A

COMPLETE TREATISE

ON

CAST AND WROUGHT IRON

BRIDGE CONSTRUCTION,

INCLUDING

IRON FOUNDATIONS.

LONDON: E. AND F. N. SPON, 16, BUCKLERSBURY.

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NEW YORK ......D. Appleton and Co.; Bailliere Brothers,
John Wiley.
PARIS................Fowler
ST. PETERSBURG : R. Watkins.

W. HUMBER, DIR. London, E & F. N. Spon. 16, Bucklersbury. PRINTED BY CUNDALL & DOWNES.

ROYAL ALBERT BRIDGE.

OVER THE RIVER TAMAR,

SALTASH.

I. K. BRUNEL, ESQ^RE F.R.S. ENGINEER.

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CAST AND WROUGHT IRON

BRIDGE CONSTRUCTION,

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IRON FOUNDATIONS.

IN THREE PARTS,

THEORETICAL, PRACTICAL, AND DESCRIPTIVE.

ILLUSTRATED BY NUMEROUS EXAMPLES

BY

WILLIAM HUMBER,

VOL. I.—TEXT.

LONDON:
E. & F. N. SPON, 16, BUCKLERSBURY.
1861.

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ILLUSTRATED BY NUMEROUS EXAMPLES,

DRAWN TO A LARGE SCALE.

BY

WILLIAM HUMBER,

ASSOC. INST. C.E., MEM. INST. MECH. ENG.

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1861.

LONDON:
PRINTED BY GEORGE WATSON, 5, KIRBY STREET,
HATTON GARDEN.

TO

THOMAS BRASSEY, ESQ.

A GENTLEMAN WHOSE NAME HAS LONG BEEN HONOURABLY CONNECTED WITH RAILWAY CONSTRUCTION

AND ENGINEERING PROGRESS;

AS A RECOGNITION OF THE MANY ACTS OF KINDNESS, RECEIVED AT HIS HANDS,

DURING YEARS SPENT IN HIS SERVICE.

THIS WORK IS RESPECTFULLY DEDICATED,

BY HIS FAITHFUL SERVANT,

THE AUTHOR.

LIST OF BRIDGES ILLUSTRATED.

Name of Bridge.	Description of Bridge.	Name of Engineer.
Rochester	Cast Iron Arch, Common Road over the Medway	Sir W. Cubitt, F.R.S.
Rochester	Plate Girder, Swing Bridge do. do.	Do.
Standish	Cast Iron Arch, Midland Railway	W. H. Barlow, Esq., F.R.S.
New Westminster	Cast and Wrought Iron Arch do. do.	T. Page, Esq.
Victoria	Wrought Iron Arch, Pimlico Railway, over the Thames	J. Fowler, Esq., F.R.S.
Staines	Plate Girder, Staines and Woking Railway do.	J. Gardner, Esq.
Trent Lane..........	Plate Girder, Nottingham and Ambergate Railway..........	J. Underwood, Esq.
Saltwater River......	Box Girder, Australian Railway	— Darbyshire, Esq.
Taptee	Warren Girder, Bombay, Baroda, and Central India Railway..	Leut.-Col. Kennedy.
Ebro	Trellis Girder, Pamplona and Saragossa Railway, Spain	Sig. Don A. Retortillo.
Jumna	Trellis Girder, Road and Railway, East India Railway	J. M. Rendel, Esq.
Morecambe Bay......	Trellis Girder, Kent and Leven Drawbridges, Ulverstone Railway	J. Brunlees, Esq.
Beelah and Deepdale..	Trellis Girder, South Durham Railway	T. Bouch, Esq.
Londonderry	Trellis Girder, Swing, Road and Railway..................	J. Hawkshaw, Esq., F.R.S.
Charing Cross........	Trellis Girder, Charing Cross Railway	Do.
Lerida..............	Lattice Girder, Barcelona and Saragossa Railway, Spain	Sig. Don Pingdollers.
Alcanadre	Do. do. do.	Do. do.
Murillo	Lattice Girder, Common Road, Spain	—
Carlos Gomes	Lattice Girder, Common Road, Rio de Janeiro..............	A. M. de Oliviera Bulhoes.
Windsor............	Bowstring, Great Western Railway	I. K. Brunel, Esq., F.R.S.
Shannon............	Bowstring Swing Bridge, Midland Great Western Railway	G. W. Hemans, Esq.
Saltash	Bowstring and Suspension, Cornwall Railway	I. K. Brunel, Esq., F.R.S.

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PREFACE.

In bringing the present work before the engineering profession and the scientific public generally, it is requisite to give some account of the motives which have induced me to prepare it.

Some years since, previous to the publication of my "Practical Treatise on Cast and Wrought Iron Bridges and Girders," I was impressed with the want of a treatise which should supply in a sufficiently condensed form such information as is constantly desired by Civil Engineers, Manufacturers, and Engineering Students; I, therefore, conceived the idea of preparing a work which should, as far as possible, supply this information, and I was not discouraged in my project, but on the contrary, I have been assured from time to time, that such a work could not fail to be appreciated, if its preparation were properly conducted.

It was then necessary for me to consider what steps should be taken in order to ensure the practical utility of my work, when I at once perceived, that a careful perusal of all the important works already written was absolutely indispensable; and besides this, information of quite a practical character had to be collected from all quarters; then a comparison of the theoretical and practical matter thus collected must be established, and the whole duly digested, rearranged, extended, and put into such a form as might render it ready for reference. This, then, is a very brief outline of what lay before me, and the fact of my being dependent in a great measure upon others for some practical and descriptive details, would sufficiently enhance the difficulties to be surmounted, and some consideration showed me that the preparation of such a treatise would be a work of years, however diligent I might be.

It appeared to me, however, that whilst I was engaged on my principal labour, I might furnish a forerunner which might, in the meantime, be found of great practical utility, and also show me, whether I should be justified in the publication of a more complete work; wherefore, I then issued my "Treatise," above mentioned, the reception of which has shown that I was not mistaken in supposing that such a work was much needed.

I have thus fully explained the nature of the above work, in order to show my readers, that the treatise now before them, is not a continuation of the subjects there considered; nor has it, in fact, any connection with that work, as no matter which I then published will reappear in the following pages.

I will now speak of the present treatise, and offer a few remarks upon the means which I have used in its preparation.

To render this work useful to practical men, formulæ of the most simple description must necessarily be inserted, and they must, moreover, be so placed, as to be ready whenever they may be required; but it must first be decided, what formulæ are really general, and this is a point which does not, strictly speaking, admit so readily as might be expected, of determination; and it leads us to examine the value of what are termed general formulæ, which examination results in the following conclusions:—

A general formula applies to a certain element of a structure, so long as that element varies in size only, but if it varies in proportion, the formula will no longer strictly obtain.

A practical general formula will give results approaching the truth, for some variations of form as well as of size.

A formula, to be valuable, must be accurately deduced from principles in themselves absolutely true; all *allowances* being made at one operation, which may be regarded as the conversion of a purely theoretical formula into a practical theoretical formula.

Any one using a practical formula should be capable of varying it in some degree, in order that it may be the more applicable to varying conditions; and he should also test its accuracy in the first instance for his own satisfaction, and any work treating of such formulæ should provide the reader (so far as is reasonable) with the processes by means of which the formulæ are obtained.

These conclusions tend to show that formulæ, however practically they may be stated, and however applicable they may be to one series of cases, are totally useless in an analogous series of cases, unless the person employing them is fully acquainted with the *fundamental principles* from which they are deduced, and upon which they depend; for it is *this* knowledge that constitutes the capability to judge of the applicability of a formula in a modified form to some particular case, for the solution of which it was not originally designed.

I, therefore, deemed it essential to the completeness of my work, to enter fully into the mathematical investigations, the object of which is the determination of the principles involved in the construction of various bridges; but, at the same time, it was requisite to keep these investigations totally distinct, in order that they might in no way interfere with the practical matter, the comprehension of which they are intended to facilitate; hence I have embodied these principles in the "Theoretical" part, which I have placed first, because it appears evident that principles should first be studied, and subsequently their application. The above reasons have induced me to insert this part, some introduction to which may not be amiss in this place.

My object has been, in the first part, to treat every subject separately, and by those means which are most capable of subsequent practical application, embracing every case that can occur, and to condense my investigations into the smallest space consistent with a complete explanation and elucidation of principles; but in some instances, which we will shortly indicate, practical cases occur which cannot be with accuracy theoretically treated, and in these cases we must have recourse to experience alone, being sometimes guided by formulæ, which we are certain will give us a great excess of strength.

In order to comprehend the manner in which a body will resist any kind of stress, it is absolutely necessary that we should have some notion of the constitution of the body which resists such stress, and this is the first step in our physical researches, but it is also the most difficult, if we must recognise every circumstance under which such a body exists.

The constitution of hard bodies is considered in the first Chapter, upon Cohesion and Elasticity; but I will here define the limits of my investigation. I have regarded the bodies of which we treat as constituted of a number of ultimate atoms, but I have regarded all the atoms as similar; I will, therefore, say, that in the application of my remarks to complex bodies, each molecule must be regarded as a compound molecule formed of different atoms.

This there will be no difficulty about, especially as I pay but little attention to the form of the ultimate molecules, basing my examination upon the supposition that each molecule, whether it be simple or complex, has a sphere of attraction, and a sphere of repulsion.

I have regarded each ultimate atom as at rest, that is to say, I have stated nothing to the contrary; but although that is a point which matters not, as far as my present purpose is concerned, I by no means wish my readers to conclude that I regard them as at rest, for the contrary condition appears far more plausible, when we come to examine bodies with regard to their behaviour when subjected to the action of heat; and also with regard to the affinities of the different forms of matter for each other.

If the ultimate molecules be in motion, let their motion be bounded by the spheres by which we have represented the atoms themselves, everything else remaining as I have put it.

Having arrived at a conclusion with regard to the effects producible on matter by a direct strain, the remainder of my theoretical part is sufficiently simple, for it consists merely of an explanation, or exemplification, of the methods to be used for the reduction of various kinds of strain, to direct strain; but it, nevertheless, requires some comment.

All the Chapters, from the second to the ninth inclusive, are of such a character, that I could not altogether dispense with some of even the more complicated processes of mathematical analysis; but I have, as far as possible, excluded all but the most simple of these processes, so that the calculations contained in those Chapters, may be fully comprehended by those who have but a moderate knowledge of mathematical science.

I may further state, that in these Chapters I have been careful to examine a great number of cases, and to place the resulting formulæ by themselves, so that they may be readily serviceable to those who may not have leisure to enter fully into every detail of the calculations.

I have observed above, that cases occur in practice which cannot be conveniently investigated, and I may here give as an example of such cases, the circumstance of a continuous girder, with one of its spans only loaded over a short part of its length; fortunately, however, the point is not of material importance.

It will doubtless be remarked, that in treating the case of a straight girder generally, I do not commence at the load itself, and immediately show its results upon that part of the structure with which it is in contact, although as all strains must pass through such parts to the piers, it would seem most natural so to do; but on the contrary, I calculate the effects of the loads placed upon the structure, from the reaction of, or effect produced upon the points of support. To treat the subject in a different way, may or may not be possible; but I did not attempt it, feeling that the introduction of methods of reasoning totally different to those which have already been received, would be very undesirable in a work pretending to a practical character, and intended for reference; but I, notwithstanding, thought it equally undesirable, to pass unnoticed so important a point, wherefore I have introduced into the sixth Chapter, which is devoted to the consideration of Shearing Strains, an investigation of the immediate relations existing between the downward pressure of the load on a beam, and the direct strain on the horizontal fibres of such beam; but I have there limited myself to the determination of the mode in which vertical force produces horizontal strain, without at all inquiring into the numerical relations of the two forces.

In the second or "Practical" part of my treatise, which although containing fewer Chapters, is of much greater extent than the foregoing, I have attempted to explain in a clear and concise manner, every important process in the Construction of a Bridge.

In the eleventh Chapter will be found the useful results of the whole of the Theoretical part; the application of formulæ to each kind of structure being exemplified; nor have I stopped here, but moving farther, I have carefully considered the manner of designing those numerous details, in which theory is of but little use, nor is the relative economy of the various forms now employed for bridges neglected, for I regarded all this matter as essential, and in fact, this part may be considered as the most important section of the work, for the subjects contained in it are indispensable to the practical man.

I have allotted a distinct Chapter to the subject of Joints, in order to afford an opportunity of showing the great importance of these details, and I may here observe the undeniable fact, that the total success of any structure, is dependent on the correctness of principle adhered to in designing the joints.

It may be desirable to remark, that in our Chapter upon Foundations, I consider only such foundations as are entirely, or in great part constructed in iron; omitting such foundations as are of masonry, although they be applied to iron bridges.

Having in the two first parts, explained the principles of Bridge Construction, the method of designing bridges in accordance with such principles, and the means by which the bridges when designed are to be actually constructed: examples are required to show the results practically obtained, and also to illustrate the forms which experience only will enable us to judge of, such as bracing, etc., which cannot readily be calculated, and of these a collection is illustrated, and explained, in the third or "Descriptive" part of the treatise.

These examples have been selected, so as to show, as far as possible, the means adopted in practice (whether good or bad, I do not pretend to decide), to surmount obstacles, both large and small; many, however, of the structures illustrated, have withstood tests of the most severe description; but as regards others, and especially those of very recent construction, a decided opinion would certainly be premature, although I may be inclined in one way or the other.

Our readers must not, therefore, consider, that I suppose all the structures illustrated to exhibit *perfection*, because I merely intend them as indicative of *practice.*

I have been careful not to insert any illustrations, which have to my knowledge been previously published in the English language.

No one can regret more than myself, that not one of the numerous structures erected by our late lamented chief, Mr. Robert Stephenson, is illustrated in either this, or my previous work; but the omission has not resulted from any neglect on my part, as I have made repeated applications for information with regard to some of those structures, but without success.

I was also desirous of giving more complete information on many works, than I have done; but in this I have been disappointed, as, in many cases, information which would have been acceptable, has been withheld.

I may here mention the foundations of Saltash Bridge, as an example of those descriptions which are incomplete; but on this subject I have not been furnished with particulars, for the reason that a paper upon those foundations, is proposed to be read at the Institution of Civil Engineers.

The Tables and Diagrams, on Strength of Materials, have been calculated and compiled from the most authentic and recent experiments; and those which are not of an experimental character, have been computed with the greatest care, in order to ensure trustworthy results.

To Sir William Cubitt, Messrs. J. Hawkshaw, J. Fowler, G. W. Hemans, W. Fairbairn, P. W. Barlow, T. Bouch, J. Brunlees, Lt.-Col. Kennedy, T. Page, R. P. Brereton, J. Gardner, J. Underwood, C. de Bergue, J. Cochrane, J. H. Porter, E. T. Bellhouse, and Messrs. Kennard, Bros., I beg to tender my thanks, for their kindness in allowing me free access to the drawings, specifications, etc., of the bridges illustrated; as also to Messrs. H. Wakefield, W. Wilson, G. Harrison, M. E. Wesley, and E. Gilkes, for their valuable information.

To my assistant Mr. Francis Campin, I am indebted for his able assistance throughout the work; as also to Mr. J. B. Walton, and my other assistants, in getting up the various drawings; to all of whom I beg to tender my sincere thanks.

W. H.

LONDON,
July, 1861.

PART I.

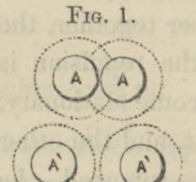
THEORETICAL.

CHAPTER I.

ON COHESION AND ELASTICITY.

The first subject to which our attention is directed, is the nature of the force by which any solid body is enabled to retain its form. This force is called the attraction of cohesion, or cohesive force, and exists between the ultimate or indivisible atoms of which any substance is composed, and is, in properties, very similar to the attraction of gravitation. The force of cohesion only exercises itself within a certain sphere, and if the constituent particles be removed without this sphere, the body is divided, and the cohesive force can only be brought into operation again, in such substances as will allow the divided parts to be brought into extremely close contact. In liquids, the sphere within which the cohesive force continues to act is very much larger than in solids, but possessed of an inferior amount of tension, hence it appears that if we can reduce pieces of a solid body temporarily to the liquid state, it will become much easier to reunite them, but they will still when again solidified, have the same amount of strength or cohesive force as at first. We may illustrate the overcoming and re-establishment of the cohesive force in the following manner:—Let a piece of bar-iron be stretched until the particles of some part of the bar are removed asunder to such a distance, as to be beyond the sphere of their mutual attraction; or in other words, let the bar be broken, it will be impossible to reunite it while cold, but if raised to a white heat, the sphere of action of cohesion will be so enlarged that the bar may by great pressure or repeated blows be reunited, and if the pieces be fused, the weight of the liquid metal will be sufficient to restore the action of the cohesive force. In all substances there is a certain play allowed to the atoms, which extends from actual contact to the greatest distance at which the cohesion remains uninjured, and this is called the elasticity of the substance, it is illustrated in the accompanying sketch. Let A A, Fig. 1, represent two ultimate atoms of any substance, which may be supposed to be spherical, and let them be in actual contact, the dotted circles representing the limits of the spheres of attraction, it is evident that in this case the particles cannot be brought into closer proximity to each other, and the body so constituted will be incapable of compression, but will allow of a certain amount of separation of the particles limited in extent by the position A′ A′.

Fig. 1.



Again, let us suppose a body composed of particles A′ A′, placed as shown, and exerting a mutual repulsion limited to the sphere shown by the dotted circles, they may be forced together, but when the pressure forcing them towards each other is removed, they will repel each other to their original position, and there, all action will cease, the attracting force being supposed to be nothing. Actual experiment shows us, that bodies may be compressed to a certain extent, or extended, and the body will yet recover its original form and size, when the force extending or compressing it ceases to act, hence we may conclude that the particles are retained in their normal positions by a repulsion opposed by an attraction, and this condition may be represented as at A″ A″, in which the black circles represent the ultimate atoms beyond which the spheres of repulsion extend to the first dotted circles; without, is the sphere of attraction, extending to the largest dotted circles, and the proportion between the resistances to compression and extension will appear to depend upon the following formula:

let r = radius of atom,
$r_{\prime}$ = do. sphere of repulsion,
r'' = do. do. attraction,
T = resistance to tension,
C = do. to compression,

then;

$$r' - r : r'' - r' :: C : T$$
$$\therefore \frac{r'' - r'}{r' - r} = \frac{T}{C}.$$

There is also another movement allowed among the particles of some substances, distinct from that caused by the elasticity, the nature of which is shown at Fig. 2. Let A represent the normal arrangement of the atoms forming some solid body, and let pressure be applied until the two atoms, a, b, are brought to the position shown in B, they will not then return to their former position, but there will be a permanent alteration of form and size in the body of which they are composed. Solid substances possessed of this property of allowing the atoms to slide over each other, are termed malleable. Where this property exists to a great extent, the matter formed by the particles is liquid, and in many cases gaseous. The above property may be imparted or increased by heat, from which it appears to depend upon temperature alone, as bodies possessing it naturally may be deprived of it by a reduction of temperature. Some bodies do not become malleable, but pass from the solid to the liquid state direct, owing, probably, to their crystalline character. Having thus briefly laid down a theory of the constitution of matter generally, we will proceed to consider the effects of this constitution. The mechanical forces to which the materials used in construction are subject, may be reduced to tension, compression, and shearing, all others being derived from these. These forces are all of them external, and are opposed to the internal forces of attraction and repulsion, which internal forces accommodate themselves to external strain by means of the elasticity of the material. Let A A, Fig. 3, represent two atoms of any body in a state of equilibrium; in order to obtain this we find, from the principles of statics, that there must be two forces in operation, of equal intensities and acting in opposite directions. One of these forces is the action, and the other the reaction, and, in the present case, these forces are the mutual attraction and repulsion of the atoms which, when the body is unacted upon by any external force, exactly balance each other. Let us now suppose two equal external forces acting in the direction of the arrows, the atoms will be brought closer together, the intensity of their mutual repulsion increasing as they approach each other; and when the repulsion is equal to one of the external forces, plus the mutual attraction of the atoms, they will become stationary, as the action and reaction will then be equal. One of the external forces may be a weight, and the other the resistance of some solid body supporting the atoms A A. When the weight is removed, the equilibrium of forces will again be disturbed, the repulsion of the atoms being greater than their attraction; they will therefore separate until the equilibrium is again restored. If care be taken not to exceed a certain limit, the above process will occur experimentally exactly as it is described; but if this limit, termed the limit of elasticity, be exceeded, the form of the body operated upon will undergo a permanent alteration. If we suppose a number of atoms to be arranged as shown in Fig. 4, and a pressure applied to the first, in the direction of the arrow, it will evidently press the first atom towards the second, until the excess of repulsion between the atoms is equal to the pressure; but under these circumstances, the repulsion between the two first atoms will be greater than between the second and third, wherefore these

Fig 2.

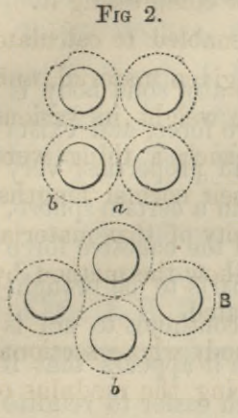


Fig. 3.

