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SOME COMMON ERRORS

IN

IRON BRIDGE DESIGN

KERNOT

On Some Common Errors in Iron Bridge Design.

To the Director of the Testing Laboratory with the Authors compliments

ON SOME COMMON ERRORS IN IRON BRIDGE DESIGN.

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On Some Common Errors in Iron Bridge Design.

In the Australian Colonies, as in other parts of the world, there is a large and increasing number of iron (in which is included steel) bridges. These bridges are of ages varying up to about fifty years. Many of them were designed at a time when the proper mode of proportioning the various parts was but imperfectly understood, while in some the material and workmanship is by no means up to the modern standards. Unlike wine, bridges do not improve with age—on the contrary, corrosion is always going on, sometimes rapidly, but generally very slowly, but no less surely, and is bound sooner or later to cause a perceptible diminution in strength. It is also thought by many that there is a tendency for the metal in course of time to become hard and brittle and so less able to endure shocks. Thus the bridges are without doubt growing weaker with effluxion of time. Meanwhile the loads they have to endure show a distinct tendency to increase. Steam rollers, traction engines, and other specially heavy loads, undreamt of at the time our earlier bridges were designed, are now common, while locomotives, with the universal call for more power, become constantly larger and heavier, and powerful continuous brakes, unknown when the earlier bridges were built, introduce longitudinal stresses of serious magnitude. From these combined causes it is plain that the margin of safety is steadily diminishing, and it is only a question of time for the point of absolute danger to be reached.

Again, there is reason to believe that many parts of the older bridges are excessively and unnecessarily strong while other portions are weak, and that the general arrangement of parts is often far from the most economical.

It appeared, therefore, that a criticism of existing bridges would be useful not only to the designer of new structures anxious to avoid the defects of the older ones, but also, and perhaps in an even greater degree, to the man who has received a legacy of imperfect structures from his predecessors, which he is desirous of utilizing as far as possible by judicious repairing and local strengthening, for it is to be noted as a good point of many of our defective bridges that they are like chains, most of the links of which are abundantly strong while occasionally a very weak one is found, which governs the strength of the whole and that thus a comparatively inexpensive local reinforcement may improve the whole structure to a very large and valuable extent.

I shall now proceed as briefly as is consistent with clearness to point out what I consider to be the principal errors in structures that have come under my notice, and indicate how their defects may be remedied, if remediable, in existing and avoided in future structures.

1. *Disproportion of foundation area to load carried.*—If a foundation is too small it gives way partially or wholly, injuring or destroying the structure; if too large it stands but represents waste of money. In every instance however some slight yielding when the load is applied takes place, and it is desirable, especially if continuous girders are employed, that all the supports should yield equally. Hence all foundations should be proportioned to the load carried—that is to say, under full load the pressure per unit area on the supporting material should be throughout equal. In calculating this pressure, it is to be remembered that it is not the total load on the foundation surface that is to be considered, but the excess over the load that existed previously. For example, at the great Hawkesbury Bridge, N.S.W., it has been stated, that the pressure on the foundation is 10 tons per square foot, and this is obtained by dividing the total weight of the structure by the area of foundation. But in order to reach the depth required a very large quantity of earth had to be removed, and the foundation was relieved to that extent. The true or effective pressure on the foundation is therefore the difference between these two amounts, and actually is only 5 tons per square foot. This I submit is the correct way of stating foundation pressure.

There is a further qualification, however, and that is the allowance for the effect of friction of earth upon the sides of a bridge cylinder or caisson, and if this be taken into account, the pressure on the base is still further reduced. This friction is somewhat variable and has been stated as high as 800 and as low as 50 lbs. per square foot in different strata.

Directing our attention to existing structures, great discrepancies appear in the size of cylinder foundations, not only between one structure and another, but between different piers of the same structure. For example the Intercolonial Railway Bridge at Albury consists of two continuous spans of 160 feet each, carried on three piers, each consisting of two cylinders of 10 feet diameter. The centre pair of these cylinders carry $\frac{10}{16}$ of the load, while the two end pairs together carry only $\frac{6}{16}$. Thus, while two cylinders carry a load represented by the number 10, four of equal size are provided to carry a load of 6 only, and these four are further surrounded by earth to a much greater height than the central ones, and therefore receive greater frictional support. It cannot, I think, be disputed that the bridge would have been both cheaper and safer had the end cylinders been reduced to 6 feet diameter, or even less, for then any yielding would have been approximately equal throughout, and the distribution of bending moment in the continuous girders consequently undisturbed. Similar remarks will apply to the Railway Bridges at Wagga, Bathurst, and Aberdeen, described in the Report of the Royal Commission on Railway Bridges, N.S.W., 1886. In all of these the terminal cylinders though carrying less than half the load, and more favourably circumstanced in other respects, are just as large in diameter as their heavily loaded companions, see Fig. 1, which represents to scale the railway bridge at Aberdeen, N.S.W. A reference to numerous successful cylinder and caisson bridge foundations leads to the conclusion that the subjoined are safe foundation pressures, the most unfavourable combination of load, wind and flood, being employed in the calculation. Rock 10 tons per square foot at least. Fine compact sand at considerable depths, 6 tons per square foot. Very good clay 5 tons per square foot. Ordinary sand, clay, or loam 1 to 3 tons per square foot. Knowing then the superincumbent load and the nature of the material there should be no difficulty in proportioning the cylinders of future bridges. As for those in existence, nothing can be done, but as they usually err on the side of excess there is not much cause for alarm.

2. *Excessive and disproportionate size of columns.*—By the term column is meant that part of the structure extending from the foundation to the girder seat. Its size is often made equal to that of the foundation, but there is no necessity that this should be the case, for while the size of the foundation depends on the resistance of the material upon which it rests, that of the column depends upon the material of which it is made and which sometimes offers a greater resistance per square inch than the foundation does per square foot. For the sake of lateral and frictional support, the cylinder is usually, and properly carried up the full size from the foundation to the surface of the ground. Above this, however, there is no reason why it should not be as economically designed as any compression element of the superstructure. In many of the older bridges the columns are of most unnecessary size, adding seriously to the cost of the structure, and impeding the flow of water in the case of river bridges in an undesirable manner. This is certainly the case with the older New South Wales railway bridges already referred to, and also with some in Victoria. As examples of what has been successfully done in the way of reducing this part of the structure to reasonable and economical proportions, two structures may be cited. The first is the Johnston Street Bridge, Collingwood, near Melbourne, shown in Fig. 2. This is an iron bridge built about 20 years since by C. Rowand, Esq., C.E., to replace a large timber arch that failed through decay. It consists of three spans of nearly 60 feet each, extending between the stone abutments of the old timber arch, and having as intermediate supports wrought iron columns filled with concrete, which for slightness present a most extraordinary contrast to the usual practice at the time it was built. Their dimensions are as follow:—

Height from top of cast iron cylinder to girder seat	45 feet
Diameter	2 feet
Thickness of metal	$\frac{5}{16}$ inch
Dead load for each column... ...	40 tons
Live load for each column	50 tons

Each pair of these columns supports an area of bridge decking 70 feet long and 32 feet wide.

The proof of the practical success of these columns is in every way most conclusive, for not only is the bridge on an important main road with heavy traffic, but it is also at the part of the Yarra where the hydraulic conditions are of the severest kind. During the great flood of July, 1891, when two iron bridges were washed away and hundreds of suburban dwellings inundated, the water stood at the level shown in Fig. 2. The gradient of the flood surface for 50 chains above the bridge was at the rate of over 5 feet per mile, the hydraulic radius about 30 feet, and floating timber and other wreckage abounded. Nevertheless these slender columns stood absolutely uninjured, and that, although the bracing between them is by no means as massive as, in my opinion, it should be.

The second example is the bridge carrying the North-Eastern Railway over the Racecourse Road, Flemington, near Melbourne. The railway is double line and is traversed by a busy suburban traffic propelled by tank engines of 49 tons weight. The bridge is situated at the entrance of the Newmarket Station and is exposed to the constant action of the Westinghouse brake. There are two spans of 51 feet each (discontinuous), four main girders to each span, and the central support consists of four columns each made of four $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ angles of mild steel, with single rivetted lacing. The foundations are of Victorian bluestone, a 3 inch cube of which crushes with 40 tons pressure, and are $2\frac{1}{2}$ feet square for each column. The compressive stress on the metal of the angles is 4 tons per square inch. The columns are 15 feet high from stone foundation to girder seat and are 18 inches square.

Strange to relate a second railway, carrying a practically identical traffic crosses the same road at a short distance, and here the columns are of cast iron filled with cement, 2ft. 3in. diameter, 1 inch thick, and the girders 44 feet span. Judging from experiments made with the University testing machine it would take 300 tons to crush a column of the former bridge and 4000 tons to crush one of the latter, and yet the latter carries a smaller load than the former.

There is no doubt whatever that considerable economies are to be effected in the design of columns of future bridges by abandoning old arbitrary methods, and treating the problem scientifically.

As a further instance of economical design, Kinzua Viaduct, New York State, U.S.A., may be quoted, a section of a wrought iron column of which is shown in Fig. 3. Two of these columns form the support intermediate to a span of 61 and one of 38 feet, carrying a main line of railway 4 ft. $8\frac{1}{2}$ in. gauge. Each column is 279 feet high and is braced laterally at intervals of 31 feet. Further comment is needless to show how excessively wasteful bridge columns in Australia have been in cases too numerous to mention.

3. *Girders supported in an unfavourable manner.*—Under this heading come many defective arrangements. The first of these is when a girder is supported at the extreme end, it being possible to support it at a more favourable point. It does not seem to have been generally recognised by engineers that the extremity of a girder is a most unfavourable point of support, giving rise to bending moments and shears of maximum value, and therefore should not be adopted except under the most cogent conditions. If the points of support of a uniformly loaded beam are moved towards the centre, the most surprising diminution both of bending moment and shear takes place, and when the supports are distant from the ends by ·207 of the length the maximum bending moment is reduced to ·172 and the maximum shear to ·59 of what it is when the supports are terminal. As in the great majority of beams including rectangular sections of timber and rolled girders of usual proportions, the strength is regulated by the moment and not by the shear, this means that a uniform beam supported at what I propose to call the "efficient points," is very nearly *six times* as strong as a similar one supported at the ends, the load being uniformly distributed. With a live or variable load, such as a crowd, the advantage is not so great, the stress range being somewhat increased, but even allowing for this in accordance with the Weyrauch vibration formula, the strength of a beam carrying an equal live and dead load—a very usual case—is increased about threefold by its being supported at the efficient points instead of the ends. Thus

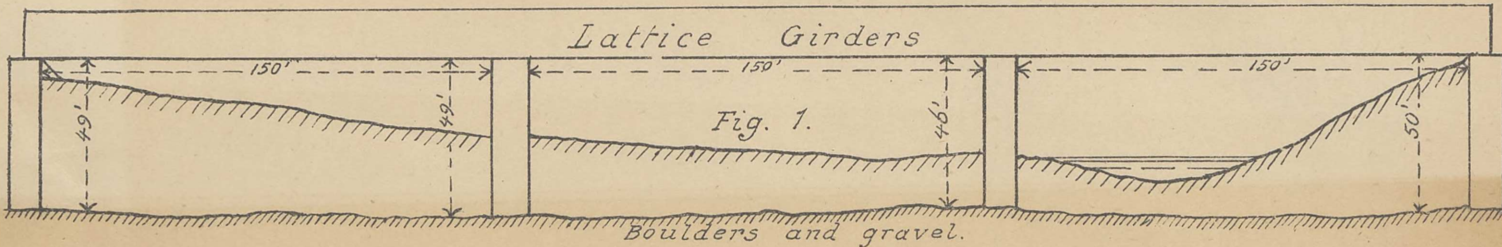


Fig. 1.

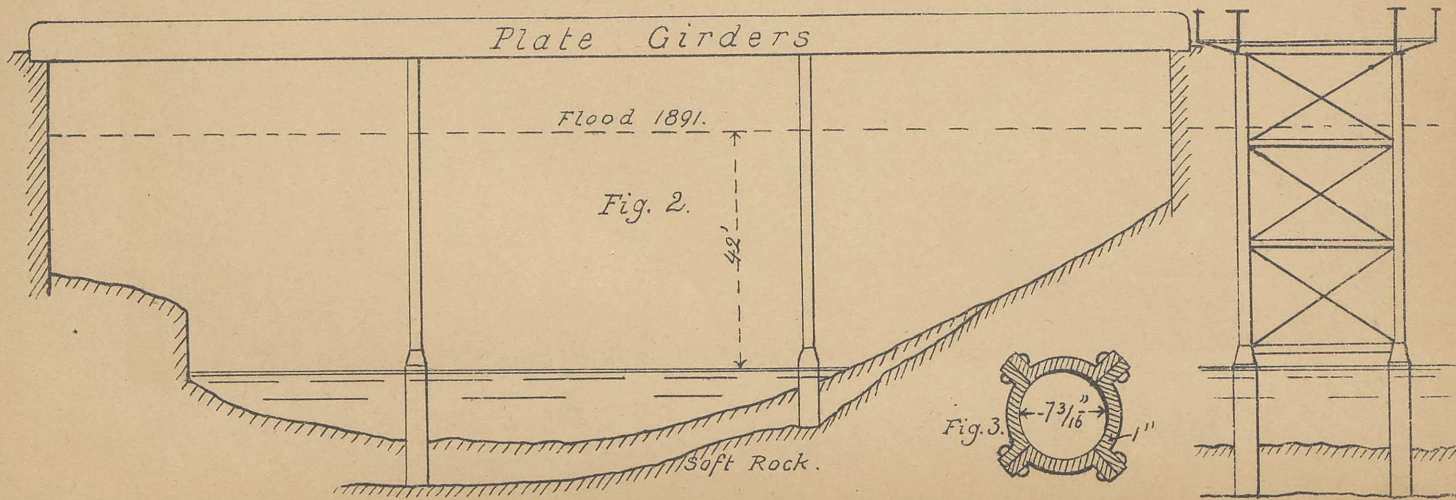


Fig. 2.

