A WORK

ON

BRIDGE BUILDING:

CONSISTING OF

TWO ESSAYS,

THE ONE ELEMENTARY AND GENERAL, THE OTHER GIVING

ORIGINAL PLANS,

AND

Practical Details

FOR

IRON AND WOODEN BRIDGES.

BY S. WHIPPLE, C. E.

UTICA, N. Y.
H. H. CURTISS, PRINTER, DEVEREUX BLOCK.
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PREFACE.

The first part of this work was published in the beginning of the present year, having been written about a year before. At the time of that publication, the preparation of the subsequent portion of the work, had not been commenced, nor had its preparation and publication been fully determined on, though an intimation of such a purpose, was therein expressed.

Some fifty or sixty copies of the first Essay were circulated among friends, and persons who, it was thought, would feel an interest in the subject, for tbe purpose of calling attention to a matter on which I had bestowed a considerable amount of study and investigation, and on whieh I believed myself in possession of some valuable information.

The encouraging reception which that Essay met with from a number of engineers and others of high standing, whose opinions on the subject were worthy of the first consideration, determined me to suspend the distribution of more copies, until I could prepare and publish an additional part, or Essay, going more into practical detail than had been previously done, which it was thought, would add greatly to the utility and value of the work, and render it, in some measure, worthy of the attention of those interested in the subject treated of.

Under these circumstances, the work does not show that unity of design, and uniformity of plan, which would be expected in a work prepared and published all at the same time. But, though this may detract a little from the external appearance of the book, it is believed not to impair its convenience or utility.

It will be seen that the first part is illustrated by wood-cuts, distributed through the body of the work, as nearly as might be to where the figures are respectively described and referred to. It was found not so convenient to carry out the same plan in the second Essay, and therefore, I have obtained lithographic figures to illustrate that part, which are annexed to the end of the work. The figures to the last part are distinguished by numbers continued on from those in the first part; and when reference is made in the last part, to figures in the first, the pages are usually given where such figures will be found.

The numbers of the articles or sections, also, run from the first part to the second, in the same system. By the way, the numbers to articles X.

and XI., are omitted in the print. The former should be on page 9, before the word "Again," about two-thirds down the page: the latter, before the word "Discarding," at the second paragraph of page 11. No. XXVII. is also omitted at the beginning of page 32, and XXVIII. at the first paragraph from the top, page 33.

Several other trifling typographical errors found way into the first Essay, none of which, however have been observed, worthy of notice, except, that in some of the copies, at the end of the 7th line, 9th page, $\frac{1}{3}\frac{ad}{bd}$ occcurs where it should be $\frac{1}{3}\frac{ad}{b'd}w$, while in most of the copies, only the accent to the *b* is wanting. Also, in the 6th line from the bottom of page 45, the word *pieces* should be prices. I may as well in this place, refer to a few errors that occur in the second Essay, which, as well as the preceding, it will be well to turn to and correct with a pencil before reading the work. First, the word *and*, at the beginning of the 9th line, 58th page, should be transposed between the figure 10, and the word *cylinders*, in the same line. Second, the letter *c*, (referred to in the 13th line from the bottom of page 67,) should be on the cross bar nearest to *a* and *b*, Fig. 13, Pl. I. Third, in the 7th line of page 77, *cb* and *cd* should be *ab* and *cd*. The other errors that have been noticed, are merely mistakes of a letter or two, which the reader will readily correct as they meet his eye. Further remarks or explanations are not deemed necessary in this place.

In offering this little work to the Engineering profession, I would not be suspected of presuming that the subject has been exhausted, or that a want which has been long and deeply felt in this branch of the profession, is fully satisfied. I may, however, be allowed to hope, that my labors in the field will have been the means of effecting one step of advancement towards the attainment of so important a desideratum.

AN

ESSAY

ON

BRIDGE BUILDING:

CONTAINING

ANALYSES AND COMPARISONS

OF THE

PRINCIPAL PLANS IN USE,

WITH

INVESTIGATIONS AS TO THE

BEST PLANS AND PROPORTIONS,

AND THE

RELATIVE MERITS OF

WOOD AND IRON,

FOR BRIDGES.

BY S. WHIPPLE, C. E.

MATHEMATICAL AND PHILOSOPHICAL INSTRUMENT MAKER.

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AN ESSAY

ON

BRIDGE BUILDING.

I. A Bridge is a structure for sustaining the weights of carriages and animals in their transit over a stream, gulf, or valley.

Bridges are built of various plans and dimensions, according to the circumstances and objects of their erection. My present purpose is, after a few remarks upon the general nature and principles of Bridges, to attempt some analyses and comparisons of the respective qualities and merits of various general plans, with a view of deducing practical results as to a judicious and economical choice and application of materials in the construction of those important erections.

II. The force of gravity, on which the weight of bodies depends, acts in vertical lines, and consequently, a body can only be prevented from falling to the earth by a force equal and opposite to that with which gravity acts on the body. This resisting force must not only act vertically upward, but the line of its action must pass through the centre of gravity of the body it sustains. All the forces in the world, acting parallel with, or perpendicular to, the vertical passing through its centre of gravity, could not prevent an ounce ball, (concentrated to the point of its centre of gravity,) from falling to the centre of the earth, unless it were a horizontal force capable of giving the ball a projection, such that the centrifugal tendency should equal or exceed gravity: a kind of force which could never be made available towards preventing people from falling into the water in crossing rivers, consequently of no use in bridge building.

In fact, nothing but a continuous series of unyielding material particles, extending from an elevated body downward to the surface of the earth, can hold or sustain that body above the earth, by vertical and horizontal action alone, either separately or in combination.

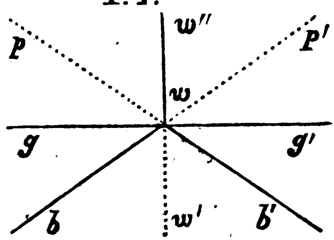
III. Suppose a body, no matter how great or small, placed above the earth, with an unfathomable abyss, or an inaccessible space beneath it. Attach as many cords to it as you please, strain them much or little, (only horizontally,) the body will fall nevertheless. Thrust any number of rods, with whatever force you may, horizontally against it, still the body will fall. Moreover, the space beneath being inaccessible, there is no foundation or foothold on which to rest a post or stud, that may directly resist the action of gravity, and the lines of all other vertical forces or resistances pass by the body without affecting it. In the situation here supposed, the body can only be prevented from falling by oblique forces, that is, by forces whose lines of action are neither exactly horizontal, nor exactly vertical.

Attach two cords to the body, draw upon them obliquely upward and outward, in opposite directions, or from opposite sides of the abyss, with a certain stress, and the body will be sustained in its position. Apply two rods to it obliquely upward, of a proper degree of stiffness, in the same verticle plane, on opposite sides of the perpendicular, a certain thrust exerted on those rods will prevent the descent of the body.

IV. Here, then, we have the elementary idea, the grand fundamental principal in bridge-building. Whatever form of structure be adopted, the elementary object to be accomplished is, to sustain a given weight in a given position, by a system of oblique forces whose resultant shall pass through the centre of gravity of the body, in a vertically upward direction, in circumstances where the weight cannot be conveniently met by a simple force in the same line with, and opposite to, that of gravity.

To dwell a little on this elementary idea, let us suppose $b.b'$ [Fig. 1,] to represent the banks of a river, or the abutments of a bridge, and gg' the line of transit for carriages, &c.; and suppose a load of a certain weight w, to have arrived at a point centrally between $b.b'$. The simplest method of sustaining the weight is, perhaps, either to erect two oblique braces, bw $b'w$, or suspend two oblique chains, $pw.p'w$ from fixed supporting points at $b.b'$ or $p.p'$.

F.1.



It is not necessary that the weight be exactly at the angular point w, of the braces or chains, but it may be sustained by simple suspension at w' below, or simple support at w'', and such obliquity may be given to the chains or braces, as may be most economical, a consideration which will be taken into account hereafter.

V. Thus we see how a weight may be sustained centrally between the banks of a river, or the extremities of a bridge. But the structure must not only provide for the support of the weight at this point, but also at every other point between $b.b'$ or $g.g'$, and it is obvious that the same plan and arrangement will apply at any other point as at the centre, with only the variation of making the braces or chains of unequal lengths.

This, however, would require as many pairs of braces or chains as there were points between $g.g'$, a thing, of course, impracticable. We therefore resort to the lateral strength and stiffness of beams—phœnomena with which all have some acquaintance, and without digressing in this place to investigate their principles and causes, I will merely assume, as a fact sustained by all experience, that for sustaining weights between two supporting points, upon nearly the same level, a simple beam affords the most economical means, until those points exceed a certain distance assunder, which distance will vary with circumstances, but in bridge-building will seldom be less than 10 to 14 feet, where timber beams are employed. Hence, for bridges of a length of 12 or 14 feet, usually, nothing better can be employed than a structure supported by longitudinal beams, with their ends resting on abutments or supports upou the sides of the stream. Of course, I shall not be understood here, as having any regard to stone or brick arches. For though these are advantageously used for short spans, and in deep valleys, where the expense of constructing high abutments for supporting a lighter superstructure would exceed or approximate to that of constructing the arch, it is my purpose to speak only of those lighter structures, composed mostly of wood and iron, and supported by abutments and piers of stone, or by piles, or frames of wood.

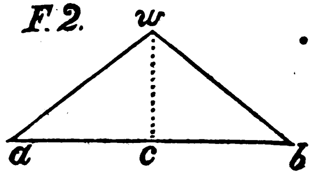
Having then, adopted the use of beams for supporting short distances, it is only necessary on longer stretches, to provide support for a point once in 10 or 14 feet, by braces, &c. from the extremities, and for the intermediate points, depend on beams or joists extending from one to another of the principal points provided for as above.

VI. For a span of 20 or 30 feet, it would seem that no better plan could be devised, than to support a transverse beam midway between abutments, by two pairs of braces or suspension chains, one pair on each side of the road-way; this transverse beam affording support for longitudinal beams extending therefrom to the abutments. When suspension chains are used, it is usually called a suspension bridge. If braces be employed it is termed a trussed bridge.

VII. Before advancing further, it will be proper to refer to a fact which has not yet been taken into account, though one of the utmost importance.

The sustaining of a weight by oblique forces, gives rise to horizontal forces for which it is necessary to provide counteraction and support, as well as for the weight of the structure and its load. The braces aw and bw, in supporting the weight w, [Fig. 2,] act in the directions of their lengths with a certain force, which bears the same relation to $\frac{1}{2}w$ as wa or wb bears to wc, and

F. 2.



their action at each of the points a and b, is the same as would be that of a perpendicular force equal to $\frac{1}{2}w$, and a horizontal force outward equal to $\frac{1}{2}\frac{ac}{wc}w$. Therefore the abutments must be calculated to withstand this horizontal force, or some sort of connection or ligature must be provided between a and b, capable of withstanding it, in which case the action upon each abutment is simply a vertical pressure equal to $\frac{1}{2}w$, omitting the weight of the structure.

This horizontal action is called the horizontal thrust, and if the abutments be relied on to sustain it, they will require to be better and more strongly built than when they only sustain the weight.

This thrust always has place when a weight is sustained by any means except by a continuous series of solid bodies extending from it directly downward to the earth. Hence the necessity, in all cases of indirect sustension of weights, of having, at least, 2 pieces or parts, either inclining in opposite directions, or having unequal inclinations, that their horizontal actions may neutralize one another; otherwise the body would move in the direction of the greater horizontal thrust.

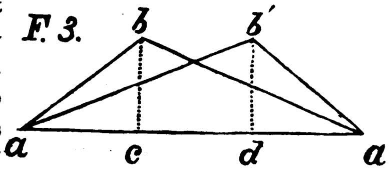
The horizontal thrust has place even in the case of a simple beam, being exerted by the upper part, and counteracted by the lower.

In the case of suspension bridges, the horizontal action of the chains is inward, and a counteracting outward thrust is exerted by that portion of the earth between the points of attachment; or it may be met and opposed by a rigid body extending from one point of suspension to the other.

Whether the horizontal thrust may be more economically sustained by the abutments or otherwise, will depend on circumstances, and can more easily be determined in a subsequent part of the investigation.

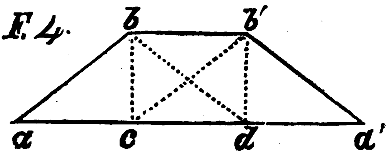
VIII. In structures exceeding about 25 feet, or probably 30 feet at most, the length of joist from the centre to the ends, would require so great a size to give them the requisite stiffness, that their weight and cost would be quite objectionable. It becomes expedient, then, in such cases, to provide support for more than one principal point, or transverse bearer. A superstructure of from 30 to 40 feet, may be constructed with two cross bearers, sustained by two trusses, with two pairs of braces each, as may be seen in Fig. 3, and it will probably be best, in such short spans, at least, to provide connecting ties between the feet *a.a′*, of the braces. The cross bearers may be at *b.b′* or suspended below at *c.d.* The thrust upon each brace, will be as the length of the brace divided by the perpendicular *bc* or *b′d*, and the stress upon the tie *aa′*, inversely as *bc*. Hence the propriety of giving the trusses a considerable elevation. Unless connected over the top, however, they become top-heavy, if carried too high, and are with difficulty sustained in an erect position. They should, therefore, usually be connected across the top, and secured by lateral bracing when designed for heavy loads.

F. 3.



IX. The above is the simplest form for a truss to sustain two cross bearers. It is not, however, the most economical. A better, as well as a more common form of a truss for two bearers is that shewn in Fig. 4.

To compare this form with that in Fig. 3, suppose a weight, *w*, placed at each of the points *b.b′*. In virtue of those two weights, there will be a vertical pressure equal to *w*, exerted at *a*, through the medium of the brace



ba, and a horizontal thrust equal to $\frac{ac}{bc}w$, and the same at a'. Similar weights at bb', in Fig. 3, would produce the same vertical pressure, but the thrust would equal $\frac{2}{3}\frac{ac}{bc}w+\frac{1}{3}\frac{ad}{bd}w$ But $ad=2ac$ and $b'd=bc$. Hence, by substitution, we have the horizontal thrust equal to $\frac{4}{3}\frac{ac}{bc}w$, which is one-third greater than in case of the truss shown in Fig. 4, having the same general dimensions.

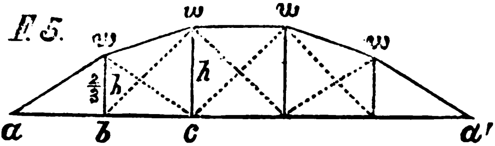
Hence truss [Fig. 3,] requires one third more material in the tie aa', or one-third more power of abutments to sustain thrust, than truss [Fig. 4,] and a similar excess of material in braces; and as an offset, truss 4 requires the additional braces bd, $b'c$, possessing about one-third the strength of the main braees ab,$a'b'$. This still leaves the balance in favor of truss 4, aside from the saving of horizontal thrust. Hence truss 4 is decidedly preferable, except where the abutments sustain the thrust. For it will readily be seen that the horizontal thrust of the braces bd, $b'c$, in truss 4, cannot act directly on the abutment, whereas the whole thrust of all the braces in truss 3, may be directly sustained by the abutments.

Again, let us suppose two trusses, 5 and 6, of the same length and height, with four bearing points at equal distances, and loaded at each of these points with a certain weight w.

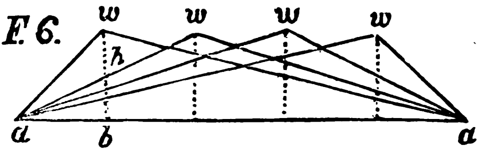
In truss 5, let aw, ww. &c. be sub-chords of an arch of which aa' is the primary chord, and let $aa'=5$, and consequently ab, bc, &c. each$=1$. Also let the vertical $wb=\frac{2}{3}h$, and the vertical $wc=h$. Now the points $a.w.w.$ &c. a' are very nearly in a circular arch, and nearly in equilibrio under the pressure of the weights $w.w.$ &c.—The horizontal thrust produced by the weights on this truss is $2w\frac{ab}{\frac{2}{3}h}=\frac{2w}{\frac{2}{3}h}=\frac{3w}{h}$

In truss 6, let each of the weights *w.w*, &c. be sustained by an independent pair of braces *wa*, *wa'*, of the same

F. 5.



F. 6.



height$=h$. Then each pair of braces will exert a certain horizontal thrust which will be the same in both directions, and the sum of which, will shew the whole thrust produced by all the weights, and will be equal to $\frac{4w}{5h}+\frac{3}{5}\frac{2w}{h}+\frac{2}{5}\frac{3w}{h}+\frac{1}{5}\frac{4w}{h}=\frac{4w}{h}$ which is $\frac{1}{3}$ greater than in the case of truss 5.

Hence the truss 6 would require more material in about the same ratio to support the weight with the same proportionate stress, besides that the long braces in truss 6 would be unable to bear the same stress in proportion to their cross section, as the shorter pieces *aw*, *ww*, &c., in truss 5.

But the truss 6 has the advantage of being able to sustain itself when part of the weights are removed, as each one is sustained independently of the rest; whereas truss 6 would be thrown out of its equilibrium in such a case, and could not stand without additional support. This may be afforded by diagonal braces, or ties between each two of the points *w.w*, &c., as shewn by the dotted lines, which will require about one-third as much material as is required to sustain the horizontal thrust independently of abutments, or one-third as much as the arch *a*, *w.w*, &c. contains, as will appear further on.

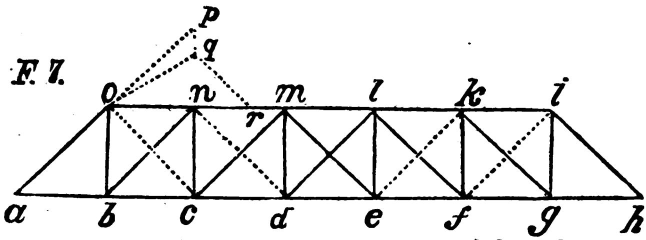
Hence, the result of this comparison is analogous throughout, to that obtained from the comparison of trusses 3 and 4. And when we take into the account the disadvantage at which the long braces in truss 6 must act, and the general practical conveniences of the two plans, it is probably risking little to assume, as a settled fact, that the truss containing the arch, and diagonals between the main points of support, is decidedly preferable to that in which each cross bearer is sustained by an independent pair of braces.

Discarding, then, the plan of independent bracing, as in trusses 3 and 5, as inferior to others, I will proceed to a comparison of the arched truss with another form of trussing, by a sort of cancel-work, in which each oblique piece extends from one bearing point to another, between two parallel horizontal ribs or stringers, bounding the truss at top and bottom.

Fig. 7 exhibits a canceled truss. In order to analyze and determine the merits of this kind of truss, I will first consider it to be loaded at each of the bearing points *o*, *n*, *m*, *b*, *k*, *i*, with a certain weight *w*. Now, supposing the diagonals, except *ao* and *hi*, to act by tension, and the verticals by thrust, it is obvious that the parts in the upper rib between *o* and *i*, will act by thrust, and the parts between *a* and *h*, by tension; also, that the diagonals *ao* and *hi* will act by thrust, and of the others, the dotted ones only, act in this condition of the load, while the rest are relaxed and useless. In order, then, to ascertain the stress upon each part, I say firstly, at the point *a*, we have an equilibrium between three forces, viz: the thrust of *ao*, the tension of *ab*, and the resistance of the abutment, or support on which the point *a* rests, which last is equal to $3w$, (not regarding the weight of the truss;) consequently, supposing *ab*, *bc*, &c., to be each equal to *bo*, the tension on *ab* is manifestly also equal to $3w$, and the thrust on *ao* is equal to $3w\sqrt{2}$.

Again, at *o*, we have an equilibrium of four forces, exerted in the directions of the lines *ao*, *on*, *ob* and *oc*, of which *ao* has just been found equal to $3w\sqrt{2}$, and *ob* is equal to *w*, being simply the weight resting at the point *o*. Then, extending *ao* and taking $op=ao$ to represent

F. 7.



$3w\sqrt{2}$, letting fall the perpendicular $pq=\frac{1}{3}ob$, we have *oq*, representing the resultant of *ao* and *ob*, which reduces the forces to 3. Then drawing *qr* parallel with *oc*, *qr* obviously represents the tension of *oc*, which is equal to $2w\sqrt{2}$, and *or* represents the thrust of *on*, which is equal to $5w$. The tension of *bc* is $3w$, the same as that of *ab*, since the parts *ob* and *bn* have no action.

Nextly, at the point *c*, the tension of *oc* has a horizontal action equal to $2w$, which added to the tension of *bc* ($=3w$) gives $5w$ as the tension of *cd*. The tension of *oc* has also an upward action equal $2w$, counteracted by the thrust of *cn*, which, of course, is also equal to $2w$. One half of this thrust of *cn*, is counteracted by the weight *w* at *n*, and the other half by the oblique action of *nd*, which, of consequence, is equal to $w\sqrt{2}$, and exerts a horizontal force equal to *w* upon the part *nm*. But *nm* is also acted on in the same direction by the thrust of *on*, before shewn to be equal to $5w$, consequently the thrust of *nm* is equal to $6w$.

At *d*, the horizontal action of *nd*, ($=w$) in addition to the tension of *cd*, ($=5w$) gives $6w$ as the tension of *dc*. The upward action of *nd* ($=w$) just sustains the weight *w* at *m*, through the medium of *dm*, and the thrust of *nm* ($=6w$) is counteracted by the thrust of *ml*, which, of course, must also be equal to $6w$.

The stress on the remaining parts is the same as on those analogously situated on the opposite side of the centre, which we have just determined.

Now, by going through with similar analyses, with the weights successively removed, beginning at *o*, and then by removing from both ends at once, &c., it will be seen that the maximum stress falls on the horizontal parts, and upon *ao* and *hi*, when the truss is loaded throughout. That with the weight at *o* removed, we have the maximum stress on *oc*, $=\frac{15}{7}w\sqrt{2}$, and on *nc*, $=\frac{15}{7}w$. With the weights removed from *o* and *n*, we have the maximum stress on *nd*, which is $\frac{10}{7}w\sqrt{2}$, and on *dm*, which is $\frac{10}{7}w$.

The maxmium on *me* $(=\frac{6}{7}w\sqrt{2})$ occurs when the weights are removed from *o.n* and *m*. On *lf*, $(=\frac{3}{7}w\sqrt{2},)$ when *o.n.m.* and *l* are unloaded, on *k.g*, $(=\frac{1}{7}w\sqrt{2},)$ and on *i*.g, $(=\frac{1}{7}w)$ when the point *i* alone is loaded.

The maximum stress on each part, then, may be arranged in tabular form, thus:

Acting by Thrust. Parts.	Max. Thrust.	Acting by Tension. Parts.	Max. Tension.
ao and *ih*	$3w\sqrt{2}$	*ac* and *fh*	$3w$
on and *ki*	$5w$	*cd* and *ef*	$5w$
nm, *ml*, and *lk*	$6w$	*de*	$6w$
ob and *ig*	$\frac{1}{7}w$	*oc* and *if*	$\frac{15}{7}w\sqrt{2}$
nc and *kf*	$\frac{15}{7}w$	*nd* and *ek*	$\frac{10}{7}w\sqrt{2}$
md and *el*	$\frac{10}{7}w$	*me* and *ld*	$\frac{6}{7}w\sqrt{2}$
		lf and *mc*	$\frac{3}{7}w\sqrt{2}$
		nb and *kg*	$\frac{1}{7}w\sqrt{2}$

Now, multiplying the length of each of these parts by the maximum stress under all the conditions of the load, it will give us the means of making a near comparison of the amount of material required on this plan, with that required in another plan in which the pieces are similar in general dimensions, and exposed to like forces as in this plan.

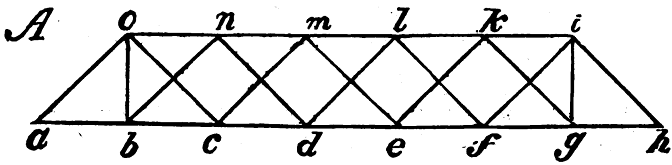
To simplify the calculation, let $ab=ob$, and represent the unit of length, which will give the length of the diagonals each$=\sqrt{2}$.

Our products then, will be,—

For parts exposed to a Crushing Force.	
ao and *ih*	$12\ w$
Parts between *o* & *i*	$28\ w$
Verticals	$7.42w$
Total,	$47.42w$

For parts exposed to Tension.	
Parts between *a* & *h*	$28w$
Diagonals	$20w$
Total,	$48.w$

XII. There is another modification of the cancelled truss, which dispenses with the vertical pieces, except, perhaps at the ends, or at the first bearing points from the ends, and I proceed to give an analysis of the action of its various parts.



In Fig. A., let the parts be so arranged and connected, that the lower stringer *ah* and the verticals *ob* and *ig*, may act by tension only, *ao* and *ih* by thrust only, and the remaining diagonals by either thrust or tension.

A weight *w* at *o* will exert a thrust on *ao* equal to $\frac{6}{7}w\sqrt{2}$, & on *oc*, *me*, *kg*, & *i.h*, each, equal to $\frac{1}{7}w\sqrt{2}$, and a tension upon *cm* and *ek* equal to $\frac{1}{7}w\sqrt{2}$: also a tension equal to $\frac{1}{7}w$ on *ig*.

A similar weight at *n* will exert a thrust$=\frac{5}{7}w\sqrt{2}$ on *nb* and *oa*, and on *nd*, *lf*, and *ih*, a thrust$=\frac{2}{7}w\sqrt{2}$; a tension equal to $\frac{5}{7}w$ on *ob*, and a tension$=\frac{2}{7}w\sqrt{2}$ on *dl*, and *fi*.

A weight *w* placed at *m*, will exert a thrust$=\frac{4}{7}w\sqrt{2}$ on *mc* and *oa*, and a thrust$=\frac{3}{7}w\sqrt{2}$ on *me*, *kg*, and *ih*; a tension$=\frac{4}{7}w\sqrt{2}$ on *oc*, a tension$=\frac{3}{7}w\sqrt{2}$ on *ek*, and one equal to $\frac{3}{7}w$ on *ig*.

The effects of a weight *w* at *l*, will be, a thrust$=\frac{3}{7}w\sqrt{2}$

on *ld*, *nb*, and *oa*, and one equal to $\frac{4}{7}w\sqrt{2}$ on *lf* and *ik*, a tension $=\frac{3}{7}w\sqrt{2}$ on *dn*, one equal to $\frac{3}{7}w$ on *bo*, and one equal to $\frac{4}{7}w\sqrt{2}$ on *fi*.

A weight w at k, will produce a thrust$=\frac{2}{7}w\sqrt{2}$ on *ke*, *mc*, and *oa*, and one equal to $\frac{5}{7}w\sqrt{2}$ on *kg* and *ih* ; a tension$=\frac{2}{7}w\sqrt{2}$ on *em* and *co*, and one equal to $\frac{5}{7}w$ on *ig*.