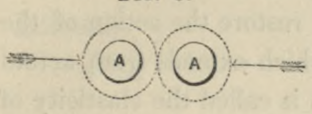
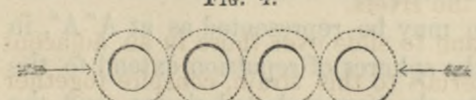


Fig. 4.



will approach each other until the equilibrium is restored, and the same will occur between all other atoms in the same line infinitely, whence it is evident that a force will compress a body through a certain fraction of its length, regardless of its length; thus, if a weight, W, will shorten a body 3 feet long to the amount of 1 inch, the same weight will shorten a body of the same material and lateral dimensions to the amount of $\frac{1}{36}$ of its length, whatever that length may be. On the other hand, it will require twice W to shorten twice the number of parallel lines of atoms to the same extent, and thrice W to produce the same effect on three times the number of lines. Hence, as the number of parallel lines of atoms in any homogeneous body may be considered as equal for equal sectional areas of the body taken at right angles to those lines, we conclude that the force necessary to produce a given amount of compression in any substance varies as the sectional area of the body compressed.

We also find that the amount of compression varies directly as the force producing that compression.

The same laws hold good for extension as for compression; but if in either case the limit of elasticity of the material be exceeded, these laws fail, and the body is compressed or extended unequally, and does not recover its normal size and form on the removal of the force extending or compressing it.

It is very desirable in practice, to have some fixed data by which we may be enabled to calculate the amount of compression or extension which any given weight will produce in any given material, and on this account experimentalists have measured the extension produced by a given weight in various substances, and, by the foregoing laws, have calculated the weights which would (if such a thing were possible), extend bars of those bodies of 1 square inch in sectional area to twice their normal lengths. These numbers supply the required data, and each one is called the modulus of elasticity of that material for which it is calculated, and is usually stated in pounds avoirdupois. We will now show the method by which the extension or compression of any substance for a given weight may be calculated.

It is required to find the amount of elongation or compression produced in a body whose sectional area is a square inches, and its length l inches, by a force equal to p pounds, E being the modulus of elasticity of the material.

Because E will produce an extension equal to l inches, the force to produce an extension of one inch $= \frac{E}{l}$, for each square inch of sectional area, therefore, to produce an extension of one inch, the area being a, the force must be

$$= \frac{a\,E}{l}$$

and to produce an extension of n inches it will require a force $= \frac{n\,a\,E}{l}$

or if p represents this force

$$p = \frac{n\,a\,E}{l}$$

wherefore

$$n = \frac{p\,l}{a\,E},$$

from which expression we may obtain the amount of elongation produced by a force p, acting on a body whose sectional area is a, and length l.

If it be required to find the amount of extension or compression in terms of the length, we may obtain a formula for it by an obvious transposition of the terms of the last expression, thus,

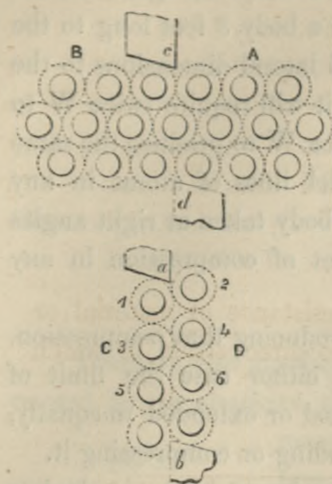
$$\frac{n}{l} = \frac{p}{aE}$$

We will next turn our attention to the operation of the force termed shearing strain.

This strain occurs in the case of plates of any material rivetted together, the plates when strained in the direction of their surfaces, exhibiting a tendency to cut, or shear the rivets.

The effect of shearing strain, is to cause the atoms in one plane to slide over those in an adjacent plane, but many circumstances connected with the resistance of materials to this strain, conspire together to render an exact understanding of its effects somewhat difficult to be obtained.

Fig. 5.



Let A B, Fig. 5, represent a number of atoms subject to a shearing strain, by the approach of the cutting edges *c d*, it is evident that these cutting edges will act as wedges, and will also compress the material at the same time, thus it will tend to force the atoms asunder, partly by sliding them past each other, partly by forcing them asunder in a line at right angles to the motion of the cutting edge, and partly by driving those atoms upon which it immediately acts, in between those below it, from this it appears that with the use of sharp edges, there are two circumstances by which the amount of force requisite to shear the material will be regulated, first, the resistance exercised by the particles against being forced asunder in a line at right angles to the motion of the cutting edge, and second, their resistance to being forced past each other.

Now, let C D, Fig. 5, represent two planes of atoms which are to be sheared asunder by the square edges *a* and *b*, and let *b* be fixed and *a* be forced down upon the plane C. In the first place, the atom No. 1 next to *a*, will be forced downwards, carrying with it the atom No. 2 until the repulsion exerted between the atoms Nos. 2 and 4 is in excess of the tangential attraction between atoms Nos. 1 and 2, these atoms will then separate, and the excess of repulsion between Nos. 2 and 4, will cause the atom No. 2, to slide past No. 1 back to its original position; the atom No. 1 being now no longer upheld by the tangential attraction of No. 2, will be forced down towards No. 3, and a similar action to that between atoms No. 1 and 2, will take place betwen Nos. 3 and 4, and this action will be repeated between each pair of atoms, until the body be entirely separated.

From the foregoing considerations, it is evident that the force requisite to shear any body, depends solely upon the number of atoms to be separated, and this number will be in exact proportion to the area of a section of the body taken through a plane extending from one shearing edge to the other, we may therefore conclude that the force required to shear any body of the same material, will depend solely upon the area to be sheared, and will be in direct proportion to it. Having examined the action of shearing strain, we may readily account for the different results of it, such as breaking, instead of being cut off. When the material allows of being slowly cut by an uniform pressure, it is evident that from the great range of elasticity, the particles are severed separately, first, the upper layers and then the lower, but when the body splits on the application of a shearing edge, it may be accounted for, by conceiving the range between the particles to be so small, that they are all severed instantaneously, there being no space for one to be slid away from its neighbour without disturbing all the others. In the case of bodies which are cracked, partly through their thickness, the shearing action appears to have proceeded to a certain depth and then stopped, to allow of which any bodies must be possessed of a certain range of elasticity, and as almost all bodies admit of being cracked, we may consider that they are all possessed of some elasticity, whence the splitting or instantaneous division would be impossible, we must therefore conclude that in this latter case the separation of the various particles from each other proceeds with such rapidity, as to produce the appearance of being simultaneous in every pair of atoms.

It must be borne in mind that shearing strain is quite different from cutting action, when the latter is accomplished with acute edges which act as wedges by insinuating themselves between the particles of which the body is composed, and thereby dividing them.

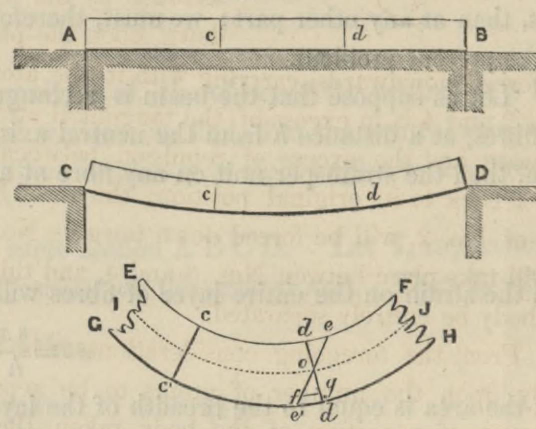
CHAPTER II.

RESISTANCE OF MATERIALS TO TRANSVERSE STRAIN.

In order to obtain an expression which will give us the value of the transverse resistance of material or resistance to lateral strain, we must base our investigations upon the resistance to direct strain to which all lateral strains must be reduced.

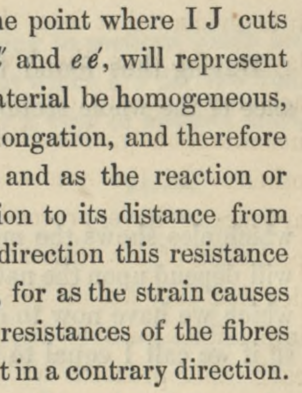
Let A B, Fig. 6, represent a perfectly straight beam, supported at each end, and also along its whole length. Let two lines, *c* and *d*, be drawn at right angles to the horizon, which may be considered to represent two planes intersecting the beam at right angles to its length, then all the fibres contained between these planes will be of the same length. Let all the supports, except the two end ones, be now removed, the beam will then have to support its own weight, by which it will be deflected as shown at C D, whereby the sections *c* and *d*, will be partly rotated, the upper fibres being shortened, and the lower ones extended. Let E F G H, represent a portion of the deflected beam, then, because the fibres at the top of the beam are compressed, and those at the bottom are extended, it is evident that where the strain changes from compression to tension, there will be no strain at all, and the length of the fibre occupying that position will remain unaltered. Let I J represent that section of the beam at which there is no strain, this section is called the neutral axis, or more properly, the neutral surface of the beam, and on each side of this, the strain increases towards the top and bottom of the beam. Through the point where I J cuts $d\,d'$, let $e\,e'$ be drawn parallel to $c\,c'$, then the spaces included between the lines $d\,d'$ and $e\,e'$, will represent the amount of strain on fibres at various distances from the neutral axis, and if the material be homogeneous, these spaces will be plane triangles. By examining the figure, we find that the elongation, and therefore the strain on any fibre, is in direct proportion to its distance from the neutral axis, and as the reaction or resistance of the fibre is equal to the strain, its resistance is also in direct proportion to its distance from the neutral axis. Before proceeding further, we must pause to ascertain in what direction this resistance is to be exerted, and we at once find that it must be exerted in a circular direction, for as the strain causes the planes $d\,d'$, and $c\,c'$, or $e\,e'$, to revolve round a point in the neutral surface, the resistances of the fibres must, when the strain is removed, cause these planes to revolve round the same point in a contrary direction. Each fibre acts at a certain distance from the axis, and the greater its distance from the axis, the greater will be its effect; thus, a fibre acting with a distance or leverage equal to six inches, will be three times as great as if its leverage were two inches. We may thus illustrate the forces of strain and resistance as they occur within the beam.

Fig. 6.



Let A B C D, Fig. 7, represent a beam fixed in a wall at A C, and let a weight W be attached to the end of the beam; we will consider the strain and resistance at any point, *o*. In this case, the fibres between *o f*, will be in tension, and those between *o g* in compression, and the lines, *f o e* and *g o e* may be regarded as two bent levers joined at *e*, and working on the fulcrum *o*, we shall then have two forces, which if the beam be strong enough to bear, the load will be equal and opposite.

Fig. 7.



On the one hand, we shall have the weight W drawing the point *e* downwards, and acting round the fulcrum o with a leverage equal to $o\,e$, and, on the other hand, we have the tensile and compressive resistances on the short arms, $f\,o$, $g\,o$, acting round the fulcrum o, with various leverages, holding the point e up, if, therefore, we call any tensile or compressive resistance s, and its leverage x, and equate the contrary forces, we have

$$W \times oe = (s' \times x') + (s'' \times x'') + (s''' \times x''') \text{ etc.,}$$

where s' is the resistance of the first fibre, and x' its leverage, s'' and x'' being the same for the second fibre, and so on. $W\,x.\,o\,e$ is called the moment of strain, and the other side of the equation is called the moment of resistance, for which last we will proceed to find an expression which may be readily applied to practical purposes. We must now return to Fig. 6, and to the subject of our investigation. We find that the strain on any fibre varies as to its distance from the neutral axis, and also that its moment varies in the same ratio, and it is, therefore, impossible to determine practically, any portion of the material, however thin a section may be taken, that will suffer an equal strain all over its cross section, but on the contrary, it will suffer a greater amount of extension or compression on the side furthest from the neutral axis, than at any other part; we must, therefore, have recourse to the infinitesimal calculus for the solution of the present problem.

Let us suppose that the beam is rectangular, its breadth being represented by b, we will take a layer of fibres, at a distance h from the neutral axis, the direct strain on which is s, per square unit of sectional area, then the strain per unit on any fibre at a distance x from the neutral axis will be

$$= \frac{s\,x}{h}$$

and the strain on the entire layer of fibres will be

$$= \frac{s\,x}{h} \times \text{area of layer};$$

but the area is equal to the breadth of the layer multiplied by its depth; the breadth is b, and the depth being infinitesimally small is represented by $d\,x$, and, inserting these in the above, we obtain

$$\frac{s}{h}\,b\,x\,d\,x$$

and the moment of this strain, which is obtained by multiplying the strain by its leverage x,

$$= \frac{s}{h}\,b\,x^2\,d\,x.$$

Having thus found the moment of resistance for any fibre, we can easily obtain the moments of resistance of the whole section, for if we call M the moment for the section above the neutral axis,

$$\frac{d\,M}{d\,x} = \frac{s}{h}\,b\,x^2$$

which also shows the moment of resistance of any section below the neutral axis. The quantities s and h will depend upon the nature of the material, and the depth of the beam, and the only part of the expression which we have now to deal with, is $b\,x^2\,d\,x$, this is the differential of what is called the moment of inertia, so if we call I equal to the moment of inertia of the section,

$$\frac{d\,I}{d\,x} = b\,x^2.$$

We will now proceed to integrate the last expression, for the various sections which occur in the construction of beams or girders.

In order to determine the limits between which x is to be integrated in the expression,

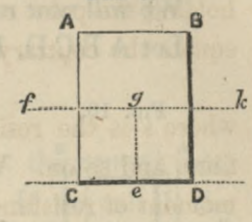
$$I = b\int x^2 d\,x$$

we must first determine the position of the neutral axis. The neutral axis will be so placed that the moment of the compressive strains is equal to the moment of the tensile strains, and it will pass through the centre of gravity of the section which may be found as follows:

Let A B C D, Fig. 8, represent a section of a rectangular girder, it is required to find its centre of gravity.

We will first take the moment of a small area, with regard to an axis at C D. Let x equal the distance of the area from C D and b, its breadth, then the moment of this area will be,

Fig. 8.



$$= b\,x\,d\,x$$

and the moment of the whole rectangle found by integrating this expression between d, the depth, and o, the distance of the edge of the section from the axis,

$$= b \int_0^d x\,d\,x = \frac{b\,d^2}{2}$$

but the moment of the whole rectangle is equal to its area multiplied by the distance from the the axis round which its moment is taken, or

$$= \underline{b\,d} \times \underline{g\,e}$$

$$\therefore \underline{b\,d} \times \underline{g\,e} = \frac{b\,d^2}{2}$$

$$\underline{g\,e} = \frac{d}{2}$$

This expression shows the distance of the centre of gravity, and, therefore, of the neutral axis from the bottom of the girder. In this, and all other symmetrical sections, we find, that the neutral axis passes through the centre of the section. The neutral axis or surface is always at right angles to the direction of the strain in any cross section.

We will now determine the moment of inertia of the same section A B C D. Let I_0 represent the moment of inertia of the section A Bfk; fk showing the position of the neutral axis. Then I_0 will be obtained by integrating the differential of I between $\frac{d}{2}$ and o thus,

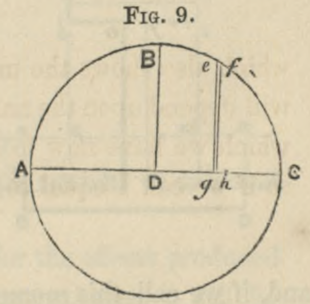
$$I_0 = b \int_0^{\frac{d}{2}} x^2\,d\,x = \frac{b\,d^3}{24} \qquad (1).$$

but the moment of inertia of the whole section will be equal to the sum of the moments of inertia for A B fk, and C Dfk, or 2 I_0 therefore,

$$I = 2\,I_0 = \frac{b\,d^3}{12}$$

This formula is the base or type of one class of expressions, viz., those referring to sections which are made up of rectangles, but before entering upon the consideration of those sections which are derived from the rectangle, we shall obtain a general expression for the moment of inertia for a type of another class, viz., the circle.

Fig. 9.



Let A B C, Fig. 9, represent a section, of which $efgh$ is a narrow element and A c the neutral axis. Let I_0 = the moment of inertia of $efgh$,

$$eg = y, \quad Cg = x.$$

Then from a former equation we obtain, replacing x by y, and b by dx,

$$I_0 = \int_0^y y^2\,dy\,dx$$

$$= \frac{y^3}{3}\,d\,x$$

and if I = moment of inertia of B D C,

$$I = \tfrac{1}{3} \int_0^x y^3\,d\,x$$

but from the equation to the circle, if a = radius

$$y = (a^2 - x^2)\,\tfrac{1}{2}$$

$$y^3 = (a^2 - x^2)\,\tfrac{3}{2} \text{ and}$$

$$I = \tfrac{1}{3} \int_0^a (a^2 - x^2)\,\tfrac{3}{2}\,d\,x$$

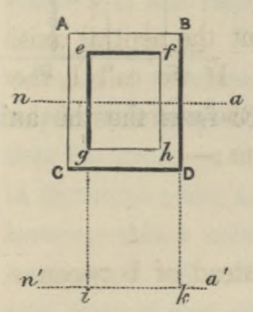
$$= \frac{\pi\,r^4}{16}$$

Hence the moment of inertia of the whole circle will be $I = \frac{\pi r^4}{4}$

We will now return to the moment of inertia for beams of rectangular sections.

Let A B C D, Fig. 10, represent a hollow rectangular beam, of which $n\,a$ is the neutral axis, let also

FIG. 10.



$AB = b,\ ef = b',\ AC = d,\ eg = d'$

$I =$ moment of inertia for the section

$I_0 =$ „ „ „ A B C D

$I_1 =$ „ „ „ $e f g h$

then will

$$I = I_0 - I_1 = \frac{bd^3}{12} - \frac{b'd'^3}{12} = \left\{\frac{bd^3 - b'd'^3}{12}\right\}$$

now let the neutral axis be removed to $n' a'$ then will

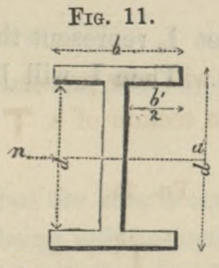
$$I = \left\{\frac{b(d + Dk)^3 - b.\,Dk^3}{12}\right\} - \left\{\frac{b'(d' + gi)^3 - b'gi^3}{12}\right\}$$

and if we make, $D\,k = e$ and $g\,i = e'$

$$I = \frac{b}{12}\left\{(d + e)^3 - e^3\right\} - \frac{b}{12}\left\{(d' + e')^3 - e'^3\right\}.$$

Let Fig. 11 represent a section of a girder, symmetrical in form, with horizontal flanges, the notations as shown in the figure.

FIG. 11.



The neutral axis $n\,a$ will pass through the centre of the section, and this case will be similar to the last, thus:

$$I = \frac{b\,d^3 - b'd'^3}{12}.$$

The following formula will give the moment of inertia generally for symmetrical forms derived from the above. Let $I_0 =$ moment of inertia of circumscribing rectangle,

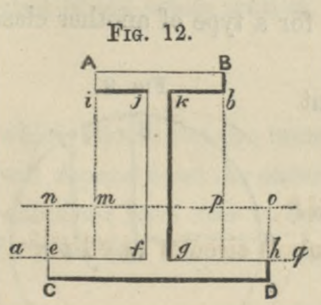
$I_1 + I_2 + I_3 \ldots\ldots\ldots + I_n =$ sum of the moments of inertia of all the open spaces, such as $e f g h$ in Fig. 10, etc., then,

$$I = I_0 - \{I_1 + I_2 + I_3 \cdots\cdots + I_n\}.$$

We will now examine some cases in which the sections are not symmetrical.

Let $A\,B\,b\,k\,g\,h\,D\,C\,e\,f\,j\,i$ represent a section of a flanged girder, the bottom flange being larger than the top flange, it is required to find its moment of inertia. The first step to be taken in the present case will be the determination of the position of the neutral axis, for which purpose we will ascertain the distance of the centre of gravity of the section, from an assumed axis of moments $a\,q$.

FIG. 12.



The moment of the area $e\,h$, C D, about the axis $a\,q$, will be,

$= \int CD\,x\,dx$, where x varies from o to h, D.

$$\int_o^{hD} CD\,x\,dx = \frac{CD \times \overline{hD}^2}{2}.$$

and, if we call this moment about $a\,q = M'$ then $M' = \frac{CD + h\overline{D}^2}{2}.$

Let the moment of the remaining area of the section about $a\,q = M_2$, then, by a similar process to the above, we find,

$$M_2 = \frac{AB \times \overline{Af}^2 - (ij + kl) \times \overline{if}^2}{2}.$$

But these moments act in contrary directions, wherefore if $M =$ moment of entire section about $a\,q$

$$M = M_2 - M_1.$$

But the distance $o\,h$ of the neutral axis from the axis $a\,q$ (see p. 7).

$$= \frac{M}{\text{area of section.}}$$

The area of the entire section will be

$$= (\mathrm{A\,B} \times \mathrm{A}f) - \{(ij + kl) \times if\} + (\mathrm{C\,D} \times h\,\mathrm{D})$$

therefore,

$$o\,h = \tfrac{1}{2}\frac{\{\mathrm{A\,B} \times \mathrm{A}f^2 - (ij + kl) \times if^2\} - \{\mathrm{C\,D} \times h\,\mathrm{D}^2\}}{(\mathrm{A\,B} \times \mathrm{A}f) - \{(ij + kl) \times if\} + (\mathrm{C\,D} \times h\,\mathrm{D})}:$$

= distance of neutral axis from the axis of moments assumed at $a\,q$.