Fig. 3.

it will be seen that there is an enormous advantage in supporting both main and cross girders at their efficient points, or as near to those points as is possible, and this is true in all cases but most especially so when the dead load is large.

Should it be obligatory to support the ~~beam~~ at one extremity, the position of the other support being optional ~~obtained~~, the efficient point is found to be ·29 of the length from the other end, and the maximum bending moment is almost exactly one-third of what it would be were both supports terminal, the load being uniformly distributed.

Should it be inconvenient to adopt so large an overhang as above mentioned, considerable advantage may still be obtained with a very moderate amount. For example, a uniform beam uniformly loaded and supported at points one-eight of its length from the ends endures a maximum bending moment of only one-half of what it would if supported at the extremities.

As an instance in which an overhanging end might have been advantageously applied certain bridges on the Melbourne and Coburg Railway may be cited. Here, as shown in Fig. 4, the terminal support of the girder is a brick pier imbedded in the embankment, and the girder seat is surrounded with earth in an undesirable way. By moving the pier to the position shown in Fig. 5, the bending moment would be reduced, and the girder seat rendered accessible.

A case of unfavourably arranged support is illustrated in Fig. 6, which represents two bridges each crossing six lines of railway in Yarra Park, Melbourne. The girders are of the lattice type, supported at the ends by a double system of standards or legs. Calculation shows that the compressions on the diagonals AB and CD are equal. But the tension on AC is equal to the compression on AB as their horizontal resolved parts balance at A. Hence the vertical resolved part of AC balances that of CD at C, and the leg CE carries no weight whatever, except the actual small floor load at C.

Now, had the bridge been arranged as at Fig. 7, the main girder would have been shortened by ten per cent., the stresses throughout the remainder would have been reduced considerably, the leg FG would have been saved and the whole structure largely reduced in length, thus economising flooring, hand-railing and area of ground occupied. This, it may be added, has been done in several more recent structures of a similar kind.

A third case of unfavourable support is where the girder seat is not placed centrally to the cylinder or column. This causes the stress to be greater on one side of the column than the other, and involves a tendency for the column to lean over if in soft ground. An instance of this is to be seen in an important bridge illustrated in "Engineering," vol. 43, p. 117.

Such errors as these are usually irremediable in existing structures. It is desirable, however, to guard against their repetition in future designs.

4. *Imperfect expansion apparatus.*—Variations of temperature affect all metal structures, and unless properly provided for may cause extra stresses of serious amount in the metal work and dislocation of the brickwork or masonry of the supports, such dislocation unfortunately being only too apparent in not a few existing structures. The extreme temperature range in Melbourne from the Observatory thermometer records is about 150° Fahrenheit, but it is not probable that the change of temperature of considerable masses of metal work such as used in bridges will be more than 120°. As iron and steel expand
49°C. not quite one part in 800 between freezing and boiling, a range of 180°, the expansion for 120° will be one part in 1200 or 1 inch in 100 feet. This simple and easily remembered rule is safe for the Victorian climate, but would not suffice for places such as New York, where the temperature range is greater in both directions than in Melbourne.

A metal structure then should be anchored or fixed at some definite point and be allowed free movement everywhere else. The point of anchorage may advantageously be near the centre, so as to divide up the motion. If this is not convenient the anchorage should be at the firmest or most solid support. If for example one end of a bridge be on solid rock and the other on a tall and somewhat flexible support, the former should be made the anchorage. At the Victoria Street Bridge, near Melbourne, the opposite course was adopted and the whole bridge was thrown somewhat out of position by movement of the support and involved the necessity of its being detached and moved back to its original position—a troublesome operation.

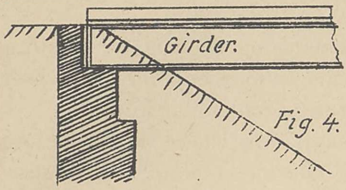


Fig. 4.

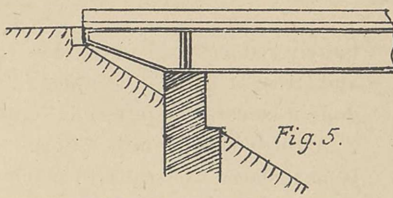


Fig. 5.

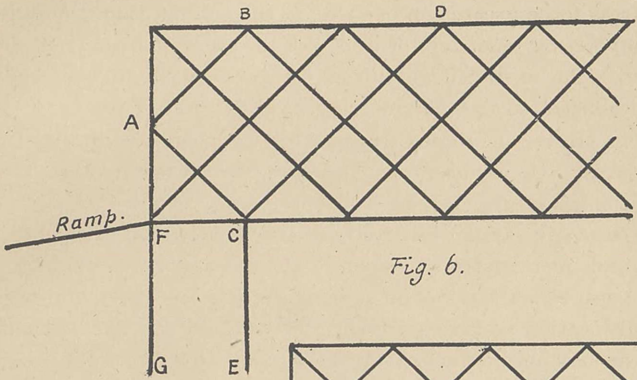


Fig. 6.

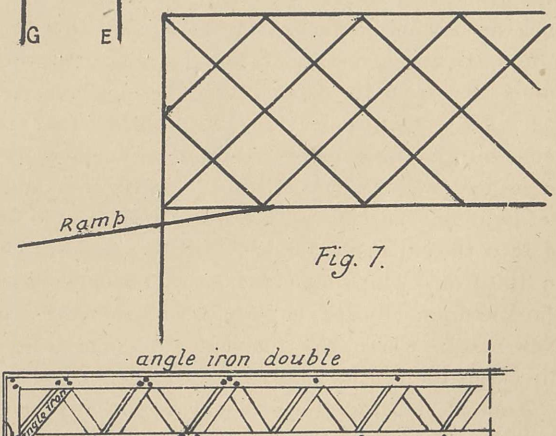
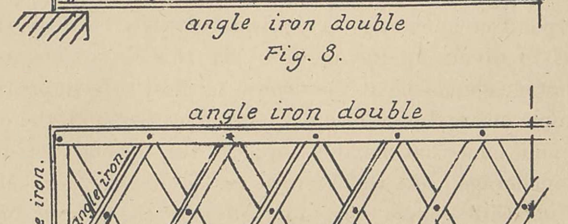


Fig. 7.

angle iron double



angle iron double

Fig. 8.

angle iron double

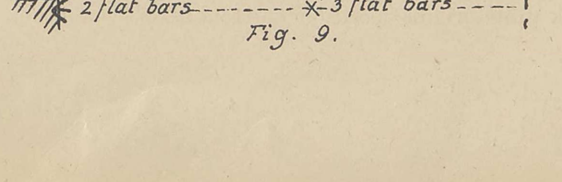


Fig. 9.

Girders of less than 100 feet span are usually supported on planed surfaces of metal which slide one upon the other. In some cases pressures of over a ton per square inch have been imposed on such surfaces with success, but usually the pressure is much less. Larger bridges are as a rule supported on rollers and it is desirable that these be of ample diameter and so placed as not to be choked up with dirt or injured by moisture. Rollers 4 inches diameter are allowed $\frac{1}{2}$ ton load per inch length by good American authorities, but this pressure is often exceeded in English practice and apparently with impunity. Whether the sliding plates or rollers should be surmounted by a rocking arrangement is a point on which practice varies. In theory the rocker is undoubtedly correct, giving a perfectly definite point of support, and obviating the unequal pressure on the rollers due to the slope of the deflected girder. Many excellent and experienced authorities however omit it. The Great Hawkesbury Bridge, N.S.W., for example, has no rockers, though carried out in the best possible style in other respects. On the other hand, the bridge over the Yarra, carrying the Port Melbourne and St. Kilda Railways is carried everywhere on rockers. The fact that in the former case the girders are very deep and therefore stiff, while in the latter they are shallower, may perhaps justify the difference.

It is often forgotten by bridge designers that expansion takes place transversely as well as longitudinally, and that roller systems should be arranged accordingly. One point of support being fixed, all the others should be provided with rollers acting in directions *radiating* from the fixed point.

Tall thin columns, such as those at Johnston Street Bridge, previously referred to, do not need expansion arrangements as a rule, the column itself being capable of springing an inch or two without injury. In dealing with the expansion arrangements of existing structures, I would suggest that they be kept clean and lubricated and protected from dust and moisture, and that all impediment to free motion be removed.

We next have to consider the main girders constituting a large and costly part of the structure. These are of various types, including plate girders, box or tubular girders, closely latticed girders of the type used 30 years ago, and the many modern forms of open girders including the X girder, the N girder, the W or Warren girder, the Pratt & Whipple girders so deservedly popular in America, and of late years, frequently adopted by English engineers, and other forms too numerous to mention. Some of them are of equal depth throughout, the top and bottom surfaces being parallel, while others vary in depth, having one or sometimes both surfaces curved. To discuss all these various forms in detail would require a very large volume indeed, far beyond the dimensions of the present *brochure*. All that can be here attempted is to clearly state the leading principles of strong and economic construction, and indicate how they have been transgressed in times past. These are as follow :

(*a*) In beams or girders having continuous plate webs the material should be concentrated as far as possible from the neutral axis so as to give a maximum moment of resistance, and only sufficient material be left in the connecting web to enable the whole mass to act as one beam.

(*b*) The web should be sufficiently stiffened or reinforced so as to enable it to bear transverse compression at points of support, or of heavy isolated load.

(*c*) In open web girders the structure should consist of a continuous series of triangles connecting the points of application of external force, the sides of which should be perfectly straight, and the angles common, and which should be so arranged that determinate equations of equilibrium may be obtained for every angular point.

(*d*) The various bars composing the frame should be proportioned to the stress they undergo, should be efficiently jointed, and if in compression should be of such a section as not to evade their duty by lateral bending or wrinkling.

(*e*) In all forms of girder no unnecessary material should be used in any part, and the proportions and arrangements of parts should be such as to give the greatest possible strength for a given amount of material and workmanship.

Let us now see how existing structures fail to comply with the above conditions. In connection with main girders the following errors have come under my notice.

5. *Insufficient depth.*—The extreme top and bottom elements, or chords as they are generally called, of all structures performing the functions of a beam are stressed in inverse proportion to their distance apart. Hence the deeper the girder the less the stress upon and requisite sectional area of these parts. As a matter of pure and bald theory the quantity of material in the web is independent of the depth, so that the most economical girder is one infinitely deep with chords infinitely small and web infinitely thin. Such a result as this is of course valueless as a guide to practice except as showing that as ample a depth as other considerations permit should be chosen.

For many years English engineers, following apparently the example of Fairbairn, adopted depths of $\frac{1}{12}$ to $\frac{1}{15}$ of the span—involving very heavy chord sections. The Americans however showed that it was possible to follow theoretical indications much more closely without incurring practical difficulties, and erected many efficient and economical structures with depths of $\frac{1}{5}$ to $\frac{1}{10}$, the average being about $\frac{1}{8}$. Of late years English practice has been approaching to American, though somewhat hesitatingly. This is of course in cases when the depth is not restricted by such considerations as head way or flood level.

As practical illustrations of the defect of insufficient depth, I could refer to, first, the earlier foot passenger bridges over the railways in the vicinity of Melbourne. These are excessively shallow Warren or lattice girders surmounted by gas pipe handrails, which in no way add to the strength. A model of one of these at the Prahran Railway Station was made and broken down at the University, and also a model containing the same metal, of decidedly simpler construction and double the depth. The latter was found to be 70 per cent. stronger than the former, and further, had the advantage of dispensing with the handrail, being itself deep enough for the purpose of a parapet (see Figs. 8 and 9).

The cross girders of the recently erected Tower Bridge London, constitute another example of the same peculiarity, having a depth of only $\frac{1}{20}$ of the span. Consequently the chords are enormously massive, consisting of, in some places, seven layers of plates rivetted together, an undesirable arrangement, as it is very difficult to effect satisfactory rivetting through so many superposed layers of metal, and, further, there is some doubt as to whether the outer layers really do their fair share in resisting the bending moments.