A weight w at i, will exert a thrust$=\frac{6}{7}w\sqrt{2}$ on *ih*, one equal to $\frac{1}{7}w\sqrt{2}$ on *if*, *ld*, *nb*, and *oa* ; a tension$=\frac{1}{7}w\sqrt{2}$ on *fl* and *dn*, and one equal to $\frac{1}{7}w$ on *bo*.

Then, collecting all those partial effects upon the various parts together in tabular form, representing thrust by the sign — and tension by +, they will stand in order, thus:

For *oa* & *ih* each $(-\frac{6}{7}-\frac{5}{7}-\frac{4}{7}-\frac{3}{7}-\frac{2}{7}-\frac{1}{7})w\sqrt{2}=-\frac{21}{7}w\sqrt{2}=-3w\sqrt{2}$

ob & *ig*.... $(+5+3+1)\frac{1}{7}w=\frac{9}{7}=w \quad +1.286w$

nb & *kg*.... $(-5-3-1)\frac{1}{7}w\sqrt{2}=- \quad \frac{9}{7}w\sqrt{2}.=-1.286w\sqrt{2}$

mc & *lf* $+\frac{1}{7}w\sqrt{2}-(4+2)\frac{1}{7}w\sqrt{2}= \quad +,142w\sqrt{2}-,857w\sqrt{2}$

ld & *me* $+\frac{2}{7}w\sqrt{2}-(3+1)\frac{1}{7}w\sqrt{2}= \quad +.286w\sqrt{2}-,571w\sqrt{2}$

ke & *nd* $+(1-3)\frac{1}{7}w\sqrt{2}-\frac{2}{7}w\sqrt{2}= \quad +,571w\sqrt{2}-,286w\sqrt{2}$

if & *oc* $+(2+4)\frac{1}{7}w\sqrt{2}-\frac{1}{7}w\sqrt{2}= \quad +,857w\sqrt{2}-,142w\sqrt{2}$

The quantities with the sign + prefixed shew the maximum tension, and those with the sign—the maximum thrust or crushing force to which these parts respectively can be exposed, and the differences, the effects produced by the whole uniform load.

For the action of the horizontal parts, it is manifest that the tension of *ab* is equal to horizontal part of the thrust of *oa*, that is, equal to $3w$. On *b.c* it is equal to the same, + the horizontal action of *nb*, that is,$=3w+1.286w$ $=4.286w$.

cd sustains this same force, + the horizontal part of the action of *oc* and *cm* under the uniform load, that is, $=(4.286+,715+,715)\ w=5.716w$.

de sustains the tension of *cd*+the horizontal action of *nd* and *dl* under the uniform load,$=(5.716+571)\ w=$ $6.287w$.

ef, *fg*, *gh*, sustain the same forces as *cd*, *bc*, *ab*, as just determined.

In a similar manner we determine the thrust of the parts between *o* and *i* to be, for *on* and *ki* equal to 3*w*+, 714*w*=3.714*w*; for *nm* and *bk*, equal to 5,28*w*, and for *mb*, equal to 6.28*w*.

Now, multiplying the lengths of the different parts by the maximum stress upon each, we have, for parts exposed to

Thrust,		**Tension.**	
ao and *ih*..........	12.000*w*	From *a* to *h*.......	32.290*w*
From *o* to *i*........	24.260*w*	Verticals *ob* & *ig*...	2.571*w*
Other diagonals....	12.568*w*	Diagonals.........	7.424*w*
Total,.........	48,828*w*	Total,........	42.285*w*
Detuct for parts that act by thrust and tension, principally tension,	1.712*w*	Detuct for parts acting by thrust and tension, principally thrust,	1.712*w*
	47.116		40.573*w*

This shews a difference of about 8 per cent in favor of truss A. over truss 7.

But it will be observed that in truss A. the number and lengths of pieces exposed to thrust are greater, which, in a measure, will counteract the seeming advantage. This plan should, doubtless, be preferred in many cases, especially for wooden bridges.

XIII. Fig. 8 represents an arched truss, of the same general parts and dimensions, and in the same condition as to load, &c., as truss Fig 7.

When loaded throughout equally, the arch is supposed to be in equilibrio without any action of the diagonals.

Now, it is obvious that the three forces meeting at the point *a*, viz, the resistance of the support, the thrust of *oa*, and the tension of *ab*, are to one another as *ob* : *oa* : *ab*. But the first is equal to 3*w* (neglecting, as in the case of truss 7, the weight of the structure,) consequently the two latter are respectively equal to $3w\frac{ao}{bo}$ and $3w\frac{ab}{bo}$.

These quantities may be calculated with the utmost precision by trigonometry. But for practical purposes generally, they may be determined with sufficient accuracy by geometrical construction, and are found to be $3w\frac{ao}{bo}=6.53w$, and $3w\frac{ab}{bo}=5.83w$. Hence, making $ab=1$ we have $\frac{3w}{bo}=5.83w$ and $\frac{3}{bo}=5.83$. Then substituting this value of $\frac{3}{bo}$ in the expression $3w\frac{ao}{bo}$, we have the thrust of $ao=5.83ao.w$.

The number 5.83 is peculiar to this particular proportion of truss, *i.e*, having 6 principal bearing points, and $md=\frac{1}{7}ah$.

Now, it is manifest that *ah* throughout sustains the same tension as the part *ab*.

Then, drawing *np* parallel with *ao*, the three forces acting at *o*, will be as *ao* $(=pn,)$: *on* : *op*. But the thrust of $ao=3w\frac{ao}{bo}$. Therefore the thrust of $on=3w\frac{on}{ob}=\frac{3}{ob}\ on.w=5.83onw$.

In like manner we find the thrust of $nm=583nm.w$, and the thrust of $ml= 5.83w$.

Now we find by analysis that these quantities represent the maximum stress upon all the parts of this arch *a.o.h*, and the chord *ah*, in any of the conditions of the load supposed in relation to truss 7, and multiplying the length of each part by its maximum stress, the chord *a.h*, gives a product $7\times 5.83w=40{,}81w$.

For the arch we have $5.83w\times(ao^2+on^2+nm^2)2+5.83w$. But $ao=1.12$ (as determined by geometrical construction,) $on=1.05$ and $nm=1.01$, and substituting these values in the above expression, we have $5.83w\ (1.25+1.10+1.02)2+5.83w,=45.13w$=the sum of the products of the lengths of the different portions of the arch, multiplied by the maximum thrust exerted by each respectively.

XIV. In order to ascertain the maximum stress upon the diagonals and verticals, (the former acting by tension and the latter by thrust, as in truss 7,) we commence by removing the weight from i. The pressure upon the support at h, then, will be diminished by $\frac{6}{7}w$, and upon the support at a, by $\frac{1}{7}w$, and the former becomes $3w-\frac{6}{7}w = \frac{15}{7}w$. The three forces acting at h, therefore, will be represented by $\frac{15}{7}w\frac{gh}{ig}$ for the tension of gk, and $\frac{15}{7}w\frac{ih}{ig}$ for the thrust of ih.

Then, taking iq on hi produced, by any scale, to represent the thrust ih, and drawing qr parallel with if, till it meets ik in r, it is obvious that the three forces acting at i, viz, the thrust of ih and ik, and the tension of if, will be represented respectively by the sides of the triangle i, q, r, parallel respectively with the directions of those forces, and may be measured with scale and dividers, or calculated trigonometrically; and it will be seen that the pressure upon the support h, enters as a factor in the expressions for all these forces. But this factor is manifestly the greatest possible in the case here supposed, except when the whole, or a part of the weight at i were restored, in which case the part if would obviously be relieved of the whole, or a part of the stress it sustains when the whole weight at i is removed. Hence it follows, that the maximum stress on if, is when all the bearing points but i have their full load, and the point i is without load.

Now, taking $fs=qr$ and drawing $s.t$ parallel with gf, st will represent the horizontal, and ft the vertical effect of the action of fi; the latter effect being counteracted by the thrust of $k.f$, and the former, in addition to the tension of fh, is resisted by the tension of ef. Therefore, ft represents the greatest stress which kf can receive from ef alone. But it is also liable to the tension of lf, which will act in conjunction with if when k is loaded, and i and l are unloaded, in which case if will have less than its maximum action. It is found, however, by analysing the forces under various conditions of the load, that the maximum action on kf, occurs when if or fl, one or the other, is under a maximum stress, and is nearly the same in both cases.

XV. If i and k be unloaded, the pressure on the support at h, is equal to $\frac{10}{7}w$, & the thrust of ih $=\frac{10w \times ih}{7ig}$. Taking iq' by the scale to represent this quantity, draw $q'r'$ parallel with if, and $q'r'$, compared with the same scale, will give the stress of if, from which, as in the former case, we obtain ft' to represent the thrust of kf. Also ir' represents the thrust of ki. Then, taking $kv=ir'$, on $i.k$ produced, raising the vertical $vx=ft'$, and joining kx, the line kx will represent the resultant of the forces kv and ft', and xy, drawn parallel with ke, will represent the tension of ke, which is its maximum strain. Also, ky represents the thrust of kl, and taking $ez=xy$, and drawing za' parallel with ef, we have ea' to represent the maximum thrust of el.

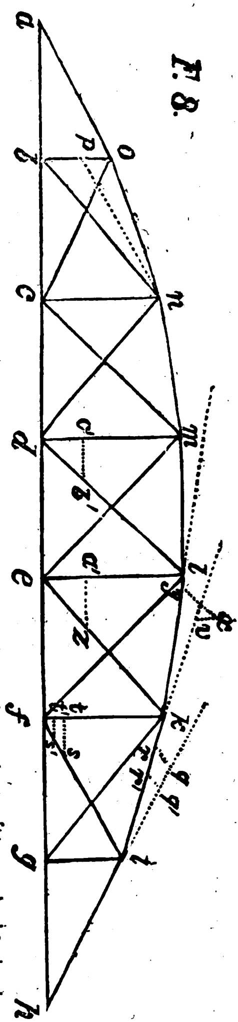


Fig. 8.

By a similar process, removing the weight from l, we obtain db' and dc' as the maximum action of dl and dm, and so of mc, nb, &c.

Or the analysis may be commenced at a instead of h, varying the process as the tyro in dynamics will readily see the circumstances to require; the same results will be obtained, except what may arise from inaccuracies of construction and measurement.

The best plan, perhaps, is to commence the analysis at both ends. and by the agreement or disagreement on meeting near the middle, a check will be afforded against errors. This method will also prevent errors from accumulating to the same extent as when the analysis is pursued throughout the whole length in one direction.

XVI. By this kind of analysis we have the means of determining, to any necessary degree of accuracy, the stress upon every part of the truss under any given circumstances, whereby, knowing the strength of the material employed, the structure may be judiciously proportioned in all its parts.

By careful geometrical construction and analysis, in the manner above explained, I obtain for the maximum strain of the diagonals as follows: Of *bn* and *cm*, 1.1*w*, of *dl*, 1.25*w*, of *ek*, 1.2*w*, and of *fi*, 1.05*w*; and to obtain the products of these quantities multiplied by the lengths of the pieces, it will be sufficiently near, the pieces being nearly of the same lengths, to multiply the aggregate length by the mean maximum stress, which gives the aggregate product=15.07*w*. For the verticals I obtain 3.70*w*.

Hence the products for the whole truss will stand,

Exposed to Thrust,		**Exposed to Tension.**	
Arch, (Art. 13).....	45.13*w*.	Chord *ah*...........	40.81*w*.
Verticals,..........	3.70*w*.	Diagonals,.........	15.07*w*.
Total for truss 8,....	48.83*w*.	Total,..............	55.88*w*.
Total for truss 7, (See Art. 11.) ..	47.42*w*	Total for truss 7,...	48.00*w*.
Difference,.........	1.41*w*.	Difference,.........	7.88*w*.

Shewing that a truss on the plan seen in Fig. 8, will require about 16 per cent more material to withstand tension, and nearly 3 per cent more to withstand a crushing force, than one on the plan shewn in Fig 7.

This is a legitimate mode of comparison generally, as regards those parts exposed to tension. But with respect to those exposed to a crushing force, it is only applicable where, as in the present case, the pieces exposed to that force may be nearly of the same dimensions. Hence the comparison between trusses Figs. 7 and 8, is more fair and reliable, than between either of these and truss A.

The verticals may be dispensed with in the arched truss, by disposing the diagonals so that they may act by thrust or tension, but I will not stop in this place to examine into the effects of that modification.

XVII. We have hitherto had regard only to the forces and effects produced by the load, independently of the weight of the structure. For the effects of the latter, it is sufficient to regard this weight as included in *w.w*, &c., as far as it regards those parts that sustain their maximum stress when the load is uniform throughout. But the weight of the structure being constant, and not variable like that of the additional load, its effects are always the same, and confined to those parts which act under the uniform load. Hence, it is proper to calculate the effects of uniform weights, $w'w'$, &c., one at each of the bearing points, where the weight of the structure may be regarded as concentrated. In this case, w' of course, will be equal to the quotient of the whole weight of the structure divided by the number of bearing points+1, the 1 being added because the two end supports sustain directly a weight equal to that which acts at each of the bearing points *o*, *n*, *m*, &c.

The effects of these weights, $w'w'$, &c. on the parts affected by them, added to the maximum effects of the variable weights *w.w*, &c., as above determined, will give the whole stress to which the various parts will be liable, and which they should be adequate to sustain, as regards truss 8 and truss A.

A. There is another effect of the weight of the structure, as it respects truss 7, which should be taken into account in practice, though it does not very essentially alter the result of this general comparison.

The counter diagonals *nb*, *mc*, *lf*, and *kg*, [Fig. 7,] can advantage in presenting the bearing points *o*, *n*, *m*, &c., in the same horizontal plane, whereas truss 8 requires elevated supports for the road-way towards the ends.

XX. If the line of transit be from *a to h*, the effects will be somewhat modified from what has been shewn above. In truss 7, transferring the weight from *o* to *b*, when *nb* does not act, will produce a tension upon *ob* equal to *w*. Therefore *ob* and *ig* will require to be so apportioned and connected as to sustain this force. At the points *c*, *d*, *e*, *f*, the weights would be sustained by the diagonals, producing exactly the same stress as they produce when at *n*, *m*, &c., through the medium of the verticals. Therefore the latter would be relieved, each, of a weight equal to *w*, without any additional stress on the other parts. Hence the transfer of the weights from the upper to the lower level, would have the effect to add $2w$ to the products for tension, and diminish by $4w$ those for thrust; an effect of no great importance, but rather favorable as far as regards the truss alone.

With respect to truss 8, it was seen (Art. 14) that the maximum thrust on each vertical, occurred when that vertical was loaded at the top, and since that thrust, in all cases was less than *w*, it follows that a transfer of the weight to the bottom of the vertical piece, would destroy all the thrust, and convert the action to one of tension.—Still there is a case where the vertical sustains a thrust which is not destroyed by the weight as in the case just stated. When *i* and *k* [Fig. 8] are unloaded and the other bearing points loaded, we see that *k.f* sustains a thrust represented by *ft′*, which becomes the maximum when the load is at the bottom of the truss, and for truss 8 it varies from ,35w to 50w. This amount being still farther reduced by the weight of the road-way, becomes very small, and the same material necessary to support the tension, will, in general, be adequate to support the thrust, if so connected as to be able to resist by both tension and thrust; for each vertical should possess strength sufficient to sustain the whole weight applied at its lower extremity, since the diagonals are supposed to be only capable of sustaining the effects of inequality in the load. Therefore the transfer of the road-way from the top to the bottom of the truss, would have the effect to diminish the representative products for material exposed to thrust by $3.7w$ (=the amount due to the verticals,) and increase those for tension by $w \times$ the aggregate length of verticals, which for the truss under consideration is equal to $4.72w$, a change so trifling as not essentially to vary the results of the former comparisons.

It is proper to remark, that, strictly speaking, the whole of the weight of the structure can not be regarded as applied either at *o, n, m,* &c, or *b, c, d,* &c. But the more considerable part of it being where the road-way is placed, I have considered it sufficient for my present purpose, to regard it all as concentrated in the main transverse beams or bearers, whether at *o, n, m,* &c. or at *b, c, d,* &c.

XXI. To pursue the comparison between truss 7 and 8, when the road-way is at the bottom, and the trusses are not sufficiently high to admit of being tied and secured across the top, the cancelled truss being higher (except in the middle) is somewhat more top-heavy, and may be more liable to yield laterally, though this cannot amount to a serious disadvantage.

The arched truss, moreover, may, by some, be thought to have a more graceful and agreeable appearance than the cancelled truss. I will not take upon myself to decide on this point, except by remarking, that, to a person who comprehends the principles and properties of different kinds of structures, in a case where strength is the grand desideratum, that plan of structure which secures this in the greatest degree, with the least amount of material and expense, will generally excite the most pleasing sensations in the mind.

XXII. Again, in cases where the truss is sufficiently high to admit of being secured and braced laterally at the top, the cancelled truss is better suited to such an arrangement than the arched truss, which, though high enough in the middle, will not admit of the lateral security being extended towards the ends without parts extending above the arch expressly for that purpose.

XXIII. On the whole, it would seem that the relative cost of trusses upon the plans shewn in Fig. 7 and 8, would be nearly proportioned to the amount of material in each respectively. Hence I conclude, that though for bridges of moderate length, particularly common road bridges, and others where the road-way passes near the lower part of the truss, the arched truss may be preferable, still, as a general rule, the cancelled truss, as shewn in Fig. 7, may be constructed to sustain a given weight, through all the changes to which the load of a bridge is usually liable, with from five to ten per cent less expense than the arched truss, and should be preferred for long spans, especially for rail-roads, and where the track passes over the top.

With regard to truss A., as compared with truss 7, the former manifestly has some 8 per cent the advantage in all cases where the diagonals act by thrust, but otherwise the advantage will be counteracted, mostly, by the greater length of some of the parts acting by thrust in truss A.

XXIV. But though I say, judging from the above examinations and comparisons, such results and advantages, in the use of the cancelled truss, may be obtained, I am nevertheless constrained to say, that, though the general plans and principles of this truss have been extensively used in bridge building, still, for reasons which I will not, in this place, undertake to explain, the principles have never, (to my knowledge or belief,) been applied with that skill and economy in the proportions of the different parts of the structure, which are necssary to secure the advantages above pointed out as practicable.

For instance, the bridges built on the plans generally known as Howe's & Long's plans, are essentially the same in their general features, as I have shewn in Fig 7. But instead of being built with each part proportioned to the maximum stress to which it is liable, the upper and lower ribs or stringers have been of the same dimensions throughout, or nearly so; whereas, the stress varies, as above shewn, some 50 per cent or one-half. The verticals and diagonals have been made nearly of the same size throughout, or increased towards the ends to a far less degree than the comparative stress to whtch they are subjected in those places, demands.

Now let us suppose the truss Fig. 7 so constructed that every piece or portion in each class, be of sufficient size to withstand the maximum stress for any of the parts of those classes respectively, and the upper rib extending to the same length as the lower, and the products taken for each portion into the maximum stress for the class to which it belongs. We should have

Products for Thrust,		**Products for Tension,**	
Top Rib,	$6 \times 7 = 42.w$	Bottom Rib,	$42.0w$
Two end Diagonals,	$12.w$	Six main Diagonals	$25.7w$
Verticals, nearly	$13.w$	Six counter do. ½ size,	$12.8w$
Total,	$67.w$	Total,	$80.5w$

Or, supposing the diagonals to act by thrust and the verticals by tension, (Howe's plan,) it will stand,

For Thrust,		**For Tension,**	
Top Rib	$42.w$	Bottom Rib,	$42.w$
Eight main Diagonals,	$48.w$	Six Verticals, nearly	$13.w$
Six Counter braces, half size,	$18.w$	Total,	$55.w$
Total,	$108.w$		

Corresponding amounts, if proportioned according to the stress upon each part, (diagonals acting by thrust,)

For Thrust,		**For Tension,**	
Tob Rib,	$22w$	Bottom Rib,	$24w$
Diagonals	$32w$	Verticals,	$18w$
Total,	$54w$	Total,	$42.w$

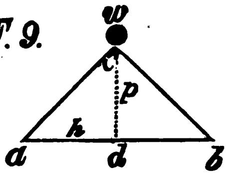
Only one-half as much material exposed to thrust, and a trifle more than three-fourths as much exposed to tension. This difference is still greater where the length of the truss has a greater ratio to the height.

The result here shewn is obtained on the supposition that the cross section of each piece may be reduced in the same ratio as the stress, whether it act by thrust or tension. With respect to the portion sustaining tension, this is essentially true. But a piece exposed to a crushing force in the direction of its length, requires that its length should not exceed a certain ratio to its diameter, in order that it may sustain a maximum. But in practice, the ratio of the length to the diameter is usually much greater than that which gives the greatest power of resistance. It follows, then, that in pieces of the same length, the power of resistance diminishes considerably faster than the cross section. Consequently the amount of saving indicated above, could not all be made available with regard to the parts acting by thrust, though a large portion of it might.

XXV. Having established the great excellence of the cancelled truss, on the plans shewn in Fig. 7 and Fig. A., for sustaining the weight of bridges & their loads, and pointed out the proportions which should subsist between its several parts when the general dimensions and proportions are given, it is proper to endeavor to ascertain the most economical proportion between the height and length of the truss, and the most advantageous angle, or inclination of the diagonals.

The office of the diagonal is, to serve as a medium through which the resistance of the abutment is made to sustain the weight of bodies not situated over the point of support; as where *ac* and *bc* are made the media through which the weight of the body *w* is transmitted to, and sustained by the two supports *a* and *b*, [Fig 9,] situated at certain equal distances on opposite sides of the vertical passing through w. Now the minimum amount of material required to sustain the weight with a given stress proportioned to the cross section, is when $a\,c\,b$ is a right angle. To demonstrate this, join ab, and draw the perpendicular cd. Then, the thrust on ac and bc each, is to $\frac{1}{2}\,w$, as the line ac (or bc) is to cd, that is, equal to $\frac{1}{2}w\frac{ac}{p}$, making $p=cd$. But making

F. 9.



$ad=h$, $ac=\sqrt{(h^2+p^2)}$ and $\frac{1}{2}w\,\frac{ac}{p}=\frac{1}{2}w\frac{\sqrt{(h^2+p^2)}}{p}$ and this multiplied by ac (or $\sqrt{(h^2+p^2,)}$) is proportional to the amount of material required to sustain the weight w, upon the supposition above laid down, being the stress multiplied by the length of the piece. Therefore, when p has such a value that the product $\frac{1}{2}w\frac{\sqrt{(h^2+p^2)}}{p}\times\sqrt{(h^2+p^2)}=\frac{1}{2}w\;(\frac{h^2}{p}+p,)$ is a minimum, the least possible amount of material will support the weight.

Then, differentiating the function $\frac{h^2}{p}+p$ in which p is the variable, and deducing the value of p when the differential$=0$ we obtain $p=h$, and consequently acb is a right angle, and ac and cb each incline 45° to the horizon.

XXVI. A. But a more important problem is, having the length and height of the truss given, to determine the horizontal reach of the diagonal which will give the greatest degree of economy.

This will present two cases, according as the diagonals act by tension or thrust. We will first consider them as acting by tension.

Now, the greatest economy in the use of a given amount of material in a diagonal acting by tension, is, manifestly, when the weight it can sustain, multiplied by the horizontal reach, gives the greatest product. This may be called the capacity of the material, and of course, will be directly as the cross section and the horizontal reach, and inversely as the rate of strain.

Then, if we make p=the height, and h=the horizontal reach, $\surd(h^2+p^2)$ will be equal to the length of the diagonal, and the amount of material being given, the cross section will be as $\frac{1}{\surd(h^2+p^2)}$. Moreover, the rate of strain will be, manifestly, as $\frac{\surd(h^2+p^2)}{p}$ or (p being constant,) as $\surd(h^2+p^2)$

Hence we have the capacity as $\frac{h}{\surd(h^2+p^2)} \div \surd(h^2+p^2)$ or as $\frac{h}{(h^2+p^2)}$, and that value of h which gives the maximum value of this expression, is the most advantageous. This is found to be when $h=p$, as in the case above.

Therefore this is the most economical position for the diagonals in all cases where they act by tension, as far as depends on those pieces alone.

XXVI. B. For the case of diagonals acting by thrust, the use of the diagonal or brace, being to carry or transfer the vertical action of certain weights towards the ends of the truss from the intermediate points, and the amount of weights sustained by each diagonal being, in general, proportioned to the distance of its upper end from the centre of the truss, it is obvious that the weight sustained by the brace is essentially the same, according to its distance from the centre, whatever be the number and inclination of the braces. The economy, then, in the use of material for braces, or diagonals acting by thrust, is directly as the horizontal reach of the brace (which determines the number,) and inversely as the amount of material in each brace, necessary to sustain the given weight.

Now the amount of material (m) is directly as the acting force or stress, and inversely as the power of resistance, $(r.)$ But the stress is as the length of the brace, (being equal to the weight sustained, multiplied by the length of brace and divided by the height (p) of the truss,) and the power of resistance is as the cube of the diameter $[d]$ divided by the square of the length, that is, as $\frac{d^3}{h^2+p^2}$*

If we take then, d' to denote the diameter of the brace when $h=0$, r becomes r', and is as $\frac{d'^3}{p^2}$, and since the power of resistance $[r]$ should be as the stress produced by the weight, we have the following proportion:

$$p : \sqrt{(h^2+p^2)} :: \frac{d'^3}{p^2} : \frac{d^3}{h^2+p^2}$$

Whence we have $\frac{pd^3}{h^2+p^2} = \frac{d'^3\sqrt{(h^2+p^2)}}{p^3}$, and $p^3d^3 = d'^3(h^2+p^2)^{\frac{3}{2}}$. Extracting the cube root of the last equation it becomes $pd = d'\sqrt{(h^2+p^2,)}$ whence $d' : d :: p : \sqrt{(h^2+p^2)}$. Therefore the diameter should be proportioned to the length of the brace, and the quantity of material as the cube of the length.

Hence, the greatest economy is when the horizontal reach, or the base of the triangle, divided by the cube of the hypothenuse, gives the greatest quotient, *i.e*, when $\frac{h}{(h^2+p^2)^{\frac{3}{2}}}$ is at the maximum.

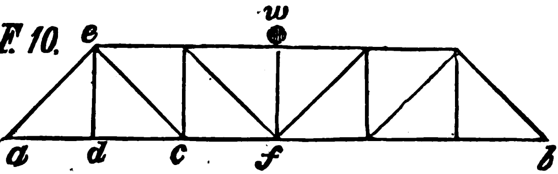
This is found to occur when $h = ,75p$ very nearly. It is also found by computation, that if h be made equal to p and $\frac{1}{2}p$ successively, the economy would be about 9 per cent less than when $h = \frac{3}{4}p$, being about the same in both cases.

Therefore, other things the same, it is the best economy to place the diagonals acting by thrust, so that the horizontal reach shall equal $\frac{3}{4}$ of the perpendicular; but considerable deviations may be made from the rule, when required by other considerations, without essential detriment to economy in the diagonals.