We may now readily find the moment of inertia of each part of the section about the neutral axis, and the sum of all these will be equal to the moment of inertia of the whole section. If we call I_0 the moment of inertia of the upper part of the section, then, by subtracting the sum of all the moments of the open spaces from that of the circumscribing rectangle, we obtain the required value, thus:—

$$\mathrm{I}_0 = \frac{\{\mathrm{A\,B} \times \overline{\mathrm{B}p}^3\} - \{(ij + kl) \times lp^3\}}{3}$$

which is obtained by integrating equation (1) between the limits o and $(\mathrm{B}p - lp)$ instead of between o and $\frac{d}{2}$.

If we call I_1 the moment of inertia of the lower part of the section, we obtain,

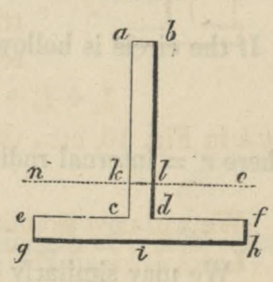
$$\mathrm{I}_1 = \frac{\mathrm{C\,D} \times \overline{\mathrm{D}h}^3}{3},$$

And if I be the moment of inertia of the whole section we obtain by summation,

$$\mathrm{I} = \mathrm{I}_0 + \mathrm{I}_1 = \tfrac{1}{3}\{(\overline{\mathrm{A\,B}} \times \overline{\mathrm{B}p}^3) - (ij + kl) \times lp^3 + (\mathrm{C\,D} \times \overline{\mathrm{D}h}^3)\}.$$

As **T** irons are occasionally used where beams of but little strength are required, we will now proceed to obtain an expression for the moment of inertia of this section. Let Fig. 13 represent a section of a **T** iron, we must ascertain the position of the neutral axis.

Fig. 13.



We will assume an axis of moments of gravity at $g\,h$, the moment of gravity of the section

$$= \frac{(a\,b \times b\,i^2) + (e\,c + d\,f) \times f\,h^2}{2}$$

dividing this by the total area of the section, we obtain the distance of the neutral axis from the axis $g\,h$, thus, $n\,o$, being the neutral axis.

$$g\,n = \tfrac{1}{2}\frac{(a\,b \times b\,i^2) + (e\,c + d\,f) \times f\,h^2}{(a\,b \times b\,d) + (g\,h \times h\,f).}$$

Let I_0 represent the moment of inertia of the upper part of the section, I_1 that of the lower, and I that of the whole section, then,

$$\mathrm{I}_0 = \frac{a\,b \times a\,k^3}{3}$$

$$\mathrm{I}_1 = \frac{(g\,h \times i\,l^3) - (e\,c + d\,f) \times d\,l^3}{3}$$

and

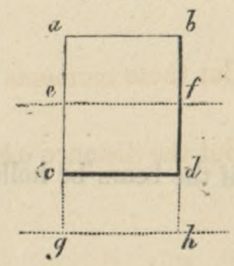
$$\mathrm{I} = \mathrm{I}_0 + \mathrm{I}_1 = \tfrac{1}{3}\,\{(ab \times ak^3) + (g\,h \times i\,l^3) - (ec + df) \times dl^3\}$$

We will now direct our attention to sections which are composed of, or contain circular or elliptical elements.

As in the cases of which we are about to treat, we shall have occasion to consider the effects produced upon the values of the moments of inertia of certain sections by changes in their neutral axes, it will be necessary first to consider the nature of such changes.

Let $a\,b\,c\,d$, Fig. 14, represent a rectangular element, $e\,f$ its neutral axis, and $g\,h$ any other axis distant k from $e\,f$.

Fig. 14.



Let I = moment of inertia of the section referred to $e\,f$.

I_0 = moment of inertia of the section referred to $g\,h$.

Then,

$$\mathrm{I} = \tfrac{1}{12}\,a\,b\,.\,b\,d^3$$

$$\mathrm{I}_0 = \tfrac{1}{3}\,a\,b\,.\,b\,h^3 - a\,b\,d\,h^3$$

if we call $a\,b = b$, and $b\,d = d$

$$\mathrm{I} = \tfrac{1}{12}\,b\,d^3$$

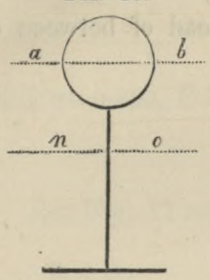
$$I_0 = \frac{1}{3} b \left(\frac{d}{2} + k\right)^3 - \frac{1}{3} b \left(k - \frac{d}{2}\right)^3$$

$$= \frac{b\, d^3}{12} + b\, d \,.\, k^2.$$

Hence we find that the moment of inertia, taken with regard to the neutral axis, is equal to that taken with regard to any other axis, plus the whole area of the section multiplied by the square of the distance between the axes.

Having now obtained the desired expression, we will proceed to cases in which we shall require it. Let Fig. 15 represent a section of which the moment of inertia is required, $a\, b$ being an axis passing through the centre of gravity of the circular part, and $n\ o$ the neutral axis of the section.

Fig. 15.



The only part of the calculation which we shall here perform, will be that which refers to the circular portion of the section, the method of treating the remainder having been detailed above.

From a former calculation we find that the moment of the inertia of a circle about its axis of symmetry, or,

$$I_0 = \frac{\pi\, r^4}{4}$$

where $\pi = 3{\cdot}1416$ and $r =$ radius of the circle. But if I equal the moment of inertia of the circle about $n\ o$ $\qquad I = I_0 + \pi\, r^2\, k^2$

k being equal to the distance between the axes $a\ b$ and $n\ o$.

$$\therefore I = \frac{\pi\, r^4 \times \pi\, r^2\, k^2}{4}$$

$$= \frac{\pi\, r^2}{4} \{r^2 + 4\, k^2\}$$

If the circle is hollow, then,

$$I_0 = \frac{\pi}{4} \{r^4 - r'^4\}$$

where $r' =$ internal radius, and

$$I = \frac{\pi}{4} \{r^4 - r'^4\} + \pi\, k^2 \{r^2 - r'^2\}$$

We may similarly obtain the moment of inertia of an ellipse under various circumstances,

Let $\quad I_0 =$ moment of inertia of an ellipse about its minor diameter,

$I\ =$ moment of inertia about any other parallel axis,

$k\ =$ distance between the axes,

$b\ =$ semiminor diameter,

$a\ =$ semimajor diameter,

then,

$$y = \frac{a}{b} (b^2 - x^2)^{\frac{1}{2}}$$

$$\therefore I_0 = \frac{4}{3} \cdot \frac{a^3}{b^3} \int (b^2 - x^2)^{\frac{3}{2}}\, dx$$

$$= \frac{\pi\, b\, a^3}{4}$$

and

$$I = \frac{\pi\, b\, a^3}{4} + \pi\, b\, a\, k^2$$

$$= \frac{\pi\, b\, a}{4} \left\{a^2 + 4\, k^2\right\}$$

but if the beam be hollow

$$I = \frac{\pi}{4} \left\{b\, a^3 - b' a'^3\right\}$$

where b^1 = internal semiminor diameter,
a^1 = internal semimajor diameter,

$$I = \frac{\pi}{4}\left\{ b\,a^3 - b'\,a'^3 \right\} + \pi\,k^2 \left\{ b\,a - b'\,a' \right\}.$$

From the foregoing formulæ, we can readily obtain the moment of inertia of any of those sections which occur in practice, hence we may now proceed to determine that form and proportion which will exert the greatest amount of strength, for a given quantity of material. It is at once evident that the moment of resistance of any section increases with its distance from the neutral axis, but there are other elements in the calculation which are not of a theoretical nature, such as the proportions at which the materials will stand, their liability to fracture by splitting, etc.

We may, however, in the present chapter, compare the values of the moments of inertia for various forms having the same sectional area. Let us first compare the circle and the square, their moments being taken with regard to their axes of symmetry, that of the square being parallel to its sides.

Let I = moment of inertia of a circle,
I_0 = moment of inertia of a square,
a = side of square,
r = radius of circle,

then,

$$I_0 = \frac{1}{12}\,a^4$$

$$I = \frac{1}{4}\,\pi\,r^4$$

If C represent a constant

$$\tfrac{1}{12}\,a^4\,C = \tfrac{1}{4}\,\pi\,r^4$$

$$\therefore C = 3.\,\frac{\pi\,r^4}{a^4}$$

but if the areas be equal,

$$\pi\,r^2 = a^2$$

$$\therefore C = \frac{3.\,r^2}{a^2} = \frac{3}{\pi}$$

Therefore the strength of a solid cylindrical beam is equal to the product of the strength of a solid square beam of the same sectional area, and $\frac{3}{\pi}$. Let us now suppose the two beams to be hollow, the diameter of the circular one being equal to the side of the square.

d = external diameter or side,
d_1 = internal diameter or side,

then,

$$I_0 = \frac{1}{12}\left\{ d^4 - d'^4 \right\}$$

and,

$$I = \frac{\pi}{64}\left\{ d^4 - d'^4 \right\}.$$

Let C represent the required constant, then if the plates are of equal thickness,

$$C = \frac{3.\,\pi}{16}$$

but if the areas are equal,

Let d_2 = internal diameter of circle,

$$\therefore I = \frac{\pi}{64}\left\{ d^4 - d_2^4 \right\}$$

and,

$$d^2 - d'^2 = \frac{\pi}{4}\left\{ d^2 - d_2^2 \right\}$$

and,

$$d^2 + d'^2 = \frac{\pi}{4}\left\{d^2 + d_2^2\right\}$$

$$d^4 - d'^4 = \left\{d^2 - d'^2\right\} \cdot \left\{d^2 + d'^2\right\}$$

$$d^4 - d_2^4 = \left\{d^2 - d_2^2\right\} \cdot \left\{d^2 + d_2^2\right\}$$

$$\therefore C = \frac{3}{16}\frac{\pi\,(d^2 - d_2^2).(d^2 + d_2^2)}{(d^2 - d'^2).(d^2 + d'^2)}$$

$$= \frac{3}{16}\frac{\pi}{}\frac{4}{\pi}\frac{(d^2 - d'^2).(d^2 + d'^2)}{(d^2 - d'^2).(d^2 + d'^2)}$$

$$= \frac{3}{4}$$

Hence the circular beam, whose diameter is equal to the side of the square beam, their areas being equal, has $\frac{3}{4}$ the strength of the square.

We will now suppose the beams to be so proportioned, that the area of the external circle is equal to that of the external square, and the area of the internal circle equal to that of the internal square.

Let d_2 = external diameter,

d_3 = internal diameter,

the other notations remaining as in our last calculation.

$$I_0 = \frac{1}{12}\left\{d^4 - d'^4\right\}$$

$$I = \frac{\pi}{64}\left\{d_2^4 - d_3^4\right\}$$

but,

$$d^2 = \frac{\pi}{4} d_2^2$$

$$d'^2 = \frac{\pi}{4} d_3^4$$

$$\therefore C = \frac{3}{\pi}$$

which was the result obtained from the comparison of solid, circular, and square beams.

We will now compare the moments of inertia of beams having rectangular and elliptical sections. First, let the beams be solid, and

b = breadth of rectangle,

d = depth of rectangle,

v = vertical axis of the ellipse,

h = horizontal axis of the ellipse.

Let also,

$$\frac{b}{d} = \frac{h}{v}$$

I_0 = moment of inertia of the rectangle,

I = moment of inertia of the ellipse,

then,

$$I_0 = \frac{1}{12} b\, d^3$$

$$I = \frac{\pi}{64} h\, v^3$$

If C be the required constant,

$$\frac{b\, d^3}{12} C = \frac{\pi}{64} h\, v^3$$

but the areas being equal,

$$bd = \frac{\pi}{4} h v$$

$$\therefore C = \frac{3}{4}\frac{v^2}{d^2} = \frac{3}{4}\frac{4}{\pi} = \frac{3}{\pi}.$$

Hence the elliptical beam bears the same proportion as regards strength to that having a rectangular section, similar to a rectangle circumscribed about the ellipse, as the circular beam bears to the square beam.

We will now complete the expression for the strength of any beam.

From former notations we find that if s be equal to the greatest strain per square unit of sectional area to which the beam may be subject, and h the distance of the extreme, and therefore most strained fibre of the section, the moment of resistance of the section will be

$$= \frac{s}{h} I$$

where I is the moment of inertia of the section.

If we determine the value of s, we may readily obtain the moment of resistance of the section by replacing it in the above expression.

It may not be undesirable here to insert the value of the moment of resistance for various sections, observing, that when the section is symmetrical with regard to the neutral axis, h is equal to half the depth of the section.

For beams of rectangular section,

Let b = breadth,
d = depth,
k = area of section,
s = greatest strain per unit,
M = moment of resistance,

then,

$$M = \frac{s}{h} \cdot \frac{b d^3}{12}$$
$$= \frac{s}{\frac{d}{2}} \cdot \frac{b d^3}{12}$$
$$= \frac{1}{6} \cdot s\, k\, d.$$

If the beam is hollow, let

b' = internal breadth,
d' = internal depth,

$$M = \frac{s}{h} \cdot \frac{b d^3 - b' d'^3}{12}$$
$$= \frac{1}{6} \cdot \frac{s}{d} \left\{ b d^3 - b' d'^3 \right\}.$$

Some authors have simplified the last formula, by assuming where the plates are thin, that we may safely consider that $\frac{d^1}{d} = 1; \frac{b'}{b} = 1$, but the error thus introduced into the calculation is too considerable to be overlooked.

We will select a case whereby to exhibit the amount of this error, previously observing that if the breadths are equal the above formula becomes,

$$M = \frac{1}{6\,d} \cdot s\, b \left\{ d^3 - d'^3 \right\}$$

which upon the above supposition may be simplified thus,

$$M = \frac{1}{6\,d} \cdot s\, k \left\{ d^2 + d d^1 + d'^2 \right\}.$$
$$= \tfrac{1}{2}\, s\, k\, d.$$

The amount of error $= \frac{1}{6\,d} \cdot s\, k \ \left\{ (d^2 + d\,d' + d'^2) - (3\, d^2) \right\}$

$$=\frac{1}{6}.\, s\, k\left\{d' + \frac{d'^2}{d} - 2d\right\}$$

and the ratio of the error to the moment of resistance obtained by the formula,

$$\mathrm{M} = \tfrac{1}{2}\, s\, k\, d$$

$$\text{varies as} \quad \frac{d'}{d} + \frac{d'^2}{d^2} - 2$$

$$\text{or} \quad = \frac{1}{3}\left\{\frac{d'}{d} + \frac{d'^2}{d^2} - 2\right\}$$

Let $d = 15$ inches, and $d' = 13$, then the above ratio,

$$= \frac{1}{3}\left\{\frac{13}{15} + \frac{169}{225} - 2\right\}$$

$$= -\ 0.1274 \text{ etc.}$$

or the error will be about $\frac{1}{8}$ of the strength obtained by the above formula, and it has a minus sign, wherefore we must substract about $\frac{1}{8}$ in this case.

We will now consider the case of a larger girder:—

Let $d = 90$ inches, $d' = 86$, then,

$$\frac{1}{3}\left\{\frac{d'}{d} + \frac{d'^2}{d^2} - 2\right\} = -\ 0.00263.$$

Here it is evident that the error is too small to be worthy of consideration.

Although the process which we have adopted has enabled us to obtain a formula sufficiently accurate to be applied to small girders with a correction, and to large girders without requiring any corrections, the manner in which this has been adopted to attain the same end by certain mathematicians is not attended with so accurate results. We may thus compare the two processes,

$$\mathrm{M} = \frac{1}{6\, d}.\, s\, b\, \{d^3 - d'^3\}$$

we first replace the area of the section by k,

$$\mathrm{M} = \frac{1}{6\, d}.\, s\, k\, \{d^2 + d'\, d + d'^2\}$$

then making $d = d'$ we obtain,

$$\mathrm{M} = \tfrac{1}{2}.\, s\, k\, d.$$

By the former process,

$$\mathrm{M} = \frac{1}{6\, d}.\, s\, b\, \{d^3 - d'^3\}$$

making $d = d'$

$$\mathrm{M} = \frac{1}{6}.\, s\, k\, \{d + d'\}$$

$$= \frac{1}{3}.\, s\, k\, d.$$

Which involves an error in some cases equal to half the value of the entire expression, and the larger the beam, the greater will be the error.

The formula which we have adopted, may be received as true for girders in which the thickness of the plate is not more than $\frac{1}{35}$ of its distance from the neutral axis, as in that case the excess will only be about $\frac{1}{50}$ of the calculated strength.

It will be observed that the same result is obtained by regarding the plate as a single layer of fibres, when we should obtain its moment by multiplying its total direct resistance by its distance from the neutral axis.

If we are desirous that the error which may exist, should be in deficiency instead of excess, we may insure this by replacing the larger depth by the less, when

$$\mathrm{M} = \tfrac{1}{2}.\, s\, k\, d'.$$

In a similar manner, we may obtain the value of the moment of resistance of any section, but we cannot resolve these expressions into those forms in which they will be practically applied, until we have investigated the manner in which the strains produced in girders by those loads to which they may be subject, operate, and for these practical formulæ, we must refer our readers to that section in which we treat of the practical application of the expressions which are the result of our investigations.

From the foregoing formulæ, the moment of resistance of any rectangular beam may be calculated; but we will here insert the general equation for an isolated rectangular element.

Let b = breadth of rectangle,
d = distance from neutral axis to top of rectangle,
e = distance from neutral axis to bottom of rectangle,

then,

$$M = \frac{s}{3h} b \{d^3 - e^3\}$$

$$= \frac{s}{3d} \cdot k \{d^2 + d' e + e^2\}$$

if $d = e$, this involves no considerable error.

$$M = s\,k\,d.$$

The moment of error is the same as above.

Let us now apply the same process to circular and elliptical beams.

Let r = external radius,
r' = internal radius,
$2a\ 2a'$ = external and internal vertical axes,
$2b\ 2b'$ = external and internal horizontal axes.

Then for a solid circular beam,

$$M = \frac{s}{h} \cdot \frac{\pi r^4}{4}$$

$$= \frac{s}{4} \cdot \pi r^3$$

$$= \tfrac{1}{4} \cdot s\,k\,r$$

for a hollow beam,

$$M = \frac{s}{h} \cdot \frac{\pi}{4} \{r^4 - r'^4\}$$

$$= \frac{s}{4h} \cdot k \{r^2 + r'^2\}$$

$$= \tfrac{1}{4} \cdot s\,k \left\{ r + \frac{r'^2}{r} \right\}$$

For a solid elliptical beam,

$$M = \frac{s}{h} \cdot \frac{\pi}{4} b\,a^3$$

$$= \tfrac{1}{4} \cdot s\,k\,a$$

for a hollow elliptical beam,

$$M = \frac{s}{h} \cdot \frac{\pi}{4} \{b\,a^3 - b'\,a'^3\}$$

$$= \frac{1}{s} \cdot s\,\pi \left\{ b\,a^3 - \frac{b'\,a'^3}{a} \right\}.$$

CHAPTER III.

THEORY OF STRAIGHT GIRDERS.

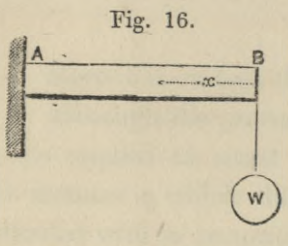
In the present chapter, we shall confine our attention to the examination of the direction and intensity of transverse strain, when resisted by a straight girder of one span or opening, under three different conditions, viz.:—

Supported only, with one end fixed, and with both ends fixed, calling those strains which make the beam convex on its upper surface + and *vice versâ.*

We will first examine the most simple case.

Let A B, Fig. 16, represent a beam firmly fixed by one end into a wall, the other end being left perfectly free. It is required to find the moment of stress occasioned by a weight W, at any point distant x from the weight. Then if M is equal to this moment it is evident that,

Fig. 16.



$$M = W x$$

therefore the section of greatest strain is at the fixed end of the beam, where $x = l$, the length of the beam, and,

$$M = W l.$$

Now let the weight be removed, and an uniform load of w per lineal unit be laid upon the beam, then the weight which can act upon any point distant x units from the end of a beam, is,

$$w = x$$

and the distance of the centre of gravity of this weight from the given point will evidently be,

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

therefore the moment of strain is,

$$M = w x \times \frac{x}{2} = \frac{w x^2}{2},$$

and at the fixed end the section of greatest strain

$$M = \frac{w l^2}{2}.$$

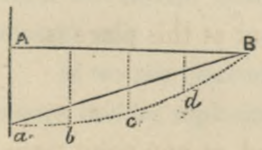
If these two loads act simultaneously,

$$M = W x + \frac{w x^2}{2} = \frac{x}{2} \left\{ 2 W + w x \right\}.$$

By calculating the moment of stress for various points, and laying down ordinates representing these moments, we may obtain a line of moments which will give the moment of stress at any point, and in the first case, this line will be straight, because the strain varies directly as x, but in the second it will be a curve, the ordinates of which vary directly as x^2.