Had the Tower Bridge cross girders been supported at the efficient points, which lie at the edge of the footpath, the reduction in material and weight required would have been enormous, and it is not clear that there is any insuperable objection to this being done. As examples of excessive shallowness and most unfavourable mode of support these girders are very notable.

This defect is obviously incurable in existing structures.

6. *Unfavourable disposition of material for enduring bending moment.*—This is an infraction of requirement *a* on page 12, and is occasionally seen in the older type of box girder. A notable example is the Railway Bridge at Penrith, N.S.W., the section of which is shown in Fig. 10. Here there are four chords instead of two, and those nearer the neutral axis represent a most unfavourable disposition of material. The intermediate chords are 10 feet apart and the extreme ones 13. Consequently, first, the stress on the intermediates is only $\frac{10}{13}$ of that on the extremes, and as it acts at only $\frac{10}{13}$ of the distance from the neutral axis, the value of every square inch of metal is only $\frac{10^2}{13^2}=\frac{100}{169}$ or not much more than half of what it would have been if placed at the extreme distance. In this way the bridge is loaded with a vast quantity of metal which performs only about 60 per cent. of the duty it should. Further, this arrangement involves the existence of cells 18 inches square, and nearly 600 feet long, which are most objectionable from the point of view of inspection, painting and repair. Fortunately, this form of girder is now quite obsolete, and the error is not likely to be repeated.

7. *Uniformity of chord section throughout the length.*—This uniformity is justifiable on grounds of simplicity in rolled beams and the smaller varieties of built girders when a pair of angle bars of convenient size suffices for the maximum chord section. But when one or more plates have to be added, to carry these throughout the whole length regardless of variations of chord stress is absurd. In some of the earlier plate girder bridges on

the Victorian Railways this has been done to a most remarkable extent. At Kororoit Creek, on the Geelong Railway, there is a double line bridge of 80 feet span, the chords of the girders of which consist of two $3\frac{1}{2}$ x $3\frac{1}{2}$ x $\frac{3}{4}$ angles and three 24 x $\frac{3}{4}$ plates continuous throughout, and representing an enormous waste of metal towards the ends of the girders. As a pleasing contrast to this the new steel bridges over the Inkermann and Balaclava Roads on the Brighton Railway, near Melbourne, may be noted. Here the plates are arranged just as the stress requires and for the last few feet of each end are omitted, the angle bars forming the whole chord. Figs. 11 and 12.

8. *Insufficient connection between chords and web.*—The stress in the chord of a plate web girder is given with approximate accuracy by dividing the bending moment by the depth. If this stress be determined at two points say one foot apart, different values will be obtained, the difference between which will be the horizontal or longitudinal shear for that foot in length tending to separate the chord and web. This has to be resisted by the rivets. As the bending moment varies rapidly near the supports, and slowly near midspan, this shear will vary correspondingly. Hence, for equal strength throughout, the pitch of the connecting rivets should vary from the centre to the ends, being very large at midspan, and becoming smaller and smaller as the supports are approached. To carry this out exactly as calculation requires would involve too much complicated measurement for practical conditions, hence one or at most two variations of pitch must suffice, portions of the work being a little over-rivetted to secure uniformity. The great Penrith Railway Bridge, N.S.W., shows a notable neglect of this requirement. The rivetting at the point A in Fig. 10 being small and uniformly pitched throughout, the rivets are seriously over-stressed for a distance of 40 feet on each side of the piers, and for 10 feet at each end of the bridge (which is continuous over three equal spans). This grave defect was discovered by Professor Warren of Sydney University and verified by the writer. It is fully dealt with in the Report of the Royal Commission on Railway Bridges of N.S.W., 1886, and the proper remedy, viz., the replacement of the $\frac{3}{4}$ inch rivets by 1 inch rivets for the distances mentioned, pointed out. But

though this effective remedy could be applied at a mere nominal cost and without any interruption of the traffic, the N.S.W. Railway authorities have persistently refused to yield to the urgent representations of the Royal Commission, of Professor Warren, and of the writer. Should disaster ensure the responsibility rests with them.

In the later work of the Victorian Railway Department this variation of horizontal shear has been consistently recognised as is evidenced by the varying pitch of the rivets in the girders on the bridge over the Yarra on the Port Melbourne Railway and elsewhere. The considerations applying to the row of rivets connecting the web and chord angles of these girders also apply to those connecting the chord angles with the chord plates, and a similar variation of pitch is required here. It is to be noted, however, that these latter rivets perform a smaller duty than the former and so may be smaller, or at larger pitch without ~~render-~~ reduc-ing the strength of the girder—Strange to say in Penrith Bridge these rivets, B in Fig. 10, are *larger* than those at A, though dealing with the shear consequent upon the stress variation in a portion of the chord only, while those at A deal with the whole —a notable anomaly. It is to be added that the stress on the vertical rows of rivets in a plate web is identical with that in the contiguous portions of the horizontal rows, and that the diameter and pitch should be the same for both. To illustrate this point Fig. 13 has been prepared, representing a theoretically rivetted girder with terminal supports and uniformly distributed load. Existing girders ought to be examined and computed to see if there is any weakness as to horizontal and vertical shear, and, if there is, rivets should be cut out a few at a time, holes enlarged and bigger rivets inserted, as recommended by the Royal Commission in the case of Penrith Bridge.

9. *Vertical stiffeners absent or wrongly placed.*—Requirement (*b*), page 12. These vertical stiffeners are added at intervals along the web of a plate or box girder for one of the following purposes:—

1. To prevent the thin web from being crushed by the local vertical pressure due to the reaction of a support or a concentrated load.

2. To check the tendency of the web to buckle under the diagonal compression that pervades it, being most intense near supports and least midway or thereabouts between them.

To comply with the former condition we need a ~~massive~~ massive vertical pillar at the end of every ordinary girder, and a still more massive one at the piers of continuous girders or those with overhanging ends. Strange to say, however, these verticals are often seen inserted when there is no need for them, and omitted where there is the maximum stress. A notable instance is to be seen in a small continuous three span bridge on the North-Eastern Railway about two miles from Melbourne.

Should a very heavy concentrated load, as for example a large column in a building, be imposed at any particular point, this column should be continued as a vertical stiffener to the bottom of the girder. This case does not however often occur in bridge-work.

These stiffeners are usually made of uniform section from top to bottom of the girder. This may be justified by convenience of construction, but is not required for strength. As we pass from the bottom to the top of the girder at a point of support, or from the top to the bottom at a point of concentrated top load, the compression in the vertical gradually discharges itself into the web in the form of a shear, or its equivalent, a set of diagonal compressions and tensions, and thus dies away. Hence the vertical at a support should be of the full section required by the reaction of that support at the bottom and diminish to nothing at the top, and that at a concentrated top load of full section for the load at top diminishing to nothing at the bottom. In large girders this fact may be made use of to save material. The other use of verticals in plate girders is to prevent the web from buckling or being thrown into waves by the diagonal compression due to the shear.

Rankine in his "Civil Engineering" treats the web as a long column tending to buckle under the diagonal compression, and measures the length on an angle of 45 deg. between top and bottom chords, or between vertical stiffeners, whichever happens to be the smaller. He then applies the excessively high safety factor of six. There are two most serious errors in this treatment. First, the diagonal tension, which is approximately equal to the diagonal compression, has a powerful tendency to prevent buckling or undulation and so improve matters, and second, to apply a safety factor of six to a long column, whose failure is due to lack of stability and not of strength is unscientific as is shown in my paper on this subject in the "Transactions of the Royal Society of Victoria," Vol. XV., p. 14. There is no doubt that the resistance of such a web to buckling is at least five times as great as the Rankine treatment will allow, and this conclusion is confirmed by Professor Warren's elaborate analysis of the web stresses of Penrith bridge, in the Royal Commission Report previously quoted. It is there shown that, according to Rankine, the safety factor of some parts of the web is less than unity under ordinary traffic, and yet this bridge has now been in existence for more than thirty years and has shown no sign of weakness.

To determine the exact strength of a thin plate web against buckling is a question of much difficulty and obscurity, but there is no possible doubt that it is immensely greater than Rankine's imperfect method of computing indicates, and that in a vast number of instances the thinnest metal that it is desirable to use from the point of view of corrosion, and practical convenience of construction is abundant to resist the tendency to buckle due to the shear.

The following rules may be laid down as sound for arranging verticals, and proportioning webs.

1. A vertical pillar at each point of support of section proportioned to the reaction of that support should extend from bottom to top of the web, but it needs the full section only at the bottom and may taper to nothing at the top.
2. This pillar should be placed fairly on the centre of the support and not as is sometimes seen at or near one edge.
3. The web should have a vertical sectional area of one square inch for not more than 2 tons of vertical shear for wrought iron, and $2\frac{1}{2}$ tons for mild steel.
4. Vertical stiffeners of T section should be placed wherever any considerable concentrated load is imposed on the top chord.

Bridges defective in these respects should be reinforced by rivetting on the necessary additional parts.

Leaving the girders with continuous plate webs we next have to consider those in which the chords are connected by some arrangement of bars forming an open lattice or trellis work of some kind. These have enjoyed a great popularity for bridges of the largest size, and also for smaller ones, where the load being light it was difficult to design an economical plate girder without using a web undesirably thin for practical conditions.

In these as in the plate web girders the fault of insufficient depth and consequently needlessly large chord sections has been very prevalent. At Cremorne on the Melbourne and Brighton Railway there is a lattice girder bridge 140 feet span and 10 feet deep, built about forty years ago. Some ten years since, it was required to build a second bridge at one side of it to carry the Gippsland Railway, and, although there was a desire for the sake of appearance to keep the two structures of uniform depth, the advantage of increasing it was so great that the new bridge was made 20 feet effective depth for the same span.

Apart from this defect the principal faults to be found in girders of this class are—

10. *Incomplete triangulation.*—As stated on p. 12 requirement *c*, every framed structure should consist of a complete and continuous series of triangles, the triangle being the only polygon whose figure is fully determined when the length of its sides are fixed. Such a structure is subject to longitudinal tensions and compressions only, and is free from bending moment and shear, and so utilizes most advantageously the material of which it is composed. Now in actual existing structures glaring departures from this rule are sometimes seen, as in the case of the shore girders of the old footbridge over the Yarra at the Botanical Gardens, Melbourne (Fig. 14). Here it must be obvious to anyone having the slightest knowledge of the subject that a single strut, as shown by the dotted line, would have carried the triangulation to its proper termination, and been far cheaper and in every way better than the complex and costly arrangement of plates, angles and rivets actually employed. As this structure is about to be removed its defects have no further interest. In future structures they should, however, be avoided.

11. *As an error in the opposite direction to the last redundancy may be next quoted.*—This fault is very wide-spread, and has received a good deal of defence from influential quarters. But such defence has usually been more in the direction of palliation than justification. Redundancy may be defined as a duplicate system of triangulation connecting identical points. In such a case the stresses in the bars become indeterminable by statical calculation, and can be computed only by a much more complex and less satisfactory method based upon the elastic deformations of the various parts, and the result is likely to be vitiated by variation in the coefficient of elasticity, and to a still more serious degree by imperfections of workmanship invisible in the completed structure. To explain more fully, a redundant structure minus certain of its bars presents a complete system of triangulation. Suppose now that the remaining bars are by accident made a little too long or too short, and are forced into their places with violence, a set of stresses of possibly great severity is induced throughout the structure which may modify profoundly the result of any calculation. Hence redundancy, while rarely, if ever, of any real advantage, may lead to most undesirable consequences. As a gigantic example of this defect, the Charing Cross Railway Bridge over the Thames at London, illustrated in "Humber's Iron Bridges," and shown in outline in Fig. 15, may be noted.

Minor instances of this fault are very common, and to forbid redundancy absolutely would mean condemning many otherwise meritorious designs. We may, however, I think, say first, that other things being equal or nearly so, preference should always be given to non-redundant arrangements, and second, that if for sufficient reasons redundant ones be adopted special care should be taken that they are put together in a perfectly unstressed condition. In French practice this fault is very prevalent, and a gigantic example of it is to be seen in the Eiffel Tower, a structure which, like the Charing Cross Bridge, would undoubtedly be much improved by the removal of numerous costly parts.

12. *Curvature of members.*—Considering that every part of a properly designed framed structure is a simple strut or tie subject to longitudinal stress only, the necessity of absolute straightness

is an immediate consequence. Strange to say, however, owing to some peculiar warp of the human mind, many persons persist on fanciful grounds in increasing the cost and diminishing the strength of structures by the introduction of curved members. A curious example of this is to be seen in the evolution of the frame of the modern bicycle. For years this vital part was made of ridiculous shapes presenting complex curves utterly contrary to scientific principles, and the result was, despite the most liberal employment of material, straining, weakness, and frequent fracture. The modern diamond frame, every part of which is perfectly straight, is thoroughly scientific, and, with half the material of the earlier frames, is far more rigid and absolutely free from fracture under ordinary and reasonable use.