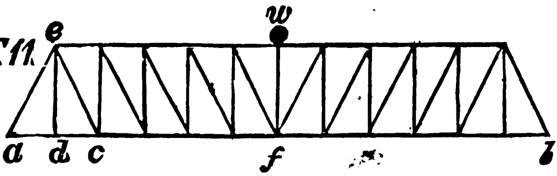
* This formula is not fully sustained by experiment, but is, perhaps, sufficient for the present investigation.

It is now proper to ascertain the comparative effects of different positions of the diagonal upon the horizontal parts, and verticals when employed. For that purpose, take two trusses, [Fig. 10 and 11,] of the same height and length, but in which the diagonals have different inclinations; and for simplicity, let truss 11 have twice as many

F. 10.



F. 11.



diagonals as 10 Now it is manifest that a given weight w, on the centre of these trusses, will produce a vertical pressure$=\frac{1}{2}w$ upon each of the supporting points a and b, and a horizontal action upon ad, equal to $\frac{1}{2}w\,\frac{ad}{ed}$, also that each diagonal from a to w, will exert the same horizontal action, the whole amount of which will be sustained by the different portions of the top and bottom ribs, the action of which will be increased towards the centre, where it will become equal to $\frac{1}{2}w\frac{ad}{ed}\times n$, n being the rum-ber of diagonals affected by w between a and w.

But the length of the truss being the same, n is inversely as the length of ad, or equal to $\frac{af}{ad}$, and substituting this value for n, we have $\frac{1}{2}w\,\frac{ad}{ed}\times\frac{af}{ad}=\frac{1}{2}w\frac{af}{ed}$ for the stress in the centre of the horizontal parts, produced by the weight w; an expression from which ad has been eliminated, and consequently the distance *ad* has no effect upon the stress of the horizontal parts in the centre, and by the same reasoning it is shewn that the same is true in relation to any other part of the horizontals, except in case of Fig. 11, the increments are added at shorter intervals, and in proportionally smaller quantities, from the end towards the centre. Hence, in general, there is no difference in the stress of the horizontal parts, whether the diagonals have one inclination or another.

With regard to the verticals, that part of their stress which they receive from the diagonals, is equal to the vertical action of those diagonals, and is the same for a given weight, whatever be their inclination. On the vertical *wf*, the pressure is received directly from the weight. But on the next adjacent vertical, on either side, one-half the same pressure is received through the intervening diagonal, and transmitted to the next, and so on to the end. Consequently, the aggregate action of the verticals produced by the weight w, is equal to $w+\frac{1}{2}wn$, taking n for the number of verticals receiving their strain through the medium of the diagonals, and which is equal to the whole number less 3 when the number is odd, and the verticals act by thrust, as in Fig. 10 and 11.

Hence, the aggregate stress of verticals increases and diminishes with their number; and economy as regards those parts, would require the diagonals to be inclined to a less angle with the horizon than that required by economy as regards diagonals.

We have seen, however, (Art. 26 B.) that by placing the diagonals at 45° when they act by thrust, we lose about 9 per cent in the economy of those parts, and we now see that such an arrangement increases the economy on verticals to a considerable extent by diminishing their number, the actual amount depending on the number, and therefore not deducible by a general rule.

It will, however, be a near approximation to the truth to assume, that by inclining the diagonals at 45°, the loss on diagonals will be very nearly compensated by the gain on verticals; hence this would seem to be about the most advantageous arrangement for diagonals acting by thrust, in combination with verticals acting by tension.

In case of diagonal ties and vertical struts, a trifling saving of material could be effected by increasing the horizontal reach of the diagonal beyond an equality with the perpendicular, but scarcely sufficient to compensate for the sacrifice in the simplicity of figure, unless other considerations conspire to favor such a departure from simplicity.

XXIX. Nextly, as to the proper height to be given to the truss, we have seen, [Art. 27,] that the horizontal thrust produced by the weight w in the centre of the truss, is equal to $\frac{1}{2}w\frac{af}{ed}$, that is, directly as the length and inversely as the height of the truss. Now this being also true with respect to all other positions of the weight, it follows that the horizontal parts have their powers of sustaining the action of the weights, increased directly as the height.

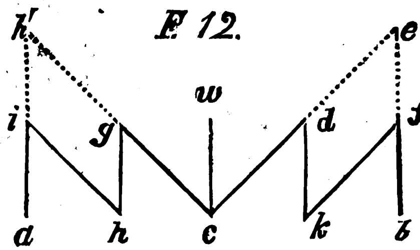
With regard to the other parts, those which act by tension, remain the same in amount, for though increased in length, they are diminished in number in the same ratio, other things the same, as far as it respects the diagonals, and the stress of the verticals received through the medium of the diagonals. For illustration the weight w, [Fig. 12,] will require the same length of diagonals, and the same stress upon them, to transmit its pressure from w to a and b, whether it be done through the parts w, c, g, h, i, a, or w, c, h', a, and so of the half transmitted to b. Also the same length and stress of verticals. But these, acting by thrust when the diagonals act by tension, have their power of resistance diminished with increase of length, and consequently will require an increase of cross section, such that the cube of the diameter may have a constant ratio to the square of the length.* If the length

* See Note foot of page 31.

be multiplied by a, the diameter must be multiplied by $a^{\frac{2}{3}}$, and the cross section by $a^{\frac{4}{3}}$, a factor greater than a.

Moreover, that part of the stress, produced by the weight directly upon the vertical, amounting to about $\frac{1}{2}$ the aggregate maximum stress, manifestly requires an increase in the cube of the diameter proportional to the increase in the square of the length, with an addition for increase of weight sustained proportioned to the diminution of number. Therefore if the height of truss be multiplied by a, the diameter must be multiplied by $a^{\frac{2}{3}}$ which requires the amount of material to be multiplied by $(a^{\frac{2}{3}})^2 \times a = a^{\frac{7}{3}}$, a factor greater than the square of a.

F 12.



Hence, since the verticals, when they act by thrust, require an increase of material greater than the simple increase of the height, for one-half, and greater than the increase in the square of the height for the other half, we may fairly assume an average increase for those parts, about as the increase in the square of the height of the truss.

It will be seen, moreover, that the diagonals at the ends always act by thrust, and that their aggregate thrust is equal to the whole weight of load and structure, divided by the natural sine of the angle those parts make with the horizon, and when they incline at 45° the above named sine$=1 \div \sqrt{2}$. Hence the thrust of the parts in question$=$ the whole weight multiplied by $\sqrt{2}$. Therefore, the height being multiplied by a, the amount of material in the endmost braces, will be multiplied by $a^{\frac{4}{3}}$, besides what is compensated by the increased horizontal reach. It will, therefore, probably, not be too much to assume, both for verticals and diagonals throughout, an increase in weight and cost, in the ratio of the increase in height, especially, taking into account the additional lateral support the high trusses will require.

Upon this assumption, since the horizontal parts diminish with increase of height, and the verticals and diagonals increase in expense in the same ratio to their respective amounts, the minimum expense of material would occur when the height were such that the horizontal parts be equal to all the others.

But in Art. 12, we found that for truss 7, in which the height is equal to $\frac{1}{7}$ the length, the representative quantities for horizontals were to those of the other parts, about as 56 to 40 or as 7 to 5. Therefore if we add $\frac{1}{6}$ to the height, it becomes $\frac{1}{7}+\frac{1}{42}=\frac{1}{6}$ of the length. Then diminishing the horizontals by $\frac{1}{6}$ we have $56-\frac{56}{6}=46\frac{2}{3}$, and increasing the other parts by $\frac{1}{6}$ we have $40+\frac{40}{6}=46\frac{2}{3}$.—Hence it would seem that the height should be $\frac{1}{6}$ of the length.

XXX. This, of course, is not given as a rigorously demonstrated point. A formula might be deduced, perhaps, by a more elaborate process, which, when comprehended, would be more satisfactory. But to introduce all the considerations by which the question is affected, would render any formulas upon the subject necessarily very complicated, and of comparatively little value in practice. There are considerations which go to render it quite improbable, that any simple ratio of the height to the length would be applicable with advantage in all circumstances.

In short spans, where great strength is required, it would often be found advantageous to carry the height to $\frac{1}{4}$ the length, a proportion almost impracticable, and certainly not advisable, in long spans, from the fact that they would become top-heavy, be more affected by wind, and other lateral influences and tendencies.

Again, in stretches of moderate length, where less strength is required, a less height, even as low as $\frac{1}{10}$ or $\frac{1}{12}$ the length, might be preferable, as having a better appearance, offering less obstruction to the prospect, &c. But for a structure of great strength, a height never less than $\frac{1}{8}$ and seldom less than $\frac{1}{6}$ of the length, can be adopted without sacrifice of economy.

Another consideration to be taken into account in fixing the height of the truss is, the distance between bearing points. In trusses of single cancels, or where the diagonals cross but once, the height must be equal to the distance between bearing points, or the diagonals must cross at oblique angles, which we have seen is unfavorable to economy in those parts, except that when they act by tension they may slightly approach the horizontal, and when by thrust, the perpendicular, without disadvantage. But if the bearing points be too far apart, it will require very heavy joists and longitudinal timbers in the road-way, besides that the top rib will require a greater cross section, being composed of longer pieces.

All these points and considerations will tax the skill and judgment of the engineer to adjust them properly in various circumstances, and I do not propose entirely to supersede his labor, nor will the limits I propose to myself in this essay, allow me to pursue this branch of the subject further.

XXXI. I have alluded to the plan of sustaining the horizontal thrust of bridges by the abutments, &c. It would seem that this might be done to a very great saving of material. But when a bridge is loaded with a full load for one-half of its length only, the abutment nearest to the loaded half sustains about $\frac{3}{4}$ of the load, and the other only $\frac{1}{4}$, and the thrust at the two ends is in the same proportion, as it regards trusses on the plans shewn in Fig. 7, 8, and A. Now the abutments can only sustain equal horizontal action from the structure, as they only act and re-act upon one another through the medium of the structure. Therefore the difference of thrust, amounting to nearly one-half the maximum, must be counteracted within the superstructure itself. Hence, but little more than half the thrust due to the maximum load *can* be sustained by the abutment.

Add to this consideration, that of the practical difficulty of making the abutment act in conjunction with the means provided within the superstructure, so that the latter may not at any time be subjected to a strain they are inadequate to bear, and the apparent advantage of relying on abutments to sustain thrust, is reduced to a small amount, if not entirely annihilated.

The principle, however, may be made available for bridges of light burthens, and for others, some assistance may be derived by bracing from the abutments. But in general, for the stronger class of bridges, as rail-road bridges, &c., it appears proper to adopt the principle, that the superstructure should be self-sustaining, only requiring a direct support for its own weight and that of its load, acting by vertical pressure, from the abutments and piers.

XXXII.. There is a plan, however, as we have already seen, in which every part of the weight, wherever situated, produces the same thrust at both ends. This is where each bearing point has an independent pair of braces extending from it, one to each abutment, as shewn in Article 11, Fig. 6. The objections to that plan are, that the amount of thrust is about one-third greater than on other plans, and the great length of a part of the braces; faults which very much impair, if not entirely destroy the value of the plan; at any rate, it is only applicable to short spans.

XXXIII. Having decided upon the most suitable forms and proportions for bridge trusses, I will say a few words in regard to the material best adapted to the purposes of bridge building. We have seen that the materials in a bridge truss, are principally subjected to two kinds of action, that of tension and that of thrust. The lateral action should always be avoided in the main parts of the truss.

It is obvious then, that those materials best calculated to resist these kinds of force respectively, should, when practicable, without the sacrifice of economy, be employed in the situations where those forces are respectively exerted.

For instance, when the diagonals act by tension, the top rib, (or the arch, in case of the arched truss,) and the verticals, should be composed of the material best adapted to sustaining a crushing force, while the lower rib or stringer, and the diagonals, should be of the best material for supporting tension.

Wood and iron, as before remarked, are the only materials that have been employed in bridge building, (I refer only to the superstructures,) to an extent worthy of notice, and it seems reasonable to conclude, that on these, we must place our dependence.

Cast iron will resist a greater crushing force than any other substance, whose cost will admit of its being used as a building material. Steel has a greater power of resistance, but its cost precludes its use as a material for building. Wrought iron resists nearly equally with cast iron, but its cost is twice as great, which gives the cast iron entirely the advantage. On the other hand, wrought iron resists a tensile force nearly four times as well as cast iron, and 12 or 15 times as well as wood, bulk for bulk.

Not only are these the strongest materials, but they are also the most durable. In fact, with proper precautions, they may be regarded as imperishable.

It would seem, then, that wrought iron for tension, and cast iron for thrust, were the *best* materials that could be employed for building bridges. But wood, though greatly inferior in strength and durability, is much cheaper and lighter, so that making up with quantity for its want of strength, and by frequent renewals for its want of durability, it has hitherto been almost universally used in this country for bridge building, and in the scarcity of means, and the unsettled state of things in a new country, where improvements are necessarily, to a great extent, of a temporary character, this is undoubtedly the most economical material for the purpose.

But it is believed that the state of things has now assumed that degree of settled permanency in many parts of this country, and available means have accumulated to that extent which renders it consistent with true economy to give a character of greater permanence to our improvements, and in the erection of important works, to have more reference to durability, even at the cost of a greater present outlay; and in this view of the subject, it seems highly probable that one of the channels in which this tendency of things will develope itself, will be in the extensive employment of iron in the construction of important bridges. With this impression, I proceed to some general comparisons as to the relative cost and economy of wood and iron as materials for bridges.

XXXIV. Cast iron resists a crushing force some 20 times as much as wood, consequently it will only require $\frac{1}{20}$ as much of the former to resist a given force, provided it can be put into a form in which its liability to flexmre and yielding laterally, is not greater than that of wood. This can be accomplished, in part, by giving the iron a hollow form, so as to make the diameter of the pieces approximate to an equality with twenty times the same amount of wood, which must generally be used in a simple rectangular or cylindrical form.

Assuming, then, that a cubic foot of cast iron, will do the same work as 15 cubic feet of wood, (after making allowance for the necessarily smaller diameter of the iron) we can institute a comparison which would seem, upon the surface, to shew the relative economy of the two materials.

A cubic foot of cast iron, manufactured for the work, will cost about $13,00. 15 cubic feet of wood in a bridge will cost, say $6,00. Whence it appears that the cast iron is more than twice as expensive, in the first outlay, for sustaining a crushing force, as wood.

Again, a cubic foot of wrought iron in the work, say 450 lbs. at $7\frac{1}{2}$ cts.,—$34.

Wood is about one-fifteenth as strong as iron. But about one-half of its fibres must be separated, in order that the other half may be so connected in the structure as to be available to their full strength by tension. Hence, it will take some 30 feet to equal one of iron; for which, say it will cost $12; shewing a difference of a little less than three to one; the average for both kinds of iron, reckoning equal quantities of each, being about 2.6 to 1.

To offset against this, we have the superior durability of the iron, which, as before observed, may be regarded as per-durable, whereas wood requires frequent renewals, at a cost each time equal to the first outlay. Now the first cost of iron is sufficient to provide for the first cost of the wood, and nearly two renewals. Besides this, money, though an inanimate substance, is nevertheless, in these usurious times, made to be exceedingly prolific; insomuch that with good husbandry, it is found to double itself once in ten or twelve years, according to the hard face of the lender, and the hard fortune of the borrower.

Assuming 5 per cent per annum as the net income of money invested, the term of time in which the $1\frac{60}{100}$ dollars saved in the wooden structure, will require to produce one dollar for renewal, will shew the time that wood ought to last, to be equal to iron in economy. $1\frac{60}{100}$ dollars at compound interest, will yield at 5 per cent, one dollar in a little less than ten years.

Therefore, if an imperishable iron structure cost 2.6 times as much as one of wood, and the latter last but ten years, and money will net 5 per cent compound interest, the two materials are nearly on a par as to economy.

Now experience has shewn that wooden bridges, unprotected by roofing and siding, seldom last with safety, over eight years, or thereabouts; and the more there be expended to increase the durability, the less surplus capital will be left to invest towards renewals. Hence the iron would seem to have the advantage.

XXXV. But the above comparison is too superficial and general to be entitled to a great deal of confidence, except, perhaps, as it regards the sustaining of a given weight by a simple post, or suspending it by a bar or rod of iron or wood. In the complicated assemblage of pieces forming the superstructure of a bridge, there are numerous other facts and considerations which materially vary the results. First, there is a difficulty in connecting pieces of timber in such a manner that every part may be proportioned to the strength required of it, to the same extent as can be done with iron. Second, it is frequently necessary to use considerable quantities of iron in bolts and fastenings, for putting together a structure of wood requiring great stability. Third, wood soon looses a portion of its strength by partial decay, and consequently requires additional strength in the beginning, that it may be safe for a time after decay has commenced.

Hence, but little can be predicated upon the simple general comparison of wood and iron as to strength and cost, relative to the comparative economy of the two materials for bridge building.

It is only by comparing the results of actual experience, or, where this has not been had, by comparing the results of detailed estimates, upon well matured plans, founded on well established principles, that a satisfactory conclusion can be arrived at.

With regard to wooden bridges, much experience has been had, and the reasonable presumption is, that a good degree of economy has been attained in their construction. But the idea of building iron bridges in this country is of recent date, and but little has been experimentally proved in relation to their cost and qualities.

XXXVI. This much, however, my own experience has demonstrated. Having received Letters Patent for an "Iron Trussed Bridge" upon the general plan of the arched truss shewn in Fig. 8, and constructed two bridges thereon, over the enlarged Erie canal, (72 and 80 feet span,) one of which has been in use for six years, it may be regarded as a demonstrated fact, that bridges may be sustained by iron trusses. Also, that the cost, for the above class of bridges, is only about 25 per cent more than the same class of bridges of wood, as *heretofore built*, under the most favorable circumstances, on the Erie Canal. That the iron portion, constituting some ¾ of the whole as regards expense, in the iron bridge, gives fair promise of enduring for ages, while the wooden structure can only be relied on to last 8 or 10 years.

Upon these facts experimentally established, I found the following comparison:

A common road bridge of 72 feet span, (the usual length for the enlarged Erie Canal,) will cost, with iron trusses,

7000 lbs. of cast iron—3 cts.,	$ 210
6000 wrought do, manufactured for the work, at 7 cts.,	420
Timber labor and painting,	230
Superintendence and profit,	80
Whole first cost,	$ 940
$175 will renew the perishable part once in 9 years, to produce which, at 5 per cent, will require a capital of	320
Total for a perpetual maintenance,	$1260

With wooden trusses fastened with iron,

Timber, labor, paint and profit,	$ 550
2000 lbs. of iron fastenings,	150
Whole first cost,	$ 700
(Some have cost $1000 or $1200, and taken 3 to 4000 lbs. of iron.)	
To renew $550 worth of perishable material, once in 9 years at 5 per cent, compound interest, will require	1000
Total for perpetual maintenance,	$1700

Shewing a clear ultimate saving of $440, in favor of the iron structure.

The reason of the apparent difference between this result and that arrived at (Art. 34) from the general comparison of the cost, &c., of wood and iron, is, that the bridges here referred to, have been constructed with a very large amount of iron fastenings, and with large quantities of casing and painting for protection and appearance.—Were the comparison confined strictly to the expense of timber work in the sustaining parts of the trusses, the result would be found not to differ so essentially from that of the general comparison.

The above estimate of $700 for the first cost of a 72 ft. wooden bridge, though considerably below the average cost of canal bridges of that decription, is nevertheless believed to be greatly above the minimum for which bridges may be built, dispensing with parts which are not essential to strength. It is probable that bridges may be built for $500, as about the minimum, of equal strength and convenience, and nearly the same durability as those hitherto built upon the Erie Canal Enlargement at a cost from $800 to $1000. Upon this supposition, which may be regarded as an extreme case in favor of wood, the comparison will stand thus:

First cost of wooden structure,...........	$ 500
Capital invested at 5 per cent to produce $500 once in 9 years for renewal,....	909
Total for perpetual maintenance,.........	$1409
The same for iron structure, as above,....	1260
Balance in favor of the iron bridge,......	$ 149

Finally since theoretical calculation and general comparison shew a *probable* advantage, for a long term of time, and experience, as far as it has gone, shows a *decided* advantage in favor of iron, it would seem very unwise to discard the latter, without, at least, a fair trial of its merits. If in the first essays at iron bridge building, the iron bridge has competed so successfully with wooden bridges, improved by the experience of ages, may not the most satisfactory results be anticipated from an equal degree of experience in the construction and use of iron bridges ?

XXXVII. Presuming the affirmative to be the only rational answer to the above question, I have arranged the details of plans for carrying into practice the preceding principles and suggestions in the construction of railroad bridges of iron.

I have also made careful detailed estimates of the expense of bridges of different dimensions and in different circumstances, some of the more general results of which I will here state.

In proportioning the parts of a rail-road bridge, I have assumed that it may be exposed to a load of 2000 lbs. per foot run, for the whole or any part of its length, in addition to its own weight, and in case of tension, have allowed one square inch cross section of wrought iron for every 10,000 lbs. of the maximum strain produced upon every part by such weights, acting by dead pressure. In case of thrust, or crushing force, I have allowed one square inch cross section of cast iron for every 12,000 lbs., acting on pieces, (mostly in the form of hollow cylinders,) of a length equal to 18 diameters, and a greater amount of material, where the ratio of the length to the diameter is greater ; always having regard to practicability, as well as theoretical proportions, in adjusting the dimensions of the parts.

My estimates made upon these bases have fully satisfied me, that a bridge of 100 feet span, with the track sustained upon the top, will cost about $2000, or $20 per foot, assuming the present pieces of iron, (1846) in ordinary circumstances. If the track pass near the bottom of the trusses, the expense will be increased two or three dollars a foot.

For a span of 140 feet, by a liberal detailed estimate, I make, in round numbers, a cost of $4000. For 70 feet, I make by actual estimate, and liberal allowances for contingencies, a cost from 900 to $1000, according to circumstances.

Thus it will be seen, that actual estimate makes the cost of a single stretch of any length, very nearly as the square of the length, as should be expected from the nature of the case.

Hence, knowing the cost of a span of any given length, we readily deduce that of a span of any other length, in similar circumstances, with reliable certainty.

These estimates of cost provide for superstructures entirely of iron, except the string timbers to support the iron rail. By the substitution of wooden cross bearers, a considerable saving may sometimes be effected without incurring serious inconveniences.

But though my investigations have forced upon me the conviction, that in general, where strong and durable bridges are required, iron should be preferred in their construction, still, there is a multitude of cases where wooden structures should be preferred, especially in sections of country comparatively new, where timber is plenty and capital scarce; and where improvements must necessarily be of a more temporary character. With this view of the subject, I have given much attention to the details of wooden bridges, and with a good deal of investigation and experiment, have arranged plans which are confidently believed to possess important advantages over the plans generally in use.

I may therefore be excused the expression of a belief, that I might be able to render valuable services to those interested in the construction of important bridges, generally, and with this conviction I make the proffer of my services, wherever they may be desired, for the purpose of furnishing plans, superintending the construction of bridges, either of wood or iron, and of consultation generally, in matters pertaining to this, as well as other subjects in Mechanics and Civil Engineering.

I would add that I design shortly to publish, as a sequel to what precedes, should circumstances seem to warrant it, a set of working plans for bridges of different lengths and descriptions, both of wood and iron, with details, specifications, and general remarks. S. WHIPPLE.

ESSAY No. II

ON

BRIDGE BUILDING:

GIVING

PRACTICAL DETAILS

AND

PLANS

FOR

IRON AND WOODEN BRIDGES.

BY S. WHIPPLE, C. E.

UTICA, N. Y.
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1847.

ESSAY NO. II.

In the preceding Essay, to which the following is intended as a supplement or continuation, I have endeavored to give a short and comprehensive general view of the subject, and to ascertain and point out the best general plans and proportions for the main longitudinal trusses or side frames of Bridges, and their several parts. The side trusses may be regarded as vastly the most important parts of the structure; since the strength and sufficiency of these being secured, there is little difficulty in arranging the remaining parts. I propose now to go more into the details of the matter, and give such practical explanations and specifications as to the strength of materials, the methods of joining or connecting the several parts or pieces, both in the main trusses and the other parts of the structure, illustrated by the necessary diagrams and plans, as, it is hoped, will enable the young engineer or the practical builder to proceed with judgment and confidence in this important branch of the profession.

I will first take up the subject of

Iron Bridges.

XXXVIII. Iron has the power of resisting mechanical forces in several different ways. It may resist forces that tend to stretch it asunder, or forces that tend to compress and crush it; the former producing what is usually called a *positive*, and the latter, a *negative* strain. Or it may be exposed to and resist forces tending to produce rupture by extending one side of the piece, and compressing the opposite side; as where a bar of iron supported at its extremities in a horizontal position, is made to sustain a weight in the middle, which tends to stretch the lower part and compress the upper. This is called a *lateral* or *transverse* strain.

Iron may also be acted on by forces which tend to force it asunder laterally, in the manner of the action of a pair of shears. This kind of strain is less important than either of the preceding, and especially in bridge building, it seldom, if ever, will more than partially have place.

XXXIX. With regard to the simple positive and negative strength of iron, it is only necessary for me to state in this place, that, as the result of a multitude of experiments, a bar of good wrought iron an inch square will sustain a positive strain of about 60,000 lbs. on the average; and a negative strain, in pieces of a length not exceeding about twice the least diameter, of about 90,000, and in about the same ratio for greater or less cross sections.*

Cast iron resists a positive strain equal to from 15,000 to 30,000 lbs. to the square inch, but usually not over 18,000. In fact, it is seldom relied on to sustain this kind of strain, and its power of resistance in this way is not so well determined as in the case of wrought iron. But cast iron resists a negative strain even better than wrought iron, its power of resistance being from 80,000 to 140,000 lbs.; seldom less than 100,000 to the square inch, for pieces of a length not exceeding twice the least diameter. But in pieces of such dimensions as must usually be employed in bridges, fracture would take place by lateral deflection, with a much smaller force than would crush the material. It is therefore necessary to take into account the length and diameter, as well as the area of the cross section, in order to determine the amount of negative strain which a piece of cast iron, or any other material, may be relied on to sustain.

* Wrought iron yields considerably under a much less pressure than 90,000 lbs. to the square inch; but it becomes hardened by the compression, and has its power of resistance increased.

XL. The cause of lateral deflection by forces applied at the ends and tending to crush a long piece, is supposed to be a want of uniformity in the material, and a want of such an adjustment of the forces, that the line joining the centre of pressure at the two ends, may pass through the centre of resistance. These elements are liable to considerable variation, and can not be very closely estimated in any case. Therefore, the absolute power of resistance for a piece of considerable length, can not be deduced by calculation from the simple positive and negative strength of the material, but resort must be had to direct experiment upon the subject; and even considerable discrepancies should naturally be expected in the results of experiment, unless the lengths of pieces experimented on be very considerable.