This curve will be a parabola. The lines of strain may be thus laid down. Let A B, Fig 17, represent the horizontal line of the beam drawn to scale. To trace the line of strain produced by a constant weight, we have these data. At B, where $x = o$, $y =$ the ordinate $= o$, or the line of strain, coincides at this point with A B, the axis x, at A $y = W l$ and the line is straight, at A set off A $a = W l$ and join a B, then a B is the line of strain.

Fig. 17.



To lay down the curve for a continuous load, we must calculate the value of y for several points, and join the loci thus obtained a, b, c, d B, is the curve.

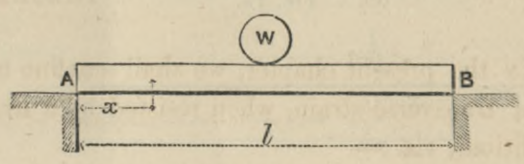
The curve produced by the simultaneous action of the two loads may be similarly traced.

If a rolling load act upon the above girder proceeding from B to A, the triangle included by the lines A B, A a, a B, will continually diminish, but the proportion between its sides will be constant.

These formulæ will apply to cantilevers, etc.

We will now consider the effects produced by various kinds of loads upon a girder supported at each end. Let A B, Fig. 18, represent a beam supported at each end, and acted upon by a central load W, the span of the girder being l. It is required to find the moment of strain at any point distant x from A, between A and W.

Fig. 18.



We can find two equations for the required moment the first involving only the reaction of the support A, and the second involving the downward tendency of the load round the given point, and the upward reaction of the support B.

As the load is central, each support will bear one half of it, therefore because action and reaction are equal and opposite, the reaction of either support,

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

hence calculating from A, the moment of stress becomes,

$$M = -\frac{W x}{2}$$

and calculating from B, we have,

$$M = W\left\{\frac{l}{2} - x\right\} - \frac{W}{2}\left\{l - x\right\}$$

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

The maximum strain will be at the centre, where $x = \frac{l}{2}$, and,

$$M = -\frac{W l}{4}$$

The line of moments may be traced by setting down the moment at the centre of the span, and joining the point thus found to A and B by straight lines.

Let the weight W be moved to any position distant z from A, then the part of the weight supported by the pier A, will be,

$$= -\frac{W(l-z)}{l},$$ and that borne by the pier B,

$$= -\frac{W z}{l}$$

the moment of stress at a point distant x from one of the supports will be if the point is between A and W,

$$M = -\frac{W(l-z)x}{l}$$

if the point is between B and W,

$$M = -\frac{W z}{l} \cdot x$$

x being measured from A in the first case, and from B in the second; if it is measured from A in both cases, the second equation becomes,

$$M = -\frac{W z}{l}\{l - x\}.$$

The point of greatest strain will be that immediately beneath W, and the moment at this place is,

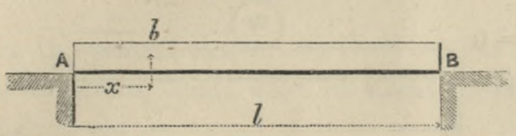
$$M = -\frac{W(l-z)}{l} \cdot z$$

This expression will also give the moment of strain produced by a moving load, at any point over which it may be passing, and the curve of moments for a moving load which may be calculated by this formula will be elliptical.

Having obtained the general equation for the moment of strain produced by a load acting on one part of a girder, we will now examine the effects produced by a continuous or distributed load upon a girder, supported at both ends.

First, let the girder support a load, W, which is equally distributed throughout its entire length, and let the load per lineal unit, or $\frac{W}{l} = w$.

Fig. 19.



Let A B represent the girder thus loaded, its own weight being the load. It is required to find the moment of stress at any point b distant x from the pier A.

We have the two moments about the point b, one caused by the weight between A and b acting downwards, and another depending upon the reaction of the pier A which acts upwards, therefore the difference between these moments will be equal to the required moment.

The weight acting downwards round b is A b which

$$= w x$$

having a position sign, as it tends to render the beam convex on the upper surface, the distance of its centre of gravity from b is $\frac{x}{2}$ and its moment $= \frac{w x^2}{2}$

The reaction of the pier A tends to render the beam concave on its upper surface, and is equal to half the weight of the load, and acts at a distance x from the point b, therefore its moment

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

and the moment of strains at the point b is

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

we can readily find the value of x which gives the maximum strain, for if there is a maximum,

$$\frac{d M}{d x} = 0$$

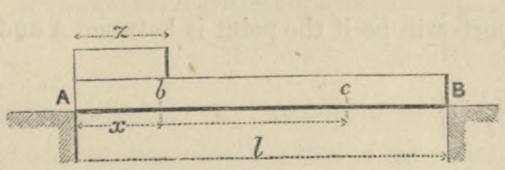
$$\frac{d M}{d x} = w x - \frac{w l}{2} = 0$$

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

or the maximum strain is at the centre of the span, where

$$M = \frac{w l^2}{8} - \frac{w l^2}{4} = - \frac{w l^2}{2} = - \frac{W l}{8}$$

Fig. 20.



Let the girder now be supposed to be partially loaded as shown in the accompanying Fig. Let the load be equal to w per lineal unit, and let it extend from the pier A through a distance equal to z. Then the weight upon A and therefore its reaction,

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

and that on B,

$$= - w z . \frac{\frac{z}{2}}{l}$$

and the moment at any point distant x from the pier A is, when x is less than z.

$$M = \frac{w x^2}{2} - w x z . \frac{l - \frac{z}{2}}{l}$$

$$= \frac{w x}{2} \left\{ x - \frac{2 l z . - z^2}{l} \right\}$$

when x is greater than z,

$$M = -wz.\frac{\frac{z}{2}}{l}\left\{l - x\right\}$$

$$= -\frac{wz^2}{2}\left\{1 - \frac{x}{l}\right\}$$

Proceeding as before to find the value of x, which corresponds with the greatest strain, we have

$$\frac{dM}{dx} = wx - wz.\frac{l - \frac{z}{2}}{l} = 0$$

$$\therefore x = z - \frac{z^2}{2l}$$

If the girder be loaded along half its length, $z = \frac{l}{2}$ and at the point of maximum strain,

$$x = \frac{l}{2} - \frac{l^2}{8l} = \frac{3l}{8}.$$

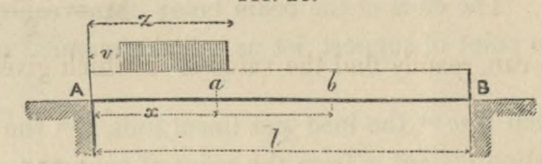
$$\text{If } z = \frac{3l}{4} \therefore x = \frac{3l}{4} - \frac{9l^2}{32l} = \frac{15l}{32}$$

$$\text{If } z = \frac{l}{4} \therefore x = \frac{l}{4} - \frac{l^2}{32l} = \frac{7l}{32}.$$

We will now suppose the load to be placed along a part of the girder distant from both points of support.

Let A B represent the girder thus loaded, the shaded part in the figure indicating the load, one end of which is at a distance v, and the other at a distance z from the pier A. The reaction on the pier A.

Fig. 21.



$$= -w\left\{z - v\right\}\left\{\frac{l - (v - \frac{z-v}{2})}{l}\right\}$$

$$= -w\left\{\left(z - \frac{vz + z^2}{2l}\right) - \left(v - \frac{v^2 + vz}{2l}\right)\right\}$$

$$= -w\left\{z - \frac{z^2}{2l} - v + \frac{v^2}{2l}\right\}$$

and the reaction on the pier B,

$$= -w\left\{z - v\right\}.\left\{\frac{v + \frac{z-v}{2}}{l}\right\}$$

$$= -w\left\{\frac{vz + z^2}{2l} - \frac{v^2 + vz}{2l}\right\}$$

$$= -\frac{w}{2l}\left\{z^2 - v^2\right\}$$

which if the entire load = W

$$= -\frac{W}{2l}\left\{z + v\right\}.$$

If the moment of strain at any point such as b is required, the point being under a part of the girder which is not loaded, it may be found by multiplying the reaction of the nearest pier by the distance of the point from that pier, thus for b we obtain,

$$M = -\frac{W}{2l}\left\{z + v\right\}.\left\{l - x\right\}$$

$$= -\frac{W(z+v)}{2} + \frac{W(z+v)x}{2l}$$

But if the point be situate under the load as at a, we consider the effect of that part of the load intercepted between a and the pier.

The moment of strain at a will be

$$M = \frac{w(x-v)^2}{2} - w\left\{z - \frac{z^2}{2l} - v + \frac{v^2}{2l}\right\} x$$

$$= \frac{w}{2}\left\{x^2 + v^2 - 2zx + \frac{xz^2}{l} - \frac{xv^2}{l}\right\}.$$

This may be considered the general equation, and we may from it deduce the conditions of greatest strain.

The curve of moments may be traced as above directed for each of these cases, it will be a parabolic curve when the beam is uniformly loaded all over, but when the beam is partially loaded the line of moments will be curved under the load, but the other part or parts will be straight lines.

By varying the valves of v and z, but keeping the value of $(z - v)$ constant, we may from the last equation obtain the curve of maximum moments for a continuous moving load.

We may, by using the foregoing formulæ either separately or combined, obtain the curve of moments on a beam loaded in any manner, provided that it is not fixed at either end but merely supported.

We will now proceed to examine the effect of a distributed load on a beam of which one or both ends are firmly fixed.

The end or ends of the beam being fixed, a tangent to the beam will at the point of support be horizontal, therefore the angle contained between this tangent and the axis on which x is measured will be nothing if therefore we call this angle α, and the deflection of the beam y, at the point of support.

$$tan\, \alpha = \frac{dy}{dx} = o.$$

The ends of the beam being immoveable, it is evident that there will be a strain on the beam above the point of support, let us call the moment of strain at one of these points,

$$= M'$$

Then if $w =$ the load per lineal unit, $l =$ the span of the girder, and $M =$ the moment of strain at any section distant x from the point of support.

$$M = M' + \frac{wx^2}{2} - \frac{wlx}{2}.$$

To find M$'$ we must for the present assume an equation which will subsequently be proved (see Deflection), viz., that if ϵ is equal to the moment of inertia of the section multiplied by the modulus of elasticity of the material.

$$\epsilon\frac{d^2y}{dx^2} = M$$

$$\epsilon\frac{d^2y}{dx^2} = M' + \frac{wx^2}{2} - \frac{wlx}{2}$$

$$\epsilon\frac{dy}{dx} = M' + \frac{wx^3}{6} - \frac{wlx^2}{4}$$

the constant is nothing because when $x = o$, $\frac{dy}{dx} = o$, make $x = l$.

$$\epsilon\frac{dy}{dx} = o = M'l + \frac{wl^3}{6} - \frac{wl^3}{4}$$

$$\therefore M' = \frac{wl^2}{12}$$

and $$M = \frac{wl^2}{12} + \frac{wx^2}{2} - \frac{wlx}{2}$$

at the centre where the strain is a maximum,

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

$$\therefore M = \frac{wl^2}{12} + \frac{wl^2}{8} - \frac{wl^2}{4} = -\frac{wl^2}{24}$$

As the moment is plus at the point of support and minus at the centre of the girder, there must be some intermediate point at which it is nothing ;

$$\text{then} \quad \frac{w\,l^2}{12} + \frac{w\,x^2}{2} - \frac{w\,l\,x}{2} = o$$

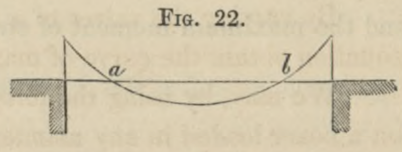
$$x = \frac{3{\cdot}5\,l \pm 2\,l.}{7}$$

Therefore, each point at which the strain is nothing is $\frac{1{\cdot}5}{7}$ of the span from the nearest abutment, or $\frac{2}{7}$ of the span from the centre.

As the curvature of the beam changes at these points, they are called points of contrary flexure.

It has been observed, that at some places on the beam the moment of strain will be plus, while at others it is minus; the curve of moments should therefore be laid out as shown at Fig. 22, where all above the axis of x is plus and all below it minus.

Fig. 22.



a and b are the points of contrary flexure. When the moment of strain is plus, the upper fibres of the beam will be extended and those below the neutral axis compressed; but if the moments have a minus sign, the reverse will take place.

As the moment of resistance is equal to the moment of strain, we must assume its sign according to that of the strain. We will now consider the beam as being fixed at one end and supported at the other, retaining the same notations as in the last case. Let R = the reaction of the pier on which the girder is only supported, and from which x is measured, then

$$\mathrm{M} = \frac{w\,x^2}{2} - \mathrm{R}\,x$$

if $x = l$, M = M′; or

$$\mathrm{M}' = \frac{w\,l^2}{2} - \mathrm{R}\,l.$$

$$\therefore \mathrm{R} = \frac{w\,l}{2} - \frac{\mathrm{M}'}{l};$$

replacing R in the first equation, we have

$$\epsilon.\frac{d^2 y}{d\,x^2} = \mathrm{M} = \frac{w\,x^2}{2} - \frac{w\,l\,x}{2} + \frac{\mathrm{M}'x}{l},$$

$$\frac{d\,y}{d\,x} = \frac{w\,x^3}{6\,\epsilon} - \frac{w\,l\,x^2}{4\,\epsilon} + \frac{\mathrm{M}'x^2}{2\,l\,\epsilon} + \text{constant.}$$

When $x = o$, the constant $= \frac{d\,y}{d\,x} = tan\ \alpha$, therefore $tan\ \alpha$ is the constant, and we must determine its value ; integrating the last equation, we obtain

$$y = \frac{w\,x^4}{24\,\epsilon} - \frac{w\,l\,x^3}{12\,\epsilon} + \frac{\mathrm{M}'x^3}{6\,l\,\epsilon} + tan\ \alpha\ x. + .\ \text{constant.}$$

When $x = o$, $y = o$, therefore, in this case the constant is nothing, and when $x = l, y = o$, therefore,

$$y = o = \frac{w\,l^4}{24\,\epsilon} - \frac{w\,l^4}{12\,\epsilon} + \frac{\mathrm{M}'\,l^2}{6\,\epsilon} + tan\ \alpha\ l,$$

$$\therefore tan\ \alpha = \frac{w\,l^3}{24\,\epsilon} - \frac{\mathrm{M}'\,l}{6\,\epsilon};$$

therefore,

$$\frac{d\,y}{d\,x} = \frac{w\,x^3}{6\,\epsilon} - \frac{w\,l\,x^2}{4\,\epsilon} + \frac{\mathrm{M}'\,x^2}{2\,l\,\epsilon} + \frac{w\,l^3}{24\,\epsilon} - \frac{\mathrm{M}'\,l}{6\,\epsilon}.$$

When

$$x = l, \frac{d\,y}{d\,x} = o = \frac{w\,l^3}{6\,\epsilon} - \frac{w\,l^3}{4\,\epsilon} + \frac{w\,l^3}{24\,\epsilon} + \frac{\mathrm{M}'\,l}{2\,\epsilon} - \frac{\mathrm{M}'\,l}{6\,\epsilon},$$

from which we obtain

$$\mathrm{M}' = \frac{w\,l^2}{8}.$$

Replacing M′ by its value in the expression for the moment of strain, we have

$$M = \frac{w\,x^2}{2} - \frac{w\,l\,x}{2} + \frac{w\,l\,x}{8}$$

$$= \frac{w\,x^2}{2} - \frac{3\,w\,l\,x}{8}$$

$$= \frac{w\,x}{8}\left\{4\,x - 3\,l\right\}$$

We will now proceed to find the points of maximum strain and contrary flexure.

When M is a maximum $\frac{d\,M}{d\,x} = o$

$$\frac{d\,M}{d\,x} = w\,x - \frac{3\,w\,l}{8} = o$$

$$x = \frac{3}{8}\,l,$$

and the maximum moment of strain will be

$$= \frac{9\,w\,l^2}{128} - \frac{9\,w\,l^2}{64}$$

$$= -\frac{9\,w\,l^2}{128} = -\frac{w\,l^2}{14}\text{ nearly.}$$

It will be observed that in this and the foregoing cases we have given the maximum minus strains only, as the maximum plus strains are evidently those immediately over the piers.

The girder fixed at one end will have but one point of contrary flexure, where $M = o$

$$o = \frac{w\,x^2}{2} - \frac{3\,w\,l\,x}{8}$$

$$x = \frac{3}{4}\,l.$$

Let us now compare those parts of these girders which are subject to a minus strain with girders supported only, of spans equal to the length of those parts and equally loaded.

When the beam is fixed at both ends that part which is subject to minus strains has a length equal to $\frac{4}{7}\,l$, hence the strains should be equal to those on a supported girder whose span equal $\frac{4}{7}\,l$.

If $s =$ span of supported girder the moment of strain at the centre is

$$M = -\frac{w\,s^2}{8},\; s = \tfrac{4}{7}\,l,\; \therefore M = -\frac{16.\,w\,l^2}{392}$$

$$M = -\frac{w\,l^2}{24\frac{1}{2}}$$

which is nearly equal to that obtained previously.

The error in this last expression is caused by our assuming 3·5 as the square-root of 12, instead 3·4641, etc., on account of its simplicity: had we adopted the latter, the distance of each point of contrary flexure from a point of support would have been,

$$= \frac{1{\cdot}4641}{6{\cdot}9382}\,l$$

which differs but little from the value adopted, viz.:

$$\frac{1{\cdot}5}{7}\,l.$$

In comparing the supported girder with that fixed at one end, and supported at the other, we must make $s = \frac{3}{4}\,l$, then at the point of greatest strain

$$M = -\frac{w\,s^2}{8} = -\frac{9\,w\,l^2}{128} = -\frac{w\,l^2}{14}\text{ nearly,}$$

which is the exact value of the maximum moment of strain on the girder, fixed at one end and supported at the other.

We must now examine the condition of the other parts of the fixed girders, viz., those which are subject to a plus strain and which are included between the points of support and contrary flexure.

These parts of the girder will exhibit the same effect as a beam of the same length, loaded in the same manner, fixed at one end and free at the other, they are subject at once to a continuous, and also to a concrete load, the continuous load in the case of the girder fixed at both ends, being

$$=\frac{1{\cdot}5\,w\,l}{7}$$

and the concrete load being that portion of the load upon the part of the girder subject to a minus strain, which passes on to the pier upon which the segment we are now treating of is fixed, this load

$$=\frac{2\,w\,l}{7}.$$

The moment of strain at the point of fixture will, therefore, be

$$M'=\frac{2{\cdot}25\,w\,l^2}{98}+\frac{3\,w\,l^2}{49}=\frac{8{\cdot}25\,w\,l^2}{98}$$

which is nearly

$$=\frac{w\,l^2}{12}$$

the small error being occasioned by our method of calculating the positions of the points of contrary flexure as above stated.

By the same method the moment of strain on the fixed end of a girder, of which one is fixed and the other supported, is found to be

$$M'=\frac{w\,l^2}{32}+\frac{3\,w\,l^2}{32}=\frac{w\,l^2}{8}$$

which is the same as was obtained by another process. We might examine some other cases of loads on these fixed girders, but as the results obtained are more curious than practically useful, we do not consider it desirable to occupy our readers attention with them.

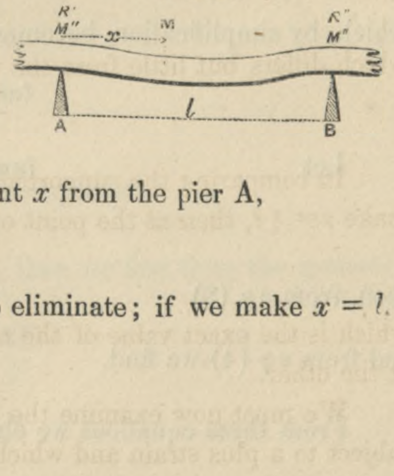
CHAPTER IV.

THEORY OF CONTINUOUS GIRDERS.

The strains to which continuous girders are subject are similar to those upon girders with fixed ends, and when the spans or openings between the points of support and the loads are so proportioned that tangents to the curve of deflection at points directly over the points of support are horizontal, each span of the continuous girder may be regarded as an isolated girder with its ends fixed.

Let A. B. Fig 23, represent one opening of a continuous girder, subject to a continuous load equal to w per lineal unit.

Fig. 23.



Let R' = reaction of pier A, R'' = ditto of pier B, M', M'' moments of strain over these piers a' a'' the angles contained between the axis of x and the tangent to the girder at A and B l = span of girder.

It is required to determine the moment of stress M at any point distant x from the pier A,

$$M = M' + \frac{w\,x^2}{2} - R'x$$

we have in this equation two constants R' and M' which it is our object to eliminate; if we make $x = l$. $M = M''$, therefore,

$$M'' = M' + \frac{w\,l^2}{2} - R'\,l$$

$$\therefore R' = \frac{w\,l}{2} + \frac{M' - M''}{l}$$

replacing R′ by its value in the first equation, we have

$$M = M' + \frac{w x^2}{2} - \left\{ \frac{w l}{2} + \frac{M' - M''}{l} \right\} x.$$

This is the general equation when M′ and M″ are known, and we will now find an equation by means of which one of these constants may be determined when the other is known,

$$\varepsilon \frac{d^2 y}{d x^2} = M' + \frac{w x^2}{2} - \left\{ \frac{w l}{2} + \frac{M' + M''}{l} \right\} x.$$

By integrating this equation we obtain

$$\frac{d y}{d x} = \frac{M' x}{\varepsilon} + \frac{w x^3}{6 \varepsilon} - \left\{ \frac{w l}{2} + \frac{M' - M''}{l} \right\} \frac{x^2}{2 \varepsilon} + \text{constant}$$

if we make $x = o$ the constant $= \frac{d y}{d x}$ which is the tangent to the angle contained between the tangent to the beam and the axis of x at the point of support.