Perhaps the most extraordinary and inexcusable instance of this fault in modern times is to be seen in the huge and costly Jubilee Bridge over the Hoogly in India, described and discussed in the "Proceedings of the Institution of Civil Engineers," Vol. XCII., 1888, an outline of the terminal panels of which is given in Fig. 16. Here one of two things ought to have been done, either AB should have been made perfectly straight, or the space C should be plated over, the latter being the only practicable remedy now. This grave fault was animadverted upon by myself in the "Engineer" of 5th June, 1885, and afterward by Professor Max am Ende before the Institution of Civil Engineers, London. A second and still more recent example is seen in the Warburton Bridge over the Manchester Ship canal, illustrated in "Engineering" of 26th January, 1894.

But while condemning as strongly as possible such designs as the Jubilee and Warburton Bridges, I would make an exception in favour of many bowstring and hogback girders such as the new Cremorne Bridge on the Gippsland Railway near Melbourne. Here the top chord forms a bold and graceful curve, very pleasing to the eye, but the panel points where the web members come in are so numerous that the curvature in each panel or length of the chord is imperceptible. If the chord was made polygonal, each panel being straight, its outline would not differ visibly from what it is at present. A chord curved at a large radius and divided into panels so short as to be practically straight, cannot be reasonably objected to. From this point of view also, the magnificent 350 foot span girders at Indooroopilly, Queensland are justifiable.

13. *Eccentricity.*—The condition laid down on page 12 that a properly designed framed structure should consist of a series of triangles having common angles, involves the necessity of any three or more bars meeting at a point being so arranged that all their mean fibres pass accurately through that point, the mean fibre being defined as the line passing through the centre of gravity of the cross section of each bar. If these mean fibres or gravity lines, as they are sometimes called, do not meet truly, bending moments and shears are set up in the bars, and loss of strength ensues, unless obviated by the introduction of additional material. Very great laxity is sometimes shown in this respect, as may be seen from Fig. 17, representing portion of a large girder recently erected over Primrose Street, in connection with the enlargement of the Liverpool Street Railway Station, London, illustrated in the "Engineer," of 21st August, 1896. Here it is to be noted that the end pillar, instead of being placed centrally over the rocker support, is placed most eccentrically so as to concentrate the stress on one edge instead of distributing it equally, and further, that the mean fibres of the last diagonal and end pillar meet far above the upper surface of the top chord, instead of as they ought in its mean fibre. Another glaring case is shown in Fig. 18, representing part of a bridge near Windsor, N.S.W., inspected some years ago by the writer. Recent Australian practice appears fairly free from these inexcusable faults, which is more than can be said for English practice.

A most disastrous accident took place ten years ago near Boston, U.S.A., causing the destruction of a train, the loss of twenty-five lives, and injury to about one hundred persons. This all started from the failure of the improperly designed bridge hanger shown in Fig. 19. This hanger had ample sectional area for the direct pull, but that pull was imposed so eccentrically as to give rise to a bending moment which increased the stress several times. This and other cases show the necessity of guarding most carefully against this fault. Few persons realise that in a rectangular section a deviation of the centre of stress

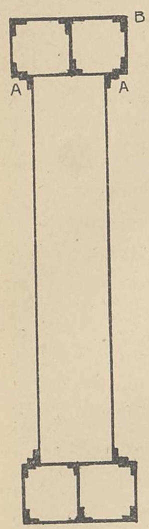


Fig. 10.

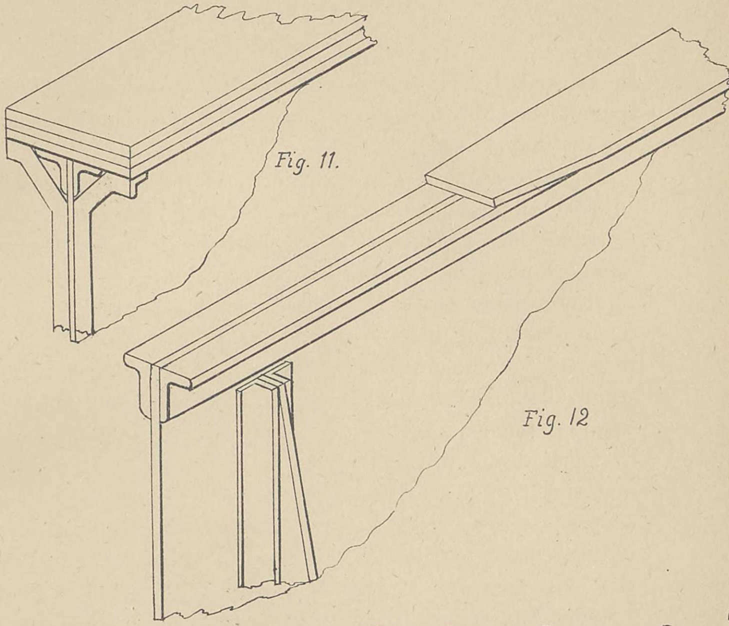


Fig. 11.

Fig. 12

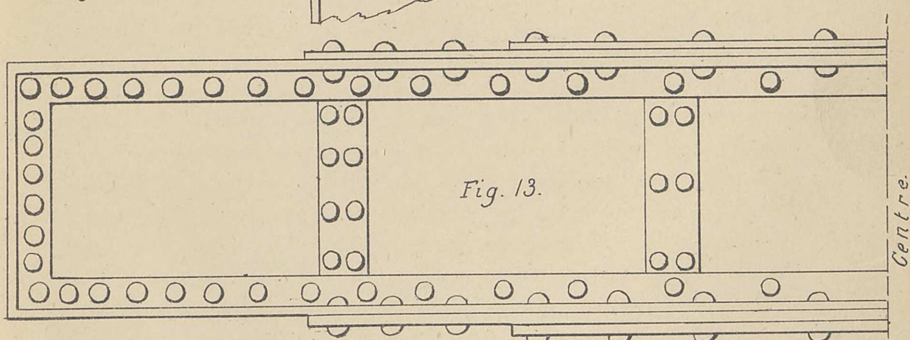


Fig. 13.

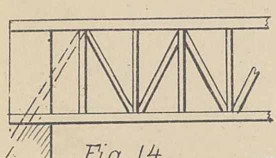


Fig. 14.

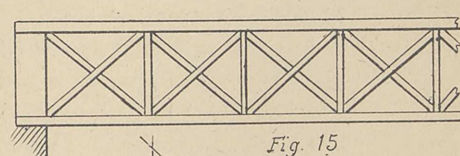


Fig. 15

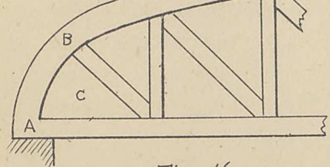


Fig. 16.

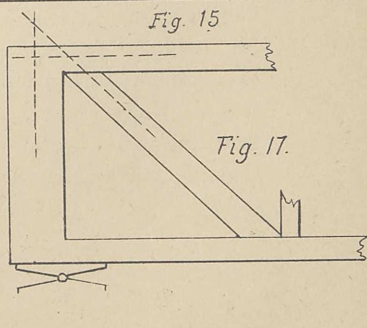


Fig. 17.

from the centre of figure by one-sixth of the width of the bar, doubles the stress on one side and reduces that on the other to zero.

Eccentricity should be most carefully avoided in designing new structures, and when it exists in old ones, special local strengthening should be applied, unless computation shows the parts to contain a sufficient excess of material to resist the moments and shears involved as well as the direct stresses.

14. *Unscientific and wasteful end pillars.*—There are multitudes of girders in existence which are reasonably and consistently designed as far as chords and web members are concerned, but which have the terminal verticals of excessive size, and most complicated construction involving great and altogether unnecessary increase in the weight and cost of the girder. No defence or excuse for this anomaly has ever reached the writer's ears, but its vitality is remarkable, as it appears not only on bridges built thirty or forty years ago, but even in quite recent structures, including some of those crossing the Manchester Ship Canal. One example is shown in Fig. 14, where the end pillar or box contains at least three times as much material as would be needed for a proper terminal diagonal or "batter brace," to use the American term. A second example is shown in Fig. 20, representing part of a bridge over the Rio Verde, South America, the work of a late president of the Institution of Civil Engineers. Here the end pillar AB has 98 square inches sectional area against 21 square inches in the adjoining diagonal BC, which bears 40 per cent. more stress, and 20 square inches in the vertical CD, which endures a compression three-fourths of that on AB. Were AB reduced to 25 square inches area, which would be most abundant, 11 per cent. of the weight and cost of the whole girder would have been saved. There is, however, a still better arrangement originated by Whipple, an American Engineer, and used in America for nearly half a century past. It is to abolish the bars AB, BC, BD, and insert a compression diagonal AD as shown by dotted lines. This will leave the compression on the top chord unchanged, and the stress on all except the end panel of the bottom chord, which at present is unstressed, though of massive section, and will reduce the stress on CD largely, thus saving no less than 17 per cent. of the

weight of the girder. The remarkable economy due to replacing the clumsy end vertical by a sloping compression piece or "batter brace" of theoretical section is now somewhat tardily being recognised by English Engineers. It has been universal practice in America since the time of Whipple.

This is an error to be avoided in future designs. In existing bridges it simply means waste of money, but does no further harm.

15. *Unduly numerous systems of triangulation*—This is a very common fault of the lattice girders of 30 or 40 years ago. A multiplicity of small bars, many of which for practical reasons have to be much larger than calculation requires, which involve a large amount of complicated workmanship and often are not placed in proper relation to the points of attachment of the cross girders, constitute a usual characteristic of early practice. Modern work, however, usually avoids this fault, and as it in most cases where it exists means waste and not weakness, it is not a matter needing any present action.

16. *Inefficient forms of compression member.*—In consequence of the high compressive resistance of iron or steel the transverse dimensions of compression parts of girders have to be comparatively small relatively to their length, consequently the tendency is to fail by buckling or long column action, rather than by direct crushing, and the strength attained depends largely on the success with which this tendency is combatted. When the column is so short as to fail by true crushing, the form of section is immaterial, provided only the required area is present, but when it is longer the form of section has a most profound influence on the resistance. The best form is that which presents the greatest resistance to bending laterally, either of the column as a whole or of any constituent part. Hence a good compression member must approach in form to an efficient beam, but as it may bend in any plane and not in one only, it must be an efficient beam in every direction. The ideal long column is a hollow cylinder which is a fairly and equally good beam in every direction. The greater the diameter the greater the resistance to bending as a whole, but there is a limit to desirable increase of diameter and consequent reduction of thickness owing to the tendency of very thin tubes to give way by wrinkling or

corrugation of the thin metal. Figure 21 shows a full size sample of bicycle tube that has been crushed in the University testing machine, its original dimensions being shown by dotted lines—the metal is ·035 thick and the ultimate load 4400 lbs. Unfortunately the circular tube is very unsuitable for the convenient attachment of other parts, and consequently has been generally rejected by the designers of framed structures, although the great Forth Bridge and the modern bicycle frame are instances to the contrary. A further reason for rejecting the circular tube is, in structures of ordinary size, the inaccessibility of the interior for purposes of inspection, cleaning, painting, and repair. To describe and discuss all the sections that have used for compression members for chord and web purposes would extend this paper far beyond permissible limits. It must therefore suffice to enunciate general conditions to be complied with, and point out instances of conspicuous transgression of these conditions.

(*a*) A good compression section should have a large radius of gyration in every direction.

(*b*) If the column is prevented from bending in one plane by the attachment of other parts, and is not so prevented, or not so effectively prevented from bending in a plane at right angles to the first, it should, if its radius of gyration varies, be placed with its maximum radius of gyration in the second plane.

(*c*) Thin unsupported edges should be avoided, or, if unavoidable, should not be counted as part of the effective section, as they are very liable to buckle.

(*d*) Flat surfaces should not be made too thin in proportion to their width. A proportion of 1 to 30 is quite small enough. If this be passed the central part of the flat face becomes of little value for resisting compression.

(*e*) If two compression members be connected together with the object of preventing their bending in the plane of the connecting pieces, those pieces should be arranged so as to constitute an efficient web system, forming with the two compression pieces a complete girder designed for resisting bending.

We now proceed to cite cases of infraction of these rules, with consequent loss of strength.

Figure 22 represents one girder of a small over bridge at the Spencer Street Railway Station, Melbourne. It is 45 feet span and about 4 feet deep. All the diagonal web members are of 3 x $\frac{3}{8}$ inch iron. These are suitable enough for tension purposes, but most inefficient in compression, having a radius of gyration across the plane of the girder of only ·1 inch or about $\frac{1}{300}$ of the length. This unfortunate girder further presents the faults of redundancy, owing to the presence of unnecessary verticals, and of absurdly heavy and complicated end pillars.

Fig. 23 represents an amended design in which the more heavily stressed compression diagonals are made of angle iron having a radius of gyration many times greater than that of the flat bars, and in which the faults of redundancy and disproportionate ends are avoided.