In respect to pieces, however, having their lengths equal to twenty or more times their diameters, a considerable degree of uniformity is found in their powers of negative resistance, and the following formula, deduced theoretically, though not fully sustained by experiment, will sometimes be useful in determining the relative powers for pieces of similar cross sections, but different dimensions. The power of resistance is as the cube of the diameter directly, and as the square of the length inversely, i. e., R is as $\frac{d^3}{l^2}$.

The manner of obtaining this formula may be readily illustrated by Fig. B., Pl. 1, in which *adb* represents a post loaded at *a*, so as to bend it into a curve, of the half of which, *cd* is the versed sine. It is obvious that in this condition, the convex side of the post is exposed to tension, (or, at least, to less compression than the other,) and the concave side to compression, and that the effect of the load at *a*, towards breaking the post at *d*, is as the versed sine *cd*, which is as the square of *ab*. But the power of the post to resist rupture transversely, is manifestly as the cross section of the post, (i. e. as the square of the diameter,) multiplied by the diameter. Hence the power is as the cube of the diameter. Now, the ability of the post to sustain the load at a, is directly as the power to resist rupture just determined, and inversely as the mechanical advantage with which the load acts, above seen to be as the square of the length of the post. Hence the formula.

XLI. The following scanty table of experiments will give some farther light upon this subject, and may be worth something in the absence of better data.

TABLE OF EXPERIMENTS

On the negative strength of Cast Iron.

No. of Ex'pts.	*Form of piece.*	INCHES. *Diam.*	*Length*	*Wt. in lbs.*	*Weight borne.*	*Breaking wt.*	*REMARKS.*
1	Cylinder.	$\frac{5}{16}$	9.	.16	990	1002	Broke $\frac{1}{10}$ in. from centre.
2	"	"	"	"	978	990	Broke $\frac{1}{2}$ in. from centre.
3	Square.	$\frac{1}{4}$	"	.15	803	854	Deflected cornerwise, & flew out without breaking.
4					914	938	Broke in half a minute, not cornerwise, $\frac{1}{3}$ inch from centre.
5	Cylinder.	$\frac{5}{16}$	7.1	.126	1417	1437	Broke in 3 seconds $\frac{1}{40}$ inch from centre.
6	"	"	"	"	1377	1397	Broke $\frac{15}{16}$ from centre. This piece was flattened by flask not shutting true, and had been straightened with the hammer where it broke
7	"	"	4.5		2580	2580	Broke in 1 minute into 4 pieces of nearly equal lengths. Piece of same as last experiment.
8	"	"	4.5		3218	3218	Broke in $\frac{1}{2}$ minute into 3 pieces, in centre and 1 in. from centre.
9	Square.	$\frac{1}{4}$	4.5		2813	2838	Broke in $\frac{1}{4}$ minute, $\frac{1}{16}$ in. from centre; deflection parallel with sides.

XLII. From experiments 7 and 8 of the above table, it appears that cast iron will sustain at the extreme, in cylindrical pieces, whose lengths equal about $14\frac{1}{2}$ diameters, a negative strain of from 41,000 to 51,000 lbs. to the square inch, say an average of 46,000 or more. Square bars, according to Expt. 9, length equal to 18 times the width of the side, will sustain about 45,000 to the square inch.

Now a hollow cylinder, of a thickness of metal not exceeding about $\frac{1}{15}$ of the diameter of the cylinder, according to calculation, has a transverse strength about 50 per cent greater to the square inch, than a square bar whose side equals the diameter of the cylinder. Hence, a hollow cylinder whose length equals 18 times its diameter, ought to sustain a negative strain of 67,500 lbs. to the square inch.

It should be observed, however, that direct experiments upon the lateral strength of the pieces used for arriving at the results and conclusions above stated as to negative strength, shewed them to possess uncommon strength transversely, even to from 30 to 50 per cent greater than the fair average transverse strength of cast iron, as will be seen hereafter.

Therefore, I do not consider it proper to estimate the negative strength of hollow cylinders of the proportions above stated, at more than from 40,000 to 50,000 lbs. (say 46,000) to the square inch.

Now, the hollow cylinder being evidently the form best adapted to sustain a negative strain, or a transverse strain in all directions, it would be well worth the while to have a set of thorough experiments, to determine accurately their actual strength. This has never, as far as I have learnt, been done, and therefore I shall assume the above estimate on the subject, as probably, not very far from the truth, subject, however, to correction whenever the facts and evidences shall be obtained, upon which the correction can be founded. In the mean time, since we know not the exact ratio between the greatest safe practical strength and the absolute strength of iron, and therefore should, in practice, keep considerably within the limits of *probable* safety, it becomes a matter of less importance to know the exact absolute strength, though this, of course, is desirable.

XLIII. It will be seen moreover, by the table of experiments, that two cylindrical pieces of 9 inches in length bore, the one 990 lbs. and the other 978 lbs., giving a mean of 984 lbs.

Now by the formula, R as $\frac{d^3}{l^2}$, these same cylinders reduced to 4.5 inches in length, should sustain four times as much, or 3,936 lbs. But by experiments 7 and 8, we see that they bore only 2,580 and 3,218, giving a mean of 2,899. Whence it appears that, the diameter being the same, the strength diminishes faster than the length increases, but not so fast as the *square* of the length increases, being about half way between the two. In fact, if we examine the results of these experiments throughout, we find that the weights borne by pieces of like cross sections, whether round or square, and of different lengths, were very nearly the arithmetical mean between the results obtained by considering them to be inversely as the simple length, and as the square of the length, successively.

For illustration, take Expts. 1 and 5. If the piece 9 inches long bore 990 lbs. taking the strength to be inversely as the length, we have this proportion $\frac{1}{9} : \frac{1}{7.1} :: 990 : 1{,}255$. Then, taking the strength inversely as the *square* of the length, we have $\frac{1}{81} : \frac{1}{50.41} :: 990 : 1{,}591$. Taking the mean of these two results, we find $(1{,}255 + 1{,}591) \div 2 = 1{,}423$. This is the weight which, according to the rule, the piece in Expt. 5 should have borne, and it varies only 6 lbs. (less than $\frac{1}{2}$ of one per cent) from what it actually did bear.

Again, take Expts. 1 and 8, in which the lengths were as 2 to 1. Supposing the weights to be inversely as the lengths, and as the square of the lengths successively, and taking the mean of the results, we have $(1,980+3,960) \div 2 = 2,970$, which is 248 lbs. less than the weight borne in Expt. 8. But it is also 390 lbs. *greater* than that borne in Expt. 7, by a piece of similar dimensions, but an inferior specimen. It does not seem, therefore, that the rule is much at fault.

From these facts and others of a similar nature which have come under my observation, I give the following as a good practical rule for determining the negative power of resistance for pieces of similar cross sections, after knowing from experiment the power of a piece of given dimensions, and similar cross section. *Make the power of resistance as* $\frac{D^3}{L^2}$ *and as* $\frac{D^3}{L}$ *successively, and take the mean of the results thus obtained, as the true result;* D representing the diameter (or side of the square if the cross section be square,) and L, the length of the piece.

XLIV. This rule will apply to pieces whose lengths are from 15 to 40 times as great as their diameters, and perhaps for greater lengths, although in bridge building, greater lengths will seldom be used.* But as the length is reduced to 8 or 10 times the diameter, or less, it is manifest that the power of resistance increases in a less ratio than that given in the rule, and even less than inversely as the simple length. For we see by the table of experiments that a square piece, length equal to 18 diameters, (Expt. 9,) bore at the rate of 45,000 lbs. to the square inch, which is about one third of the actual crushing weight for the strongest iron. But according to the rule, a piece of half the length, or equal to nine diameters, should sustain 135,000, which is about the maximum for cast iron, whereas experiment shews that the power of resistance increases till the length is reduced to about 2

* It is probable that for greater lengths than 40 diameters, the formula given article 30, would be more nearly sustained than where the length is less.

diameters. I would therefore apply the rule to square pieces above 15, and to solid cylindrical, above 12 diameters. From 15 to 10 diameters for square pieces, and from 12 to 8 for cylinders. I would call the strength as $\frac{D^3}{L}$, and from those lengths down to 2 diameters, I would add pro rata, according to the differences between the weights, or resistances determined as above for square and pieces of 10, cylinders of 8 diameters in length, and the absolute crushing weight of the iron; i. e., if a square piece whose length equals 10 diameters bear m pounds, and the crushing weight for pieces of 2 diameters or less be n pounds, to obtain the resistance for a piece of 9 diameters, I would call it equal to $m+(n-m)\div 8$, and for 8 diameters, I would take $m+2\ (n-m)\ \div 8$, and so on.

XLV. I havo already observed that in practice, materials should be exposed to much less strain than their absolute strength is capable of sustaining for a short time.

This is a fact well known to every body who has had experience in building, or has reflected upon the subject, and the reasons for it are, perhaps, sufficiently obvious, still, I will mention some of them.

Firstly, there is a great want of uniformity in the quality and strength of materials of the same kind, and no degree of precaution can always guard against the employment of those containing defective portions, possessing less than the average strength.

Again, when materials are exposed to a strain, although it be but a small part of what they can bear, a change is produced in the arrangement of their particles, whereby they frequently, if not generally, become weakened, especially if they be frequently exposed to such process. Hence, it often happens that a piece will break with a less strain than it has previously borne without apparent injury.

Now, there are no means of estimating exactly the allowance necessary to be made for either of these circumstances. Consequently, we can not determine with certainty how much of any given material may be relied on with safety, to sustain a given force. We must therefore lean to the safe side, with the greater *penchant*, in proportion as the consequences to be apprehended from a failure are the more disastrous. The failure of a bridge is liable, in almost all cases, to be a serious affair, even to the imminent hazzard of life and limb. They should therefore, be constructed of such strength as to render failure quite out of the range of probability, if not absolutely impossible.

It is generally considered that wrought iron of a good quality may be relied on to sustain a stress equal to one fourth part of its absolute capacity, with very nearly perfect safety, with reasonable care in selecting, and guarding against defective parts. I have therefore adopted the rule in practice, of estimating good wrought iron to be capable of sustaining a positive strain of 15,000 lbs. to the square inch of cross section.

XLVI. With regard to cast iron, it is not economical to employ it to sustain tension, and whenever it may be exposed to that action, I would not rely on it for more than about 4,000 lbs. to the square inch. I am confidently of opinion, however, that where cast iron is exposed to a crushing force, in pieces of such length as to be deflected and broken laterally, it may be loaded with safety to one third of its absolute average capacity. If a piece exposed to a negative strain have a defective part it does not diminish its power of resistance to the same extent as when it acts by tension. The power of negative resistance being inversely as the *fleche* or deflection produced by a given weight, and the fleche depending on the stiffness of the piece throughout its whole length, the power is manifestly only diminished as the amount of defect multiplied by the ratio of the length of the defective part, to the whole length; that is, if the piece be defective to the loss of one fourth of its stiffness, for that part of its length to which is due one tenth part of the deflection, the deflection will only be increased by $\frac{1}{4}\times\frac{1}{10}=\frac{1}{40}$, and the power of resistance is diminished in the same ratio, whereas the power of positive resistance would be diminished by $\frac{1}{4}$.

The effect of a negative strain, moreover, is believed not to be so deleterious to the strength of iron, as that of a positive strain, though I can refer to no particular facts in corroboration of the opinion.

On the whole, I am induced to estimate the power of cast iron hollow cylinders of a length equal to 18 diameters, to resist a negative strain, at 15,000 lbs. to the square inch of cross section. Solid cylinders, same length, 8,000, and solid square pieces 10,000.

There is another useful form of cross section for sustaining negative strain, which is that of a cross, thus +. The power of resistance for this shaped piece, is essentially the same per square inch, as that of the solid square which will just contain the figure, and may be estimated at the same rate, calling the side of said square the diameter.

The T formed section will also be sometimes convenient, and possesses about the same power as the X formed section, and may be estimated as a square of which the top horizontal, or the perpendicular portion is the diagonal.

XLVII. The following table, exhibiting the power of negative resistance to the square inch of cross section, for hollow and solid cast iron cylinders, and solid square pieces, under which class may be included the X and T formed sections, of different lengths, from 2 to 40 diameters, the resistance being estimated at what is considered safe in practice, will be found convenient in estimating and proportioning the parts in bridges and other structures designed to sustain a negative strain. For Rail Road bridges, $\frac{1}{4}$ should be deducted from the quantities in the table.

Length in diameters.	Power of resistance to the square in. in lbs.		
	Hollow Cyl.	Solid Cylinder.	Square, X, & T section.
2	33333	33333	33333
4	31438	29722	29950
6	29545	26111	26566
8	27652	22500	23183
10	25759	18000	19800
12	23866	15000	16500
14	20457	11755	14143
16	17900	9562	11953
18	15000	8000	10000
20	12825	6800	8550
22	11156	5955	7438
24	9844	5250	6562
26	8787	4686	5858
28	7921	4224	5280
30	7200	3840	4800
32	6592	3515	4395
34	6072	3238	4048
36	5625	3000	3750
38	5235	2792	3490
40	4833	2610	3262

Lateral or transverse strength.

The transverse strength of bars or beams may be deduced from the positive and negative strength in the following manner:

Let *ab*. Fig. C, Pl. 1, represent a portion of a rectangular bar projecting from a wall in which it is firmly fixed. If a weight be applied at *w*, the upper portion will be stretched and the lower compressed; and where these portions meet is what is called the *neutral plane*. Experiment shews that this plane, in rectangular beams, is central between the upper and lower surfaces, or at least, very nearly so, for all elastic substances, until they approach rupture. The tendency of the weight at *w*, then is to produce rotation about the point *c*, or the line of intersection of the neutral plane with the cross section at *c*, and the cohesion of the upper portion *cd*, and the repulsion of the lower part cb, tend to resist rotation. Now, to determine the amount of this resistance, we will first consider the upper portion; and it is obvious that at every part of the cross section, the resistance to rotation is as the resistance to extension, multiplied by the distance of the part above the neutral plane. But the resistance to extension is, by the law of elasticity, as the degree or amount of extension, which is determined by the distance from the neutral plane. The parts at two inches from this plane, or the centre of motion, are extended twice as much as those at one inch, and resist twice as much. Then, denoting this distance from c by the variable quantity x, the resistance to extension for each part may be denoted by sx, and the resistance to rotation about c, by sx^2.

Again, representing the horizontal breadth of the beam by t, $t.dx$ will represent the differential of the section, and $s.t.x^2 dx$ the differential of resistance. Then integrating and making $x=cd=h$, we have the whole resistance to rotation, of the part above the neutral plane $= \frac{1}{3}\, s.t.h^3$. But $s.h$ becomes equal to the positive strength of the material when $x=cd=h$, and $t.h$ = area of the section above the neutral plane. Therefore, the power of this part to resist rotation is equal to $\frac{1}{3}$ the area multiplied by the half depth of the beam, and the absolute positive strength of the material, in case the negative strength exceed the positive.

Now, it is manifest that the part below the neutral plane exerts exactly the same amount of resistance to rotation as the part above. Therefore the total resistance to rotation about c, in other words, the resistance to rupture, is equal to $\frac{1}{3}$ the whole cross section multiplied by $\frac{1}{2}$ the depth of the beam, and by the cohesive strength of the metal; that is, equal to $\frac{1}{3}\ CtD \times \frac{1}{2}D = \frac{1}{6}CtD^2$, making $D=db$, and C=cohesion, or positive strength of the material.

If we wish to determine, then, the greatest weight which the beam is capable of bearing applied at the point *w*, we institute the following equation: $W.L.=\frac{1}{6}\ C.tD^2$, in which W=the weight, and L=the length *wd*. Or, to make the expression more general, L=the distance from the centre of motion *c*, to the line in which the force (W) acts.

From the above equation we have $W=\dfrac{C.t.D^2}{6L}$.

XLIX. This formula will enable us to determine the transverse forces which a rectangular beam will sustain, when we know the material and dimensions of the beam, and the lines in which the forces act, are parallel with the sides of the beam.

In case the positive strength of the material exceed the negative, the same formula, $(W=\dfrac{C.t.D^2}{6L})$, holds true, if we consider C to represent the negative strength of the material instead of the positive.

This formula is deduced on the supposition that the material is perfectly elastic, so as to suffer no permanent change of shape until the strain produces actual rupture. There are few substances if any, and certainly wood and iron are not of the number, that fulfil this condition so nearly but that considerable discrepancies are found between the deductions of theory and the results of experiment. Indeed, in the case of cast iron, experiment shews the transverse strength to be fully twice as great as it is made to appear by the above formula.

If in the expression $\dfrac{C.t.D^2}{6L}$, we make $L=D$, it may be reduced to $\frac{1}{6}\ CtD$, which shews that the power of the projecting end of a beam to sustain weight at a distance from the fulcrum equal to the depth of the beam, is only $\frac{1}{6}$ as great as the positive, (or negative, in case that be the smaller,) strength of the beam.

This is a convenient way of expressing transverse strength, viz: as equal to a force of so many pounds to the square inch of cross section, the force being understood as acting on a leverage equal to the breadth of the beam in the direction of the force. If we call 18,000 lbs. to the square inch the positive strength of cast iron, we may call the transverse strength, (according to the above deduction,) $\frac{1}{6} \times 18{,}000$ lbs.$=3{,}000$ lbs., meaning that a bar 1 inch square will sustain upon its projecting end, 3,000 lbs. at 1 inch from the fulcrum, and proportionally less as the distance is greater.

Now, experiment shews that it will sustain twice this amount, and frequently more, so that in reality we may reckon the transverse strength of cast iron at about 6,000 lbs. to the square inch.

I know of nothing to which to attribute this great discrepancy between theory and experiment, except a want of complete elasticity in the material.

Cast iron, when exposed to a transverse strain, suffers extension on one side and compression on the other; and the power of resistance to both these effects, increases very nearly as the amount of extension or compression till they approach a certain point or maximum, and after passing this point the power diminishes. Now it is reasonable to suppose, in fact, we can hardly suppose the contrary, that for a certain interval on each side of the maximum point, the power of resistance remains nearly stationary. But this stationary interval is reached on the positive, much sooner than on the negative side, and the inevitable consequence must be, that the neutral plane is transferred farther from the positive side, so as to preserve the equilibrium between the resistance to extension and the resistance to compression. Hence, the amount of resistance on the positive side is increased, both by the increased area of the cross section of extension, and increased leverage, or distance from the neutral plane.

Moreover, a greater portion of the fibres (so to speak) of extension, act with their full power; since, while the outside portion is passing through what we have called the stationary interval, successive portions towards the neutral plane, are reaching and approaching that interval. Hence, some considerable portion of all the fibers of extension, may act with their maximum power, whereas, if the body were perfectly elastic up to the point of actual rupture, only the outside fibers, farthest from the neutral plane, could act with absolute power, and all other parts, only in the ratio to their respective distances from said plane. To illustrate, suppose the extreme positive side, when extended one inch, reach the stationary interval, which is one inch more. It follows that, when the outside has passed to the other limit of said interval, one half of the positive portion of the bar, will be within the range of that interval, and act with its maximum power, producing one third more resistance to extension than the same fibres could afford if the body were perfectly elastic to the point of rupture.

I know of no more plausible manner of explaining the observed discrepancy between experiment and calculation upon this subject.

L. Wrought iron has something more than three times the positive strength of cast iron, upon the average, and if we consider its transverse strength to be in the same ratio to that of cast iron, the following proportion will give us the transverse strength of wrought iron: 18,000 : 6,000 : : 60,000 : 20,000.

Hence, the projecting end of a bar of wrought iron an inch square in a horizontal position, should sustain, at one inch from the fulcrum, a weight of 20,000 lbs. It begins to bend considerably, however, with about one third of this strain, and therefore, in practice, it should never, probably, be exposed to a lateral strain greater than five thousand pounds to the square inch.

The principal occasions where wrought iron will be exposed to a transverse strain in bridge builuing, will be in cylindrical pins for connecting other parts of the work, in which cases the forces will act with a certain leverage which can be nearly determined, and denoting by a the area of the cross section of the pin in square inches, and by d its diameter in inches, and by b the length between the centres of bearing at the ends, we shall have $4 \times 5{,}000\ ad \div l=$ the number of pounds the pin will sustain in the middle with safety. A pin 1 inch in diameter has a cross section of .785 inches$=a$, and if $l=6$ inches, we have, by substituting these values in the last formula, $4 \times 5{,}000 \times 1 \times .785 \div 6 = 2{,}620$ lbs., which an inch pin will bear in the middle when supported at 3 inches from the middle each way.

It will be seen that this formula regards a cylindrical bar of a given cross section to be capable of bearing the same force, acting on a leverage equal to its diameter, that a square bar of the same section can bear on a leverage equal to the width of its side; a supposition which my experiments on cast iron sustain, though differing from the result of calculation.

BRIDGE WITH THE ARCHED TRUSS.

LI. For the general plan of this truss, reference may be had to figures 5 and 8, pages 10 and 19.

The arch is composed of a number of cast iron pieces, exceeding by one, the number of principal bearing points, *b.c.d.* &c., fig. 8, page 19. In this plan I suppose the road way to be along the chord of the arch, or slightly cambred according to circumstances, or the taste of the engineer.

The cross section of the arch pieces may be of various forms, but there are only two which I would recommend; the one is the form adopted for my canal bridges, and the other is that of hollow cylinders, which is more economical, in certain cases, as for railroad bridges, requiring great strength. I will first describe the former.

aonm, Fig. 13, Pl. 1. presents a top view of the arch from the end to the centre, x and y represent enlarged cross sections at p and q. Each piece consists of two side portions, of ˥ formed section, connected by 3 or 4 cross bars of a T formed cross section ; those at the ends being so broad and thick as to possess sufficient transverse strength to sustain any weight that may be brought upon the bearing points, and having a semi-circular notch in each, so as to form a round hole where they meet, for the vertical bolt to pass through.

For a bridge of 72 feet span, in which the arch pieces are from 11 to 12 feet long, the depth of the section should be at least six inches, the top cross of the ˥ on each side about 3 inches wide, and the thickness of metal, average about $\frac{5}{8}$ of an inch, so as to give about 12 square inches of cross section. The endmost arch pieces should also have diagonal bars from 1 to $1\frac{1}{4}$ inch square. The mid rib of the intermediate cross bars of the arch pieces, should have a depth from $\frac{1}{2}$ to $\frac{2}{3}$ that of the side portions. The end cross bars should be from 4 to 6 inches wide, and from $1\frac{1}{2}$ to 2 inches thick in the middle, according to their length; or greater depth and less width would be better economy.

The cross bar *c*. should be about a foot from the end, and the side portions formed into a sort of foot, (see *a*, Fig 14, Pl. 2,) with a shoulder, *a*. for the chord chain, hereafter to be described, to act against.

The ends of the pieces are beveled according to the radius of the arch, so as to form a fair joint, and one or both of contiguous ends should have projections lapping by the joint, to assist in keeping the ends in place.

The width *ab*, should be a little more than $\frac{1}{4}$ the height of the arch, and the width in the centre, should be not less than $\frac{1}{15}$ the length of the piece.

LII. The chord *ah*, (fig. 8, p. 19,) is a chain composed of two sets of long links, extending from *a* to *b*, from *b* to *c*,

&c., and connected by pins or bars, of a length equal to *ab* Fig. 13, Pl. 1. The cross section of these chains should contain adout 6 square inches to each truss, for a 72 feet common road bridge of 16 feet width of way.

In fig. 14, Pl. 2, *A* represents a top, and *B* a side view of the chain, and its connection with the foot of the arch piece above described. The end links are twisted 90°, so as to have one end open horizontally to receive the arch piece, and the other vertically to receive the transverse pin, *C* is a side view, and *D* a cross section near the middle of the transverse pin, shewing the manner in which the vertical and diagonal parts connect with said pin.

This pin may be formed in this manner. Take a bar or plate of wrought iron, 6 to 7 inches wide, and $\frac{5}{8}$ to $\frac{3}{4}$ thick, of a length about 2 inches shorter than the pin is to be. Let this be rolled or swedged into the form of a trough, whose cross-section is shaped as seen at *E*. Then take a bar of the same or a little greater length, about 4 inches wide in the middle, and tapering towards the ends; the section in the middle having a trapezoidal form as at *F*. Let this be bent in the form seen at *G*, and placed in the trough formed as above, with its convexity upward, and the ends welded and swedged to a cylindrical form, (about $2\frac{1}{2}$ inches diameter,) to receive, or pass through the links of the chain. It will readily be seen at *C* and *D*, how the lower or troughed portion is punched for the reception of the diagonals, which cross one another, and are secured by nuts below, and how the upper part is pierced, and tapped for the vertical to screw into. This should be done before welding. The ends of the pin are slightly headed in the direction of the length of the chain, to keep the links from slipping off.

LIII. The vertical is a round bar of wrought iron of about $1\frac{3}{4}$ inches in diameter, as I have used them in bridges of from 70 to 80 feet span, the lower end screwed into the transverse pin as just above described, and the upper end having a thread cut for 9 or 10 inches, with two nuts, from 1 to 1¼ inch thick, the one to form a shoulder to hold the upper end of the diagonals, and the other, on the end to secure it above the cast iron arch.

Fig. 15, Pl. 1, is a vertical longitudinal section, shewing the arrangement at the upper end of the vertical and diagonal pieces. Each diagonal has an eye at the upper end, is bent so as to allow the vertical to pass directly through, and prevented from slipping down by the nut below. For this lower nut, a key, or a solid shoulder may be substituted with advantage. The diagonals should be of round iron about 1 inch diameter, for 72 feet span, and enlarged at the lower end as much as the screw thread cuts away.

LIV. It will be seen that these trusses, having a width of base equal ¼ or more of the height, will support themselves, laterally, without any assistance; wherefore the flooring, including the cross beams, may be entirely of wood, and renewed at pleasure without any disturbance of the iron work.

It is therefore recommended to use wooden cross beams, which may be formed of two pieces, as by slitting the beam vertically, and bolting or pinning the parts together, with the vertical bolt passing through the ends.—In this manner they may be put on after the trusses are put together. Between each two cross bearers, and between the endmost ones and the abutments, should be a pair of diagonal braces, (2 to 3 inches thick by 6 to 10 inches wide,) to prevent lateral swinging of the road way.

Upon the cross beams, longitudinal joists are placed to support the floor plank, a thing so simple, and so generally understood, that further description is unnecessary in this place.

More or less casings and finishings of wood work outside of the road way may be added, according to circumstances, or the taste of the builder.

In my 72 feet canal bridge, the height of truss is 9 feet. But the cord chains and road way have a cambre of 1 foot in the centre, leaving only 8 feet from centre of the arcn to the chord chains. A greater height, as we have before seen, would be more economical as regards vertical strength; but this proportion is thought to have a better oppearance, and, as the arches have no extraneous support laterally, it is best not to carry them too high, whereby they would be made top-heavy and unsteady. I would recommend the height of truss on this plan to be from $\frac{1}{9}$ to $\frac{1}{8}$ of the length, for road bridges, and a little more for rail roads; although for rail roads, I prefer the cylindrical arch pieces, except, perhaps, for spans of 50 feet or less.