(1) $$\therefore \frac{d y}{d x} = \frac{M' x}{\varepsilon} + \frac{w x^3}{6 \varepsilon} - \left\{ \frac{w l}{2} + \frac{M' - M''}{l} \right\} \frac{x^2}{2 \varepsilon} + \tan \alpha'$$

When $x = l$

$$\therefore \frac{d y}{d x} = \tan \alpha'' = \frac{M' l}{\varepsilon} + \frac{w l^3}{6 \varepsilon} - \left\{ \frac{w l}{2} + \frac{M' - M''}{l} \right\} \frac{l^2}{2 \varepsilon} + \tan \alpha'$$

which by simplifying becomes

(2) $$\frac{d y}{d x} = \tan \alpha'' = - \frac{w l^3}{12 \varepsilon} + \left\{ M' + M'' \right\} \frac{l}{2 \varepsilon} + \tan \alpha'$$

By integrating formula (1) we obtain

$$y = \frac{M' x^2}{2 \varepsilon} + \frac{w x^4}{24 \varepsilon} - \left\{ \frac{w l}{2} + \frac{M' - M''}{l} \right\} \frac{x^3}{6 \varepsilon} + \tan \alpha' x + \text{constant}$$

the constant is nothing in this case, for when $x = o$, $y = o$.

If $x = l$, $y = o$, therefore,

$$y = o = \frac{M' l^2}{2 \varepsilon} + \frac{w l^4}{24 \varepsilon} - \left\{ \frac{w l}{2} + \frac{M' - M''}{l} \right\} \frac{l^3}{6 \varepsilon} + \tan \alpha' l$$

which by simplification becomes

$$o = - \frac{w l^4}{24 \varepsilon} + \frac{l^2}{6 \varepsilon} \left\{ 2 M' + M'' \right\} + \tan \alpha' l$$

$$\therefore \tan \alpha' l = \frac{w l^4}{24 \varepsilon} - \frac{l^2}{6 \varepsilon} \left\{ 2 M' + M'' \right\}$$

(3) $$\therefore \tan \alpha' = - \frac{w l^3}{24 \varepsilon} - \frac{l}{6 \varepsilon} \left\{ 2 M' + M''. \right\}$$

By replacing tan α' in equation (2) by its value thus obtained we have

$$\tan \alpha'' = - \frac{w l^3}{12 \varepsilon} + \left\{ M' + M'' \right\} \frac{l}{2 \varepsilon} + \frac{w l^3}{24 \varepsilon} - \frac{l}{6 \varepsilon} \left\{ M' + M \right\}$$

which, by simplification, becomes

$$\tan \alpha'' = - \frac{w l^3}{24 \varepsilon} + \left\{ M' + 2 M'' \right\} \frac{l}{6 \varepsilon} \ldots\ldots (4).$$

Let $$\tan \alpha' = \frac{l^3}{24 \varepsilon} \theta' \qquad \tan \alpha'' = \frac{l^3}{24 \varepsilon} \theta''$$

$$M' = \frac{q' l^2}{4} \qquad M'' = \frac{q'' l^2}{4}$$

then, from $e q$ (3)

$$\theta' = w - 2 q' - q''$$

and from $e q$ (4) we find,

$$\theta'' = - w + q' + 2 q''.$$

From these equations we obtain,

$$q' = w - 2 q' - \theta'$$
$$\theta'' = w - 3 q' - 2 \theta'$$

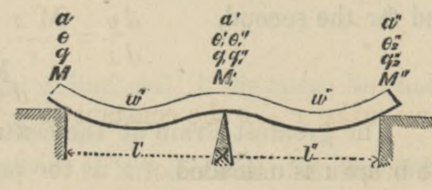
By means of these expressions we can easily find the values of $q''_1\ \theta''$ when those of $q'_1\ \theta'$ are given and $q''_1\ \theta''$ being determined, the values of $\tan \alpha''$ and M'' may also be obtained.

Having now completed the general investigation of the conditions of strain on continuous girders, we will proceed to apply the formulæ resulting therefrom, to some particular cases.

We will first turn our attention to the case of a continuous girder, supported at three points.

Let $\tan \alpha$, $\tan \alpha'$, $\tan \alpha''$, represent the values of $\tan \alpha$ at each point of support, M, M′, M″, the moments of strain at the same points, l', l'', the length of the spans, w', w'' the loads per lineal unit on the two spans.

Fig. 24.



Then for the first span we have

$$M = \tfrac{1}{4}\, q\, l'^2 = o \ldots (1) \qquad M' = \tfrac{1}{4}\, q'_1\, l'^2 \ldots (2)$$

$$\tan \alpha = \frac{l'^3 \theta}{24\, \varepsilon} \ \ldots\ldots\ldots (3) \qquad \tan \alpha' = \frac{l'^3 \theta'_1}{24 .\, \varepsilon} \ \ldots\ldots (4)$$

$$q'_1 = w' - \theta \ \ldots\ldots (5)$$

$$\theta_1' = w' - 2\,\theta \ldots\ldots (6).$$

And for the second span,

$$M' = \tfrac{1}{4} .\, q_1''\, l''^2 \ \ldots\ldots\ldots (7) \qquad M'' = \tfrac{1}{4}\, q_2''\, l''^2 = 0 \ \ldots\ldots\ldots (8)$$

$$\tan \alpha' = \frac{l''^3 \theta''_1}{24\, \varepsilon} \ \ldots\ldots\ldots (9) \quad \tan \alpha'' = \frac{l''^3 \theta''_2}{24\, \varepsilon} \ \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$q''_2 = w'' - 2\, q_1'' - \theta_1'' = 0 \ \ldots\ldots\ldots (11) \qquad \theta''_2 = w'' - 3 q_1'' - 2\theta''_1 \ \ldots\ldots (12).$$

It will immediately be observed, that from these data we can determine the values of the required constants. Let $\dfrac{l'}{l''} = m$

From the equations (2) and (7) we find,

$$\tfrac{1}{4}\, q'_1\, l'^2 = M' = \tfrac{1}{4}\, q''_1\, l''^2$$

$$\therefore q''_1 = q'_1 \frac{l'^2}{l''^2} = q'_1 m^2$$

And from equations (4) and (9)

$$\frac{l'^3 \theta'_1}{24\, \varepsilon} = \tan \alpha' = \frac{l''^3 \theta''_1}{24 .\, \varepsilon}$$

$$\therefore \theta''_1 = \theta'_1 \frac{l'^3}{l''^3} = \theta'_1\, m^3.$$

By substituting these valves in equation (11) we have

$$0 = w'' - 2\, m^2 q'_1 - m^3\, \theta'_1$$

By equation (5)

$$q'_1 = w' - \theta \quad \therefore \theta = w' - q'_1.$$

Substituting this value of θ in equation (6)

$$\theta'_1 = w' - 2\,(w' - q'_1) = 2\, q'_1 - w'$$

which being inserted in the above equation gives,

$$0 = w'' - 2\, m^2 q'_1 - 2\, m^3 q'_1 + w'\, m^3$$

$$\therefore 2\, m^2 q'_1 + 2\, m^3 q'_1 = w'' + w'\, m^3$$

$$q'_1 = \frac{w'' + w'\, m^3}{2\, m^2 + 2\, m^3}$$

q'_1 being thus obtained, we have the moment of strain over the pier from the equation,

$$M' = \tfrac{1}{4}\, q'_1\, l'^2.$$

Let R, R′ R″ equal the reactions upon the three points of support, then we find from the general equation for R, that

$$R = \frac{w'\, l}{2} - \frac{M'}{l'}, \quad R' = \frac{w'\, l'}{2} + \frac{w''\, l''}{2} + \frac{M'}{l'} + \frac{M'}{l''}$$

$$R'' = \frac{w''\, l''}{2} - \frac{M'}{l''}.$$

The equation to the curve of moments is for the first span,

$$M_0 = \frac{w' x^2}{2} - R x$$

where x is the distance of any point from the third support, for the second span,

$$M_0 = \frac{w'' x^2}{2} - R'' x$$

where x is the distance of any point from the third support.

If x be measured from the second support, the above equations become for the first span,

$$M_0 = M' + \frac{w' x^2}{2} - \frac{w' l' x}{2} - \frac{M' x}{l'}$$

and for the second,

$$M_0 = M' + \frac{w'' x^2}{2} - \frac{w'' l'' x}{2} - \frac{M' x}{l''}.$$

The greatest strain at the centre of either span will be produced when that span is loaded, and the other span is unloaded.

If the spans of the girder are of equal length,

$$q_1' = \tfrac{1}{4} (w'' + w')$$

and,

$$M' = \tfrac{1}{4} q'_1 l_1^2 = \frac{l_1^2}{16}(w'' + w')$$

if the load is the same on both spans,

$$M' = \frac{w l^2}{8}.$$

To find the points of contrary flexure, we have from the equation to the strain on the first span when x equal distance of the point of contrary flexure from the first support,

$$0 = \frac{w' x^2}{2} - R x = \frac{w' x^2}{2} - \frac{w' l' x}{2} + \frac{w l x}{8}$$

$$\therefore x = \tfrac{3}{4} l.$$

The girder may therefore be considered as divided into separate girders, each of $\frac{3}{4} l$ span, loaded with w per lineal unit, and two half beams, each $\frac{1}{4} l$ in length, loaded with w per lineal unit, and with a concentrated load at the end equal to $\frac{3}{8} w l$.

The above distribution of load will produce the greatest moment of strain above the pier, but as we have before observed, the greatest maximum strain on that part of the girder subject to minus strains, is produced by loading one span only. It would be useless to regard one span as entirely unloaded, as such a case is practically impossible, we will therefore assume a ratio between w' and w'', such as may probably exist between a girder loaded with its own weight only, and loaded with both its own weight and a temporary load.

First, let $w' = 2 w''$, then

$$q'_1 = \frac{w'' + 2 w''}{4} = \frac{3}{4} w''$$

$$M' = \frac{1}{4} q'_1 l'^2 = \frac{3}{16} w'' l'^2 = \frac{3}{32} w' l'^2.$$

To find the distance of the first point of contrary flexure from the first support we have

$$0 = \frac{w' x^2}{2} - \frac{w' l' x}{2} + \frac{3.\, w' l' x}{32}$$

$$\therefore x = \frac{13}{16} l'$$

for the first point of contrary flexure.

Let both spans be equal then for the point of contra flexure on the second span :—

$$0 = \frac{w'' x^2}{2} - \frac{w'' l'' x}{2} + \frac{3 w'' l x}{16}$$

$$\therefore x = \frac{5}{8} l.$$

And in this case the greatest minus strain on either span is thus found

$$\frac{d\,\mathrm{M}}{d\,x} = 0 = w'\,x - \frac{w'\,l}{2} + \frac{3\,w'\,l}{32}$$

$$\therefore x \quad \frac{13}{32}\,l$$

replacing x by this value we have the greatest moment of strain,

$$= \frac{169}{2048}\cdot w'\,l^2 - \frac{13}{64}\,w'\,l^2 + \frac{39}{1024}\,w'\,l^2$$

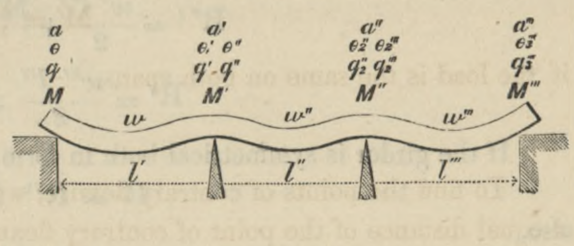
$$= -\frac{169.\,w'\,l^2}{2048} = -\frac{w'\,l^2}{12} \text{ nearly.}$$

We have thus found the greatest moments of strain, both plus and minus, and if the spans be short these data will supply the information necessary for the determination of the girder.

It would be useless here to multiply examples on the continuous girder of two spans, as we have now shown the method of applying the formulæ. We shall therefore pass on to the application of the general equations to a continuous girder of three spans.

We will employ the same notations as before, with the addition of l''', $tan\ \alpha'''$, M''', and corresponding factors for the additional span. The equations for the first span will be

Fig. 25.



$\mathrm{M} = \frac{1}{4}\,q\,l'^2 = o$ …… (1) $\mathrm{M}' = \frac{1}{4}\,q\,l^2$ ………………. (2)

$tan\ \alpha = \dfrac{l^3\,\theta}{24\,\varepsilon}$ ……….. (3) $tan\ \alpha' = \dfrac{l'^3\,\theta'}{24\,\varepsilon}$ ………………. (4)

$q'_1 = w' - \quad 2\,q - \theta$ (5) $\theta'_1 = w' - 3\,q - 2\,\theta$ ……… (6)

for the second span,

$$\mathrm{M}_{\prime} = \tfrac{1}{4}\,q''_1\,l''^2 \text{ ……… (7) } \mathrm{M}'' = \tfrac{1}{4}\,q''^2\,l''^2 \text{ …………… (8)}$$

$$tan\ \alpha' = \frac{l''^3\,\theta''_1}{24\,\varepsilon} \text{ ……… (9) } tan\ \alpha'' = \frac{l''^3\,\theta''_2}{24\,\varepsilon} \text{ …………… (10)}$$

$$q''_2 = w'' - 2\,q''_1 - \theta''_1 \text{ (11) } \theta''_2 = w'' - 3\,q''_1 - 2\,\theta''_1 \text{ ….. (12)}$$

and for the third span,

$$\mathrm{M}'' = \tfrac{1}{4}\,q'''_2\,l'''^2 \text{ ……… (13) } \mathrm{M}''' = \tfrac{1}{4}\,q^{3'''}\,l'''^2 = o \text{ ………..(14)}$$

$$tan\ \alpha'' = \frac{l'''^3\,\theta'''_2}{24\,\varepsilon} \text{ …….. (15) } tan\ \alpha''' = \frac{l'''^3\,\theta'''_3}{24\,\varepsilon} \text{ …………… (16)}$$

$$q'''^3 = w''' - 2\,q'''_2 - \theta'''_2 = o \text{ (17) } \theta'''_3 = w''' - 3\,q'''_2 - 2\,\theta'''_2 \text{ ……(18).}$$

We obtain from equation (5) $\theta = w' - q'_1$ because $q = o$; hence, replacing θ in equation (6) $\theta'_1 = 2\,q'_1 - w'_1$ from equations (2) and (7), we also find $q''_1 = m_2\,q'_1$, and from equations (4) and (9) $\theta''_1 = m^3\,\theta'_1$.

By substituting these values in equations (11) and (12) we obtain

$$q''_2 = w'' - 2\,m^2\,q'_1 - m^3\,\theta'_1$$

$$\theta''_2 = w'' - 3\,m^2\,q'_1 - 2\,m^3\,\theta'_1:$$

also, if we make $m_2 = \dfrac{l''}{l'''}$ and proceed in the same manner with the second and third spans as we have with the first and second, we shall find

$$q'''_3 = w''' - 2\,q''_2\,m_2^2 - m_2^3\,\theta''_2$$

$$\theta'''_3 = w''' - 3\,q''_2\,m_2^2 - 2\,m_2^3\,o''_2$$

By continually replacing the constants θ''_1, q''_1, etc., by their values thus obtained, we ultimately arrive at the value of q'_1,

$$q'_1 = \frac{-\,w''' + m_2^2\,(2 + m_2)\,w'' + m^3\,m_2^2\,(2 + 2\,m_2)\,w'}{m^2\,m_2^2\,\{4\,(1 + m + m\,m_2) + 3\,m_2\}}$$

if the girder is symmetrical, $m_2 = \dfrac{1}{m}$ and

$$q'_1 = \frac{-\,m^3\,w''' + (2\,m + 1)\,w'' + 2\,m^3\,(m + 1)\,w'}{m^2\,(4\,m^2 + 8\,m + 3).}$$

If all the spans are equal $m = 1$ and

$$q'_1 = \frac{-w''' + 3w'' + 4w'}{15.}$$

The value of q''_2 may be found from the equation

$$q''_2 = w'' - 2m^2 q'_1 - m^3 (2 q'_1 - w'),$$

and if all the spans are equal,

$$q''_2 = w'' - 4 q'_1 + w'.$$

For the moments of strain over the first and second piers, we have

$$M' = \frac{q'_1 l'^2}{4} \text{ and } M'' = \frac{q''_2 l''^2}{4}.$$

To find the reactions at the point of support, we must ascertain the distribution of the load on the piers. Let R′ represent that part of the load on the first span which is borne by the first support, R″ the same for the second support, R''', R^{IV}, the portions of the second span borne by the second and third supports, and R^{V}, R^{VI}, the same quantities for the third span on the third and fourth supports, then

$$R' = \frac{w' l'}{2} - \frac{M'}{l'} \qquad R'' = \frac{w' l'}{2} + \frac{M'}{l}$$

$$R''' = \frac{w'' l''}{2} + \frac{M' - M''}{l''} \qquad R^{IV} = \frac{w'' l''}{2} + \frac{M'' - M'}{l''}$$

$$R^{V} = \frac{w''' l'''}{2} + \frac{M''}{l'''} \qquad R^{VI} = \frac{w''' l'''}{2} - \frac{M''}{l'''}$$

If the girder is symmetrical both in form and load,

$$R' = R^{VI} \quad R'' = R^{V} \text{ and } R''' = R^{IV}$$

also,

$$M' = M'' \therefore R''' = R^{IV} = \frac{w'' l''}{2}.$$

The general equation to the curve of moments is,

$$M_0 = M' + \frac{w x^2}{2} - R x$$

in which the values of M′ and R must be put according to the span to which the equation is intended to apply, thus we obtain, for the first span,

$$M_0 = \frac{w' x^2}{2} - \frac{w' l' x}{2} + \frac{M' x}{l}$$

$$= \left\{ \frac{w' x}{2} - \frac{w' l'}{2} + \frac{M'}{l} \right\} x$$

for the second,

$$M_0 = M' + \frac{w'' x^2}{2} - \frac{w'' l'' x}{2} - \frac{M' x - M'' x}{l''}$$

$$= M' + \left\{ \frac{w'' x}{2} - \frac{w'' l''}{2} - \frac{M'}{l''} + \frac{M''}{l''} \right\} x$$

and for the third,

$$M_0 = M'' + \frac{w''' x^2}{2} - \frac{w''' l''' x}{2} - \frac{M'' x}{l'''}$$

$$= M'' + \left\{ \frac{w''' x}{2} - \frac{w''' l'''}{2} - \frac{M''}{l'''} \right\} x$$

in each span x is measured from the left hand support.

If all the spans are of equal length, and are equally loaded, we find,

$$q_1' = \frac{6w}{15}$$

$$q''_2 = 2w - \frac{24w}{15} = \frac{6w}{15}$$

In which case,

$$M' = M'' = \frac{q_1' l^2}{4} = \frac{w\, l^2}{10}$$

If the centre span only has a moving load, let

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

then,

$$q_1' = \frac{9\, w'}{15} = q_2''$$

therefore,

$$M' = M'' = \frac{9_1'\, l^2}{4} = \frac{9\, w'\, l^2}{60}$$

If the outer spans only support a moving load,

$$w' = w''' = 2\, w''$$

then,

$$q_1' = \frac{9\, w''}{15} = q^{2''}$$

therefore,

$$M' = M'' = \frac{9_1'\, l^2}{4} = \frac{9\, w''\, l^2}{60}.$$

If the first and second spans be loaded,

$$w' = w'' = 2\, w'''$$

$$q'_1 = \frac{13\, w'''}{15}$$

$$q''_2 = \frac{8\, w''}{15}$$

therefore,

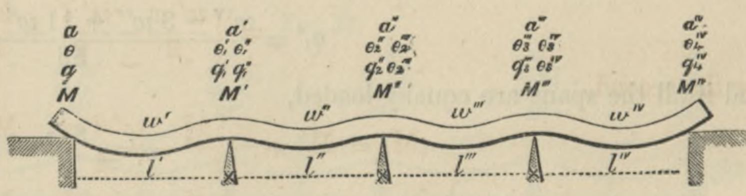
$$M' = \frac{q_1'\, l^2}{4} = \frac{13\, w'''\, l^2}{60}$$

$$M'' = \frac{q''_2\, l^2}{4} = \frac{8\, w'''\, l''}{60}.$$

When any simple relation, such as the above, exists between the different values of w, it will be found most convenient to simplify them as above, previous to making the subsequent calculations.

Girders of four spans now require our attention, and in the application of our general formulæ to this case we shall use the same notations as above for the first three, with the addition of w^{IV}, l^{IV}, $tan\ a^{IV}$, M^{IV}, and corresponding values of q and θ for the fourth span.

Fig. 26.