Inch to the foot models in iron of each of these girders were constructed at the University and tested to destruction, with the following results:—The amended design contained 16 per cent. less iron than the original owing to the omission of the massive end plates and intermediate verticals. It involved much less workmanship owing to there being less than one-third the number of rivets, and the time taken in making it was less than half that of the other. Its actual breaking load distributed along the bottom chord as in the actual bridge was 771lbs. as against 208 for the original structure. Thus with identical external dimensions, and very little change in appearance, the cost of the structure was largely reduced, and its strength increased nearly fourfold. The mode of fracture of this defective design was, as was predicted by calculation, the buckling or side ways bending of the weak compression diagonals near the end. There would be no difficulty and but little expense in increasing the strength of this structure threefold by simply clamping angle irons to the most heavily stressed compression diagonals so as to prevent their bending, and this ought to be done, and was long ago urged by the writer, but without result.

Fig. 24 represents the section of a large buttress or inclined strut, erected at great cost on the down stream side of the Victoria Street Bridge, near Melbourne, for the purpose of

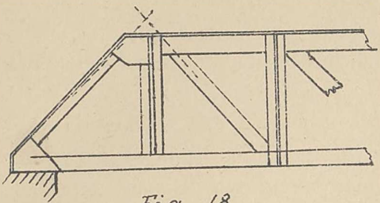


Fig. 18.

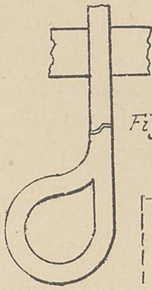


Fig. 19.

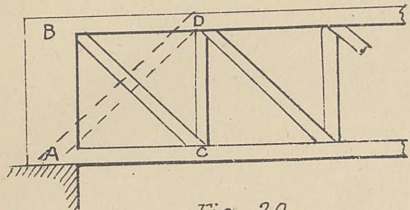


Fig. 20.

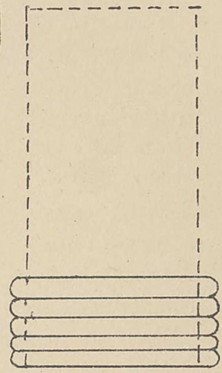


Fig. 21.

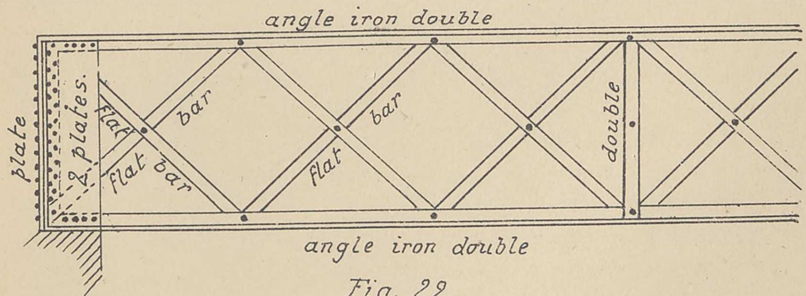


Fig. 22.

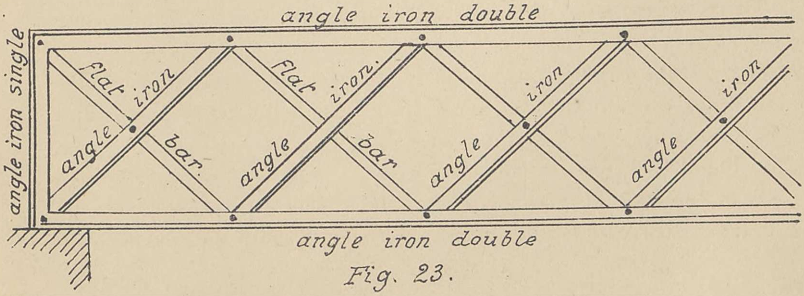


Fig. 23.

remedying an entirely imaginary lack of lateral stability. Its radius of gyration is 3·9 inches in the plane of its web, and 1 inch at right angles to that plane. It is 25 feet long and is regarded as fixed in direction at the ends, though whether this fixing is perfectly reliable is not altogether certain. It is braced against lateral bending by a costly system of bars *in the plane* of the web. Taking the most favourable view, calculation shows that it would give way under a compression of 98,000 lbs., whereas if it were turned the other way, as it easily might have been, so as to be braced in the direction in which it was weakest, its resistance would be more than 200,000 lbs. This is an instructive example of the neglect of condition *b*, p. 25.

Fig. 25 represents a very usual section of compression chord of the earlier types of lattice girder bridges in England and Australia. Here the side plates are so thin and so liable to buckle or wrinkle at the edges that it was decided in the calculations made by Professor Warren and the writer, and published in the Report of the Royal Commission on Railway Bridges, N.S.W., 1886, to ignore the outer half of their width as contributing anything to the compressive resistance of the chord. As an example of improved practice, Fig. 26 is given, representing the upper chord of the very fine steel bridge over the Yarra, on the Gippsland Railway.

Of transgression against condition *d*, I am happily unable to quote a case.

Condition *e* applies to very numerous cases of compression diagonals, where two parallel angle, T or channel bars are braced together. Fig. 27 represents an arrangement that has been adopted on the two most recently erected bridges over the Yarra, and shows that the latest practice is not always the best. The only function that the cross connections perform is to ensure that the two main bars bend the same way, and if they are of themselves inclined to do so no gain of strength ensues as compared with entirely disconnected bars. Fig. 28 shows a vast improvement with the same amount of iron and rivetting, and the dotted lines show how any lateral bending must be of a very different character to that of Fig. 27. In fact the shorter curves and more numerous nodes in the latter case are equivalent to a reduction to the effective length of the column to the distance XY, which means a valuable gain in strength. There would be no difficulty and but little expense in altering these structures now, removing the cross pieces one or two at a time when the bridge was free from live load, and replacing them by square plates as in Fig. 28. An even better but slightly more expensive arrangement would be to insert a complete triangulation similar to that shown for a different purpose in Fig. 8.

Fig. 29 represents a type of braced strut appearing in the late Sir John Hawkshaw's great railway bridge over the Thames at Charing Cross, Fig. 15, and copied thence extensively in Europe and in New South Wales. It requires but little consideration to detect the weakness of this arrangement. Suppose there is a tendency for the member to bend to one side, the rectangular panels will become rhomboidal, one diagonal being extended, and the other reduced in length. Now if these diagonals are straight they will oppose the maximum resistance to such distortion, but if bent or crooked, as in the Charing Cross Bridge, the extended one will tend to straighten, and the compressed one to become more crooked. Experiments have been made at the University on different types of compression members, and have shown that great advantage ensues from replacing this unsatisfactory double system of crooked flat bars, by a single diagonal system of straight bars, as shown in Fig. 30. This improvement might easily be applied to actual structures, taking advantage of times when they are free from live load to remove the one and insert the other.

Fig. 31 is a photograph of a number of experimental compression pieces tested at the University, arranged in order of merit. The best of these carried, in proportion to the metal it contained, rather more than twice the load that the worst did. These specimens were free to bend in their own plane, but braced at three points in a direction at right angles to their own plane, and represented compression diagonals in closely latticed girders intersected and kept from bending in the plane of the main girder by the tension diagonals. The figures at the bottom indicate the ratio of the load carried, to the weight of the structure, and thus express the relative values of the various systems. The first and third of these models have angle bars and the second and fourth channel bars for their sides.

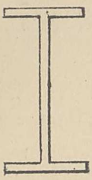


Fig. 24.

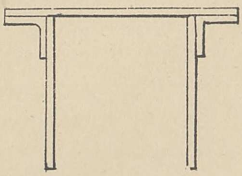


Fig. 25.

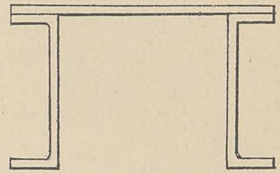


Fig. 26.

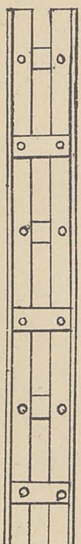


Fig. 27.

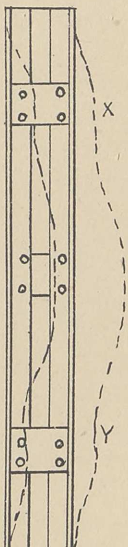


Fig. 28.

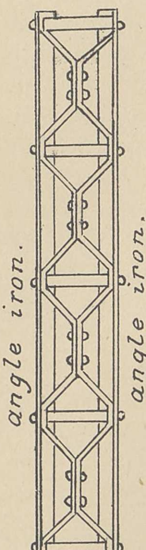


Fig. 29.

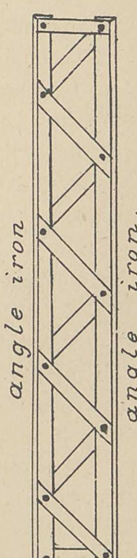


Fig. 30.

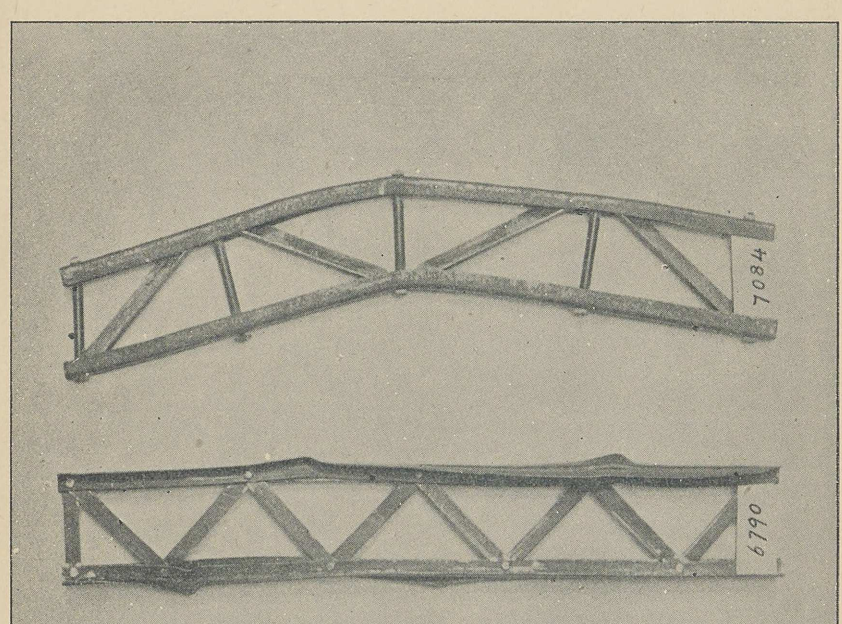


Fig. 31.

17. *Imperfect jointing of tension members.*—The form of section and ratio of length to transverse dimension of a tension member are immaterial as regards strength, and so may be arranged according to convenience. The joints, however, are the points of vital consequence, and malformation of these often leads to serious loss of strength and efficiency. Of these joints there are two principal classes, rivetted joints and eye-bar and pin joints. The first of these is most usual in European, and the second in American bridge work.

In rivetted work, the metal is used as it comes from the rolling mill and the attempt to utilize the full section for strength is not made. A rivetted joint, in other words, never claims perfect efficiency, but is always subject to some percentage of loss as compared with the pieces connected. To minimise this percentage of loss is the object in arranging the joint, and it is possible to reduce this to the proportion of area cut away by one rivet hole. Fig. 32 shows a properly rivetted joint, which should fracture at the line AB, passing through the leading rivet and giving an efficiency less than unity by the fraction represented by the diameter of one rivet hole divided by the width of the bar. It will not fracture through the following pair of rivet holes, because the tension is too much reduced by the action of the first rivet to permit it, nor through the third row for a similar reason. All this has been verified by careful experiments at the Melbourne University and elsewhere. In contrast with this, Fig. 33 shows a tension member weakened by no less than three rivet holes in one leading row, and many similar cases might be quoted. There are many more complicated forms of tension joint met with in girder work—to discuss which would occupy too much space here. Suffice it to say that the same general principles apply as in the simple cases.

The leading rivet ought to to placed in the mean fibre of the bar, which in an ordinary rectangular section is the middle of the width and the other rivets arranged symmetrically behind. Sometimes it is convenient to place the leading rivet near one side, and, as far as the writer's experiments go, the loss of strength due to such an unsymmetrical arrangement does not seem to be serious. Still it is a departure from what is obviously the right arrangement, and is therefore not to be encouraged.

In arranging rivetted joints it is recommended that there should be a width of solid metal at least equal to $1\frac{1}{2}$ diameters of a rivet between each rivet hole and its neighbour or between a rivet hole and the edge or end of the plate, for drilled work, and $1\frac{3}{4}$ for punched work, that the shearing area of the rivets be 20 per cent. in excess of the tearing area of the plate, and that the aggregate diametral bearing area or sum of the products of the diameters of all the rivets piercing a given plate into the thickness of such plates shall be not less than half the tearing area of that plate.

If a weak rivetted joint be discovered in an existing structure, and it is not possible to increase the number of the rivets, an improvement may often be made by removing the rivets one or two at a time, carefully enlarging the holes with a suitable cutting tool, and inserting larger rivets. This may be done very advantageously in punched work, for the metal removed is that which was damaged in the process of punching, and therefore of little value.