Cylindrical Arch Pieces.

LV. I proceed to describe thôse parts of the truss with the cylindrical arch pieces, which differ from corresponding parts of the truss above described.

The cylinders are cast with a length from 15 to 20 times the diameter, and a thickness from $\frac{1}{20}$ to $\frac{1}{12}$ of the diameter. The ends are beveled to the radius of the arch, and are about double thickness for 3 or 4 inches from the joint, with semi-circular notches, so as to form a round hole at the joint for a horizontal pin to pass through, to hold the vertical and diagonal parts, passing in by notches or openings in the under sides of the cylinders at the joints.

Fig. 16, Pl. 2, shews a side view and section of the joint of the arch, with its pin hole, and the connections of the verticals and diagonals. The top end of the vertical is forked, so as to admit the diagonals side by side between the branches of the fork, thus bringing the action of all the forces acting on the arch, to meet near the centre of the cylinders.

The ends of the cylinders resting on the abutments, are beveled to a horizontal plane, or adapted to foot piece so beveled, with a large pin hole, by means of which the ends of the chord chains are connected with the extremeties of the arch as seen in Fig. 17, Pl. 2.

This truss otherwise differs little from the preceding, except that the chain pins are shorter, as the arch pieces are not expanded towards the ends, or points of connection with the chains, and the end-most links of the chains do not require the twisting to form said connection.

For estimating the strain upon the different parts of the arched truss, and proportioning them accordingly, reference may be had to Articles 13, 14 and 15, and to the tables and remarks upon the strength of iron.

LVI. It will be seen that these trusses, having but small width of base, will require lateral support to preserve them in a vertical position, which may be afforded by braces running up from the cross bearers, and by tying across the top when sufficiently high. In this case, since the trusses depend for support upon the cross bearers, in some measure, and since it is desirable that the iron work form an independent system, capable of sustaining itself without the aid of any less durable material, I consider it advisable (although not absolutely necessary) to use iron cross bearers, in connection with cylindrical arched trusses.

Iron cross bearers may be formed on the same principles as the longitudinal trusses, by combining wrought and cast iron where each is respectively best suited to sustain the force it will be subject to.

Fig. 18, Pl. 3, shews an iron cross bearer, and the manner of its connections with the verticals and chain pins; which latter form a sort of nucleus for the connection of other parts.

The cross bearer has a cast iron arch-like piece standing with two feet upon two chain pins of the main bearing trusses. The verticals passing down through said feet, are screwed into the chain pins. The thrust of the arch piece is counteracted by a chord chain embracing its feet. This chain is composed of three links, a little twisted so as to lie nearly flat wise, when linked together, and long enough for each of the two connected ends to embrace the vertical bolt as seen at *a*. The vertical bolts have two nuts at the lower end, (one above and the other below the chain and diagonal,) and at the upper end, a countersink, or a common bolt head, with a key or nut below the arch piece and the diagonal; the diagonal being a $\frac{5}{8}$ or $\frac{3}{4}$ inch rod with an eye at each end, and bent so as to suffer the vertical to pass square through.

In case a nut be used under the upper end of the diagonal, the vertical must be enlarged by upsetting or otherwise, where the thread is cut, so that the nut may slip up from the bottom, or the head at the top must be replaced by a nut.

The arch piece has a T formed cross section, with such enlargements where the bolts &c. pass through, as to possess the same strength as at other parts.

The verticals of the main trusses may be of round bolt iron, with a wrought iron brace as shewn on the right hand side of the figure, or formed of two flat bars of wrought iron, (as seen edgewise on the left hand of the figure,) with a thin cast iron plate edgewise between, forming an H formed cross section, the two sides wrought, and the cross, cast iron, broad in the middle and tapering to the ends, the whole secured together by rivets or bolts passing through the wrought bars, and either through the cast iron mid-rib, (transverse holes and extra thickness at the holes, being cast in it for that purpose,) or on one side and the other alternately. The lower end of the vertical should terminate in a round part and screw to pass through the cross bearer, and enter the chain pin as before stated, and the upper end, forked and punched to connect with the arch cylinders of the longitudinal trusses.

When the virtical is formed in this way it will have more stiffness, and the supporting brace need not connect so near the top, but may be of wrought or cast iron, connecting with the vertical 3 to 4 feet or more from the bottom, and with the arch piece of the cross bearer, near the angle where the rail is placed. The brace may be cast in the same piece with the arch.

The sizes of the parts of the cross bearer will depend on the length and distance asunder. For a single track rail road bridge, they should be from 14 to 15 feet long from centre to centre of trusses. The cast iron part should be about 4 inches broad and deep, and contain about 6 square inches area of cross section, if 12 feet apart, and more or less, as the distance is greater or less than 12 feet. The cast iron brace to give lateral support to the main trusses, should have a cross section of $2\frac{1}{2}$ or 3 square inches. The chord chain should contain 2 square inches cross section for cross bearers of 3 feet depth and 12 feet apart, and more, as the depth is less or the distance asunder greater, and vice versa. The vertical bolts of the cross bearer should be about $1\frac{1}{2}$ inches diameter for 12 feet apart. The diagonals should be about $\frac{3}{4}$ inch in diameter.

LVII. The lateral support necessary to be provided for the trusses, can not be exactly estimated. It depends essentially upon the force of the wind (which can not be very severe, as so little surface is exposed in an iron bridge,) and upon the centre of resistance in the arch, not being exactly in the same vertical plane with the centre of thrust, whereby is produced a tendency to lateral flexure. If we suppose the distance of the axis of thrust from the axis of resistance, in the middle of the arch, to be $\frac{1}{2}$ the diameter of the arch cylinder, and that the axis of resistance have a circular curve from end to end, which is probably more than would ever be true in practice, the tendency to flexure laterally would not exceed the 75th part of the thrust of the arch, say for a 100 feet truss, about 1,600 lbs. distributed through the whole length of the truss. A 100 feet truss should have about 8 verticals, and if each of these possess sufficient stiffness, and be so braced as to afford 200 lbs. of lateral support to the arch, a failure in that respect would be quite improbable, if not absolutely impossible. In using wrought iron braces, then, it would be proper to make them of 1 to 1½ inch iron, according to the length.

The depth of the cross bearer from the arch piece to the chord tie, should be from 2 to 4 feet; not less than 3 feet when there is sufficient space for such a depth.

LVIII. The lateral swing of the road way, is another tendency to be guarded against, and this, like the lateral tendency of the arch, can not be exactly estimated. The best means of counteracting it is, by horizontal diagonal tie braces of wrought iron from ¾ to 1¼ inch diameter, for 100 feet span, between each two cross bearers, and these, like the vertical diagonals in the main trusses, should be larger towards the ends, and smaller in the middle of the span.

These should be furnished with swivel nuts for adjustment, (see Fig. 19, Pl. 3,) and may be formed as seen at *a*, having a head and swivel at one end, and a screw at the other, with an angle and a hole from one to two feet from the screw end. Thus formed, it may be interposed between the cross bearer and the chain pin, so that the hole may receive the vertical of the main truss, after it has passed through the foot of the cross bearer, as before described.

String timbers to support the iron rail may be placed over the upright bolt of the cross bearer. Or, truss-work resting on the uppermost nut at the lower end of said bolt, and of such height as to carry the track over the top of the cross bearer, may be employed, which will give more stiffness to the track.

The height of the truss for a rail road bridge, should be from ⅛ to ⅐ the length of span, and no cambre to the chord chains.

For a span from 40 to 100 feet, (affording from 4 to 8 bearing points, or cross bearers,) this plan is little if any inferior to any that can be adopted.

Cancelled Truss Bridge.

LIX. For a general view of the truss, see Fig. 7, page 12. In this plan the part *a.o.i.h* is composed of hollow cast iron cylinders, similar to those composing the arch in the last described truss, but not beveled at the ends, except at *o,i,a,* and *h.* The chains *ah* also, are formed and connected in the same manner; but the links of the chains increase towards the middle of the truss, as shewn to be necessary in reference to Fig. 7. The verticals act only by thrust in this plan, and consequently are of cast iron; may be tubular, or of a x formed cross section, larger in the middle, and tapering towards the ends; the upper end forked, as in the verticals of the cylindrical arched truss above described. The lower end may be cast with a wrought iron screw, and connect with the cross bearer, lateral braces and chain pin, precisely as in the said arched truss, in case of the cross bearers being placed in that position. The same manner of securing the trusses laterally, and preventing lateral swing may be adopted.

The diagonals are connected at both ends in the same manner as in the cylindrical arched truss, except that the main diagonals, or those which act when the bridge is loaded uniformly, and which are shewn by the dotted lines in Fig. 7, are in pairs, (with the counter diagonals between them,) and larger towards the ends, as the stress increases in those parts. The cross bearers are made in the same manner as above described for the cylindrical arched trusses.

The cancelled, tapezoidal truss, if rightly proportioned, is from 5 to 10 per cent cheaper than an arched truss of the same strength, and for rail road bridges, is *generally* to be preferred.

It is decidedly preferable when the track may be placed on a level with the top of the trusses, in which case, the distance between trusses need not exceed the 10th part of the length of span. Hence a considerable amount of material is saved in the construction of the cross bearers, and in lateral bracing.

The preference to be given to the trapezoidal truss in this case, is on account of the bearing points, (the joints of the cylinders,) being in the same horizontal plane, and not at different elevations as in the arch.

The cross bearer for a track on the top may be as seen in Fig. 20, Pl. 3. The cast iron portion straight, and formed to fit the cylinders at the ends, where they are secured by bolts. On the other hand, the wrought iron part, which acts as a chord chain in the other cross bearer, acts as a suspension chain in this, having its connections essentially the same in both cases. The verticals and diagonals, as well as the mode of applying the track timbers, are also the same as in the case of Fig. 18.

The lateral, or horizontal diagonals, may be as shewn in Fig. 19, with or without the hole at the angle, and passing through elongated holes near the ends of the bearer, may be secured by pin or wedge.

The chain pin for trusses sustaining the track on the top, need not be adapted to receive the vertical by a hole in the upper side, but may be formed by welding the ends of 4 square bars, rounding the welded parts to receive the chain links, and opening the middle portion for the diagonals to pass through. Fig. 21, Pl. 3, shews a middle cross section of a pin formed in this manner, with the diagonals passing through, and the vertical standing on the top.

It will be most economical to make the height of the truss in this plan, equal to $\frac{1}{6}$ the length of span, but circumstances may sometimes render it advisable to reduce it to $\frac{1}{7}$ or $\frac{1}{8}$. The diagonals should incline at 45°, though a small deviation may be made without great detriment to economy.

This plan, with the single cancel, as in Fig. 7, is good, perhaps the best, for any span under 75 feet. If carried beyond that, it would, perhaps, be best to give more length without increasing the height, by making more bearing points, to avoid the disadvantage of too long cylinders in the top rib, and long verticals.

LX. From 70 or 80, to 160 feet stretches, should be made with double cancels, or two crossings of diagonals, as seen in Fig. 23, Pl. 4. The formation and connection of the parts in this case are precisely as in the above, except as follows: 1st. The end cylinders *cb* and *cd* incline at 60° instead of 45° with the horizon, and must be beveled accordingly. Also, these parts having greater length, will usually require to be cast in 2 pieces, and will require the tie and brace *ef*, as well as lateral stiffening, which may be done as hereafter described for stiffening the verticals. Or, when the track is on the top, by a transverse bar at *e*, and diagonals (Fig. 23, Pl. 4,) from one truss to the other, which will also afford lateral support for the ends of the trusses. If the track be at the bottom, as from *a* to *d*, such an arrangement would interfere with the passage of the trains, and lateral support must be provided at the ends by braces or guys (*gg*, Fig. 23,) that may not produce such interference. With the track at bottom also, the end-most verticals will act by tension only, and such materials and connections should be provided as will afford the necessarry resistance.

With the track on the top, the end-most verticals are useless and unnecessary, if other means than the brace *ef* (Fig. 22) be used to stiffen *ab*, and the two diagonals at each end represented by dotted lines, are unnecessary to the strength of the structure in all cases, unless for very short spans. It is not to be expected however, that the proportions of a bridge on this plan will be assumed without such an analysis as to determine the amount and kind of force each part must sustain, which done, it will readily be seen what parts are necessary, and what proportions should be given them.

Another modification which the double cancelled truss requires is, an opening in the middle of the vertical, for the diagonals to pass through. For this purpose, and to give them the stiffness their great length renders necessary, without too great an expenditure of materials, they may be formed, when of considerable length, as shewn in Fig. 24, Pl. 4.

The middle or cast iron portion is in two, or may be in 4 pieces, and connected and stiffened by the 4 small bent wrought iron bolts *b.b*, &c., (two only appearing in each view,) passing through flanges formed for that purpose, and strained out in the middle by a plate as seen at *a*, interposed between the two cast iron parts forming the main body of the vertical. The castings may be tubular, or of a x formed section; if very long, the tubular form will be preferable.

The Cancelled Truss without vertical Struts.

(See Fig. A, p. 14.)

LXI. This truss may be constructed of iron, with cast iron cylinders and horizontal chains, the same as when the vertical strut is used, and the description relating to those parts need not be here repeated.

The verticals *ob* and *ig*, and the two diagonals *oc* and *if*, (the track being along *ah*,) should be wrought iron, and so connected as to act by tension only. Of the other diagonals, those which act mainly by thrust, may be formed of cast and wrought iron as shewn in Fig. 25, Pl. 5. *ab* is a view as it would appear when looking lengthwise of the truss, and *cd*, when taking a side view. It has a cast iron portion with a H formed section, a hole in the centre for the other diagonal, *ef*, to pass through, and 4 wrought iron rods, each running from the centre (where they receive the diagonal *ef* through an eye or loop made for that purpose,) to the end; i. e., two to each end. Those terminating at the upper end *b* or *d*, have an eye to receive the transverse pin connecting the diagonals with the cylinders. Those terminating at the lower end, pass through the chain pin, and are secured by screw nuts.

These rods being strained out by short screw struts as seen at *g.g*, give stiffness to the piece between the centre and the ends, and being wedged, or otherwise made fast to the crossing diagonal *ef*, in the centre, are in a condition to resist a crushing force to advantage. The wrought iron rods should be sufficient to sustain whatever tension the piece may be exposed to.

Those diagonals which act slightly by thrust, but principally by tension, may be of round wrought iron rods, of sufficient size to sustain any force that may act upon them, and should be so connected as to act by either tension or thrust, without play or motion in changing from thrust to tension, and the contrary; and for this purpose the eye at the upper end should be made snug to the connecting pin by wedging, if necessary, and the lower end, passing through the chain pin, should have a nut on both sides. These pieces passing through those above described, in the centre, are secured at that point, and thus, the better enabled to stand the small thrust they are liable to.

Before constructing a bridge on this plan, it should be considered whether the track is to be at the top or bottom. Then the kind and amount of stress upon each part, due to the weight of the structure, should be ascertained, and these forces added to, or subtracted from (according as the signs are like or unlike,) the maximum effects due to the variable load, will shew the forces each part must be capable of sustaining, and the construction may be proceeded in with confidence.

These trusses may be used with either wood or iron cross bearers, and when not high enough to be connected across the top, must be supported laterally, by braces or guy rods from the cross bearers to the cylinders. When the track is on the top, the same arrangements for lateral support may be used as in the case of the truss with vertical struts.

The preceding details, it is hoped, will be found sufficient to enable the advantages and excellencies previously pointed out as appertaining to the three general plans of trussing, (viz., the arched truss, and the cancelled, trapezoidal truss, with and without the verticals,) to be made easily available, in the construction of iron bridges for rail roads, for all spans under 160 or 180 feet. The modifications required to adapt the same to the purposes of common travel will readily suggest themselves to the practical engineer and skillful mechanic.

It is not expected that these details are so full and clear as to leave no necessity for the exercise of ability, skill, and judgment in carrying them into practice. The plan and purpose of this work would not admit of such minuteness of detail, if the time, experience and ability were not wanting to accomplish the object. Neither is it presumed that some of the details that have been given, may not be susceptible of material improvement. Still, I do entertain the fullest confidence that nothing is here given but that is highly feasible, and worthy of being used as a guide, where better plans and modes are not known.

LXII. I have not yet spoken of stretches of more than 180 feet; not that this length forms a limit beyond which it would be impracticable to construct bridges on these general principles. But for greater spans than what is here mentioned, I would suggest the following modification of the truss Fig. 7, p. 12.

It is desirable that a transverse bearer should be contained in the structure, at least, once in 12 or 15 feet. Hence, in long stretches the verticals and diagonals become very numerous, and the former being very long, and acting by thrust, must necessarily act to a great disadvantage. It is proposed therefore, to form a simple truss as in Fig. 7, of a height of from $\frac{1}{8}$ to $\frac{1}{6}$ the length, containing from 5 to 7 full length verticals, (or from 3 to 5, dispensing with the end ones,) and between each two of these, make use of suspension chains and 2 or more short verticals, when the track is to be on the top, or sub-arches and short verticals when the track is to be at the bottom.

Fig. 26, Pl. 5, shews this arrangement, one half with the track at the top, and the other with the track at the bottom. When the track is below, small suspension chains will be required to sustain the weight of the cylinders between the main bearing points, as shewn in the figure. Or there may be independent trusses, or bridges extending from one of the main bearing points to another.

I have not perfected the details of such an arrangement, but have so far considered the subject as to be satisfied of the feasibility of it, even for spans of 400 or 500 feet, not to say more. This plan admits of using *wire cables* to advantage.

It has been proposed by an eminent English engineer, (Mr. Stevenson,) to construct a tubular, or box bridge over the Menai Straights, out of plates or sheets of wrought iron about 1 inch thick; the box to be 15 feet wide, 30 feet high, and 450 feet long, and weighing upwards of 1,500,000 lbs, and it is doubted, even by the projector, whether this will be sufficient for the purpose.

Now, I have estimated, with considerable care, the probable amount of iron that would be required for a bridge of 450 feet on the plan shewn in Figure 26. In this estimate I have reckoned the safe load of wrought iron at 12,000 lbs. to the square inch, which is about 15,000 lbs. for that portion which sustains the dead weight of the structure, and 10,000 for that which supports the moveable load, instead of 10,000 lbs. for the whole, as I have adopted the rule of estimating for rail road bridges of ordinary lengths, for reasons to be explained hereafter. This course seems warrantable in the present case, since, besides that so large a portion of the strength of the material goes to support the dead weight, whatever effects may be due to the motion of the additional load, will be proportionately much less, acting on such a mass of material, than upon a short bridge of only a few tons weight.

My estimate shews a gross amount of about 560,000 lbs. in about equal portions of wrought and cast iron, to sustain 900,000 lbs. besides the weight of the structure. We may put this amount, in round numbers, at 600,000 lbs., and the tubular bridge, as above described will require at least two and a half times as much iron, all wrought, to sustain the same load, in addition to its own weight, with the same stress on the material.

As a corollary to what precedes, it may be remarked that, as a 450 feet bridge will support twice and a half its weight, one of 900 feet will weigh four times as much, and sustain twice as much, or will sustain 25 per cent. more than its own weight, with the same stress of material. Hence, no good reason appears against the practicability of making a trussed bridge of 900 feet long, (by 150 feet high,) that would be safe for ordinary travel, but not for a rail way, (unless the truss be proportionally higher.) It is probable, however, that from 4 to 5 hundred feet would form a limit, practically, beyond which it would scarcely be advisable to undertake the construction of a trussed bridge.

Effects of rapid motion of Trains, and remarks upon the forces to which Bridges may be exposed.

LXIII. I have given one fourth the absolute positive strength, and one third the negative strength, as the safe *practical* strength of iron, beyond which it is not prudent to load that material. This has reference to dead forces, and not, of course, to forces brought instantaneously to act on the material.

It is well known that, on an elastic material like iron, a given force will produce twice as much effect when instantaneously applied, as when brought gradually to act. Any one can illustrate this by taking a pound weight, holding it in contact with the dish of a spring balance, and letting it go suddenly, when it will be found to move the pointer to the two pound mark, after which it will settle and remain on the one pound mark.

Now, upon a bridge for common travel the maximum forces to be provided for are, the weight of the materials in the structure, and a crowd of men or other animals, all of which acts principally as so much dead weight. Hence, for such bridges, it is proper to consider the whole area of the road way, covered with men, which is about 100 lbs. to the square foot, as the greatest load to which the bridge can be exposed. The effects of such a load, upon the whole or any part of the platform, added to those of the weight of the structure, are the forces which the bridge should be calculated to sustain; and for that purpose, it is sufficient to provide in each part, 1 square inch cross section of good wrought iron to every 15,000 lbs. of tension to be sustained, and for thrust, to provide cast (or wrought) iron, according to the table for the safe practical strength of cast iron.

The methods of estimating the stress of the several parts of the structure, were given in full in the first part of this work.

In regard to rail road bridges, the enquiry naturally suggests itself, whether the ponderous engines and trains dashing over with lightning velocity, do not produce effects approximating to those of forces instantaneously applied? Doubtless some greater effect is produced upon some parts of a bridge by the rapid, than by the slow transit of a train. But with respect to the horizontal parts, (and the arch, in case of the arched truss,) which suffer their maximum strain under the full load of the bridge, sufficient time elapses between the first and the last portions of weight added, to obviate any tendency to mischief from the sudden application of force. With regard to the diagonals and the verticals, it is doubtful whether the same remark would not hold essentially true.

When the equilibrium of an elastic body is disturbed, it vibrates about the point of equilibrium, with longer or shorter times of vibration, according to distance of the motion and the mass of the vibrating body. If a weight be instantly left to act upon the lower end of a suspended iron rod, it will stretch the rod twice as far, abating what is due to atmospheric resistance and want of perfect elasticity, as it is capable of holding it at for any length of time. The elasticity of the rod will then preponderate, and will raise the weight nearly to its original position. The weight again will preponderate in turn, and thus the vibration will be kept up, till the want of perfect elasticity in the iron, and the resistance of the air, shall finally induce a state of rest. Now, we know that these vibrations are extremely short, as to duration of time, as well as extent of motion; quite inappreciable to the senses, unless the rod be very long; and if the weight be applied so gradually that a space of time equal to several of these vibrations, elapses from the incipiency to the end of the application, the vibration must be reduced to almost nothing, if not entirely destroyed. With regard, then, to the diagonal of a bridge, the weight has to traverse a space of 10 or 12 feet from the time it begins to act, until it comes to produce its full action, requiring, at a velocity of 60 ft. in a second, or $\frac{3}{4}$ of a mile in a minute, $\frac{1}{6}$ part of a second of time; a space greater than many of the vibrations above spoken of.

I conclude then, that theoretically, no allowance need be made for the rapidity of the passage of rail road trains, and, that if the rail be perfectly straight and even, and the motion true and equable, there is no more danger to a bridge from a rapid than a moderate transit.

But *practically*, this perfection in the adjustment of the rail and the movement, can not be secured; and consequently it is reasonable to suppose there is, in reality *some* more liability to failure by the quick passage of a rail road train, than by the same load at rest on the bridge.

Again, the adoption of iron bridges for rail roads, is a new thing, which many regard in the light of an "experiment," about which many wise doubts are entertained; and to reduce the chances of failure in this incipient stage of the matter, which would serve to weaken confidence, and delay the introduction of the change, as well as be attended with disastrous consequences to life and limb, perhaps; I have thought proper to provide bountifully of the means for sustaining all the forces, both actual and contingent, that can ever come to act in the premises.

For that purpose, I have adopted the general rule for R. R. bridges, of estimating wrought iron to sustain only 10,000 lbs. positive strain to the square inch, (only one-sixth the absolute capacity,) and cast iron hollow cylinders, of a length equal to 18 diameters, from 10 to 12,000 negative strain, or about a fourth part of the actual capacity; and in like proportion for other forms and dimensions of pieces. Consequently, in estimating for R. R. bridges, I diminish the quantities given in the table for the practical strength of cast iron by about one quarter.

As to the gross load for a R. R. bridge, the heavy engines now used in this country, comprise about 20 tons of weight within as many feet of length. The heavy freight trains weigh, in general, not over 1000 lbs. to the foot run. But I have thought proper to estimate bridges as liable to a load of 2000 lbs. to every foot, upon the whole or any part of their length.

The providing of abundant strength seems the more proper, as the tendency has been and still is, to increase the weight of engines and loads to the utmost capacity of the track, and hence an iron bridge, intended as a durable structure, should be proportioned with reference to such changes and tendencies.

I have now given such details of the plans I propose for the construction of *Iron Truss Bridges*, as appeared to me necessary to guide the skillful builder in the erection of bridges, for rail roads or common roads, of all lengths which will be likely to be undertaken upon the Truss principle. I will, therefore, leave this part of the subject for the present.

Suspension Bridges.

The general principle of Suspension Bridges was briefly alluded to in Article 6. It is not my purpose to go much into particulars with respect to this kind of bridges. My object is not to *compile* from works already before the public, but to give such results of my own labors and investigations, as are believed to possess originality and value. Not having given a large share of attention to the details of Suspension Bridges, I shall have but little to offer.

The longest spans ever built, have been on the suspension plan. In fact, this plan has advantages for very long spans, which render it practicable to construct longer stretches upon this than upon the truss principle. In the truss bridge, a large portion of the material does nothing towards directly sustaining the weight of the bridge and load. The horizontal portions, for instance, in trusses Fig. 7 and Fig. A do not directly sustain a pound of weight. They simply act and react upon one another through the medium of the oblique parts, and are essential to enable the latter to perform their functions. The chord chains also, in the arched truss, only sustain the horizontal action of the arch.

Now, all of these parts add to the weight of the structure, and proportionally to the necessary amount of material.

In the suspension bridge also, the oblique action of the suspension chains gives rise to horizontal forces which must be counteracted. But the material by which this counteraction is produced, is situated outside of the piers or abutments supporting the spans, and consequently, do not add to the weight to be sustained between said piers or abutments. Therefore, a less proportion of the strength of the material is exhausted in sustaining the structure itself.

But we have seen that the weight of a bridge increases in the duplicate ratio of the increase in length, while its power of sustaining weight, only increases in the *simple* ratio of increase in length; and the limit of the practicable length of span is, when the weight becomes equal to the whole power of the material to sustain.

It is manifest, then, that the trussed bridge will sooner reach that limit than the suspension bridge, the latter containing a less amount of material in proportion to its strength, or power of sustension.

If the trussed bridge require fifty per cent more material for a given length to sustain a given weight, than a suspension bridge, the latter will sustain its own weight till the square of its length is fifty per cent. greater than the square of the greatest length at which the former can sustain itself.

This advantage of the suspension principle, renders it worthy of some consideration for very long stretches, where the weight of the structure forms so large an item in the whole amount of weight to be supported.