We will also make,

$$\frac{l'''}{l^{IV}} = m_3$$

Proceeding, as in the two former cases, we first obtain the following equations:—

For the first span,

$$M' = \tfrac{1}{4}\, q\, l'^2 = o \ldots\ldots (1) \quad M'' = \tfrac{1}{4}\, q'_1\, l'^2 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$tan\ a = \frac{l'^3\, \theta}{24\, \varepsilon} \ldots\ldots\ldots\ldots (3)\ tan\ a' = \frac{l'^3\, \theta'_1}{24\, \varepsilon} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$q'_1 \;\; = w' - 2\, q - \theta \ldots (5) \quad \theta'_1 = w' - 3\, q - 2\, o \ldots\ldots\ldots (6)$$

For the second span,

$$M' = \tfrac{1}{4}\, q''_1\, l''^2 \ldots\ldots\ldots (7) \quad M'' = \tfrac{1}{4}\, q''_2\, l''^2 \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$tan\ a' = \frac{l''^3\, \theta''_1}{24\, \varepsilon} \ldots\ldots\ldots\ldots (9)\ tan\ a'' = \frac{l''^3\, \theta''_1}{24\, \varepsilon} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$q''^2 \;\; = w'' - 2\, q''_1 - \theta''_1\, (11) \quad \theta''_2 = w'' - 3\, q''_1 - 2\, \theta''_1 \ldots (12)$$

For the third span,

$$M'' = \tfrac{1}{4}\, q'''_2\, l'''^2 \ldots\ldots\ldots (13) \quad M''' = \tfrac{1}{4}\, q'''_3\, l'''^2 \ldots\ldots\ldots\ldots (14)$$

$$tan\ a'' = \frac{l'''^3\, \theta_2'''}{24\ \varepsilon} \ldots\ldots\ldots (15) \quad tan\ a''' = \frac{l'''^3\, \theta'''_3}{24\ \varepsilon} \ldots\ldots\ldots\ldots (16)$$

$$q'''_3 = w''' - 2\, q'''_2 - \theta'''_2\, (17) \quad \theta'''_3 = w''' - 3\, q'''_2 - 2\, \theta'''_2\, (18).$$

And for the fourth span,

$$M''' = \tfrac{1}{4}\, q_3^{IV}\, l^{IV\,2} \ldots\ldots\ldots (19) \quad M^{IV} = \tfrac{1}{4}\, q^{IV}_4\, l^{\,IV\,2} = o \ldots\ldots\ldots (20)$$

$$tan\ a''' = \frac{l^{IV\,3}\, \theta^{IV}}{24\ \varepsilon}{}^{3} \ldots\ldots\ldots (21) \quad tan\ a^{IV} = \frac{l^{IV\,3}\ \theta^{\ IV}_4}{24\ \varepsilon} \ldots\ldots\ldots\ldots (22)$$

$$q^{IV}_4 = w^{IV} - 2 q^{IV}_3 - \theta^{IV}_3 = o\ (23) \quad \theta^{IV}_4 = w^{IV} - 3\, q^{IV}_3 - 2\, \theta^{IV}_3\ (24).$$

By equating various expressions in the above series of equations, as in the previous cases, we obtain the following values:

$$\begin{aligned}
\theta'_1 &= 2\, q'_1 - w' \\
q''_2 &= w'' - 2\, q'_1\, m^2 - m^3\, \theta'_1 \\
\theta''_2 &= w'' - 3\, q'_1\, m^2 - 2\, m^3\, \theta'_1 \\
q'''_3 &= w''' - 2\, q''_2 m^2_2 - m^3_2\, \theta''_2 \\
\theta'''_3 &= w''' - 3\, q''_2 m^2_2 - 2\, m^3_2\, \theta''_2 \\
q^{IV}_4 &= w^{IV} - 2 q'''_3 m^2_3 - m^3_3\, \theta'''_3 = o \ \ldots\ldots\ (a) \\
\theta^{IV}_4 &= w^{IV} - 3 q^{IV}\, m^2_3 - 2\, m^3_3\, \theta'''_3.
\end{aligned}$$

If we now replace in the equation (a) q'''_3 and θ'''_3 by their values, and in the expression thus obtained, replace q''_2 and θ''_2 by their values, and continue replacing until we arrive at an equation containing only q'_1, and the various w's and m's, we shall find that

$$q'_1 = \frac{w^{IV} - m_3^2\,(2 + m\,3)\, w''' + m_2^2\, m_3^2\,(4 + 2\, m_2 + 2\, m_2 m_3 + 3\, m_3)\, w'' + m^3\, m_2^2\, m_3^2\,(4 + 4\, m_2 + 4\, m_2\, m_3 + 3\, m_3)\, w'}{m^2\, m_2^2\, m_3^2\,(8 + 8\, m + 8\, m\, m_2 + 8\, m\, m_2 m_3 + 6\, m_2 + 6\, m_3 + 6\, m\, m_3 + 6\, m_2\, m_3)}$$

If the girder be symmetrical with regard to the centre pier, then $m_3 = \dfrac{1}{m}$ and $m_2 = 1$

and the above becomes

$$q_1' = \frac{m^3\, w^{IV} - (2\, m + 1)\, w''' + (6\, m + 5)\, w'' + m^3\,(8\, m + 7)\, w'}{4\, m^2\,(4\, m^2 + 7\, m + 3)}$$

When all the spans are equal,

$$m = m_2 = m_3 = 1$$

and therefore,

$$q_1' = \frac{w^{IV} - 3\, w''' + 11\, w'' + 15\, w'}{56}$$

and if all the spans are equally loaded,

$$q_1' = \frac{3\, w}{7}$$

The values of q''_2 and q'''_3 may be found from the equations,

$$q''_2 = w'' - 2\, q_1'\, m^2 - m^3\,(2 q'_1 - w') \text{ and}$$

$$q'''_3 = w''' - 2\, q''_2\, m_2^2 - m_2^3\, \{\, w'' - 3\, q'_1\, m^2 - 2\, m^3\,(2\, q'_1 - w')\, \}$$

and those of M′, M″, M‴, from the equations

$$M' = \tfrac{1}{4}\, q'_1\, l'^2,\ M'' = \tfrac{1}{4}\, q''_2\, l''^2,\ M''' = \tfrac{1}{4}\, q''_3\, l'''^2.$$

We may further simplify the value of q'''_3 by considering the girder as symmetrical when,

$$q''_3 = w''' - w'' - 2\, q''_2 + 3\, q'_1\, m^2 + 4\, m^3\, q'_1 - 2\, m^3\, w'$$

when all the spans are equal

$$q''_2 = \frac{3\, w''' + 3\, w'' - w' - w^{IV}}{14}$$

$$q'''_3 = \frac{11\, w''' - 3\, w'' + w' + 15\, w^{IV}}{56}$$

If all the spans are loaded equally

$$q''_2 = \frac{2w}{7} \text{ and}$$

$$q'''_3 = \frac{3w}{7} = q'_1.$$

The general equation for the reaction on any pier produced by any span, is

$$R = \frac{w\,l}{2} + \frac{M' - M''}{l}$$

where M′ is the moment of strain over the pier for which the reaction is taken, and M″ the same for the point of support at the other end of the span.

The equation to the curve of moments is $M_0 = M' + \frac{w\,x^2}{2} - R'\,x$

R′ being the reaction on the first support.

Applying this to the four spans we have for the first

$$M_0 = \frac{w'\,x^2}{2} - \frac{w'\,l'\,x}{2} + \frac{M'\,x}{l'}$$

$$= \left\{ \frac{w'\,x}{2} - \frac{w'\,l'}{2} + \frac{M'}{l'} \right\} x$$

for the second,

$$M_0 = M' + \frac{w''\,x^2}{2} - \frac{w''\,l''\,x}{2} - \frac{M'x - M''x}{l''}$$

$$= M' + \left\{ \frac{w''\,x}{2} - \frac{w''\,l''}{2} - \frac{M' - M''}{l''} \right\} x$$

for the third,—

$$M_0 = M'' + \frac{w'''\,x^2}{2} - \frac{w'''\,l'''\,x}{2} - \frac{M''\,x - M'''\,x}{l'''}$$

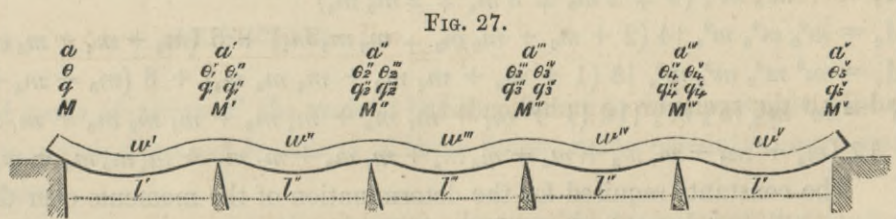
$$= M'' + \left\{ \frac{w'''\,x}{2} - \frac{w'''\,l'''}{2} - \frac{M'' - M'''}{l'''} \right\} x$$

and for the fourth,

$$M_0 = M''' + \frac{w^{IV}\,x^2}{2} - \frac{w^{IV}\,l^{IV}\,x}{2} - \frac{M'''\,x}{l^{IV}}$$

$$= M''' + \left\{ \frac{w^{IV}\,x}{2} - \frac{w^{IV}\,l^{IV}}{2} - \frac{M'''}{l^{IV}} \right\} x$$

Fig. 27.



We will now pass to the last special case which we shall here consider, viz., that of a girder of five spans. We shall adopt the foregoing notations for the first four spans with the addition of l^V $tan\ \alpha^V$ M^V w^V $m_4 = \frac{l^{IV}}{l^V}$ and corresponding values of q and θ.

Our preliminary equations obtained as before, will be:—

For the first span,

$$M \quad = \tfrac{1}{4}\,q\,l'^2 = o \;\ldots\; (1) \qquad M' \quad = \tfrac{1}{4}\,q'_1\,l'^2 \;\ldots\ldots\ldots\ldots\ldots\; (2)$$

$$tan\ \alpha = \frac{l'^3\,\theta}{24\,\varepsilon} \;\ldots\ldots\; (3) \qquad tan\ \alpha' = \frac{l'^3\,\theta'_1}{24\,\varepsilon} \;\ldots\ldots\ldots\ldots\ldots\; (4)$$

$$q'_1 \quad = w' - 2\,q - \theta \;\; (5) \qquad \theta'_1 \quad = w' - 3\,q - 2\,o \;\ldots\ldots\; (6)$$

For the second span,

$$M' \quad = \tfrac{1}{4}\,q''_1\,l''^2 \;\ldots\ldots\; (7) \qquad M'' \quad = \tfrac{1}{4}\,q''_2\,l''^2 \;\ldots\ldots\ldots\ldots\; (8)$$

$$tan\ \alpha' = \frac{l''^3\,\theta''_1}{24\,\varepsilon} \;\ldots\ldots\; (9) \qquad tan\ \alpha'' = \frac{l''^3\,\theta''_2}{24\,\varepsilon} \;\ldots\ldots\ldots\ldots\; (10)$$

$$q''_2 \quad = w'' - 2q''_1 - \theta''_1\,(11) \qquad \theta''_2 \quad = w'' - 3\,q''_1 - 2\,\theta''_1 \ldots\ldots (12)$$

For the third span,

$$M'' = \tfrac{1}{4} q'''_2 \; l'''_2 \ldots (13) \quad M''' = \tfrac{1}{4} q'''_3 \, l'''^2 \ldots\ldots\ldots (14)$$

$$tan\, a'' = \frac{l'''^3 \, \theta'''_2}{24\, \varepsilon} \ldots\ldots (15) \quad tan\, a''' = \frac{l'''^3 \, \theta'''_3}{24\, \varepsilon} \ldots\ldots\ldots (16)$$

$$q'''_3 = w''' - 2\, q'''_2 - \theta'''_2 \; (17) \quad \theta'''_3 = w''' - 3\, q'''_2 - 2\, \theta'''^2 \ldots (18).$$

For the fourth span,

$$M''' = \tfrac{1}{4} q^{IV}_3 \, l^{IV\,2} \ldots (19) \quad M^{IV} = \tfrac{1}{4} q^{IV}_4 \, l^{IV\,2} \ldots\ldots\ldots (20)$$

$$tan\, a''' = \frac{l^{IV\,3} \, \theta^{IV}_3}{24\, \varepsilon} \ldots (21) \; tan\, a^{IV} = \frac{l^{IV\,3} \, \theta^{IV}_4}{24\, \varepsilon} \ldots\ldots\ldots (22)$$

$$q^{IV}_4 = w^{IV} - 2\, q^{IV}_3 - \theta^{IV}_3 \; (22) \quad \theta^{IV}_4 = w^{IV} \; 3\, q^{IV}_3 - 2\, \theta^{VI}_3 \ldots (24)$$

And for the fifth span,

$$M^{IV} = \tfrac{1}{4} \; q^{V}_3 l^{V2} \ldots\ldots (25) \quad M^{V} = \tfrac{1}{4} q^{V}_5 \, l^{V\,2} = 0 \ldots\ldots (26)$$

$$tan\, a^{IV} = \frac{l^{V3} \, \theta^{V}_4}{24\, \varepsilon} \ldots\ldots\ldots (27) \; tan\, a^{V} = \frac{l^{V3} \, \theta^{V}_5}{24\, \varepsilon} \ldots\ldots\ldots (28)$$

$$q^{V}_5 = w^{V} - 2\, q^{V}_4 - \theta^{V}_4 = 0 \; (29) \; \theta^{V}_5 = w^{V} - 3\, q^{V}_4 - 2\, \theta^{V}_4 \ldots (30).$$

By equating various expressions from the above series, we obtain the following equations:—

$$\begin{aligned}
\theta'_1 &= 2\, q'_1 - w' \\
q''_2 &= w'' - 2\, q'_1 \; m^2 - 2\, m^3 \; \theta'^1 \\
\theta''_2 &= w'' - 3\, q'_1 \; m^2 - 2\, m^3 \; \theta'_1 \\
q'''_3 &= w''' - 2\, q''_2 \; m^2_2 - \; m^3_2 \, \theta''_2 \\
\theta'''_3 &= w''' - 3\, q''_2 \; m^2_2 - 2\, m^3_2 \, \theta''_2 \\
q^{IV}_4 &= w^{IV} - 2\, q'''_3 \, m^2_3 - \; m^3_3 \, \theta'''_3 \\
\theta^{IV}_4 &= w^{IV} - 3\, q'''_3 \, m^2_3 - 2\, m^3_3 \, \theta'''_3 \\
q^{V}_5 &= w^{V} - 2\, q^{IV}_4 \, m^2_4 - \; m^3_4 \, \theta^{IV}_4 = (e) \\
\theta^{V}_5 &= w^{V} - 3\, q^{IV}_4 \, m^2_4 - 2\, m^3_4 \, \theta^{IV}_4
\end{aligned}$$

By replacing the values of q and θ in (e) and continually replacing them in the resulting equations, until we arrive at an expression containing only q'_1 and the various values of m and w, we shall find that,

$$q'_1 = \frac{-w^{V} + M_4\, w^{IV} + M_3\, w''' + M_2\, w'' + M_1\, w'}{N}$$

in which,

$M_4 = m^2_4 \, (2 + m_4)$

$M_3 = - \, m^2_3 \, m^2_4 \, (4 + 2\, m_3 + 3\, m_4 + 2\, m_3\, m_4)$

$M_2 = m^2_2 \, m^2_3 \, m^2_4 \, \{4\, (2 + m_2 + m_2 m_3 + m_2 m_3 m_4) + 6\, (m_3 + m_4 + m_3 m_4) + 3\, m_2\, m_4\}$

$M_1 = m^3 \, m^2_2 \, m^2_3 \, m^2_4 \, \{8\, (1 + m_2 + m_2\, m_3 + m_2\, m_3\, m_4) + 6\, (m_3 + m_4 + m_2 m_4 + m_3\, m_4)\}$

$N = m^2 \, m^2_2 \, m^2_3 \, m^2_4 \, \{16\, (1 + m_1 + m_1\, m_2 + m_1\, m_2 + m_1\, m_2\, m_3 + m_1\, m_2\, m_3\, m_4) +$
$12\, (m_2 + m_3 + m_2 m_3 + m_4 + m_3 m_4 + m_1 m_3 + m_1\, m_4 + m_1 m_3 m_4 + m_2 m_3 m_4 + m_1\, m_2 m_4) + 9\, m_2\, m_4\}.$

The constants required for the determination of the moments over the other piers will be,

$q''_2 = w'' - 2\, q_1'\, m^2 - m^3\, (2 q_1' - w')$

$q'''_3 = w''' - 2\, q''_2\, m^2_2 - m^3_2 \, \{w'' - 3\, q'_1 m^2_1 - 2\, m^3_1 \, (2\, q'_1 - w')\}$

$q^{IV}_4 = w^{IV} - 2\, q'''_3 \, m^2_3 - m^3_3 \, \{\, w''' - 3\, q''_2 \, m^2_2 - 2\, m^3_2 \, [w'' - 3\, q'_1 \, m^2_1 - 2\, m^3_1 \, (2\, q'_1 - w')\,]\}$

And the values of the moments over the pier may be found from the equations,

$$M' = \tfrac{1}{4}\, q'_1 \, l'^2 \; M'' = \tfrac{1}{4}\, q''_2 \, l''^2 \; M''' = \tfrac{1}{4}\, q'''_3 \, l'''^3 \; M^{IV} = \tfrac{1}{4}\, q^{IV}_4 \, l^{IV\,2}$$

When the girder is symmetrical,

$$m_4 = \frac{1}{m_1} \quad m_2 = \frac{1}{m_3}$$

and,

$M_4 = m^3\, (2\, m + 1)$

$M_3 = - \, m^3_3 \, (4\, m + 2\, m_3\, m + 3 + 2\, m_3)$

$M_2 = 4\, (2\, m.\, m_3 + m + m.\, m_3 + m_3) + 6\, m_3\, (m.\, m_3 + 1 + m_3) + 3$

$M_1 = 8\, m^3\, (m + 2\, m_3\, m + m_3) + 6\, m^3\, (m.\, m^2_3 + m_3 + 1 + m^2_3)$

$N = 16\, m^3\, (2\, m_1\, m_3 + m + 2\, m_3) + 12\, m^2\, (2\, m + 2\, m\, m^2_3 + 2\, m.\, m_3 + 2\, m_3 + m^2_3 + m^2\, m^2_3) + 9\, m_2$

And if all the spans are of equal length,

$$q'_1 = \frac{-w^{\mathrm{V}} + 3\,w^{\mathrm{IV}} - 11\,w''' + 41\,w'' + 56\,w'}{209} = q^{\mathrm{IV}}_4$$

$$q''_2 = \frac{45\,w'' + 44\,w''' + 4\,w^{\mathrm{V}} - 12\,w^{\mathrm{IV}} - 15\,w'}{209} = q'''_3$$

and if the spans be all equally loaded,

$$q'_1 = q^{\mathrm{IV}}_4 = \frac{88\,w}{209}$$

$$q''_2 = q'''_3 = \frac{66\,w}{209}.$$

To find the reaction on any pier over which the moment is M′, the reaction produced by any span l', is

$$\mathrm{R} = \frac{w\,l'}{2} + \frac{\mathrm{M}' - \mathrm{M}''}{l'}.$$

The general equation to the curve, for any span where M′ is the moment over the first support, and M″ that over the second, is

$$\mathrm{M}_0 = \mathrm{M}' + \frac{w\,x^2}{2} - \left\{\frac{w\,l}{2} + \frac{\mathrm{M}' - \mathrm{M}''}{l}\right\}x$$

in which M′ M″ must be replaced according to the span to which the equation is to be applied; thus

$$\left.\begin{array}{ll}
\text{For the first span.........} & \mathrm{M}' = o_1 \quad \mathrm{M}'' = \mathrm{M}' \\
\text{For the second span......} & \mathrm{M}' = \mathrm{M}' \quad \mathrm{M}'' = \mathrm{M}'' \\
\text{For the third span ......} & \mathrm{M}' = \mathrm{M}'' \quad \mathrm{M}'' = \mathrm{M}''' \\
\text{For the fourth span ......} & \mathrm{M}' = \mathrm{M}''' \quad \mathrm{M}'' = \mathrm{M}^{\mathrm{IV}} \\
\text{For the fifth span.........} & \mathrm{M}' = \mathrm{M}^{\mathrm{IV}} \quad \mathrm{M}'' = o
\end{array}\right\}\ \text{in our notations as above.}$$

It is unnecessary to apply our investigations to girders of more than five spans continuous, as such a case will seldom occur; and, when they do occur, it will be preferable to modify the last formulæ, instead of calculating others, whereby much time and labour would be consumed.

CHAPTER V.

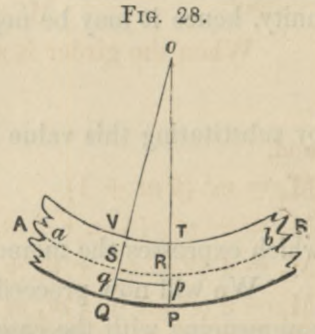
DEFLECTION.

Having explained the nature and mode of action of the various bending forces to which straight girders generally are subject, we will now examine the visible effects of these forces, made evident to our senses by the deflection of the loaded girders.

Our object is at present to determine some relation existing between the intensity of the bending force on the girder and its curvature, whereby, one of these quantities being given, the other may be found.

Let A B, in Fig. 28, represent a portion of a deflected beam, of which the neutral surface is at $a\ b$. Let P T and Q V be transverse sections of the beam infinitely near to each other, and perpendicular to the neutral surface at the points R and S. Let o be the centre of curvation of the beam.