In eyebars the attempt is usually made to secure perfect efficiency, in other words to make the joint as strong as the bar, and when the bar is very long there is great economical advantage, as the whole of the material is then utilized fully. In order to do this the proportions of the eye must be carefully attended to. These have been determined experimentally by Brunel, Sir Chas. Fox, and Berkley in England, and Shaler Smith and others in America, and their conclusions may be generally summed up as follows, averaging the results when the authorities differ, as they do, to a small extent.

(*a*) The internal diameter of the eye or diameter of the pin must never be less than two-thirds of the width of the bar, and if the bar has a thickness of more than one-fifth its width this should be progressively increased, till, when the thickness is half the width, this diameter is 1·2, and when equal, the bar having a square section, 1·9 times the width.

(*b*) The sectional area of the metal on both sides of the eye should be 1·33 that of the body of the bar for bars whose thickness does not exceed one-fifth of the width, increasing to 1·76 when the thickness equals the width and the bar becomes square. The outline of the bar must consist of easy curves, and sharp re-entering angles must be most carefully avoided. (see Fig. 37).

(*c*) The pins must be designed as beams to endure the bending moment due to the pull of the eyebars, and if this necessitates a larger diameter than the previous rule (*a*) gives, the eyes must be correspondingly enlarged.

In America the eyebar construction in its most perfect form is almost universal in the tension parts of the larger bridges, and a very magnificent example of recent American practice is to be seen in the great Hawkesbury Bridge, N.S.W.

In Europe, India, and Australia it is less usual, and where it exists is often of very defective design, as the following examples will show.

Fig. 34 represents the wind bracing of the piers of the ill-fated Tay Bridge, which was destroyed by a gale in 1879, involving the destruction of a whole railway train and every person upon it. Here no attempt has been made to form a proper eye, and only about half of the strength of the bar is really utilized.

Fig. 35 represents a tension eyebar from the Taptee Viaduct, Bombay, Baroda, and Central India Railway, of which the Gundagai Bridge, N.S.W., appears to be a copy. It is also very inefficient, utilizing less than half of the metal in the bar.

Fig. 36 represents the eyebar diagonals of the great Moorabool Viaduct on the Geelong and Ballarat Railway. The most cursory inspection will reveal how far this departs from orthodox proportions with its diminutive pin, its sharp internal angle at A leading to great local intensification of stress and its two sides in the aggregate barely equal to the body of the bar instead of being 33 per cent. greater. Experiments were made at the University on models, first in lead, then in brass, and finally in iron on a quarter full size scale, to determine the efficiency of this joint, and the results varied from 64 to 70 per cent. Further experiments by the Victorian Railway Department led to a practically identical conclusion. Considering that there are several hundreds of these bars in the structure it does seem strange that this defective form was decided upon without experiment, and also that the example of Brunel and others who had used fairly good eyebars in suspension bridges years earlier was disregarded. After some years of discussion the Railway authorities yielded to the writers representations, and condemned this viaduct as a double line bridge. It now carries a single line only, though containing abundant material if rightly disposed to make an amply strong double line structure.

In the bottom chord of this viaduct eyebars are also used, having the extraordinary peculiarity of the centre of the eye being an inch *above* the centre line of the bar. As, however, this chord is of excessive sectional area no real danger arises from this singular departure from correct practice.

Structures having defective eyebars in them can be improved only by reconstruction and replacement of the weak parts. If this is impossible, the load carried should be reduced, by narrowing the deck if a public road is carried or removing a line of rails, placing the remaining line centrally as was done at the Moorabool Viaduct.

Fig. 37 represents an eye from the Hawkesbury Bridge, and may be regarded as an example of the best and latest practice.

The joints of compression parts are not often a source of weakness. If they are solidly butted together, the only need for rivetting is to prevent lateral displacement. The usual practice, however, is to assume that the ends are not in contact, and insert sufficient rivets to carry the whole stress as in tension joints. In this case there is no need to have the single leading rivet, and the arrangement shown in Fig. 33 is admissible. Should the rivets of a compression joint be insufficient to take the stress they will distort slightly and permit the ends of the plates to come into contact, when further yielding will be effectually prevented. The practice has been sometimes adopted, and appears reasonable, of putting the girders together with but a few loosely fitting "service bolts" in the rivet holes of the compression joints, then loading them with a weight sufficient to bring the ends of the parts into perfect contact, and then rivetting up. If this method be adopted the number of rivets in compression joints may be made very small.

18. *Local weakness at intersection of web members.*—In the older form of lattice girders the web diagonals near the centre of the span are usually made of angle bars, which give a suitable

section in view of the circumstance that with varying conditions of loading they may be called upon to endure either compression or tension. When two of these angle bars intersect it is a usual practice to cut away one limb of one of them, reducing it to a simple flat bar, which is further reduced by the hole needed to receive the connecting rivet. Thus the remaining or net sectional area becomes about one-third of the gross area, and as it further is subjected to the most injurious kind of stress, that alternating from compression to tension, during the passage of the load, a very serious but easily overlooked weakness ensues. Fortunately the remedy is simple and cheap. It consists in adding a second layer or reinforcing plate to the diagonal at the weak point, extending about a foot on each side of the intersection, and connected with the unmutilated part of the angle bar with a sufficient number of rivets.

19. *Arrangements involving serious secondary stress are not uncommon, especially in English types of girders.*

The stress upon any bar of a framed structure with rigid joints is of two kinds, primary and secondary. The primary stress is that computed by the ordinary methods of analytical or graphic statics; in other words, by the successive application of the proposition known as the parallogram or triangle of forces. This investigation, provided the structure is not redundant, is simple, and the result certain, admitting no possibility of dispute. All such calculation, however, is based on the assumption that each set of bars meeting at a point is connected by a perfectly frictionless hinge joint. This assumption is by no means strictly true, even in eyebar work, on account of friction, while in structures having the joints made by complicated groups of rivets it is manifestly highly erroneous. What then is the nature and magnitude of the extra, or, as they are now called, secondary stresses, due to the friction or rigidity of joints, and how may they be minimised?

This question was first discussed by the Austrian investigator, Manderla, in 1878, but owing to the intricacy of the calculations, and the delicate nature of the experiments needed, we have so far arrived at but approximate determinations of its value.

The completest treatment that the writer has met with is that of Professor W. Ritter, of the Polytechnic School of Zurich, and was published in 1884.

Manderla's work is referred to in Bender's "Economy of Design of Metallic Bridges," New York, 1885. Bender states that he has applied Manderla's method to a number of examples, and that he has found in a "100 foot Whipple truss, 20 feet deep, a maximum secondary strain of 8 per cent." in the centre of the top chord. He also says that he has found "Secondary strains of 172 per cent." of the primary stresses in a triangular pin-jointed girder of 118 feet span and 12·5 feet deep in South Germany. Again he speaks of "secondary strains as high as 180 per cent." over the middle piers of continuous bridges. Now, all this is most unsatisfactory and alarming. Unsatisfactory, because Bender gives no drawings or detailed dimensions of his bridges, nor does he show how he arrives at his results. Alarming because the only meaning that can be attached to his words is that the secondary stresses in structures of ordinary type may be greater than, in fact, nearly double of the primary stresses, and if this be the case, the structures affected must be most imminently dangerous, indeed it is difficult to understand why they have not long since fallen.

Ritter's work, as quoted by Koechlin in his "Applications de la Statique Graphique," Paris, 1889, is much fuller and more satisfactory, and his results more intelligible and less alarming. The maximum secondary stress that he arrives at in structures of ordinary proportions, is less than 30 per cent of the primary stress. Still, his method appears to the writer to be too general, and to fail in indicating exactly at what points of a frame the severest stress is to be expected. The writer has, after much consideration, arrived at a method, which he submits as giving, without inordinate labour, a fair approximation to the secondary stress in, at any rate, the simpler types of structure. It consists of the following operations :—

(*a*) From the primary stress and sectional area of each bar, and its known modulus of elasticity, its change in length is computed. This will be an elongation or shortening, according as the primary stress is tensile or compressive.

(*b*) This change in length is exaggerated a convenient number of times. The writer increases it one hundred fold.

(*c*) The frame is then re-plotted with the altered lengths of the bars. One result of this is that the panel points of the top and bottom chords, instead of lying in straight lines, will lie in curves, which in most cases are approximately circular, but in others that have been tried, have a cusp at the centre, like a Gothic arch inverted. This cusp indicates a point of intense secondary stress.

(*d*) A smooth and regular curve (usually circular) is drawn through all these panel points, and at each panel point a tangent to this curve is drawn. This is most conveniently done by taking a well-tempered piece of spring steel and bending it so as to pass through all the points.

(*e*) Lines are drawn, making the same angles with these tangents that the diagonals or web members made with the chords, when the structure was free from stress. If two such lines drawn from the two extremities of a diagonal or web member coincide, forming one straight line, that web member is free from secondary stress. But if, as is usually the case, they do not coincide, then the diagonal, which, by virtue of its rigid attachment to the chord is tangential to these lines at its ends, must be bent into a curve, usually of double curvature, or S curve as it is often called. This curve will present a point of contraflexure and consequently of no bending moment at or near the mid-length of the bar and each part of the bar will be a cantilever, the deflection of the end of which can be measured on the exaggerated scale adopted, and from this, its width, and the modulus of elasticity, the secondary stress can be computed. This secondary stress is usually least at the centre of the bar, increasing to a maximum at the ends. To find the exact curve assumed the piece of spring steel mentioned before may be applied.

(*f*) The secondary stress of the chords is computed from the radius of the curve *d* and in most cases is approximately uniform throughout.

To carry out this method, diagrams of one of the simplest of which Fig. 38 is a greatly reduced copy, have been made, and from them the following results have been obtained :—

The smallest secondary stress so far as he has gone, the writer has found in the simple Warren girder, consisting of one series of equilateral triangles.

In this girder, assuming that the transverse dimensions of the chords and web members to be equal, and their sections symmetrical about the neutral axis, the secondary stress in the web members is about four times that in the chords.

Taking a Warren girder of 8 panels as shown in Fig. 38, of 100 feet span, with all members 1 foot wide, stressed to 6 tons per square inch with ordinary provision for partial loading, and a modulus of elasticity of 26,000,000, the secondary stress in the chords was 8 per cent. of the primary, and in the web members 30 per cent. of the primary. Any reduction in the width of any bar (as measured in the plane of the frame) without altering its sectional area, reduces the secondary stress in the same proportion. Hence it is desirable to keep the web members of such girders as narrow as other considerations will permit.

The usual trough or T section employed for chords is subject to only about half of the increase of stress, that a rectangular or other section symmetrical above and below experiences. This is due to the fact that the neutral axis lies so much closer to the side where the primary and secondary stresses are additive.

The N girder or that with alternate vertical and diagonal web members is subject to considerably more secondary stress than the Warren of equal dimensions and width of bars, especially in the vertical members. It also shows an intensification of secondary stress at the midspan if made with an even number of panels. At each of these points the secondary stress is at least 50 per cent. greater than at the corresponding points of the Warren girder.

In the X girder, Fig. 23, the distribution of secondary stress is peculiarly complicated. That in the chords is variable, attaining maxima values at alternate panel points, and vanishing, or nearly so, at the other alternate panel points, its maximum value being about double what it is on the corresponding Warren girder. That in the diagonals is about equal to what exists in the Warren girder if the diagonals are not connected at the points of intersection. If they are it is increased about 50 per cent.

At the root of a cantilever or over an intermediate support in a continuous girder, the secondary stresses, both in chords and web members, attain a value nearly twice as great as those in a girder simply supported at the ends and designed to carry distributed moving loads, such as ordinary road and railway bridges are subjected to. This is due to the fact that both chords and web members are stressed most heavily at the same time, which is not the case at the centre of an ordinary discontinuous girder where the maximum stress comes on the chords under full load, and on the web members under a load extending from one end to the centre.

The foregoing results, while very far removed from the alarming statements of Bender, nevertheless err on the side of pessimism. The subjoined facts all indicate certain sources of relief from secondary stress, which neither Ritter's nor the author's methods of investigation take account of.

1. The full stress of 5 or 6 tons per square inch is not maintained throughout any structure. Many parts are, for convenience, to obviate the use of too many different sections of metal, or to facilitate jointing, made 10, 20, or even sometimes 50 per cent. more massive than calculation requires. Hence the deformation of the structure is reduced below what it would be if more closely designed. This tends to reduce secondary stress.
2. In both Ritter's and the author's methods it is assumed that the chord sections are so much more massive than those of the web members as to completely overpower them, and compel them to accommodate their direction to that of the chords. This is by no means absolutely true. Assuming as a fair average that the web members are one-fourth as stiff as the chords, there will be a rotation of the joints, tending to reduce the secondary stresses in the web members by 25 per cent. and increase those in the chords. As the previously determined secondary stress in the

web members was, other things being equal, about four times that in the chords, this means a distinct gain.

3. By virtue of resistance to secondary stress and entirely apart from primary stress, a certain small portion of the load is carried.

4. It is doubtful if ever the tightest rivetted joints are absolutely rigid, and any microscopic yielding or adaptation in them tends to reduce secondary stress.

In order to provide for secondary as well as primary stress, with economy it is recommended in new designs.