Suspension bridges have been used with tolerable success for common travel, where the moving load is trifling, compared with the weight of the structure itself. The most important one now in use in this country, is that over the Schuylkill, near Philadelphia, being 343 feet long, and 27 feet wide. It is sustained by wire cables passing over towers at the corners of the bridge, the ends of the cables carried down obliquely and anchored into the ground outside of the towers, and the central portions hanging in a catenarean curve between the towers. The road-way or platform is suspended from these curved cables. This is the general plan of constructing suspension bridges, using sometimes wire cables, and sometimes chains formed by connecting bars of iron. Much longer ones have been constructed in Europe, even two or three times as long as the Schuylkill bridge, some of which have endured, while others have failed.

A serious difficulty in the use of suspension bridges is, the want of fixedness or stability among the parts. The curve of the chains being left to find its own equilibrium, yields to every force that tends to disturb that equilibrium, and hence arises an undulatory motion, whenever the bridge is exposed to the passage of heavy loads, or to the action of strong winds, which is frequently attended with disastrous consequences. This quality renders these bridges utterly unfit for rail road purposes, as they are usually constructed. No plan has yet, as I believe, been devised and successfully executed, to obviate the difficulty. Similar obstacles are encountered, to those alluded to in speaking of the feasibility of sustaining the horizontal thrust of truss bridges by the action of abutments and piers.

I have a plan which is believed to be original, and which is *certain* to obviate the difficulties of undulation, and render the suspension bridge as applicable to rail roads as to common roads. It is not, however, to be expected, that a bridge can be built that will sustain a rail road train of hundreds of tons, at so small an expense as will suffice for a common road bridge, seldom required to support more than a few thousand pounds.

LXV. This is my plan. Divide the stretch into a convenient number of parts, of 25 or 50 feet, or any convenient length; as by the points *b.c.d.e.f.*, Fig. D., Pl. 5. At each of these points is to be provided support for a weight, say of one ton to each foot between each two of these points, in addition to the weight of a proportionate part of the structure. The former of these weights may be represented by w, and the latter by w', and their sum by W.

Now, from *a* to *b*, let there be an inverted arched truss, the horizontal and vertical parts being of cast iron, (the former, hollow cylinders,) and the rest of wrought iron. Let the end *b*, of this truss, be sustained by a chain or cable *hb*, descending obliquely from the top of the supporting tower *ah*. If $ab = ah$, which we will suppose for the present, the chain *hb* must be sufficient to sustain a stress $= W\sqrt{2}$ and the horizontal hollow cylinders *ab*, must sustain a negative strain or thrust equal to $W +$ the action produced by the extreme tension of the curve chain *akb*. We have now provided, (considering the points *a* and *h* to be fixed,) support for the point *b*, and for the space *ab*.

For the point *c*, let fall two chains, *hc* and *ic*, the former (*hc*) being capable of sustaining a stress equal to $\frac{1}{6} W\sqrt{5}$, and *ic*, capable of sustaining a stress equal to $\frac{2}{3} W\sqrt{17}$.

For the point *d*, the two chains *hd* and *id*, each capable of sustaining $\frac{1}{2} W\sqrt{10}$, will be sufficient.

The points *e* and *f* are provided for in the same manner as *b* and *c*. It is obvious that any other number of points might be assumed, the same principle being applied to support them.

If the distances *bc*, *cd*, &c., be greater than can be economically spanned by simple beams, truss-work may be employed to advantage, either of wood or iron, upon some of the general plans, previously, or hereafter to be described in this work.

Similar provisions by bracing, &c., must be resorted to to guard against lateral swing, as is made use of in the case of truss-bridges.

In order to secure the permanency of the point *h*, and prevent the tower from being drawn inward by the tension of *hb*, *hc*, &c., a guy chain *hl*, descends obliquely, with an inclination equal to that of the resultant of all the antagonist forces, and of a strength equal to the aggregate of those forces, (or their resultant, according to the mode of arrangement.)

The chains hb, hc, hd, &c., should be supported at suitable intervals, to prevent them from assuming a curved form, by which they would, in a measure, be liable to the same difficulties from undulating motion as the common suspension bridge; though to a much less degree. This support may be given to the chains by vertical rods or studs running up from the platform, or by a curved chain $hmni$, from which those below may be suspended, and those above, supported by stiff rods. Whatever may be supported by this curved chain, will leave so much less to be sustained by other parts of the structure, and the latter may be proportionally smaller.

To estimate the quantity of iron in this structure, let us assume $ag=450$, and $ah=75$ feet$=ab$. Now, the material in each part, is as the stress multiplied by the length of the part. But the stress is as the weight sustained, multiplied by the length, therefore, the material is as the weight, multiplied by the square of the length.

Making $ab=1$, hb will be equal to $\sqrt{2}$ and $hb^2=2$. But the weight sustained by hb,$=W$. Hence the material in hb may be represented by $hb^2\,W=2\,W$.

Again, $hc^2=ha^2+ac^2=1^2+2^2=5$, and the weight sustained by hc, being $\frac{2}{3}W$, the material will be represented by $\frac{2}{3}W\times 5=\frac{10}{3}W$.

In like manner, the material in hd, may be represented by $\frac{1}{2}W\times 10=5\,W$, and the material in he, by $\frac{1}{3}W17=\frac{17}{3}W$.

Adding these amounts together, we have 16W to represent the material in hb, hc, hd, and he, and $32\,W$ for the same, together with the corresponding chains ci, di, &c.

Now, the unit in this expression$=75$ feet, and $W=150$,000 lbs. plus $\frac{1}{6}$ the weight of the structure between piers, which we will assume at 66,000 lbs., making $W=216{,}000$ lbs.

To sustain this weight at 12,000lbs. to the square inch, will require 18 square inches cross section of iron, or about 60 lbs. to the foot. This multiplied by 75, gives 4500 lbs., which substituted for W in the expression 32 W, gives 144,000 lbs., as the weight of material in the parts in question.

The tension of the guy *hl*, will be about $5\frac{1}{2}$ *W* and the length, say 250 feet$=3\frac{1}{2}$ *ab*. Therefore the material in *hl*, will be represented by $18\frac{1}{3}$ *W*, and a like amount for the opposite end, making $36\frac{2}{3}$ *W*, which gives 165,000 lbs. as the weight of material, making a total of 309,000 lbs. for main suspension chains or cables. To this should be added, probably, 20 per cent for vertical support to chains, extra weight at joints, or connections, making 371,000 lbs.

We now want 5 heavy cross bearers at *b.c.d.* &c., for

which say	12,000 lbs. cast, &	8,000 wrought,
6 truss bridges of 75 ft.,	48,000 " "	48,000 "
Cast iron saddles on towers, &c.,........	12,000 " "	
Horizontal guys and diagonals to prevent lateral swing, say		10,000 "
Add amount fr'm above..............		371,000 "
Total,............	72,000	427,000

This makes the weight of iron, in round numbers, 500,000 lbs., of which, say 300,000, are between piers. Adding to this 70,000 lbs. for rail track, makes 370,000 lbs., as the weight of that part of structure between piers. Now, the structure is calculated to sustain $6\,W=1{,}296{,}000$ lbs., from which, deducting 370,000, leaves 926,000 as the effective strength of the bridge.

In a preceding article, (62) I made the amount of iron in a 450 feet truss-bridge, by liberal estimate, about 600,000 lbs., and the effective strength 900,000 lbs.: from which it appears that the truss bridge requires about 100,000 lbs. more iron to sustain the same weight, than the suspension bridge. But it does not follow that the expense would be in the same proportion. On the contrary, the suspension bridge would cost considerably more, in consequence of being mostly of wrought iron, while the truss-bridge is composed of one half cast iron, which does not cost more than half as much for the same quantity.

Besides this, the towers to support the suspension bridge, and the pits for securing the guys, add greatly to the expense.

I conclude then, that unless some better and more economical plan can be devised for adapting suspension bridges to R. R. purposes than I have yet met with, the truss bridge is decidedly preferable for any thing less than 500 feet, unless it be in very peculiar circumstances, and such as have never come within my observation. If cases should occur warranting attempts to construct R. R. bridges of greater stretch than 500 feet, the suspension might deserve a preference. But such will occur so rarely, if at all, that I choose to leave them for the special investigation of those into whose hands their management may fall, rather than devote to them, in this work, the time and space which I consider would be more profitably occupied in matters more within the range of ordinary practical bridge building.

I will here make one remark in reference to the proposed "Niagara Suspension Bridge."

I have seen it stated that the plan proposed is, "the combination of the suspension chains, (or cables,) with a heavy cast iron arch," which, it is thought, will obviate the difficulties of undulation.

How much or how little this may differ from a truss bridge, I can not judge, without a more full description of the plan than I have yet seen. I will hazard the opinion, however, that the modification of the truss shewn in Fig. 26, is better suited to the purpose than any combination of arches and suspension chains that can be devised,

The banks of the river being very high, the bridge would be so placed as to have the rail track on the top, while a passage-way for carriages and common travel could be arranged underneath, perfectly secure from all danger, except, perhaps, that of frightening horses by the passage of trains overhead. It would probably be best, however, not to suffer horses to go on to the bridge when trains were in hearing.

WOODEN BRIDGES.

Preliminary Remarks upon the Strength of Timber, &c.

LXVI. The qualities of wood as a building material, have been extensively treated of by authors, with whose works the public have long been familiar, with a degree of ability and research to which I can make no pretensions. I shall therefore only simply state the conclusions I have arrived at, from reading and observation, with respect to the average absolute strength, (positive, negative, transverse, and to resist splitting, in certain cases,) of the timbers principally in use for building purposes, as also, the forces they will bear with safety in various circumstances. At the same time, I shall, of course, leave it to others to adopt my conclusions for their own practice, or to modify or correct them, according as their greater experience or better judgment may dictate.

Pine timber in this section of country, is perhaps to be ranked as the most valuable timber for building, in use.—White oak and some other varieties are preferred for some purposes, as being harder and somewhat stronger, and especially better calculated to bear a negative or crushing force, whether acting parallel with, or at right angles to their fibres. But in what follows, I shall principally have reference to the ordinary white pine of this country.

Some writers and experimenters estimate the absolute positive strength of pine, at 10,000 lbs. to the square inch of cross section, and the safe practical strength at one-fifth of this amount, or 2000 lbs. to the square inch. I am not in possession of facts that would warrant the conclusion that this estimate for the absolute strength is too high. But I have been induced to adopt the rule for practice, of not relying on pine to sustain more than 1000 lbs. to the inch, for that portion of the piece not cut or separated to form connections with the rest of the structure.

The negative strength of pine is not over about half the positive strength. But I consider it reliable under a crushing force, (parallel with its fibres) of 1000 lbs. to the square inch, in pieces of a length not more than five or six times the diameter. And for pieces of the length of 18 times the diameter (or side of the square) I would trust it with 700 lbs. to the square inch.

From a length of 18 down to 6 diameters, I would add *pro rata* of the difference between 700 and 1000 lbs., and from 18 diameters upward, make use of the rule given in regard to the negative strength of iron.

The following table, giving the lengths of pieces in terms of the sides of their cross-sections, or their diameters in the directions in which they are liable to be deflected, and the number of pounds to the square inch which they may be relied on to sustain with safety, may be of utility in practice. For lengths above 40 diameters, the strength is reckoned as the cube of the diameter divided by the square of the length.

Length.	*Pounds.*	*Length.*	*Pounds.*	*Length.*	*Pounds.*
6	1000	24	460	50	143
8	950	26	410	55	118
10	900	28	370	60	100
12	850	30	336		
14	800	32	307		
16	750	34	283		
18	700	36	263		
20	600	40	229		
22	520	45	176		

Transverse Strength of Wood.

XLVII. Pine will bear a transverse strain of 1500 lbs. to the square inch of cross section; i. e., the projecting end of a beam will sustain, at a distance from the fulcrum equal to the depth of the beam, 1500 lbs. for each square inch in its cross section. Or what amounts to the same, a beam 1 inch square upon supports 2 inches apart, will sustain in the middle, a weight of 3000 lbs.

I have therefore, adopted 250 lbs. to the square inch, as the safe load for pine when acting transversely; and to calculate the safe load for a projecting beam, this quantity must be multiplied by the cross section and the depth, and divided by the distance of the load from the fulcrum.

For the safe load in the middle of a beam supported at the ends, take four times the above amount, or 1000 lbs., multiply by the cross section and the depth, and divide by the length between supports.

In order that a piece of timber may act by tension, it is necessary to cut off a portion of the fibres, so as to form a head, or shoulder reversed from the end, for the stretching force to act against; and that the strength of the piece may be made available for as great a portion of the length as may be without having the shoulder split off, it is important to know the power of the material to resist that kind of action.

Let *ab*, Fig. 27, Pl. 6, represent a shoulder by means of which the stick is made to act by tension. The area *ab* should contain about 1 square inch for every thousand pounds to beapplied to it. Now if *ab* be too nigh the end of the stick, the part *abcd* will be split and thrust off from the end. I find by experiment, that, to produce this effect upon straight grained pine, requires a force of nearly 600 lbs. to the square inch in the area of cleavage, *efcb*. I would therefore, in practice, allow 1 square inch for every 100 lbs., which would require *ef* to be equal to 10 *ae*.

If the shoulder be in the central part of the stick, as by a mortice or pin-hole, two cleavages must be made from the hole to the end in order to force the part out, consequently, the distance from the hole to the end, need be only five times the width of the hole ; i. e., an inch hole should be 5 inches, and a two inch hole, 10 inches from the end.

Timber is sometimes liable to be crushed by forces acting transversely to its fibres. If the pressure be applied to the whole side of a piece, the pressure should not exceed 150, or at most 200 lbs. to the square inch, in practice. If the pressure act on one-half or less of the surface, it will bear from 300 to 500 lbs. to the inch without yielding so as to endanger the work. Hard timbers will bear from 20 to 50 per cent. more.

From what has been said, it follows that for a piece to act to the best advantage by tension, if the connection be made all at the same point in the length, the piece must be cut half off, so as to form an area of shoulder equal to the cross section of the remaining part of the stick. But if several shoulders be made, a less number of the fibres will require to be separated.

If, on a piece 4 inches thick, instead of a shoulder of 2 inches, at 20 inches from the end, we make two of 1 inch each, one at 10 and the other at 20 inches from the end, we have the same area of shoulder, and fifty per cent. more fibres to act by tension, which may be made available by cutting another shoulder at 30 inches from the end. Thus a greater portion of the fibers, but a less portion of the length is made available.

In the same manner, if a piece be connected by pinning, requiring two pins of 2 inches diameter, at 10 inches from the end, four 1 inch pins, two at 5 and two at 10 inches, (if stiff enough,) will give the same shoulder surface, and require the cutting of only half as many fibres ; and two more pins, at 15 inches from the end, will give three-fourths of the whole amount of fibres available. In case the smaller pins are not stiff enough, they may be of an oblong section in the direction of the strain.

A still further reduction of the depth of shoulder, or width of pin, will make a still larger portion of the *fibres* available, but not so much *length*, and experience and judgment, with a little calculation, will dictate the proper medium in this respect. The limit in theory, is, when the shoulders are infinitely small, in which case the whole amount of fibres becomes available. But in practice, I would only estimate from one-half to two-thirds as available for tension. This reduces the safe load for pieces acting by tension, to from 500 to 700 lbs., (I usually take the former,) to the square inch, for the whole cross section.

I shall not, perhaps, find a more suitable place to make a few general remarks upon the merits and use of pins for connecting pieces of timber. I am much in favor of the use of *iron* pins and bolts for this purpose, especially for connecting pieces that cross, or meet one another at an angle. Wooden pins in such cases, do not possess sufficient strength in proportion to the surface, unless made so large as to cut the timber too much. Moreover, the action on the pin tends to crush it laterally, in which direction the hardest timbers do not offer so much resistence as the ends of the fibres to which they are opposed.

Where pieces are connected with their fibres parallel, wooden pins or keys with the cross sections elongated in the direction of the grain, to give them the necessary strength, may be employed without too much cutting of the timber; but as just remarked, the key is liable to yield before the ends of the fibres cut, are taxed to their full capacity, and consequently is poorly adapted to the purpose in any case where the utmost strength is required.—But when the grain runs in different directions, the hole can not be elongated without too much cutting of at least one of the pieces. Suppose a piece to be connected by a pin between two others. The pin should be strong enough to bear as much strain as the opposed surface can sustain. Now, this can scarcely be accomplished by wooden pins, as has just been remarked. But, if sufficiently stiff, the pin may yield somewhat without injury to its strength. The transverse strength of pin timber may be taken at about 300 lbs. to the inch. The formula $4 \times 300ad \div l$, (a=cross section, and d=diameter of pin,) gives the amount that the pin will bear in the middle.—Now l, in this case, is equal to $1\frac{1}{2}$ the thickness (t) of the middle piece of timber, and the effect of the force exerted by said middle piece, is two-thirds of what the same force would produce if concentrated in the middle of the pin. We have, then, $4 \times 1\frac{1}{2} \times 300ad \div 1\frac{1}{2}t, =1200ad \div t$=strength of the pin.

But the opposed surface will bear $1000td$.

Putting this expression equal to the former, and deducing the value of d, in terms of t, it will shew the smallest diameter of a wooden pin strong enough to bear as much as the opposed surface.

This gives d a trifle (about $2\frac{1}{2}$ per cent,) larger than t.

In the same manner, $4 \times 1\frac{1}{2} \times 5000ad \div 1\frac{1}{2}t$, represents the strength of an *iron* pin in the same circumstances; and putting this equal to $1000td$ gives the most economical diameter of an iron pin$=,252t$, and the length$=2t$.

Since the pieces a and c, [Fig. 28,] require each half the thickness of the piece b, and since the diameter of the pin should be about $\frac{1}{4}t$, it will be equal to $\frac{1}{2}$ the thickness of a or c. Hence, when a piece is fastened on by spiking, the spike should have a diameter equal to $\frac{1}{2}$, and a length from 3 to 4 times the thickness of the piece spiked on, in order to secure the greatest effect, for the amount of material cut off by the spike hole.

When the end bearings of the pin act transversly to the grain, they require at least 50 per cent more extent of bearing, which increases the value of l in the formula to $1\frac{3}{4}t$, and the effect of the force upon the middle portion to $\frac{4}{7}$ of the effect of the same force acting all at the centre. The equation for the proper diameter of the pin, then, will be $4 \times 1\frac{3}{8} \times 5000ad \div 1\frac{3}{4}t = 1000td$, whence $d=,283t$, and the length, $2\frac{1}{2}t$.

Splicing.

This is an important operation, and may properly claim consideration in this place.

Fig. 29, Pl. 6, is a form of splicing in which $\frac{1}{3}$ of the fibres are made available; the depth of the locking being equal to $\frac{1}{3}$ the thickness of the stick. The length of the lap should be 10 times the depth of locking each way, or $20 \div 3 = 6\frac{2}{3}$ of the thickness of the stick.*

By two lockings, as in Fig. 30, Pl. 6, one-half the fibres are available, with a lap of 10 times the thickness of the stick.

With three lockings, on the same principle, $\frac{3}{5}$ of the fibres are available, with of a lap of 12 times the thickness, and by lapping $13\frac{1}{3}$ times the thickness, we make $\frac{2}{3}$ of the fibres available. Finally, by a lap of 20 times the thickness, and an infinite number of lockings, the whole amount of fibres would be available.

But this, of course, is a limit not attainable in practice. From $\frac{1}{3}$ to $\frac{2}{3}$, say $\frac{1}{2}$, on an average, is as much as can be reckoned on, and this is as much as can usually be made available at the end connections.

Splicing may be done by lapping with a plain scarf, as in Fig. 31, Pl. 6, and bolting, pinning and spiking the parts together. If the pins and spikes be properly arranged and proportioned, a strong splice may be formed in this way, with a less lap than on the preceding plan. But the expense will usually be greater, and on the whole, the locked splice is generally entitled to a preference.

Timbers may also be connected by iron straps and bolts, either with or without a lap. The aggregate cross section of straps should be about 1 inch for every 10 to 15,000 lbs. strain which the splice is intended to bear.

The diameter of the bolts fastening the straps, should

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* In this splice, the power being applied at the reversed shoulder, out of the line of the unbroken fibres which resist the power, the tendency is to throw the ends outward, and produce a degree of lateral action, which somewhat weakens the timber beyond the proportion to the fibres cut.

be from $\frac{1}{6}$ to $\frac{1}{8}$ the thickness of the timber, to secure the greatest effect according to the fibres cut by the bolt hole.

To connect two sticks twelve inches square, so as to make half the fibres available, take 6 straps, two feet long from hole to hole, (three straps on a side,) and six 2 inch bolts, as shewn in the figure 32, Pl. 6. The straps should contain each, 1 square inch cross section, or a little more, in all parts.

This will take, say 150 lbs. of iron at 8 cents,...$12,00.

To form a splice with 2 lockings, 10 ft. lap

will cost, for 10c.ft. timber at 30 cts.,......	$3,00	
40 lbs. iron at 8 cents,.................	3,20	
Extra labor,.........................	1,00	
	——	7,20
Difference,..................		$ 4,80

The splice with iron straps will cost near $5,00, or 68 per cent more, for a 12 inch timber, than the lap and lock splice. The same proportion may not hold for all sizes of timber, but there can be little doubt that the lap and lock plan is the better and more economical mode of splicing.

I will now proceed to the application of the preceding general facts and deductions to the construction of bridges.

It is not necessary to allude to bridges of less than 12 or 15 feet, simple beams being sufficient for such cases.

Plan of a 20 ft. Bridge for Rail Roads,

Applicable for Stretches from 15 to 25 ft.—[See Fig. 33, Pl. 7.]

In this plan, *A* represents a side, and *B*, an end view of one truss; and *C*, a top view of the bridge. The scale is distorted, in order to shew the details more distinctly, without making the figure of an inconvenient size. The lengths of pieces are on a scale of 1 to 100, and the widths or diameters, 1 to 30. The angles are in their true proportions.

The truss is composed of a horizontal stringer, 5 inches deep by 6 inches wide, two main braces 6×5, 2 pieces of hard wood plank 6×2, and about $2\frac{1}{2}$ feet long, spiked on to the lower side of the stringer at the ends; 2 bolts $\frac{3}{4}$ inch, to secure the lower ends of the braces, with clips, or washer plates under the heads and nuts; and 2 bolts, 1 inch diameter, passing from the vertex of the braces, divergently down through the transverse beam, as seen at B. Under the heads of these bolts at the vertex, is a plate fitted to the wood, about $\frac{1}{2}$ inch thick, and at least 24 square inches in area. Between the two nuts and the lower side of the beam, are plates of the same thickness and same aggregate area, to prevent the nuts from bedding into the timber.

The lower ends of the braces toe into the horizontal stringer, by two shoulders; the extreme one, $\frac{3}{4}$ of an inch deep, and the other $1\frac{1}{2}$ inches deep, and at least 20 inches from the end of the stringer.

The bolts supporting the transverse beam, diverge each at least 5° from the perpendicular, to secure the upright position of the truss, and equal tension of the bolts.

The trusses should be about 14 feet apart in the clear, and the cross beam 12 inches wide by 14 deep; or greater depth and less width, when practicable. It should be secured by bolting, spiking or other means, to the lower side of the stringer. Or, in some cases, it may be above the stringer with advantage.

For the horizontal braces shewn at *C*, 2 by 6 inch stuff is sufficient; and they should be spiked to the under side of the rail timbers.

The rail timbers should be at least 10×12 inches.

Bed pieces 3×12, on the abutments, under the ends of the trusses, may or may not be used.

This bridge will take a trifle less than 100 cubic feet of timber, including, or 55 ft. without the rail timbers; and about 135 lbs. of iron.

To adapt this plan to a greater or less span than 20 ft., the cross section of parts acting by tension should be varied in the same ratio as the length of span. Those acting by thrust, should be varied at a rate between the simple and the duplicate ratio of the variation in length.

I prefer a reference to general facts and principles, to rules and formulæ, the reasons for which can not be taken in at a view. I have given illustrations of the modes of determining the maximum forces of every kind to which each piece in the truss is liable; also, the power of resistence of the material in the various circumstances in which it may be placed. I will here add the following general directions, which, if attended to in arranging the proportions of a bridge, will be pretty likely to ensure its sufficiency for the purpose it is intended for.

Estimate the kind and amount of force to which each piece is liable. If it be tension, see that there be at least 1 square inch of unbroken fibres, for every 1000 lbs. of tension. If a negative or crushing force, parallel with the fibres, consider the length between supported points, and the least diameter of the piece, and then see whether the strain per square inch, is greater than the amount given in the table of safe negative resistence of timber, for pieces of such proportions, and if necessary, change the dimensions accordingly. See that the pressure upon the end of the grain, be not more than 1000 lbs. to the square inch, and at right angles with the grain, from 150 to 500 lbs., according as the pressure is upon the whole, or only a small portion of the surface.

As a piece can only be made to act with tension, by cutting a portion of its fibres, and applying the force to the reversed ends, be sure that at least an inch area of fibres thus cut, for every 1000 lbs. of tension, be opposed to the pressure of the straining force.

Where an iron pin secures one piece between two others, acting in the directions of the fibres throughout, see that the diameter of the pin be about ¼ the thickness of the middle piece, and extend into each of the out side pieces to the length of ½ the thickness of the middle piece. If the action of the pin be transversly to the fibres, give it at least 50 per cent more bearing surface for the same pressure, than where it acts on the ends of the fibres, and make the diameter not less than ,28 the thickness the middle piece. Where pressure acts on the reversed ends of fibres, tending to force them out or off from the end of the stick, be sure that the amount of cleavage, be not less than one square inch for each 100 lbs. of force. See that the transverse strain of timber, and the positive stress of iron, are in all parts within the limits of safety.

Finally, see that every force that tends to break or derange the structure, have an adequate counteracting force opposed to it, and attend to such minor details as cannot fail to suggest themselves while attending to the above enumerated points, and the chance of failure must be exceedingly small, if failure be even possible.

24 to 36 ft. Span.

For this range, probably no better general form of truss can be adopted, than that shewn in Fig. 4, page 9. The principal details are the same as in the above described plan of a 20 ft. bridge. The foot of the braces are secured in the same manner, the amount of bearing surface being increased to correspond with the increased pressure. The upper ends of the braces should be made square, and the upper horizontal piece, beveled so as to fit the braces. The small cross braces toe into the upper and lower horizontals, the one being 3 × 7 inches, (for a 30 feet bridge,) and edgewise to view, the other 3 × 5, flatwise to view, and passing through a mortice in the centre of the former.

The transverse bearers are the same size, and sustained in the same manner by bolts diverging downward, as in the 20 feet bridge. Also, the horizontal braces are arranged the same, but should be a little larger, except the middle pair; say $2\frac{1}{2}$ by 6 inches.

The dimensions of the parts of the truss for a 30 feet bridge, should be, for the lower stringer, about 6 inches deep by 7 wide. The braces and top piece, 7 inches square. The sizes of the cross braces have been already given. The bolts at the feet of the braces, $\frac{7}{8}$ or 1 inch in diameter. The suspension bolts, about $1\frac{1}{8}$ inches diameter.