Fig. 28.



Let a R $= x$; S R $= \Delta\, x$, and consider the lamina P Q T V to be made up of fibres parallel to S R, then $\Delta\, x$ will represent the length of any of these fibres before the beam is deflected, since the length of the neutral axis has remained unchanged.

Let $\delta\, x$ represent the quantity by which the fibre $p\ q$ has been elongated by the deflection of the beam; then the length of that fibre will be represented by $\Delta\, x + \delta\, x$.

If E represent the modulus of elasticity of the material of which the beam is composed, and Δk the area of an infinitely small element of the area of the section, then the force which must have operated to produce the elongation δx in a fibre whose original length was Δx, will be represented by

$$E \frac{\delta x}{\Delta x} \Delta k.$$

Let the radius o R be represented by R and the distance Rp by z, then by similar triangles

$$\frac{o\,p}{o\,R} = \frac{p\,q}{R\,S} \text{ or } \frac{R + z}{R} = \frac{\Delta x + \delta x}{\Delta x} \text{ hence}$$

$$1 + \frac{z}{k} = 1 + \frac{\delta x}{\Delta x} \therefore \frac{z}{k} = \frac{\delta x}{\Delta x}$$

Substituting this value in the expression which represents the pressure that must have operated to produce the elongation of the fibre $p\,q$ and representing this pressure by Δ P, we have

$$\Delta P = E \frac{\delta x}{\Delta x} \Delta k = \frac{E}{R} z. \; \Delta k$$

and the moment of this force about the neutral axis, is

$$= \frac{E}{R} z^2 \, \Delta k$$

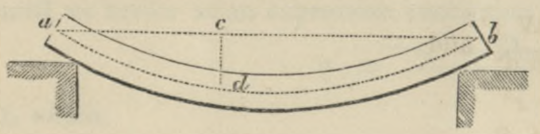
If we integrate this expression, and represent the sum of the moments of resistance of the fibres by M, we find that

$$M = \frac{E\,I}{R}$$

where I represents the moment of inertia of the section.

This formula gives the moment of the elastic force at any section of the beam in terms of the radius of curvature at that point, but as this would be practically useless, we must transform this expression into one involving the elements of the curve of deflection.

Fig. 29.



Let $a\,c\,b$ represent the original position of the neutral axis of a beam, and $a\,d\,b$ its position when the beam is deflected.

$a\,c = x$ and $c\,d$ which represents the deflection of this point $= y$. By the application of the principles of the differential calculus to the circle we find that if the curve is concave to the axis of x and R is the radius of curvature.

$$R = \frac{\left(1 + \frac{d\,y^2}{d\,x^2}\right)^{\frac{3}{2}}}{-\frac{d^2 y}{d\,x^2}}$$

$$\therefore \frac{1}{R} = -\frac{d^2 y}{d\,x^2} \left(1 + \frac{d\,y^2}{d\,x^2}\right)^{\frac{3}{2}}$$

But $\frac{d\,y}{d\,u}$ represents the tangent to the angle contained between the axis of x and a tangent to the curve, and as the deflection of beams is usually very small $\left(\frac{d\,y}{d\,x}\right)^2$ is inconsiderable when compared with unity, hence it may be neglected, then

$$\frac{1}{R} = -\frac{d^2 y}{d\,x^2}$$

by substituting this value in the expression for the moment of the elastic forces, we obtain

$$M = -E\,I \frac{d^2 y}{d\,x^2}$$

which expresses the moment of the elastic forces in terms of the co-ordinates of the deflection curve.

We will now proceed to find equations to the curves of deflection for beams variously circumstanced, commencing with the case of a beam fixed at one end, free at the other, and loaded with a weight W at a point distant l from the fixed extremity of the beam. Let $\mathrm{E\,I} = \varepsilon$, then because the moment of resistance is equal to the moment of strain at any point distant x from end of beam,

$$-\varepsilon \frac{d^2 y}{d x^2} = \mathrm{W}\, x$$

integrating this equation, we obtain

$$\varepsilon \frac{d y}{d x} = -\frac{\mathrm{W}\, x^2}{2} + c,$$

when $x = l \, \frac{d y}{d x} = o$ as the beam is horizontal at its fixed extremity, there $c = \frac{\mathrm{W}\, l^2}{2}$

again integrating, we find

$$\varepsilon\, y = -\frac{\mathrm{W}\, x^3}{6} + \frac{\mathrm{W}\, l^2\, x}{2} + c$$

the constant in this case, is $-\frac{\mathrm{W}\, l^3}{3}$, for when $x = l$, $y = o$, therefore the equation to the deflection curve, is

$$y = \frac{\mathrm{W}}{\varepsilon} \left\{ \frac{l^2\, x}{2} - \frac{l^3}{3} - \frac{x^3}{6} \right\}$$

the sign minus signifies that y is measured downwards from the axis of x.

Let the same beam now be supposed to be loaded with an equally distributed weight of w per lineal unit

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

$$\varepsilon \frac{d y}{d x} = -\frac{w\, x^3}{6} + \frac{w\, l^3}{6}$$

$$y = \frac{w}{\varepsilon} \left\{ \frac{l^3\, x}{6} - \frac{x^4}{24} - \frac{l^4}{8} \right\}$$

by substituting the value of the moment of inertia we obtain the deflection of any form of beam, thus for a square beam, we have

$$y = -\frac{\mathrm{W}\, x^3}{6\, \mathrm{E}_3} \times \frac{12}{b\, d^3} = -\frac{2\, \mathrm{W}\, x^3}{\mathrm{E}\, b\, d^3} \text{ and}$$

$$y = -\frac{w\, x}{12\, \mathrm{E}} \times \frac{12}{b\, d^3} = -\frac{w\, x^3}{\mathrm{E}\, b\, d^3}$$

where b is the breadth and d the depth of the beam. For circular beams, if $d =$ the diameter and a the sectional area of the beam,

$$y = -\frac{\mathrm{W}\, x^3}{6\, \mathrm{E}} \times \frac{16}{a\, d^2} = -\frac{8\, \mathrm{W}\, x^3}{3\, \mathrm{E}\, a\, d^2} \text{ and}$$

$$y = -\frac{w\, x^3}{12\, \mathrm{E}} \times \frac{16}{a\, d^2} = -\frac{4\, w\, x^3}{3\, \mathrm{E}\, a\, d}.$$

We will now apply our formulæ to girders supported at both ends and loaded at the centre. In this case,

$$-\varepsilon \frac{d^2 y}{d x^2} = -\frac{\mathrm{W}}{2}\, x$$

$$\varepsilon \frac{d y}{d x} = \frac{\mathrm{W}}{4}\, x^2 + c$$

when $\frac{d y}{d x} - = o\ x = \frac{l}{2}$ for at that point y is a maximum,

$$\therefore \mathrm{C} = -\frac{\mathrm{W}\, l^2}{16}$$

$$\varepsilon \frac{d y}{d x} = \frac{\mathrm{W}}{4} \left\{ x^2 - \frac{l^2}{4} \right\}$$

$$\varepsilon\, y = \frac{\mathrm{W}}{4} \left\{ \frac{x^3}{3} - \frac{l^2\, x}{4} \right\}$$

which is the equation to the curve of deflection, which at the centre of the span becomes

$$y = \frac{W\,l^3}{48\,\epsilon}.$$

We shall now apply this last formula to beams of various sections.

For a square beam, if b = breadth and d = depth

$$y = -\frac{W\,l^3}{48\,E} \times \frac{12}{b\,d^3} = -\frac{W\,l^3}{4\,E\,b\,d^3}.$$

For circular beams of a = area, and d = diameter,

$$y = -\frac{W\,l^3}{48\,E} \times \frac{16}{a\,d^2} = -\frac{W\,l^3}{3\,E\,a\,d^2}.$$

And for any other form of section the value of y may be found in a similar manner.

Let the load be now distributed uniformly over the whole length of the beam, then

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

$$\epsilon\frac{d\,y}{d\,x} = -\frac{w}{2}\left\{\frac{x^3}{3} - \frac{l\,x^2}{2}\right\} + c$$

but when $\frac{d\,y}{d\,x} = o,\ x = \frac{l}{2}$

$$\therefore c = \frac{w}{2}\left\{\frac{l^3}{24} - \frac{l^3}{8}\right\} = -\frac{w\,l^3}{24}$$

therefore, $\epsilon\frac{d\,y}{d\,x} = -\frac{w}{2}\left\{\frac{x^3}{3} - \frac{l\,x^2}{2} + \frac{l^3}{12}\right\}$

by integrating again,

$$\epsilon\,y = -\frac{w}{2}\left\{\frac{x^4}{12} - \frac{l\,x^3}{6} + \frac{l^3 x}{12}\right\}$$

$$= -\frac{w}{24}\left\{x^4 - 2\,l^3 x + l^3 x\right\}$$

which is the equation to the curve of deflection.

The deflection at the centre will be,

$$\epsilon\,y = -\frac{5\,w\,l^4}{384}$$

this will become for rectangular beams,

$$y = -\frac{5\,w\,l^4}{32.\,E\,b\,d^3}$$

and for circular beams,

$$y = -\frac{5.\,w\,l^4}{24.\,E\,a\,d^2}$$

The radius of curvature of the beam will not be actually the same for every point in the span, but will be least at the centre, and increase towards the extremities, at which points it becomes infinite.

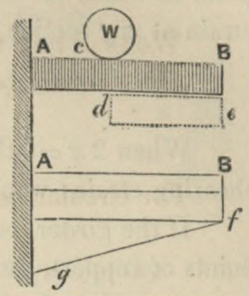
The above formulæ are, however, sufficiently accurate for all practical purposes, to any of which they may be applied according to the above method.

CHAPTER VI

ON SHEARING STRAIN.

We have hitherto confined our attention to that action of transverse strain, which produces flexure of the body acted upon, which is usually called horizontal strain, on account of its being resolved into forces acting directly upon the horizontal laminæ of the beam. We are now about to investigate another effect of transverse strain.

Fig. 30.



Let A B, Fig. 30, represent a beam, which we will assume to be composed of a number of sections, shown by the vertical lines. If a load W be placed upon the beam, it will evidently tend to cause these sections to slide one over the other, whereby a portion of the beam would be forced away from the other, as shown at the dotted lines, *d e*. As this force acts directly without modification of any kind whatever, it must necessarily affect the material in proportion to its intensity or the weight of the load, hence, in the present case, the shearing force on any section of the beam between *c d* over which the load is placed, and A the point at which the beam is fixed to the wall is exactly equal to the weight W, and at A the weight of the load is transmitted to the point of support. Let us now take another case where the load, instead of being concentrated over one section is uniformly distributed, being equal to w per lineal unit of the beam's length, then commencing at the outer extremity of the beam we find that at the section of B*f* no shearing strain exists, but if we take any point distant, x from B*f*, we have there a shearing strain equal to the load carried by such section, which is,

$$= w\,x$$

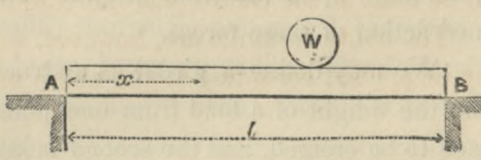
From this it appears, that the shearing strain increases under an uniform load towards the point of support of direct ratio to x if $x = l$, the length of the beam then will be the shearing strain $= w\,l$, the total load on the beam, which is at this point transmitted to the pier, hence to lay down a diagram of the shearing strain, all that is necessary consists in making the line at the point of support, equal to the total load on the beam, and joining the extremities of this line and the beam by the line *g f*, then the shearing strain at any point will be represented by an ordinate to the straight line at that point.

From the foregoing remarks, we at once observe that the shearing strain at the point of support is equal to the weight supported, or to the reaction of the support, and if the load producing this strain be concentrated on one point, the shearing strain continues unchanged on every section from the point of support to the load, but if, on the contrary, the load is uniformly distributed over the whole length of the beam, the shearing strain gradually diminishes, being at any section equal to the amount of load transmitted through that section to the pier which will be equal to the load beyond such section, or to the total load on the pier, minus the load between the pier and the section at which the shearing strain is taken, hence we have an easy method of finding the shearing strain at any point in the beam, when the manner in which the load is applied is known, and also the amount of weight supported by the pier.

We will now, by the means we have just pointed out, examine the amount of shearing strain on girders variously circumstanced.

Let A, B, Fig. 31, represent a beam supported at both ends, and loaded at any point distant x from the pier A, with a weight W, it is required to find the shearing strain at B, but because the load is concentrated upon one point in the beam, therefore the shearing strain at any section between B and W is equal to that at W, or if S_B equal the shearing strain at any point, between the load and the pier B.

Fig. 31.



$$S_B = W\frac{x}{l}.$$

It may similarly be shown that if S_A represent the shearing strain at any section between the pier A, and the load W,

$$S_A = W - W\frac{x}{l}.$$

Let the load be now supposed to be uniformly distributed and equal to w per lineal unit, then the total load will be $= w\,l$, and this will be equally divided between the two points of support, therefore the shearing strain at each point of support is,

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

But we have shown above, that the strain at any section is equal to the load transmitted through such section to the pier, or to the total load on the pier, minus the load included between the pier and the point at which the strain is taken, let x represent the distance between these points, and S the shearing strain at any section, then will

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

When $2\,x = l$ the shearing strain is nothing, and this occurs at the centre of the span, hence the shearing strains would be the same if the beam were divided in the centre.

If the girder be fixed at both extremities the load will still be equally distributed between the two points of support, therefore the shearing strains will remain the same as they were when the ends of the beam were free, hence the shearing strain is not affected by conditions which alter the horizontal strain, unless they also alter the distribution of weight upon the points of support.

Let the girder be supported at one end and fixed at the other, let $M' =$ the moment of strain over the pier, $R' =$ the reaction over the same pier, and $R'' =$ the reaction over the opposite pier, then w being the weight per lineal unit, and l the span.

$$R' = \frac{w\,l}{2} + \frac{M'}{l} \quad R'' = \frac{w\,l}{2} - \frac{M'}{l}.$$

These quantities will be equal to the shearing strains at the points of support. The strain at any point distant x from the pier over which M' exists, is,

$$S = \frac{w\,l}{2} + \frac{M'}{l} - w\,x$$

which is true for any value of x, but it will be found that at the point of greatest *horizontal* strain the shearing strain will always be nothing. If x is measured from the pier which supports the free end of the girder, then,

$$S = \frac{w\,l}{2} - \frac{M'}{l} - w\,x.$$

In a similar manner we may readily find the shearing strain at any part of a continuous girder, having previously determined the weight or reaction on the points of support, which may be done by means of the equation,

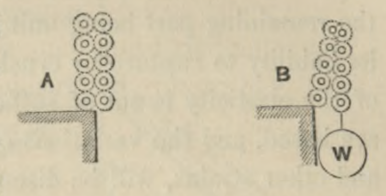
$$R' = \frac{w\,l}{2} + \frac{M' - M''}{l}$$

where M' is the moment over the pier for which the reaction is required, and M'', that over the pier at the other end of the span, the weight thus obtained will not be the total load on the pier, but only that brought upon it by the span under consideration.

As we have in the present and foregoing chapters mentioned two distinct kinds of strain as exercising an influence upon materials subject to a transverse strain, it may be desirable here to examine the combined action of these forces.

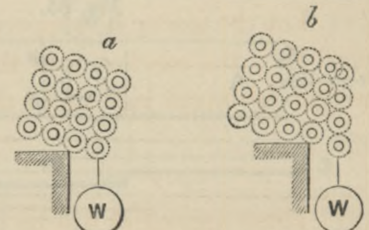
We may define a girder as a structure, the object of which is to transfer the vertical force resulting from the weight of a load from one point to another differently situated, the first being any point over the space to be crossed, and the second being that at which the girder is supported or fixed. It would appear from this, that the only force which should be brought upon a girder by a load is a shearing strain, which is transmitted from section to section, until it arrives at the point of support, but we must examine more closely into the method by which the strain is passed from one section to another. If we make a sketch of the atoms or molecules contained in two contiguous sections, and carefully examine the direction of the shearing force, we shall find that in the transmission of the strain from one section to the other, tends to alter the relative positions of these sections, so, that if they were originally parallel, they will not be so when affected by a strain. This variation of position, will, of course, occur between every pair of sections, hence the beam will become curved or deflected, and the attraction of cohesion existing between the molecules in any contiguous sections, will, when the load is removed, cause them to revolve round some axis, until they again become parallel to each other. We will thus illustrate these results of the shearing

FIG. 32.



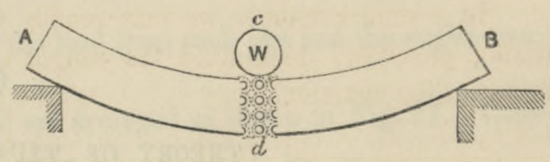
force. Let A, Fig. 32, represent two contiguous sections of a beam, one of which is resting upon a pier or other support, if a weight W be now brought to bear upon the outer section, it will tend to force it down past that one which is supported on the pier, and in so doing, to separate the molecules composing the two sections, this action will be resisted by the mutual attraction of the particles, and the distances to which they are separated will depend upon the proportion of the forces. If the end of the beam be free to move, as shown at B, the action of the load will cause it to revolve round the line of contact with the edge of the pier, and the sections will then be in the position shown at *a* in Fig. 33. It is evident, that while matters are in this condition, the weight may approach still nearer to the ground if the section is caused to revolve round an axis, situated at some point near the centre, and if the elastic resistances to tension and compression are equal, this axis will be at the centre of gravity of the section, and the two series of molecules will assume the position shown at *b*, the upper molecules being caused to approach each other, while the lower ones are separated, and this occurring between all the sections of the beam, the upper part will be compressed, and the lower part extended. In these remarks, there are two points which perhaps require a little more explanation on our part, the first consists in the assumed motion of the end of the beam about the line of contact with the pier, although this may seem in the first instance improbable, we may consider that an amount of tangential motion will be allowed of between the particles (before any great horizontal strain comes upon them), which will be quite sufficient to produce this effect. We will, however, illustrate this mode of action in another manner.

FIG. 33.



Let A B represent a beam supported at each end of which *c d* is a section, to which at present we confine our attention. If a weight W be placed over the section *c d* it will force this section downwards, and the particles in the section will attract those in the neighbouring sections; if we suppose all the other sections of the beam inseparable, it appears at once that the two ends of the girder may be caused to revolve through a slight distance round their lines of contact with the piers, thus producing compression in the upper and tension in the lower part of the beam. If we consider all the sections as separable from each other, we arrive at a curved form for the loaded beam. This illustration also explains the other point we mentioned as requiring explanation which was the position of the weight whether acting direct upon the top or bottom molecules first? If the beam was fixed at one end and free at the other, the particles in the upper parts of the sections would be most readily moveable, and would, therefore, be separated to a greater distance than those in the lower part of the beam, hence, supposing them to revolve, while their relative position is being altered, round an axis contained within the beam, the upper part will be in tension and the lower in compression. The axes of revolution will be lines in the neutral surface of the beam. We must now leave this effect of load and turn our attention to other shearing strains, the nature of which is more certain—viz., the strain upon bolts, rivets, &c. In this case the action of the strain must have a tendency to shear the material, in a manner in all respects similar to the process of shearing by a machine, and, therefore, all that we have to consider in this chapter, is the law by which the variations of the force and resistance are governed. We will first examine the force producing the shearing action; this kind of action occurs at every rivet or other similar fastening, hence its relation to the load on the girder may depend on one of various conditions, but if we take the force acting on the parts joined by the rivet, without regarding any other conditions, the shearing strain will be exactly equal to such force. The resistance to shearing strain is usually said to vary directly as the sectional area of the bolt, but this is not, strictly speaking, correct, for it is not so great. If the resistance did vary as the sectional

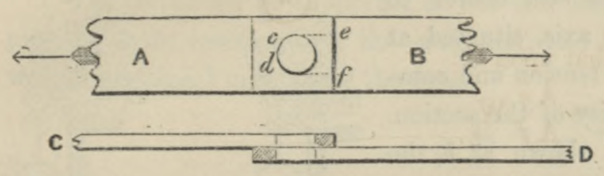
FIG. 34.



area, the whole area would be acted upon at once, and supposing the force to be sufficiently great to cause rupture it would take place instantaneously, but such is not the case, for a bolt may be cut partially through, the remaining part being uninjured, this may result from two causes—viz., the elasticity of the metal and its liability to rupture by crushing before the strain is sufficiently great to shear the material. The effect of the elasticity is not of sufficient magnitude to demand a place in the data upon which our calculations are based, and the variations of strength which depend upon the proportions between the ultimate shearing and other strains, will be discussed in a subsequent chapter.

We may now with sufficient accuracy assume, that the resistance to shearing varies as the area to be sheared, hence the strength of bolts varies as the square of the diameter of the bolt.

Fig. 35.



Other parts of the joints, besides the bolts or rivets, will also be subject to shearing strain. If in the accompanying Fig. A B represents two plates rivetted together as shown and acted upon by forces tending in the direction of the arrows, then that part of the plate *c e d f* is liable to be torn out, the metal being sheared through the section of the joint shown at *c d*, the parts at which the shearing force acts are shown by shading. From the foregoing observations, we conclude that in a joint of this nature there are three elements to be observed—viz., the diameter of the bolt or rivet, the sectional area of the plates, and the amount of overlap, the proportions of these depending upon the strength exerted in opposition to the various forces acting upon them.