(*a*) That other things being equal or nearly so, preference be given to the Warren or equilateral type of girder, with a single system of triangulation.

(*b*) That chords of trough or T section be preferred to those that are symmetrical about their horizontal neutral axis.

(*c*) That all members be kept as narrow in the plane of the girder as other conditions permit.

(*d*) That the unit stresses in the web system be made about 10 per cent. less than those adopted in the chords.

(*e*) That girders of ample depth, say not less than $\frac{1}{10}$ of the span, be preferred to shallower ones.

(*f*) That those portions of continuous girders immediately above non-terminal supports be made about 15 per cent. more massive than the corresponding parts of ordinary girders. This extra strength to taper away till it disappears at the point of contraflexure.

Existing bridges should have their stresses determined by the method previously described, and where the secondary stress and primary stress together exceed the usually permitted primary stress by more than 30 per cent., either the load should be reduced or the weak part strengthened. This strengthening must be made *without* increasing the breadth of the bar.

Girders of the N type with broad web members, such as that shown in Fig. 17, where the diagonals have a width of one-eighth of their length and are very firmly rivetted, should be looked upon with the greatest suspicion and taken in hand first, also continuous girders of all types.

As some of the above remarks may appear condemnatory of the American type of girder, of which the Hawkesbury Bridge is so magnificent an example, it is well to point out that, owing to the narrowness of the web members relatively to their length, the secondary stress is here comparatively small, not exceeding, according to the writer's method, 10 per cent., even on the assumption that the joints are perfectly rigid, which, being eye-bar and pin construction, they hardly can be, no matter how tightly fitted and bolted up.

20. *Arrangements involving severe temperature stresses.*—The existence of heavy stresses due to variations of temperature has been generally recognised in the case of metallic arches and suspension bridges, but not in the case of ordinary girders. There is, however, reason to believe that in a subtropical country like Australia, and especially in those parts where the air is dry, and so imposes but little resistance to the solar radiation, very serious stresses, amounting possibly to tons per square inch, may be produced by one of a series of parts that ought to act in unison, being exposed to the direct rays of the sun, while the others are in shade. In this way a difference of temperature of 30 deg. or 40 deg. may easily be produced, and the consequent difference of expansion will give rise to temperature stresses of 2 to 3 tons per square inch, which are cumulative upon the ordinary stresses given by statical calculation.

That these temperature stresses really exist is abundantly proved by recent experiences at the great Moorabool Viaduct on the Geelong and Ballarat Railway. Here there are four precisely similar continuous girders, each 1300 feet long. They are completely sheltered from the sun by a broad overhanging deck that surmounts them, with the exception of the lower chord of the girder on the northern side. This chord consists of a number of eyebars placed side by side, and it was observed by the officers of the Railway Department that the outside bar when highly heated by the sun actually buckled under the compression, although at a part where the heaviest tension due to the load existed. The result of this buckling was to increase the stress on the adjoining bars by probably 30 per cent. To remove this serious source of weakness, a wooden roof, the whole length of the viaduct was erected over the chord so affected.

This was undoubtedly the right thing to do, only, in the writer's opinion, an iron roof would have been more permanent and more in harmony with the monumental character and architectural pretensions of the structure.

This question of temperature stress ought to be looked into in the case of all girders that are partly or wholly exposed to direct solar radiation, and where they are found to exist in any serious degree, light screens or roofs of sheet iron should be introduced so as to ensure vital and highly stressed parts of the structure being always in the shade. As things are at present, it seems impossible to resist the conclusion that structures designed in the usual way for a working stress of 5 or 6 tons per square inch are frequently, owing to the combined effect of secondary and temperature stresses, subjected to actual stresses approaching double their nominal amount.

Further, it is recommended that girders exposed to the direct solar radiation be painted white, in order to keep them as cool as possible.

21. *Insufficient lateral bracing.*—While gravity is usually by far the greatest force acting on bridges, there are other forces that must not be overlooked operating in non-vertical directions, and which may, and as a matter of history have wrecked structures that gravity was powerless to injure. Of these forces the most important are the horizontal pressure of wind or flood against the structure, the lateral oscillation of badly balanced locomotives, and the tendency of the compression chords of the main girders to bend sideways as long columns. From this point of view, a pair of columns such as shown in Fig. 2, form a vertical cantilever, fixed at the foundation and subjected to horizontal forces from flood and wind. The columns form the chords of the cantilever, and must have sufficient sectional area to endure the consequent compression and tension, as well as the compression due to the load. In this way the bending moment is provided for. The shear requires a suitably designed web system connecting the columns, without which they cannot act together as one efficient cantilever. Now, with regard to this web system, the wildest inconsistency is found in existing bridges. In not a few cases, as for example the Charing Cross Railway Bridge, London, already referred to, it is entirely absent, although the rapid

tidal current of the Thames and the prevalence of heavy barges drifting therewith would appear to call for more than usual provision against lateral shocks. In others, as for example the railway bridge over the Murray, on the main line from Melbourne to Adelaide, it consists of very light round rods. On the Victorian Railways a fairly massive T iron bracing is usually employed, while on the older New South Wales Railway Bridges a still more massive and complex arrangement is used. The N.S.W. Roads and Bridges Branch has adopted a practice different from all the preceeding, and connects the two columns by a continuous web of sheet iron, lightened by being pierced with large oval openings. This arrangement, it is contended, gives ample strength, and is not so likely to be injured by floating logs as the others. It however involves the use of a large amount of metal and comparatively complex workmanship. Now, various and inconsistent as is the practice of Engineers in this respect, the principles of design are few and simple, and are identical with those applying to the design of framed girders. A non-redundant system of triangles, free from eccentricity, with good joints and bars massive enough to be safe from accidental blows, is all that is required. The bracing of the Johnston Street Bridge, as shown in Fig. 2, is open to serious criticism. It is in the first place redundant. The horizontal pieces are costly and complex rivetted girders, while the diagonals are angle irons of unduly light section, so light, in fact, that they can be sprung some inches by the pressure of the hand. Had the horizontal connections been left out altogether, and a fourth of the money so saved expended in making the diagonals twice as massive as at present, the structure would have been strengthened and cheapened at the same time, and the calculation of stress on its various parts made easy and certain. In this and many other cases, the bracing terminates above the summer level of the stream, leaving the bottom part, when the shear is greatest, unbraced. This course is usually excused on the ground of the difficulty of making attachments under water, and if the lower part of the columns is much more massive than the upper and well supported by the firm ground around the excuse may be accepted. It is, however, far more scientific and satisfactory to carry the bracing down to the bottom, and this has been done in two of the most recent bridges over the Yarra at Melbourne.

The lateral bracing of the girders of existing bridges is just as various and inconsistent as that of the piers—in some cases being entirely absent and in others overdone at unnecessary expense.

The older type of tubular bridges, such as the Footscray Railway Bridge, Victoria, and those at Menangle and Penrith, N.S.W., though presenting enormous surfaces to wind pressure, are invariably absolutely devoid of lateral bracing. They resist the wind simply by the resistance to bending of the main and cross girders in their weakest direction, and must be subject to extra stresses of serious amount in consequence. On the other hand, one not unfrequently sees small open lattice girders offering but insignificant surface to the wind, braced in the most costly and elaborate manner. Now, both these extremes must be wrong, and a discussion of what is really needed will be of advantage. A little consideration will show, as has been abundantly verified by experiments on models at the University, that a girder may be wrecked by lateral movement in one or more of the following ways :—

(*a*) It may fall over on its side, turning on its lower chord as an axis. This may be caused by wind or other lateral force acting on the top chord and web. The tendency will be greatest with deep and narrow girders, and becomes insignificant in the case of those that are shallow and broad. If the traffic is carried on the top of the girders this tendency may be effectively met by inserting a diagonal of sufficient section between the bottom of one girder and the top of the other at each point of support. This, with the cross girders, will keep all secure. Should, however, the traffic pass *between* the girders, as is often the case where headway is limited, such a diagonal is inadmissible. We must, therefore, make the base of the girder broad enough to secure stability, and carry a pillar up, starting the full width of this broad base, but tapering, if desired, toward the top, as indicated in Fig. 12, or, as an alternative, cross girders may be made continuous with stiff web members, so that the verticality of the plane of the latter is secured by the resistance to bending of the cross girder. Properly such cross girders should carry no load, for if loaded their deflection will affect the verticality of the main girder. This, however, can hardly be insisted upon in practice.

(*b*) The chords may be bent sideways by the pressure of wind upon the girder itself or upon objects supported by it. This pressure will be great in the case of plate web girders and of that chord of an open girder that carries the deck. It is to be met by inserting a proper system of diagonals forming with the chords of the main girder, a complete horizontal triangulation. It is certainly desirable that such a triangulation, made of stout T or angle section, be added to the older tubular girder bridges and others that do not possess it. The smaller the width as compared with the span the more important this horizontal system is. There are some cases, however, in which it is not needed. The first is when the bridge is provided with a continuous metallic deck, as is now often the case. Such deck forms a most efficient horizontal web. The second is when the bridge is built on a skew such that the cross girders attached at or near the end of one main girder meet the other main girder at a fourth to a third of the span from one end. Here the cross girders themselves form an efficient bracing.

(*c*) The compressed chord may buckle or fail as a long column. To prevent this, it must be rigidly held at frequent intervals by some system of bars preventing any small initial lateral bending from increasing. In an ordinary discontinuous girder the top chord is compressed, and if the deck is on the top, the same bracing that resists the pressure of wind on the top chord, deck, and load, will meet this requirement. If, however, the deck is on the bottom, the top chords may, if the girder be deep enough, be braced together overhead, with a complete triangulation extending from end to end. This is by far the most satisfactory method, and is usually found in the larger American and European bridges of recent construction. If the headway is insufficient to permit this overhead bracing, then the top member must be kept straight by some stiff construction continuous with the cross girders, and extending from the ends of these upward, and it is desirable that the cross girders be very deep and stiff. This is well arranged in the Toolamba Railway Bridge, Victoria, a section of which is shown in Fig. 39, and in addition the bottom chords must be kept from bending horizontally by a stiff deck or complete horizontal system of triangulation.

Very often the former of these requirements is met by projecting the ends of the cross girders some distance outside the main girders, and inserting an inclined strut from the end of the cross girder to the top of the main girder. If this system be used, these struts should be straight, of stiff section, capable of bearing compression as well as tension, and making an angle of about 30 deg. to the vertical.

The costly and complex arched connections extending overhead from one main girder to the other at the Footscray Railway Bridge, near Melbourne, and on most of the earlier railway bridges in New South Wales, are of very little value.

Existing bridges should be carefully examined, and if found insufficiently braced, have proper triangulated systems added, capable of bearing a wind pressure on the loaded structure of 20 lbs. per square foot, with a safety factor of at least 5, seeing the wind may blow from either side, rendering the question one of vibration strength.

Numerous errors occur in connection with the design and arrangement of cross girders in many early road and railway bridges, of these the following deserve notice :—

22. *Cross girders not placed in proper relation to the web system of main girders.*—A most glaring instance is to be seen in the bridge carrying the Heidelberg and Eltham Road across the Plenty River, near Melbourne, shown in outline at Fig. 40. Here the main girders are of the X type, and both triangulations are equally strong. The cross girders, however, are so placed as to discharge their load immediately upon the *alternate* panel

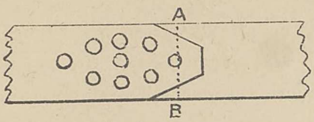


Fig. 32.

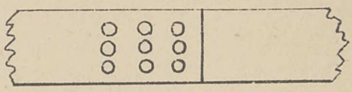


Fig. 33.

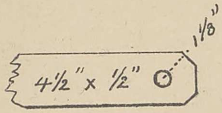


Fig 34.

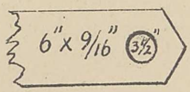


Fig. 35.

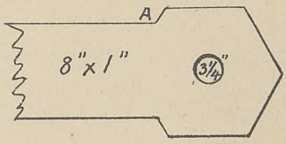


Fig. 36.

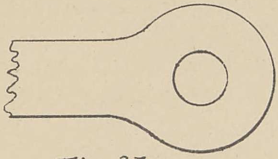


Fig. 37.

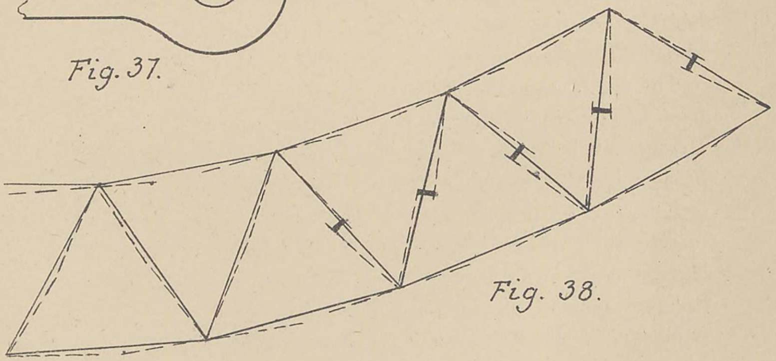


Fig. 38.