The truss should be about 7 feet from top to bottom, for the proportions of parts above given.

A 30 feet rail road bridge will require 180 cubic feet of timber, (including 64 cubic ft. of rail timbers,) and about 375 lbs. of iron.

Plan for a 40 Feet Bridge,

Applicable for any length between 36 and 48 feet, and not confined within those limits.—[See Fig. 34, Pl. 8.]

Scale for lengths of timbers, 1 to 100; other dimensions, 1 to 30.

In this truss, the upper and lower horizontal stringers *a* and *b*, are each composed of two pieces, 6 inches deep by 4 inches thick, and placed 6 inches apart.

The brace *c*, is 6×14, edgewise to view, with 6 inches cut out of the middle, at the upper end, to make room for *d* and *e*. At the lower end, it is fitted into a boxing of 1 inch depth upon the upper and inner sides of the stringers *a*, while the middle portion, 6 inches in thickness, runs down to the abutment, being beveled off even with the lower side of *a*.

The vertical, *d*, consists of 2 planks, $8 \times 2\frac{1}{2}$, placed 3 inches apart, to receive the upper end of *e*, and a 3 inch tenon formed on the lower end of *f*, between.

There is a boxing 1 inch deep at each end to let in the upper and lower stringer pieces, and the vertical pieces should extend at least 6 inches beyond the boxing. There is also a boxing in *d*, just above the stringer *a*, an inch deep, in the edge, to receive the cross bearer, and the cross bearer is boxed 4 or 5 inches deep, to receive the verticals *d*.

The diagonals *f.f*, $3\frac{1}{2} \times 7$ inches, are locked or tenoned together at the upper end, and fitted into a half inch boxing on the inside of the upper stringer *b*. In the center is a mortice 3×7 inches for the piece *e* to pass through. The lower end is reduced to 3 inches wide between the pieces *d*, and 4 inches at the end, having a head, or reversed shoulder fitting a triangular boxing inside of *d*, shown by dotted lines.

The diagonals *e.e*, are secured each by 2 bolts of $1\frac{1}{2}$ inches diameter, and a few spikes at the upper end; and at the lower end, they are halved and locked together, with a piece of 2 inch plank, $2\frac{1}{2}$ feet long. locked, bolted, and spiked onto each, and extending across the other to the end. These short pieces, being reduced to $1\frac{1}{2}$ inches thick, except for 6 inches at the upper end, which forms the locking, they just fill the 6 inch space between the stringer pieces. Two bolts of $1\frac{1}{4}$ inch iron, through stringer and diagonals, with a few spike, complete the arrangement at this point.

To secure the vertical position of the truss, a brace and tie runs up from a point in each cross beam, about $2\frac{1}{2}$ feet from the centre of the truss, to or near the top stringer, being secured at each end so as to act by either tension or thrust, with a force of some 2000 lbs. each. Perhaps the best arrangement for this purpose is, to use a 3 inch square, or 3×4 scantling for the brace, with a $\frac{5}{8}$ iron rod running beside it for the tension. These are not shewn in Fig. 34.

If the lower stringers can not be obtained of the whole length, they may be spliced by a two-lock splice between cross beams, with a piece of 2 inch plank about 6 feet long, which will extend 16 inches beyond the lap each way, well spiked on to the inside. Six inch pressed spike, ($\frac{1}{2}$ inch iron,) the points drawn about an inch for clenching, with a portion of five inch spike, may be used to advantage in this splice.

But the splicing at best, adds considerably to the expense, and it is better, when practicable, to obtain stringers of the full length, even at a considerable extra cost per foot.

The amount of materials for a 40 feet railroad bridge on this plan, is estimated at 306 cubic feet of timber, and 400 lbs. of iron.

Plan of a 60 Feet Rail Road Bridge,

APPLICABLE FROM 48 TO 72 FEET WITH ADVANTAGE.

Scale for Length of Pieces, 1 to 100 ; other Dimensions, 1 to 30.

LXIX. Figure 35, Pl. 9, shews a side view from one end past the centre.

The lower stringer *a*, is composed, as in the preceding plan, of two portions, each 9 inches deep by 5 wide, with a space of 11 inches between, from the end to the point *a*, about three feet from the second cross-beam from the end. Thence, a 2½ inch plank extending to within the same distance of the opposite end, is added to the inside of each portion, reducing the width of the space to six inches. These plank are well spiked at the ends, say with 3 spike, ⅝ square by 7 inches long, at each end of each plank, two being placed six inches apart, and 3 or 3½ from the end, and the other in the centre, and 7 or 8 inches from the end. It would be better, perhaps, that one or two of these, should be ⅝ or ¾ screw bolts.

The 5×9 part of the stringer may be spliced by the double lock splice, centrally opposite the second bearing point, or any where between the second and fourth. If spliced at the bearing point, the transverse bolts which secure the diagonals, will strengthen the splice at the same time. The splice, of course, should be farther assisted by spike and small bolts, and these should in general, be at least five times their diameter from the lockings, or from the ends of the pieces.

The top stringer is composed of two 5×8 portions, 9 inches apart, with a 2 inch plank on the inside of each, extending from the second to the fourth connecting point, or half the length of the stringer. The parts in the top stringer may be in two or more pieces, meeting by square ends, or lapped a few inches and halved, without locking.

The end braces are each composed of two pieces, 8 inches deep by 7 wide, and 5 inches apart, with a block at the lower end and the centre; the latter having a $\frac{5}{8}$ bolt passing through it, and thus connecting the two portions of the brace. The block at the lower end is secured by the main bolt connecting the brace with the ends of the stringer pieces.

The lower end of the brace is fitted into a $1\frac{1}{2}$ inch triangular boxing on the inside, and a 1 inch boxing on the upper side of the stringer pieces, as shewn by the dotted lines in the figure; the boxing on the upper side, only extending to within one inch of the outside of the stringer; the lower stringer being two inches wider from outside to outside, than the brace and the upper stringer.

The brace meets the end of the upper stringer by a bevel joint bisecting the angle formed by the two; the brace pieces being cut away upon the inside, where they interfere with the vertical.

The vertical, *v*, is composed of two pieces of 8×4, with a space of 4 inches, to admit the diagonals *b* and *c* between. Each part of the vertical has a $1\frac{1}{2}$ inch boxing at the upper end, to receive the inside of the upper stringer and main brace pieces, with two $1\frac{3}{4}$ inch bolts passing through the whole. There are also at this point, two $\frac{3}{4}$ inch iron pins, passing through the vertical pieces and the diagonal *b*, but not through the stringer or brace pieces. These pins should be, one directly under each of the bolts, but not cutting the same fibres in *b*, as are cut by the bolt holes. The vertical should extend about nine inches above the top of the stringer, and the diagonal *b*, 9 or 10 inches beyond the bolt holes. The vertical recieves the lower stringer pieces into a $\frac{1}{2}$ inch boxing, and also recieves between its two parts, a 4 inch tenon upon the lower end of the diagonal *c*, which is 4×9 inches, and stands edgewise to view. There is also an inch boxing on the edge of the vertical, just above the stringer, to receive the cross bearer; which latter is boxed 5 inches deep to receive the vertical, as in the case of the last plan. It is desirable that the centre of the beam should be brought nearer to the centre of the vertical, but a good plan for effecting it has not occurred to me. The arrangement here adopted can not, however, be very objectionable.

The vertical pieces extend 5 or 6 inches below the bottom of the stringer. Two $1\frac{1}{2}$ inch bolts placed diagonally about 7 inches from centre to centre, and passing through stringer, vertical and diagonal pieces, as shewn in the figure, complete the connection at this point.

The diagonal *b*, (being 10×4 inches,) passes through a mortice in the centre of *c*, and also through a mortice near the lower end of the diagonal *e*. The latter piece being 7×4 inches, has an oblique shoulder of $\frac{1}{2}$ an inch on each side, fitting into a triangular boxing of corresponding depth on the inside of the stringer. Two $1\frac{1}{2}$ inch bolts pass through the whole, and a $1\frac{1}{2}$ inch pin, 7 inches long, passes through *b* about midway of its width and close to the under side of *e*, with a $\frac{1}{4}$ inch plate, 3 inches long by 2 inches wide, for each end of the pin to bear on.

The cross bearer at this point is boxed on, so as to bear upon both the diagonals and the stringer pieces.

At the upper end of *c*, is a mortice 3×9 inches for the end of *d* to pass through; *d* being also 3×9. On each side of *c*, is a shoulder of 1 inch, oblique to the piece, but in a vertical line, acting against the ends of the inner plank of the top stringer; one inch of the ends of the plank being halved off, to let the piece *c* pass by with a [illegible] of 7 inches. A $1\frac{1}{2}$ inch bolt passes through the stringer, and through the centre of c and d. There are also two $1\frac{1}{4}$ inch iron pins, 7 inches long, passing through d on the upper side of c, $4\frac{1}{2}$ inches from centre to centre, with two $\frac{1}{4}$ inch plates 2×7 inches, for the ends of the pin to bear on. These bearing plates should have nail holes at the ends. The pieces in all cases, should extend beyond the bolt or pin holes, not less than five times the diameter of said holes.

The diagonal e, is halved and locked with its mate at the upper end, and fitting an X formed boxing 1 inch deep, inside of the stringer pieces, is secured by a $1\frac{1}{4}$ inch bolt through the whole.

The piece d is halved and locked with its mate at the lower end, with a piece of 2 inch plank, $2\frac{1}{2}$ feet long, locked, bolted and spiked on to each, as in the corresponding case in the preceding plan. Two $1\frac{1}{2}$ inch bolts are used at this point, each in the centre of the stringer.

The horizontal bracing is the same in this as in the preceding plan, the braces being a little larger towards the ends. 3×6 braces will be sufficient, or 3×6 at the ends and $2\frac{1}{2}\times6$ in the middle. They may be halved together at the crossing point, and spiked to the under side of the rail timbers, or one may be placed sufficiently below the other to pass without cutting either.

The bracing and tying to preserve the vertical position of the trusses, is the same as in the 40 ft. plan. In this, as well as that, the braces may be on the outside of the truss, the cross bearers extending some three feet from the centre of the truss, to support the brace and tie.—These braces are not shewn in the figure.

It is advisable in all these plans where the erect position of the truss depends on its connections with the cross beams, that the latter should be trussed, in all cases where they would not thereby occupy too much space vertically, as the spring of the beam communicates motion to the truss. This is, perhaps, less detrimental in a rail road, than a common bridge, since the steady motion of cars has not that tendency to produce vibration, which arises from the trotting of horses. Still, it is an object to secure stiffness in the cross beams in all cases.

Fig. 36, Pl. 7, shews a good form for a trussed cross bearer for a rail road bridge. All except the two short braces is 8×8 inch stuff. The two short braces 3×3 inch stuff. The long and short bolts, $\frac{3}{4}$ or $\frac{7}{8}$ inch iron, according to the distance of the cross bearers asunder. The truss should be not much less than 3 feet deep in the middle, and the line of the centre of the main braces, should meet that of the bottom timber, at the centre of the main trusses of the bridge. The boxings at the foot of the brace should be about $1\frac{1}{4}$ inches deep, for the extreme one, and about 3 inches for the other. The upper end of the main braces should be made square, and the necessary bevel all made upon the top piece. This truss, if 3 feet deep, is good for about 16 tons.

The 60 feet bridge estimates at 600 cubic feet of timber, and 1,000 lbs. of iron.

An 80 feet Bridge.

LXX. On the preceding plan, a truss of 70 feet in the clear, requires a height of 12 feet from centre to centre of the stringers. For trusses exceeding this height, it is very desirable that they should be connected at the top, as it is somewhat troublesome to support them independently in a vertical position. By adding one or two to the number of pannels, the length may be increased to near a hundred feet with a height of about 12 feet. This would answer a tolerable purpose for common roads, and even for rail roads, they are frequently built of no greater height. But we have seen that there is a loss of economy when the length of the truss exceeds about six times its height.

Rail road bridges require about 15 feet in the clear, above the top ef the rail, which takes about 18 feet height of truss, with the cross beams on the top of the lower stringers. By suspending the beams below the stringers, 16 feet trusses will give sufficient space. I propose the following plan, Fig. 37, Pl. 10, for an 80 feet bridge, which, by diminishing or increasing the length or number of pannels, or bearing points, may be suited to any span from 70 to 150, or perhaps 200 feet, with probably as good economy, to say the least, as any plan in use. The scale of this plan is the same as that of the preceding, 1 to 100 for length, and 1 to 30 for other dimensions.

This I call a *double-cancelled* truss; there being two crossings of the diagonals in the centre of each pannel, or between each two consecutive bearing points.

The lower stringer is formed mainly as in the preceding plan, being 21 inches in width horizontally, and 9 inches deep. From the end to within a foot of the first cross beam, the space between the two portions of the stringer is 13 inches. Thence, to about midway between the 2d and 3d beam, the space is 9 inches; and thence to the centre, 5 inches; and so for the other half. The outside courses are 4 inches thick for the whole length, spliced with the double-lock splice. Opposite the 9 inch space, a 2 inch plank is added, and opposite the 5 inch space, a 4 inch piece, locked by a single lock on to the 2 inch plank just mentioned, as seen in the figure.

The upper stringer is 19 inches from outside to outside, and 9 inches deep. Each half is composed of a 5 inch course on the outside throughout, with a 2 inch course on the inside, between the points over the second cross beam from each end.

The end braces *b*, are in two pieces of 9×6 inches, with a 7 inch space between; the lower end fitted into an inch boxing inside, and a 1½ inch boxing on the top of the stringer pieces; the end resting on the abutment. Between the two pieces at the foot, passes the diagonal *e*; being reduced at the end, to a width corresponding with the space that receives it. This piece also extends down to the abutment. At the top, *b* forms a bevel joint with the top stringer.

The vertical *d*, is 12 × 3 inches; passes through *e* in the middle, and *h* at the lower end, where *d* and *h* are both secured to the stringer by two 1½ inch bolts. The cross beam at this point may be let into the edge of *d*, 3 inches, and boxed, so as to bring the centre of the beam within 4 inches of the centre of *d*. At the upper end, *d* terminates in two pieces of 2 inch plank, 5 feet long, one on each side; locked ½ or ⅝ inch deep, bolted and spiked, so as to occupy a space of 6 inches from out to out, and cut away on the inside, so as to make room for *g*, which is 3 × 12 inches.

The oblique piece *f*, 9 × 4 inches, passes through *e*, *h*, and *k*; being secured at the lower end by two 1¼ inch bolts, and a 1¼ inch pin, 9 inches long, with bearing plates, 2 or 2½ × 3 × ½ inch, on the under side of *k*. At the upper end, *f* has 2 pieces of 3 × 12 inch plank about 7 feet long, locked on ¾ inch deep, and spreading so as to be 11 inches from out to out at the end; boxed 1 inch on each outside, to receive the top stringer and the brace *b*; and cut away inside, making a space of 6 inches for *d*. The piece *f* should extend 9 or 10 inches above the stringer. Two 1¾ inch bolts through the whole, and two 1¼ inch iron pins through all but the stringer and brace, will be sufficient for this point.

The diagonal *e*, 4 × 9 inches, at the upper end, has a shoulder of 1 inch on each side, cut vertically, and acting against the end of the inner plank of the top stringer. It has a 1½ inch bolt and 2 iron pins, 1¼ inch, and 7 inches long, through *i*, on the upper side of *e*, with bearing plates under the ends of the pins.

The diagonal *h*, (3½ × 9 inches,) and the top stringer, are each boxed 1 inch, to let in the width of the other; with two 1¼ inch bolts. *h* has 5 mortices, including those at the ends, as have also the other diagonals that stand edgewise to view in the plan.

The diagonal *k* (3×9 inches) is boxed in the same manner as *h*, at the upper end, and has one 1⅜ inch bolt.

The piece *n*, is 3½×7 inches except about 8 feet of its upper end, which is reduced to 3½ inches wide; and passes through its fellow *m*, and through *k*. *m* is also reduced to 3½ inches at the upper end, and at the lower, fits a boxing of 1 inch in the lower stringer, but has no shouldering; has two 1¼ inch bolts and a ½ inch spike through, and two 1 inch pins with bearing plates on the under side, through *g*.

The diagonals *i* (9×3,) and its antagonist (7×3,) at the lower end, halve and lock, with lap pieces locked, bolted and spiked, as in the case of *d.d*, in the preceding plan, making up a thickness of 6 inches, which, of course, requires a ½ inch boxing on the inside of the stringer pieces. This point has two 1¼ inch bolts. The diagonals generally, should have ½ inch spike at the crossings; or perhaps a round iron pin from ½ to ⅝ inch diameter would be cheaper and better.

The cross beams for this stretch, being 9 feet apart, should be 10×14 or 12×12 inches and put on as in preceding plans. The horizontal braces may be from 3×5 in the middle, to 3×7 at the ends of the span.

Across the top are light tie beams, say 5×7 inches, fitted on to the top stringer, similarly to the fitting of the bearing beams below; secured with small bolts or spike, with diagonal horizontal braces, 3×4, or 4×4 inches. At the ends should be lateral supporters, running down from near the upper end of the main end braces *b*, diverging from 1 in 6 to 1 in 4 outward, to the abutment, or a bed timber lying on the abutment; and properly secured at both ends. These braces should be about 6 inches square, with perhaps a steadying block and small bolt midway.

The truss in this plan is proportionally higher than is thought advisable for long spans, the height being to the length, as 1 to 4½. It is not certain whether this proportion for an 80 or 90 feet span is more or less economical than a less height, independently of affording room for the locomotive. But taking in that consideration, there can be no doubt of the propriety of giving to an 80 feet stretch, this height of truss.

Reducing the height of truss 1 foot, and the space between bearing points ½ a foot, will afford a stretch of about 75 feet, and still afford room for the engine, by a proper arrangement of track, &c.

It is not necessary that the height of truss be exactly once, twice, or thrice the space between bearing points or cross beams; even with the uniform rectangular crossings of the diagonals or cancels. The height may be once and a half, or twice and a half the length of said space. Hence, the skillful engineer will find little difficulty in arranging the proportionate height of truss, and preserving the distance between bearers within such limits as to secure economy in the use of rail timbers, &c.

The estimate for an 80 feet bridge is, 920 cubic feet of timber, and 1250 lbs. of iron.

LXXI. It is hoped that enough of detail and specification have already been given, to make the peculiarities of the plans I recommend, intelligible to those conversant with the subject, or to those desirous of acquiring a knowledge thereof. I will therefore, after some remarks upon the adaptation of these plans to sustaining the load on the top of the trusses, and to the use of common roads, dismiss the consideration of the subject for the present.

It will appear obvious enough, that all these trusses in the preceding plans, are equally well, and even better calculated to sustain the weight on the top, than at the bottom, as they have been regarded to do in what precedes. Some difference would take place, on making the transfer of weight, in the action of some of the parts. The amount of tension would be increased in some places, and thrust in others, and vice versa. This is necessary to be taken into account in apportioning the dimensions, and arranging the connections of the parts, and sufficient directions and illustrations have already been given, to enable this to be done, and the plan arranged with facility.

When the track passes over the top, the abutments or piers should be built with offsets or recesses to support the ends of the trusses, and a thin wall carried up to support the ends of the rail timbers; or, a bent of wood may be made to answer the purpose.

With this arrangement of the track, the distance between trusses need not exceed the tenth part of the length of span, by which means a considerable saving may be effected in the length and size of cross bearers.

Horizontal bracing, of course, will be requisite at the top, but not at the bottom of the trusses; as the constant tension on the lower stringers, will counteract sufficiently any tendency to swing, unless the sides be boarded up, so as to take the severe action of the wind; in which case some bracing may be necessary.

On the whole, it is manifest, that when practicable, there is a decided advantage in point of economy, in arranging the plan for the track to pass over the top.

Common Road Bridges.

LXXII. If we allow 16 ft. width of road-way, which will admit the passage of two carriages, and reckon 100 lbs. to the square foot for the maximum load of a common road bridge, the trusses for the latter would require only about three-fourths the strength that we have estimated for rail road bridges, aside from what supports the structure itself.

But the flooring of the common road bridge is heavier than the rail track, though the load of 100 lbs. to the square foot is more, probably, than one bridge in a thousand is ever exposed to. It is therefore probable, that the trusses for common road bridges do not, in general, require more than three-quarters the strength of a rail road bridge. Even much less strength, in many cases, would be entirely safe.

But it is well to lean always towards safety, and rather make a structure unnecessarily strong than too weak.— Enough, however, has perhaps been said on this head.

I would make no difference in the general plan of trusses for rail road or common bridges. Sometimes a less proportionate height will be admissable, if thought to possess any advantages in the way of appearance, or for any other consideration. I would seldom, however, make the height less than one-eighth the length of span.

A height from 12 to 14 ft. on a common road, will admit of ties across the top, and in such cases they should always be employed, with suitable horizontal bracing, and in all cases sufficient security should be provided to preserve the erect position of the trusses; for if their erectness be lost, their strength is very much impaired, and they are not only dangerous, but very unpleasant to the eye.

The flooring may be formed with longitudinal joists, upon the cross bearers, and cross planking upon the joists, or by making the bottom stringer a little larger, particularly deeper, diminishing the horizontal thickness of the cross bearers, and increasing their number, so as to leave spaces of only 2½ or 3 ft. between them, and laying the plank lengthwise or diagonally. Horizontal braces may be spiked on the lower side of the cross beams or joists, in the manner of what is sometimes called stay-lathing.

An advantage of planking lengthwise is, the more steady motion of carriages, producing less shaking of the structure; also, the less wearing of plank by toe corks. The disadvantage if any, is the liability of the wheels to wear at the joints of the plank. It would seem that this might be obviated by inclining the plank more or less out of the direct line of the wheel track. The longitudinal planking is the cheaper method, as one set of timbers, (the joists,) is dispensed with, without any considerable increase in the expense of the cross timbers. On the whole, I am at a loss for the reason of this plan of planking on bridges, not having been more extensively used.

Roofing and Siding.

LXXIII. As to the propriety of roofing and siding bridges, it may be remarked, that it no doubt adds considerably to the durability of the structure; but it also increases the weight, and consequently the requisite strength of the trusses; thereby enhancing the first cost to a considerable degree. They are much more liable to injury by the action of the wind, when roofed and sided, which is another somewhat important consideration.

If a bridge costing $1000 without covering, will last nine years, an additional investment of $1818 at 5 per cent., compound interest, will produce the means of renewal as often as necessary, so that $2818, will provide for a perpetual maintenance.

Again, supposing that the same bridge protected by roofing and siding, will last thirty years and cost $1500, including repairs in keeping on boards, &c., it will require an investment of only about $500 to provide for renewal, or $2000 for perpetual maintenance.

This shews decidedly in favor of roofing and siding, and I am inclined to think not too favorably, though the estimate of cost has been rather roughly made. For short stretches, the comparison would shew a different result up to a certain limit. The cost of roofing and siding, particularly the former, is about the same per foot run, for long or short stretches; while the cost of the supporting trusses, per foot, is nearly proportional to the length of stretches.

A 30 ft. bridge for instance, would cost about double, with roof and siding, that it would without; and allowing the former to cost $200 and the latter $100, the comparison would be about $280 for the perpetual maintenance of the uncovered bridge, against about $266 for the covered one. Hence, this must be near the dividing point; and it may be put down as highly probable, if not a decided point, that below 30 ft. there is no economy in covering bridges; but above that limit, or something near that, there is a decided advantage in "protection," increasing with the length of stretch.

Many bridges are annually swept away in this country by floods aud freshets. Where the permanency of the structure is doubtful, prudence would rather dictate that the structure be made as cheap as possible, consistently with strength and safety.

These remarks upon covering, have more particular reference to common bridges. As to rail road bridges, there is more liability to accident by fire in covered, than uncovered bridges, and the policy may be somewhat more doubtful, unless for considerable stretches.

In the construction of bridges not designed to be covered, it is believed to be worthy of recommendation to saturate the joints, and points of contact of the different pieces of timber, with oil-paint, pitch, or some other substance that may serve to harden the timber, as well as preserve it by excluding water. Those parts are liable to decay much sooner than in other places, as well as being usually the weakest parts, where the timber is cut most to form the connections.

Length of Stretch.

There are cases where the length of the stretches for a bridge is optional, and may be regulated as economy in construction may dictate. For instance, in crossing a broad stream or valley, where piers are not objectionable, as obstructing the current, &c. In such cases it is desirable to know what length of stretch is most favorable to economy.

It may be assumed in the first place, that the cost of a pier is *nearly* the same to support a short, as a long span. Hence the cost of piers will be as their number; or inversely as the length of stretches. Therefore the minimum cost for both piers and superstructure, (for a bridge of indefinite length,) will occur when the cost of a pier is just equal to that of one stretch of the superstructure,

The Dee Bridge.

A considerable degree of excitement has been caused within a few months in England, by the failure of a bridge over the river Dee, near Chester, (England,) during the passage of a rail road train, by which accident several lives were lost.

As the facts of the case may afford an instructive lesson upon the subject treated of in the preceding pages, I am inclined to state the most important of them, with a few accompanying remarks, before taking leave of the subject.

The bridge in question, was upon the plan called the "Cast Iron Girder Bridge."

In this plan, each track, or pair of ways, is supported by two cast iron Girders, or beams, with their ends resting on abutments and piers, which, in the case of the Dee Bridge, are 98 feet apart. The form of the Girders, is essentially that of the common railway bars, known as the I or I rail; the vertical "web" being two and one-eighth inches thick, by about three and one-half feet deep; the lower flange, twenty-four and one-half inches wide by two and a half thick, and the top flange, about seven and a half inches wide, by a little less than two inches thick; the whole cross section containiug 160 square inches, and the whole depth being 45 inches.

In addition to this, are wrought iron suspension bars, containing 60 square inches of cross section, running obliquely downward from a connection at the ends, about 3 ft. above the upper flange, (there being a rising portion at the ends to which such connection is made,) to the lower flange, at the joints of the cast iron part, 36 ft., or one third the length of the girder, from the ends; the casting being in three pieces, connected by bolting through flanges, and by strengthening pieces at the points. Between the two joints, the suspension bars run horizontally, just above the lower flange of the girder.

One effect of these suspension bars is, to throw a great amount of thrust, or crushing force upon the top flange, in so much that the engineers who examined and testified upon the subject, upon the inquest held in relation to the lives lost by the accident, *generally* concurred in the opinion that they rather weakened than strengthened the girder. Adopting that conclusion upon this point, which appears to be corroborated by experiment, let us examine whether the failure of the bridge, in connection with the other facts of the case, should inspire distrust, or confidence in the use of iron bridges for rail roads.

