taken as equally good for iron and for mild steel.

168. Arrangement of Rivetted Joints.—Figs. 182, 183, and 184 illustrate some of the forms of joint most usually adopted in bridge work. In Fig. 182, each leaf of the cover-plate contains three rows of rivets, with four rivets in each row, but the number will of course depend upon the requirements of each case, and it will often happen that the required rivet area cannot be obtained with less than four or sometimes five rows of rivets. At the section a the whole tensile stress will be borne by the net section of plate, whose width will in this case be equal to $B - 4d$, in which d is the diameter of the rivet hole. The plate is not likely to give way at the section b, and still less likely to break at c, because a portion of the tensile stress has been taken up by the four rivets of line a, and another portion by the rivets of line b; so that theoretically the stress in the principal plate at these respective lines a, b, and c will be as 3, 2, and 1; while the stress at the same lines in the cover-plate will be as 1, 2, and 3.

Fig.182.

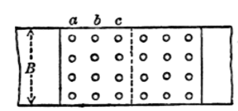


Fig.183.

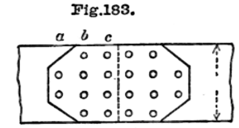
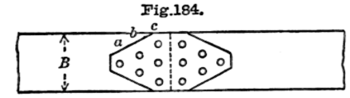


Fig.184.



[1] Vide *Proceedings of the Institution of Civil Engineers.*

Obviously, therefore, the joint may be improved, and the strength better equalised by adopting the form shown in Fig. 183, or still better that of Fig. 184. In the last-named joint, if we suppose that every rivet does its fair share of the work, we shall have the following stresses and effective areas :—

At line *a*, the entire pull of the tie will take effect on the plate-section $B-d$.

At line *b*, we shall have five-sixths of the tensile stress, and the net section will be $B-2d$.

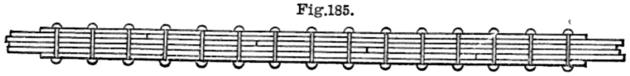
At line *c*, we shall have one-half of the tensile stress with a net section of $B-3d$.

But at this line the stress in the cover-plate will be equal to the entire pull of the tie, and as the cover is pierced with three holes its thickness, or the aggregate thickness of the two covers, must be greater than that of the principal plate.

It will not be difficult to find the requisite proportions of plate and rivet-section for a joint formed in this or in any other manner, and it is unnecessary here to discuss the matter further. The arrangement shown in Fig. 184 is a very efficient joint, but in the ordinary practice of bridge-construction it cannot often be used except in ties composed of a single bar; and in the tension flanges of girders, the rivets are generally arranged in the manner shown in Figs. 182 and 183.

To make up the necessary section in the tension flange of a girder, it is generally expedient, and often necessary, to pile together several layers of plates, as shown in Fig. 185; and in this case it is better to avoid the use of a double-leafed cover-plate at each joint, which may be done by

Fig.185.



arranging the joints of the several plates in consecutive steps as shown in the Fig., so that one long cover-plate is sufficient for the whole series of joints. The number of rivets between any two consecutive joints must of course be proportioned to the shearing stress; and if all the plates have the same thickness, the shearing stress will be the same as for the extreme leaf of the cover-plate, which forms a continuous cover for the whole series.

When the flange is very wide, it will often be more economical to make up each layer in two widths of plate, arranging the widths of the superimposed plates so that the longitudinal joints break joint with one another, as well as the cross-joints.

169. The Practical Weight of Tension Members.—For purposes of computation, the gross weight of a tension member per foot lineal may be expressed as in Art. 67 by $S\gamma_t$, in which S is the total tensile stress and γ_t the practical weight of a tie, per ton of direct stress and per foot lineal of tie, including all waste.

If t denotes the working-stress per square inch of net section, we should have for a single wire, or a tie without joints, $\gamma_t = \frac{\cdot 0015}{t}$; but to make allowance for the loss of section at rivet holes, and the weight of cover-plates and rivet heads in plate joints,—or to make allowance for the weight of the overlapping swelled heads and the pins of eye-bars, we may suppose a gross sectional area which would yield the same total weight, and which is equal to the net theoretical area multiplied by a certain coefficient κ to be derived from actual examples; and we may then write $\gamma_t = \cdot 0015 \frac{\kappa}{t}$.

The coefficient κ must include, besides the above items of loss or waste, the additional metal which is unavoidably employed when the theoretic section varies continuously from point to point, as in the flanges of parallel girders. Its value varies considerably in different types of construction and for any given form the student cannot do better than to take out the actual quantities from a type drawing. The following table may serve, however, as an approximate guide, the values being derived from actual examples of well-designed work of the different classes :—

Table 5.—*Examples of the Weight of Tension Members.*

	Coefficient κ.
1. Single wire, or ideal tie, without connections	1·00
2. Chains of suspension bridges, composed of eye-bars with pin-connections	1·20 to 1·25
3. Tie-bars with swelled heads, including pins at end-connections	1·33
4. Flat ties with rivetted joints	1·30 to 1·40
Do. including overlap at ends	1·50
5. Plate-built girder flanges, under a nearly uniform stress	1·40 to 1·60
Do. including overlap at ends	1·50 to 1·70
6. Plate-built girder flanges under a varying stress, such as the flanges of parallel girders	1·66 to 1·90
Do. including overlap at ends	1·80 to 2·00