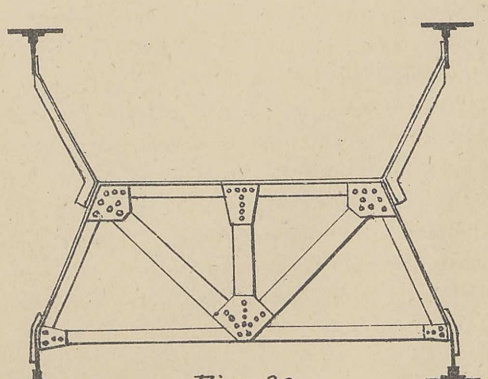


Fig. 39.

points, so that in theory, at least, one triangulation carries the whole weight. Mr. J. A. L. Waddell, a leading American Bridge Engineer, to whom photographs and particulars of this structure were sent, expresses himself as follows :—

"For unparalleled ignorance and stupidity the triangular truss illustrated bears off the palm. It is almost incredible that any man with common sense could put in a whole system of triangulation where it is impossible for it to do any work whatever."

While generally agreeing with Mr. Waddell's drastic condemnation of the absurdity of this design, I question whether his closing words are strictly correct, for the following reason: If all the load was carried on one triangulation only, the panel points connected with it would deflect more than those connected with the unloaded triangulation, and hence the top and bottom chords instead of bending in an approximately circular curve would bend in a sinous way, having numerous points of contraflexure. But such bending is impossible without evoking a considerable beam action in the chords, which are several inches in depth. Hence, some part of the load *must* be transferred to the other triangulation. How much would be so transferred it is difficult to compute, but certainly not sufficient to cause even an approximate equalization in the stress on the two triangulations. If we assume three-fourths of the total load to come on one and one-fourth on the other triangulation, we shall probably be taking a fairly favourable view. This bridge, undoubtedly, might be largely strengthened by inserting cross girders at *all* the panel points, and by replacing the thin flat diagonals near midspan of the main girders by proper double acting (strut and tie) sections capable of resisting the action of unsymmetrical loading, and this should be done at once.

Another example of a similar fault is be seen in an important bridge at Southampton, England, illustrated in "Engineering," 4th January, 1884, and criticised by the writer in "Engineering," 6th June, 1884. Here the cross girders are placed at intervals having no direct relation to the panel points of the main girder. At one point a cross girder comes exactly midway between panel points, thus bringing an unnecessary and undesirable bending moment of some magnitude upon the bottom chord.

A further instance of this fault was to be seen in the eastern extension of the Victoria Street Bridge, Melbourne, erected in 1884, and represented in Fig. 41. This, however, was remedied at the writer's recommendation.

23. *Cross girders too numerous and individually too weak.*— This fault is very commonly to be found in the earlier railway bridges of Victoria and New South Wales that have come under the author's notice. The heaviest local load to be provided for is that of the driving wheel of a locomotive, and amounts to slightly over 8 tons for one type of Victorian locomotive, and between 7 or 8 tons in five other types. In New South Wales the weights are practically the same, for, though the engines are heavier, they are supported on a greater number of wheels. But it must not be forgotten that this load is liable to be largely increased, first by the downward resolved part of the thrust or pull of the connecting rod, and second by the effect of imperfect balancing, so that 10 tons is the very least amount it would be safe to allow as the actual wheel load. Now, this load may come upon every individual cross girder in succession, and therefore every cross girder should be strong enough to resist it. At the same time there is no advantage in placing them nearer together than the minimum distance between the locomotive driving wheels, which is about 5 feet. Cross girders then, at not less than 5 foot intervals, each strong enough to carry the heaviest loaded pair of engine wheels, constitute reasonable practice, and many good recent structures correspond closely to this arrangement. On some of the earlier Victorian bridges, however, the interval is only 2 feet 6 inches, and in New South Wales, 3 feet. The stresses in these girders are found by computation to be from 7 to 12 tons per square inch—very alarming figures. Looking at many of these earlier bridges it is difficult to understand why their bottoms have not dropped out long ago. It is, however, to be remembered that some of the worst have never been fully loaded, double line bridges as at Penrith in New South Wales, and Kororoit Creek, Victoria, carrying, hitherto, only single lines of way, while in most cases heavy timber longitudinals or stout continuous decks interposed between the track and the cross girders tend to spread the load and prevent any individual girder receiving its full punishment. Reliance upon these, how-

ever, is not advisable, as bad workmanship, decay and imperfect jointing may nullify their distributing action. The best thing to do with these weak girders is to introduce longitudinal connections of great stiffness between them. An arrangement in iron or steel similar to what is known as "herring boning" between the joists of a floor was recommended several years ago by the author for Penrith Bridge, N.S.W., see Fig. 42.

24. *Improperly designed cross girders.*—These occasionally occur, especially in the earlier bridges, and seem to have arisen from an ill-advised attempt to make the cross girders of similar type to the main girders. Badly designed lattice cross girders with vital junctions made by single rivets only, and heavy compressions taken on thin flat bars of considerable length were unfortunately to be found in some early Victorian bridges. Many have now been removed, and their places taken by substantial plate girders. As an example of most improper design, Fig. 43 may be quoted, which, not many years ago, was removed, after a quarter of a century of service, from an important bridge on a busy suburban railway near Melbourne. It will be seen that the tension on bar AB is 10 tons per sq. inch, the shear on the rivet A, 8 tons per sq. inch, and pressure on the bearing area 25 tons per sq. inch—figures alarmingly high. The marvel is that such girders failed to cause appalling disaster.

It is fully illustrated here as a warning to bridge designers, and also as an encouragment; for the fact of these frightfully over-stressed constructions having carried a busy suburban traffic for a number of years without giving way, is a conclusive proof of the extraordinary endurance of the material, which was only iron, and an evidence of the wide margin of strength against unforeseen contingencies possessed by structures designed in a scientific way under usual limitations as to permissible stress.

25. *Unscientific forms of footpath brackets.*—Many railway and road bridges are provided with overhung footways outside the main girders. The brackets supporting these should be rationally designed cantilevers continuous with the cross girders, and consisting of a proper top and bottom chord, and suitable web, either of continuous plate, or triangulated series of bars. Unfortunately, however, an unhealthy desire for ornament, overlooking the fact that the really beautiful must be based upon

and grow out of that which is scientific and useful, has led many designers into extravagancies and absurdities. For example the old Church Street Bridge, Richmond, near Melbourne, originally had footpath brackets made of thin round rods bent into the curves of a ram's horn, an absurd and excessively weak form, while the recently constructed Swing Bridge at Footscray has the webs of its footpath cantilevers made of circular rings of L iron, a costly and unscientific arrangement, the exact strength of which it is impossible to compute. It is suggested that such brackets be carefully examined, tested, and if showing any signs of weakness strengthened, and that in future structures, scientific and rational forms be adopted instead of these unsatisfactory pseudo-ornamental abortions.

26. *Parapets too low, too weak, or too open.*—These are of not uncommon occurrence, and as sources of danger to the public are frequently of serious import. An examination of such parapets on bridges in or near Melbourne has revealed the fact, that some are less than 3 feet in height, while others approach 5 feet. The former are certainly dangerously low, the latter needlessly high. It is recommended that no bridge parapet should be less than 3 feet 6 inches, which is the height of the elbow of a man of medium size, while 4 feet may be taken as a maximum beyond which it is unnecessary to go. The same rule should, in the writer's opinion, be applied to the balustrades of staircases, landings, and balconies in buildings. These are usually too low, and have repeatedly been the cause of serious accidents.

No rule has been to the writer's knowledge generally accepted for the strength of parapets. He therefore proposes that to provide for the pressure of a dense crowd, they be made strong enough to endure a horizontal pressure of 100 lbs. per foot in length, applied at the top with a safety factor of not less than 3 for metal and 5 for timber.

This strength may be provided by the resistance to bending of the uprights or standards of the parapets. But if this be found to involve an undesirable amount of material, lighter standards may be employed, with sloping struts outside. These struts should be straight, inclined at an angle of not less than 20 deg. to the vertical, and extend from the projecting end of the foot-

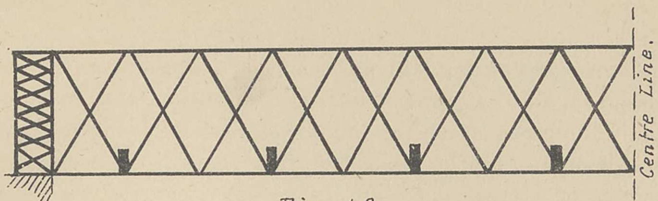


Fig. 40.

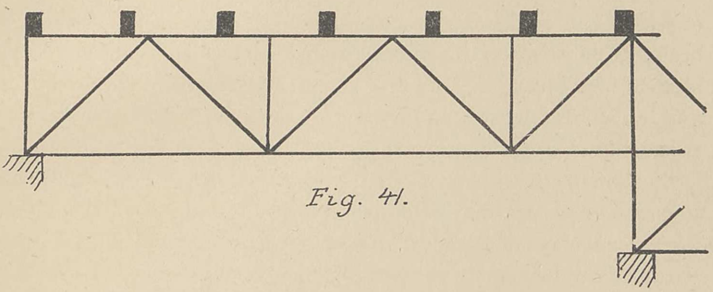


Fig. 41.

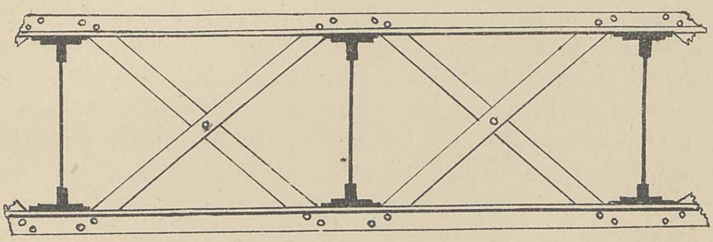


Fig. 42.

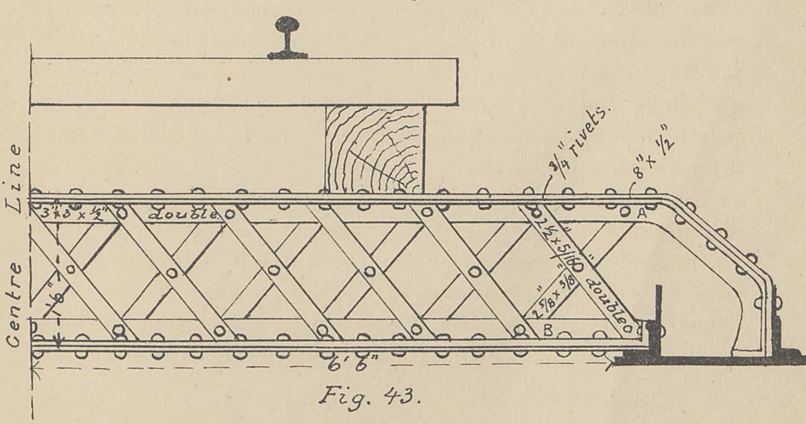


Fig. 43.

path cantilever to a point about three-fourths of the height of the parapet. They may be made of angle, T, channel, or any other stiff section of metal and securely rivetted or bolted top and bottom. In some recent and otherwise excellent bridges near Melbourne, small triangular plates of thin metal rivetted along the bottom and the vertical side have been used. These are not to be commended, as the maximum compression comes on the thin edge of the plate that is least able to endure it, and the majority of the rivets are placed where they endure but little stress and are of proportionately little service. The same amount of material and labour put in a substantial sloping strut would give far greater strength.

A parapet may be high enough and strong enough, and yet a source of danger by having openings in it large enough for children to pass through. An examination of a considerable number of such parapets leads the writer to recommend that if formed of paralled horizontal bars, these bars should not be more than 9 inches apart vertically, or if as is often the case formed of lattice work showing square openings, such openings should never be more than 1 foot square. It is recommended that all parapets accessible to the public be altered when necessary to comply with the above requirements.

In concluding this necessarily very inadequate treatment of a vast and complicated subject of extreme public importance, the writer desires to apologise for its brevity and other defects, and to express his hope that it may be the means of rendering a most important section of engineering work in some small measure more consistent, economical, and secure against accidents than it has been hitherto.

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Beregning af sekundære Spændinger i Stivede Dragerformer

a) Af de primære Spændinger og Tværsnitsarealet af hver Stang og dens Plasticitetskoef. beregnes dens Længdeforandring. (Strækning eller Sammentrykning)

b) Disse Størrelser gøres f. Ex. 100 Gange større.

c) Konstruktionen tegnes op med de ændrede Længder af Stængerne. Resultatet af dette bliver at at Knudepunkterne for Stængerne i Hoved og Fod i Stedet for at ligge i rette Linier vil ~~vil~~ ligge i Kurver som i de fleste Tilfælde tilnærmelsesvis er cirkulære. Men undertiden har Spidsen for Midten som er omvendt gotiske Buer. Disse Spidser ~~Punkter~~ angiver Punkter med stor sekundær Spænding.

O. S. V.

Kunz. On some common errors in iron Bridge design Pag 35 —

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