By calculation according to formulæ deduced from experiment, a pair of girders is estimated to be able to sustain at the extreme or breaking point, 148 tons in the centre, or 296 tons distributed uniformly over their length. In this estimate, the testifying engineers concur with little or no variation. Now, the weight of a pair of girders, is about 70 tons, or, with other parts of the structure, about 90 tons. Mr. H. Robertson, (engineer,) estimates the effects of the engine and tender as "equivalent to 32 tons in the centre," which is equal to 64 tons distributed. Add to these, "25 tons of ballast," which was put on to the bridge just before the accident, and we see there was $90+64+25=179$ tons upon the bridge, which was more than six-tenths of the actual breaking weight, supposing it all at rest, and acting equally on the two girders. But the action of the engine, and the vibration of the girders, (which must be considerable for a length between supports equal to 26 times the depth,) must be very important, though not capable of exact estimation. It is therefore not improbable that the girder was taxed to more than two-thirds of its extreme capacity under favorable circumstances.

Now, from one-quarter to one-third of the actual breaking strain, is all that any practical engineer estimates it safe to rely on cast iron to bear; whereas, in case of the Dee bridge, we see it exposed to a load of more than six-tenths of the ordinary capacity, besides the effects of motion, vibration, &c. It certainly cannot be surprising, then, that a failure took place. And so far from this failure amounting to evidence, or even an argument, against the safety of iron bridges, the fact that this is the first and only failure that has yet taken place, among "upwards of 100 similar bridges either in use or in the course of construction, in England," some of which are doubtlessly, often exposed to strains exceeding half their absolute capacity to bear, seems calculated to inspire the fullest confidence, that, properly proportioned, so as never to be exposed to a stress of more than one-fifth or one-quarter of the average strength of the metal, iron bridges may be relied on with the utmost confidence.

We see that the Dee bridge contained some 70 tons or more of iron in the girders, for a single track of 98 feet stretch,* besides 20 tons in the other work, which is nearly one-third of the actual breaking weight of the girders; and as much or more than they ever should or *could* with safety, be exposed to.

The English engineers seem to me to have erred, in deducting the weight of the structure from the *breaking weight*, instead of the *safe load* of the girders, to obtain the effective strength.

Now, that the most eminent engineers, of a most scientific nation, should be obliged to avail themselves of the extensive employment of such a plan of structure as that, is evidence enough of a prevalent want of more light upon the subject of *Bridges*. And whether my labors in the field will have aided essentially towards supplying the deficiency, it is unnecessary for me to give a more direct expression of opinion, than will be afforded by respectfully offering these humble essays to the attention of the engineering profession.

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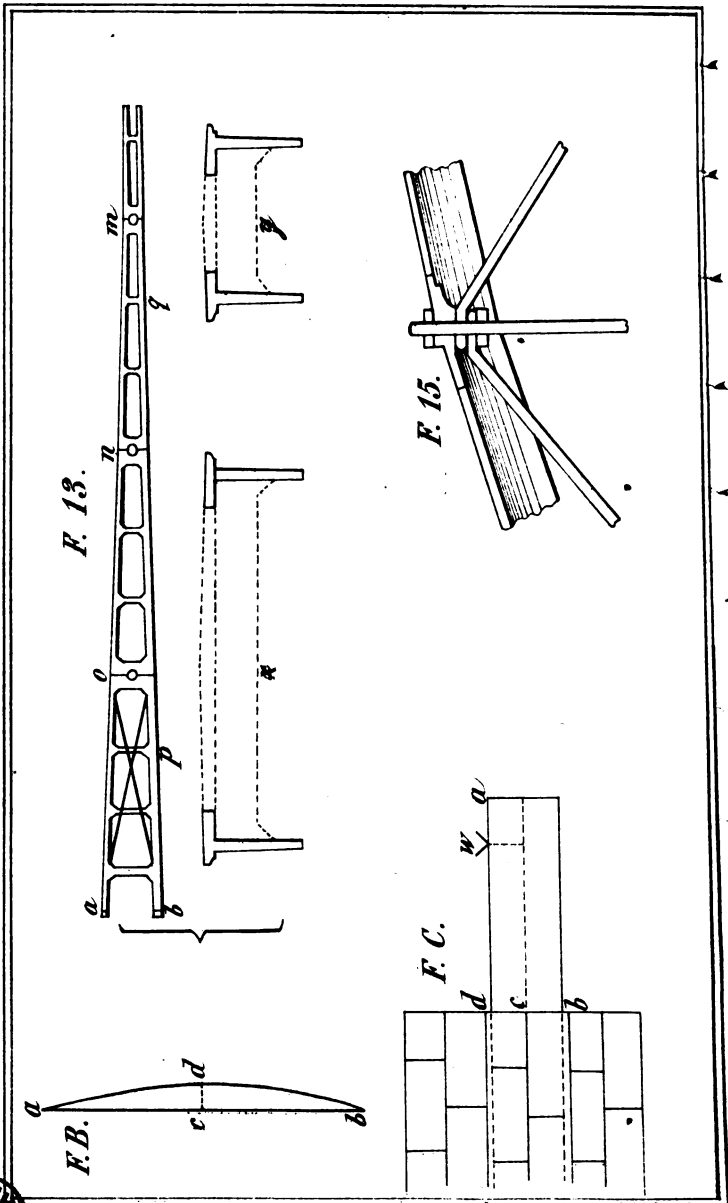
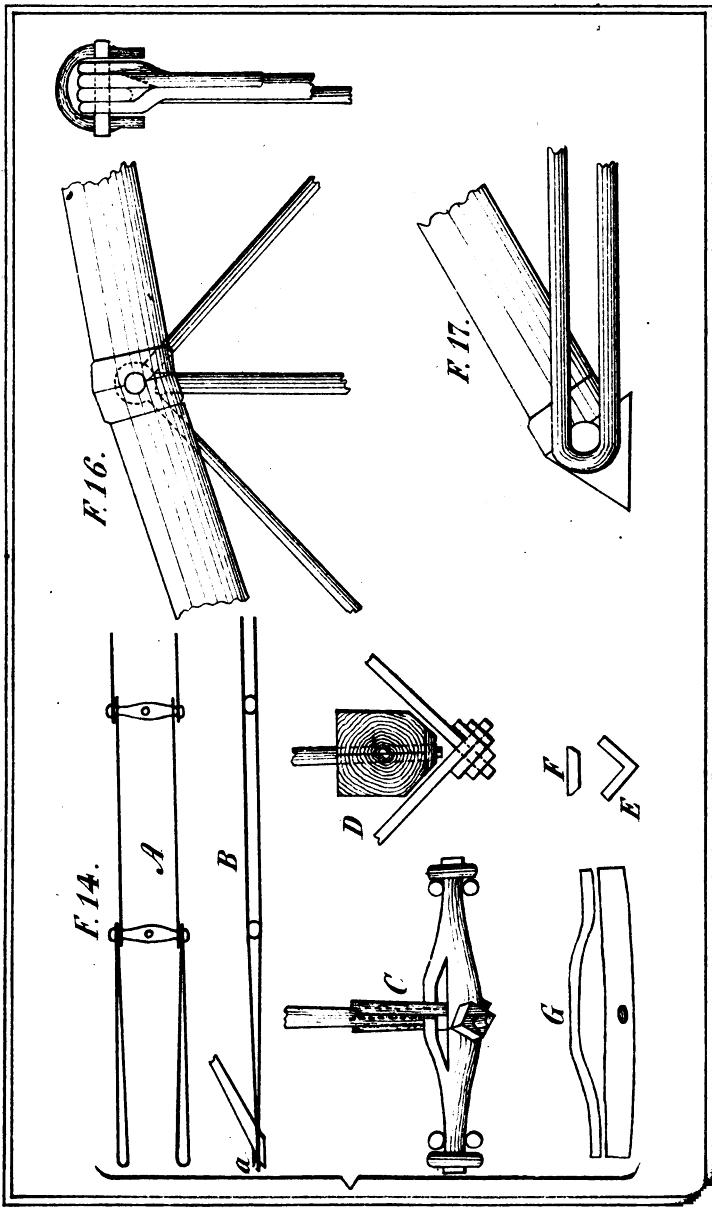
* I estimate the amount of iron for a like stretch upon my plans, at about 15 tons.


F. 13.
m
n
o
p
q
a
b
F. 15.
F.B.
a
b
c
d
F.C.
a
b
c
d
w

Pl. II.



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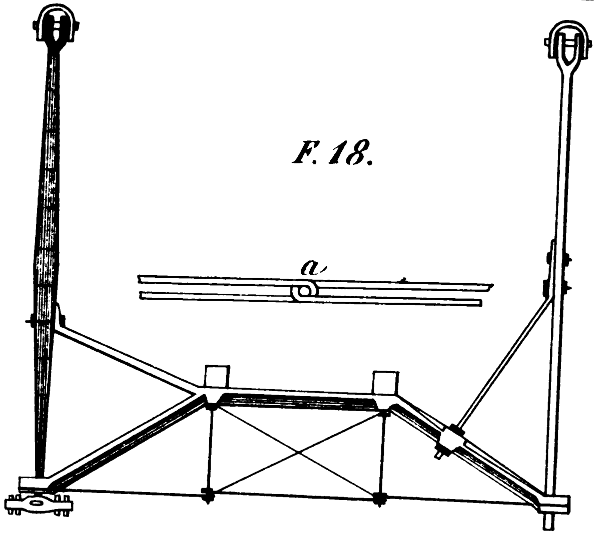

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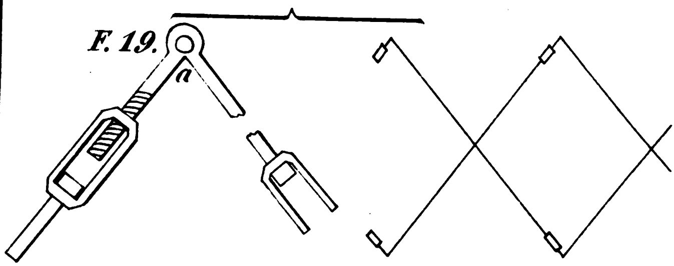

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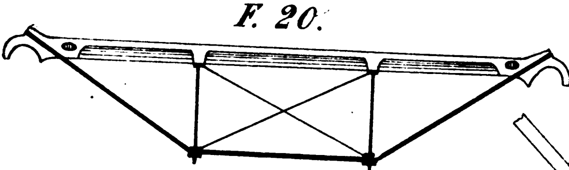

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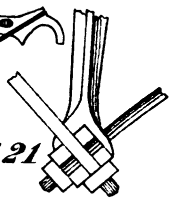

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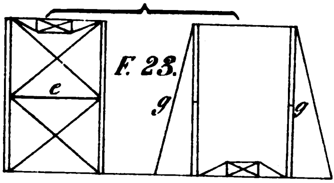

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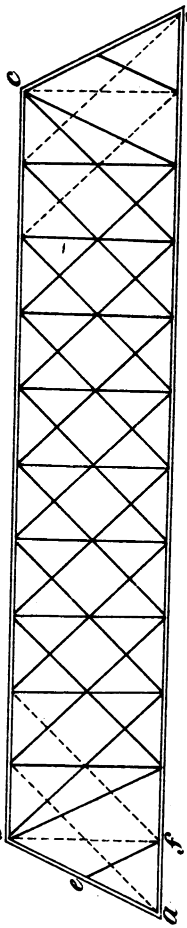

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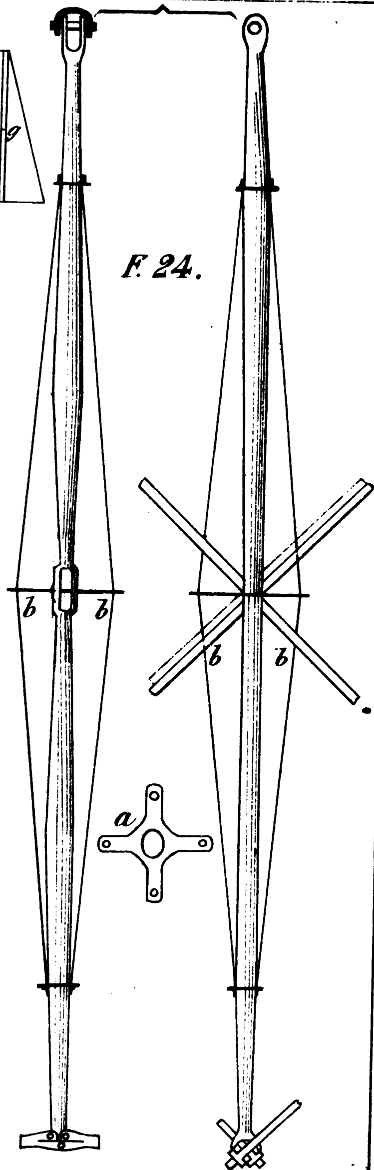

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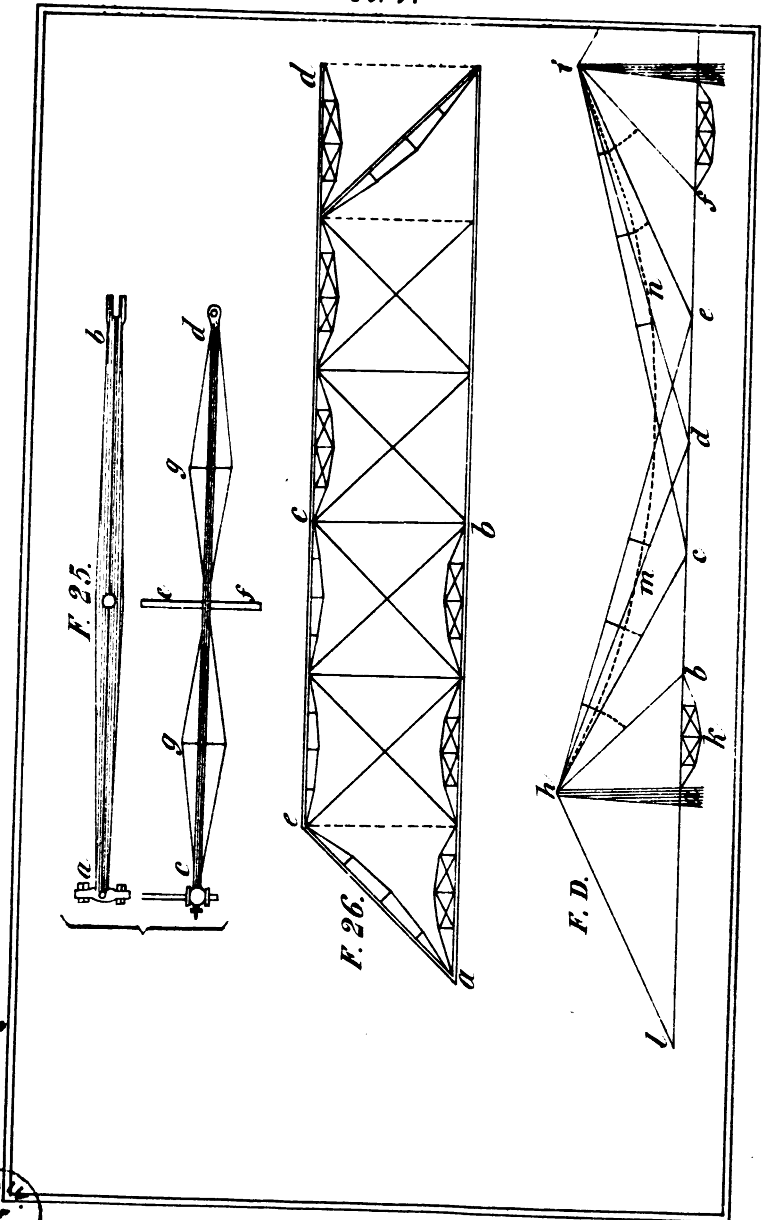
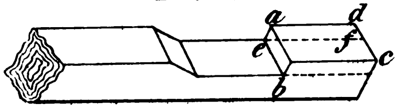

F. 25.
a
b
c
d
g
g
e
f
F. 26.
a
b
c
c
d
F. D.
h
i
k
l
m
n
a
b
c
d
e
f

Pl. VI.

F. 27.



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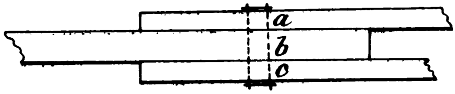

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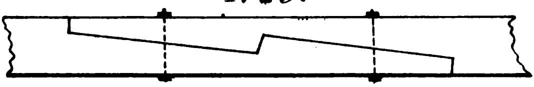

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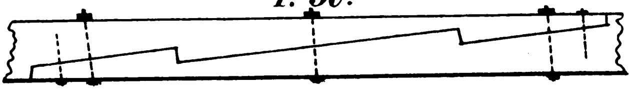

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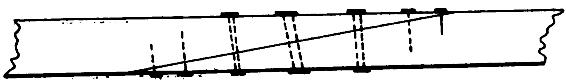

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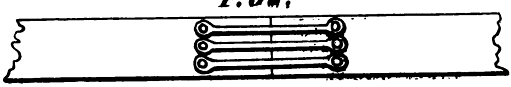

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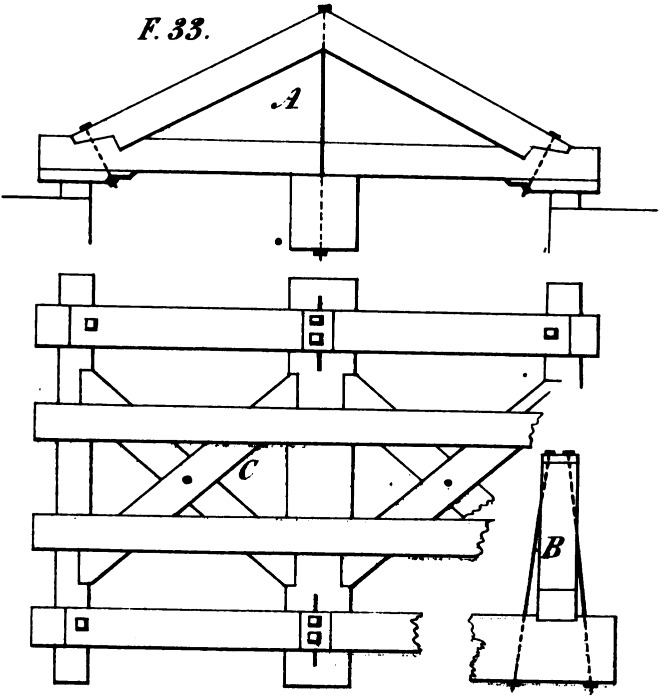

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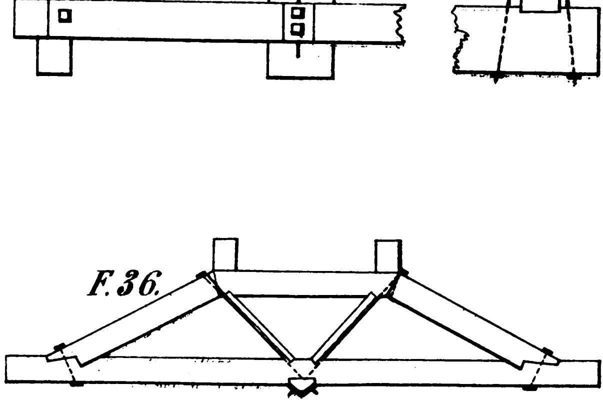

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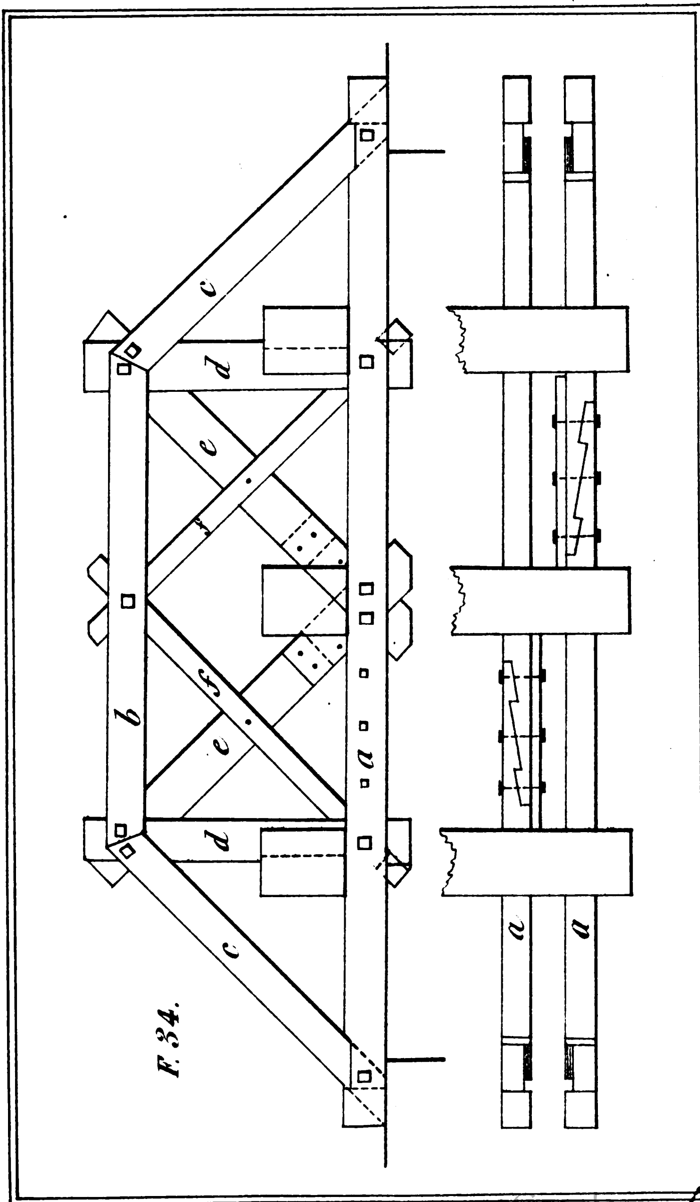
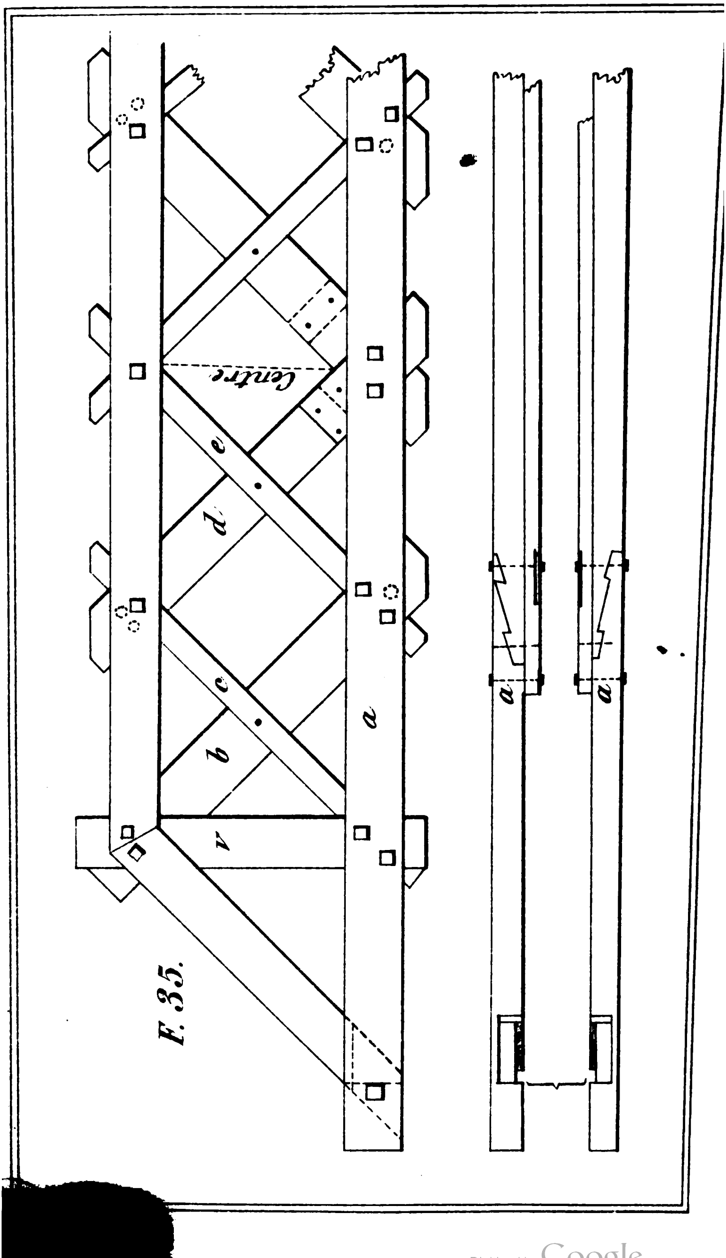


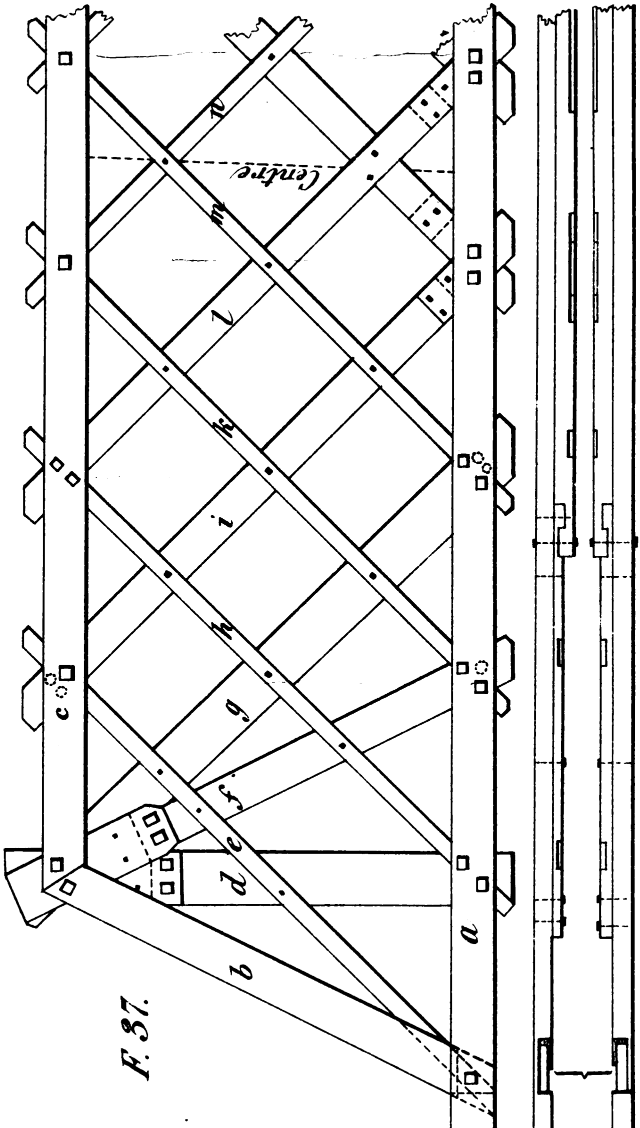
F. 34.

Pl. IX.



F. 35.

Pl. X.



F. 37.