CHAPTER VII.

THEORY OF TRUSSED AND OF LATTICE GIRDERS.

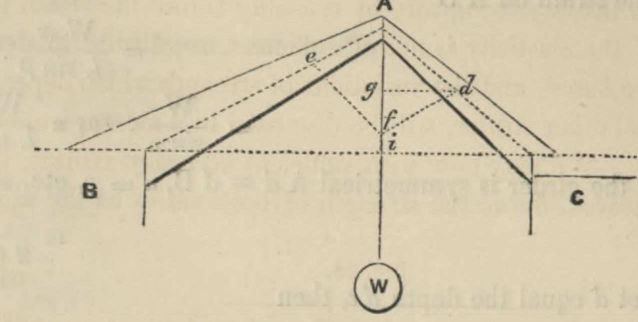
A CLEAR comprehension of the relation existing between the horizontal and vertical forces in a beam solid throughout is not very readily obtained, and this uncertainty results from the want of knowledge as regards the actual direction of the strains, for we can only assure ourselves of the existence of the horizontal strains with perfect confidence. When we attempt to analyze the laws of shearing strains, doubts arise as to the existence of strains pursuing diagonal courses from the load to the points of support. In our remarks upon shearing strain we have shown the existence of diagonal strains, but these extend only from one molecule to another. It is, however, unnecessary for us here to enter further into this question, as a girder which will sustain the horizontal and vertical strains will be sufficiently strong to resist any other to which it may be subject.

In those structures, the principles of which we are now about to investigate, the arrangement of elements is such that there can be no doubt as to the direction of the strains, for there are certain channels, so to speak, through which the strains must pass, we shall, therefore, treat the girders as acted upon by one series of forces only, which forces change their direction in passing from element to element, sometimes increasing and at others diminishing the total strain on one particular part, but so combined as to render a division of them inconvenient.

Before entering upon the calculation of the forces acting upon the girder, which, though by no means difficult, presents, at first sight, a complicated appearance, we shall select a few elementary illustrations of similar forces in order to allow us to concentrate our attention upon the resolution of the downward force of the load upon any particular part of the structure, without regarding the strains transmitted by other parts.

The principle upon which our investigation will be based, is that of the parallelogram of forces, but we shall generally solve the parallelogram by plane trigonometry.

Fig. 36.



Let A B, B C, Fig. 36, be two beams, meeting at their upper extremities, and supported on the piers B and C. Let a weight W be suspended from the point A. Complete the parallelogram A *efd*, then will the strain on A C be equal to W $\frac{Ad}{Af}$ and that on A B will be represented by W$\frac{Ae}{Af}$. From d let fall the perpendicular $d\,g$ upon A f, then if we resolve the diagonal strains upon the piers into horizontal and vertical forces, and call R′ the weight upon B and R″ that upon C:

$$R' = \frac{gf}{Af}W, \text{ and } R'' = \frac{Ag}{Af}W$$

join B C, then by the laws of the lever,

$$R' = W.\frac{iC}{BC}, R'' = \frac{iB}{BC.}$$

If W is represented by A f, then

the strain on A C = A d

the strain on A B = A e

$R' = gf \qquad R'' = Ag.$

Let the angle A dg, or A C B = α and gdf, or A B C = β,

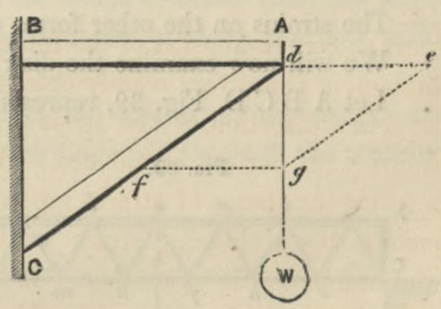
then will
$$Ad = Ag.\frac{\sin Agd}{\sin \alpha} = \frac{R''}{\sin \alpha} \text{ and}$$

$$Ae = gf\frac{\sin fgd}{\sin \beta} = \frac{R'}{\sin \beta}.$$

Which gives us the strain on each beam when the distance of the load from each pier and the angles contained by the beams and the horizon are known.

Let us now take another case, where the beams and load are arranged as shown in Fig. 37. Complete the parallelogram $fdeg$, then will the strain on A B = W $\frac{de}{dg}$ and that on A C = W $\frac{df,}{dg}$ or if dg = W, these strains become de and df. Let $dfg = \alpha$

Fig. 37.



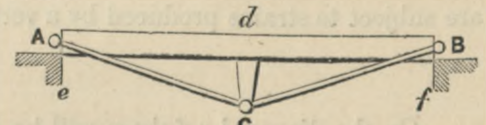
$$de = dg.\frac{\cos \alpha}{\sin \alpha} = \frac{W}{\tan \alpha}$$

$$df = dg.\frac{\sin fgd}{\sin \alpha} = \frac{W}{\sin \alpha}.$$

These two cases are types of all that occur in lattice and trussed girders.

Let A B represent a simple trussed girder, supported at the ends, on piers A, B, R′ being the weight on A, and R″ that on B. Let d C A, or C A e = α, and d C B, or C Bf = β. If the girder be loaded with a weight W placed at d,

Fig. 38.



then
$$R' = W.\frac{dB}{AB} \quad R'' = W.\frac{dA}{AB}.$$

If l equal the span of the girder, and A d = x, then

$$R' = W.\frac{l-x}{l} = W - \frac{Wx}{l} \quad R'' = \frac{Wx}{l}.$$

The strain on the tie bar A C

$$= \frac{R'}{\sin \alpha} = \frac{W}{l.\sin \alpha.}\left\{l - x\right\}$$

and that on the tie bar CB

$$= \frac{R''}{\sin\beta} = \frac{W x}{l. \sin\beta}.$$

The strain on AB

$$= \frac{W x}{l. \sin\beta} \cdot \cos\beta = \frac{W x}{l. \tan\beta}.$$

$$= \frac{W}{\sin\alpha} \cdot \cos\alpha \; \frac{W x}{l. \tan\alpha} = \frac{W}{\tan\alpha} - \frac{W x}{l. \tan\alpha}.$$

If the girder is symmetrical A d = d B, $\alpha = \beta$, etc. = etc., strain on AB,

$$= \frac{W}{2 \tan\alpha}.$$

Let d equal the depth $d\,c$, then

$$\tan\alpha = \frac{d}{\frac{1}{2} l}.$$

and the above becomes

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

Let the load be distributed and equal to w per lineal unit, then half the total load will be sustained by the piers direct, and the other half will pass through $d\,c$ and the tie bars; hence, in this case, the strains will be one half their last value, or the strain on AC:

$$= \frac{w}{2 \sin\alpha} \left\{ l - x \right\}$$

and that on CB

$$= \frac{w x}{2 \sin\beta}.$$

The strain on AB

$$= \frac{w x}{2 \tan\beta}.$$

And if the girder is uniform, this expression,

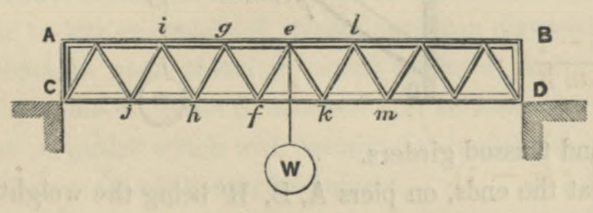
$$= \frac{w l}{4 \tan\alpha} = \frac{w l^2}{8 d}.$$

The strains on the other forms of trussed girders may be calculated in a similar manner to the above.

We will now examine the distribution of strains upon a more complicated structure.

Let ABCD, Fig. 39, represent a lattice girder, consisting of two flanges AB, CD and a number of diagonal bars so arranged that ef is parallel to gh ij, etc., and the bars inclining in the opposite direction are parallel to each other also; the two halves of the girder being similar.

FIG. 39.



Let $efk = \alpha$ and $gfh = \beta$, then ekf lmk, etc., each equal α, and lkm, etc., each equal β.

Let also a load W be placed upon the centre of the girder, then half of this load will be supported by each pier, therefore the bars in each half of the girder are subject to strains produced by a vertical force:

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

On the diagonal ef there will be a compressive strain,

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

which will be resolved into two others at the point f both tensile, one on fg

$$= \frac{W}{2 \sin\alpha} \cdot \frac{\sin\alpha}{\sin\beta} = \frac{W}{2 \sin\beta}$$

and another on fk

$$= \frac{W}{2 \sin \alpha} \cdot \frac{\sin (\alpha + \beta)}{\sin \beta}$$

$$= \frac{W}{2} \left\{ \frac{1}{\tan \alpha} + \frac{1}{\tan \beta} \right\}.$$

The first of these forces is resolved at g into two compressive forces, one on $h\,g$,

$$= \frac{W}{2 \sin \beta} \cdot \frac{\sin \beta}{\sin \alpha} = \frac{W}{2 \sin \alpha}$$

and the other on $g\,e$

$$= \frac{W}{2 \sin \beta} \cdot \frac{\sin (\alpha + \beta)}{\sin \alpha}$$

$$= \frac{W}{2} \left\{ \frac{1}{\tan \alpha} + \frac{1}{\tan \beta} \right\}.$$

Hence, on every diagonal bar making an angle α with the horizon, there will be a compressive strain,

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

and on every diagonal making an angle β with the horizon, there will be a tensile strain,

$$= \frac{W}{2 \sin \beta}.$$

And generally, every diagonal inclining downwards from the load towards the pier is in compression, and all diagonals inclining in the opposite direction are in tension.

At every junction of two diagonals making angles α and β with the horizon, a strain will be brought upon the flange

$$= \frac{W}{2} \left\{ \frac{1}{\tan \alpha} + \frac{1}{\tan \beta} \right\}.$$

At the joint c one diagonal only meets the flange, and the strain is resolved horizontally and vertically, the vertical strain $= \frac{W}{2}$, and the strain on the flange,

$$= \frac{W}{2 \sin \alpha} \cdot \frac{\cos \alpha}{\sin c\,A\,i} = \frac{W}{2 \tan \alpha}.$$

The top flange will be in compression, and the bottom flange in tension, and as there is an addition made to the strain on these members at every joint, the total strain on the flange increases from the piers to the centre of the girder, not gradually, as in a solid beam, but by increments at every junction with the diagonals.

If n = the number of struts between any point and the nearest pier, the strains on the flanges at that point may be found by summing the strains brought upon them by the diagonals, then will the tension on the bottom flange,

$$= \frac{W}{2} \left\{ \frac{n}{\tan \alpha} + \frac{n-1}{\tan \beta} \right\}.$$

And the compression on the top flange, is

$$= \frac{W}{2} \left\{ \frac{1}{\tan \alpha} + \frac{1}{\tan \beta} \right\} \cdot (n - 1).$$

Let s = the distance between two joints on either flange, l = the span of the girder, and d = the depth of the girder, then

$$S = d \left\{ \frac{1}{\tan \alpha} + \frac{1}{\tan \beta} \right\}$$

$$\frac{l}{2} = s\,(n - 1) + \frac{d}{\tan \alpha}$$

$$\therefore \frac{l}{2\,d} = \frac{n}{\tan \alpha} + \frac{n-1}{\tan \beta}$$

where n represents the number of struts in half the length of the girder. For the tension on the bottom flange at the centre of the girder, we have by substitution,

$$\frac{W\,l}{4\,d}.$$

And at any point distant x from the point of support the tension on the bottom flange

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

and for the compression on the top flange, we have

$$\frac{W\,x}{2\,d} - \frac{W}{2\,d.\,tan\,\alpha}.$$

And at the centre,

$$\frac{W\,l}{4\,d} - \frac{W}{2\,d\,tan\,\alpha}.$$

Let the load be situated at any point distant x from the pier. Then the load acting upon the bars in one part, etc., will be $= W.\ \frac{l-x}{l}$, therefore, there will be on every bar in that part which inclines downwards from the load towards the pier, a strain in compression,

$$= \frac{W}{l.\,sin\,\alpha}\left\{l - x\right\}$$

etc., and on every bar inclining in the opposite direction, a tensile strain

$$= \frac{W}{sin\,\beta}\left\{l - x\right\}.$$

On the other part of the girder the strain will be due to the action of a weight $= \frac{W\,x}{l}$, therefore if we call the angle alternate to $\beta = \theta$, and that alternate to $\alpha = \phi$, there will be a compressive strain on every bar inclining down towards B

$$= \frac{W\,x}{l.\,sin\,\theta}$$

and a tensile strain on every bar inclined in the opposite direction,

$$= \frac{W\,x}{l.\,sin\,\phi}.$$

The strains on the flanges may be found, as before, by summation, but the maximum strain will exist on these at the point at which the load is applied. We will now collect the results of the foregoing observations.

If a lattice girder consist of two flanges, connected by diagonals, as above, and be subject to a central load, bars making equal angles with the horizon will be equally strained, the bars inclined downwards from the point of load to the piers will be struts, and those inclined in the opposite direction will be ties. The strain on the flanges will be a maximum at the centre.

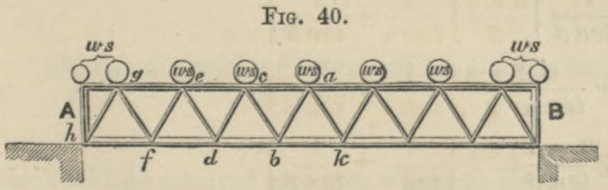
If the load is not central, all bars making equal angles on the same side of the load, will be equally strained, but the strains will be different on the two parts separated by the load.

The strain on the flanges will be a maximum at the point at which the load is applied.

If the load move from one end of the girder to the other, every diagonal bar will be alternately subject to tension and compression, but the intensities of the strains will be different.

The foregoing calculations being perfectly understood, no great difficulty will be experienced in determining the nature and intensity of the strains when the girder is subject to an uniformly distributed load.

Fig. 40.



Let A B, Fig. 40, represent a lattice girder uniformly loaded with w per lineal unit, if s be the distance between the apices of two successive triangles, the load on each triangle, except the two end ones, will be $w\,s$. The girder being loaded all over each half of the girder will carry half the load, we will, therefore, consider the strains on one half of the girder only. Let $a\,b\,k = \alpha = c\,d\,b =$ etc., and $c\,b\,d = \beta = e\,d\,f =$ etc., the load on g will be half the load between g and e, plus half the load between A and g, or

$$= \frac{w}{2}\left(s + \frac{D}{tan\ \alpha}\right)$$

where D represents the depth of the girder.

The load on $a\,b$ will be equal to half the weight at a or $\frac{w\,s}{2}$, therefore the strain on $a\,b$

$$= \frac{w\,s}{2\ sin\ \alpha},$$

this load will be transmitted to $b\,c$ and $b\,k$, the strain on the former will be

$$= \frac{w\,s}{2\ sin\ \beta}$$

this will again produce a compressive strain on $c\,d$

$$= \frac{w\,s}{2\ sin\ \alpha};$$

but this strut will also have to sustain the weight at c or $w\,s$, the strain from which will be

$$= \frac{w\,s}{sin\ \alpha}$$

therefore the total strain on $c\,d$ will be

$$= \frac{w\,s}{2\ sin\ \alpha} + \frac{w\,s}{sin\ \alpha} = \frac{3\,w\,s}{2\ sin\ \alpha}$$

this will produce a tensile strain on $d\,e$

$$= \frac{3\,w\,s}{2\ sin\ \beta}.$$

And thus we may, by summation, find the strain on every bar. They will be as follows, commencing from a, the centre of the girder:

	Strain on strut:	Strain on tie:
On the first pair	$= \frac{w\,s}{2\,sin\,\alpha}$	$= \frac{w\,s}{2\,sin\,\beta}$
On the second pair	$\frac{w\,s}{2\,sin\,\alpha} = \frac{w\,s}{sin\,\alpha} = \frac{3\,w\,s}{2\,sin\,\alpha}$	$= \frac{3\,w\,s}{2\,sin\,\beta}$
On the third pair	$\frac{3\,w\,s}{2\,sin\,\alpha} = \frac{w\,s}{sin\,\alpha} = \frac{5\,w\,s}{2\,sin\,\alpha}$	$= \frac{5\,w\,s}{2\,sin\,\beta}$
On the $(n-1)^{th}$ pair	$= \frac{(2\,n-3)\,w\,s}{2\,sin\,\alpha}$	$= \frac{(2\,n-3)\,w\,s}{2\,sin\,\beta}$
On the $(n)^{th}$, or last pair	$\frac{(2\,n-3)\,w\,s}{2\,sin\,\alpha} +$	

$$\frac{w}{2\,sin\,\alpha}.\left\{s + \frac{D}{tan\,\alpha}\right\} = \frac{(n-1)\,w\,s}{sin\,\alpha} + \frac{w\,D}{2\,sin\,\alpha.\,tan\,\alpha}.$$

There are only $(n-1)$ ties when there are (n) struts. These strains will of course be different when the end of the girder forms a complete triangle, and is suspended by the upper flange; but we shall not consider that case, as the method of calculating it is not sufficiently obvious.

The tension produced on the lower flange will be as follows:

By the first strut from the centre	$\frac{w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}$
By the second strut from the centre	$\frac{3\,w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}$
By the third strut from the centre	$\frac{5\,w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}$
By the $(n-1)^{th}$ from the centre	$\frac{(2\,n-3)\,w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}$
By the $(n)^{th}$ strut from the centre	$\frac{w}{2\,tan\,\alpha}\left\{2\,(n-1)\,s + \frac{D}{tan\,\alpha}\right\}.$

The maximum tension on the lower flange will be

$$T = \frac{w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\} \cdot \{1 + 3 + 5 + \text{etc.} + (2n-3)\} + \frac{w}{2\,tan\,\alpha}\left\{2\,(n-1)\,s + \frac{D}{tan\,\alpha}\right\}$$

$$= \frac{w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\} \cdot \left\{n-1\right\}^2 + \frac{w}{2\,tan\,\alpha} \cdot \left\{2\,(n-1)\,s + \frac{D}{tan\,\alpha}\right\}$$

as before, $s + D\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}$ therefore,

$$T = \frac{w\,D}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}^2 \cdot \left\{n-1\right\}^2 + \frac{w\,D\,(n-1)}{tan\,\alpha}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\} \frac{+w\,D}{tan^2\,\alpha}$$

$$= \frac{w\,D}{2}\left\{\left(\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right)(n-1) + \frac{1}{tan\,\alpha}\right\}^2.$$

But half the span $= (n-1)\,s + \frac{D}{tan\,\alpha}$

$$= \left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}(n-1)\,D + \frac{D}{tan\,\alpha}$$

$$= D\left\{\left(\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right)(n-1) + \frac{1}{tan\,\alpha}\right\}$$

therefore,

$$T = \frac{w\,D}{2} \cdot \frac{l^2}{4\,D^2} = \frac{w\,l^2}{8\,D}.$$

The compressive strain on the upper flange may be found in a similar manner; at the centre it will be

$$C = \frac{w\,s}{2}\left\{\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right\}\{n-1\}^2$$

$$= \frac{w\,d}{2}\left\{\left(\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right) \cdot (n-1)\right\}^2.$$

Let l' = the length of the top flange between the extreme apices, then

$$l' = 2\,(n-1)\,s$$

$$= 2\,D\,(n-1)\left(\frac{1}{tan\,\alpha} + \frac{1}{tan\,\beta}\right)$$

therefore,

$$C = \frac{w\,D}{2} \cdot \frac{l'^2}{4.\,D^2} = \frac{w\,l'^2}{8\,D}.$$

If $\alpha = \beta$ the compressive and tensile strains on the diagonals are equal. Let $\alpha = \beta = 45°$, then

$$s = 2\,d \qquad n = \frac{l.\,tan\,\alpha + D}{2\,D} = \frac{l + D}{2\,D}$$

and the strains will be on—

First tie or strut $\frac{w\,s}{2\,sin\,\alpha} = \frac{w\,D}{sin\,\alpha} = w\,D \times 1{\cdot}4142$

Second tie or strut $\frac{3\,w\,s}{2\,sin\,\alpha} = \frac{3\,w\,D}{sin\,\alpha} = w\,D \times 4{\cdot}2426$

Third tie or strut $\frac{5\,w\,s}{2\,sin\,\alpha} = \frac{5\,w\,D}{sin\,\alpha} = w\,D \times 7{\cdot}0710$

$(n-1^{th})$ tie or strut $(2n-3) = \frac{w\,D}{sin\,\alpha} = w\,D \times \frac{l - 2\,D}{D} \times 1{\cdot}4142$

$(n)^{th}$ strut $(4\,n-3)\frac{w\,D}{2\,sin\,\alpha} = w\,D \times \frac{2\,l - D}{D} \times 2{\cdot}8284$

Let $\alpha = \beta = 60°$, then $n = \frac{l\,tan\,\alpha \times D}{2\,D} = \frac{l\,\sqrt{3} + D}{2\,D}$

$s = \frac{2\,D}{tan\,\alpha} = \frac{2\,D}{\sqrt{3}}$ and the strains will be—

First tie or strut $\frac{w\,D}{sin\,\alpha\,tan\,\alpha} = w\,D \times 0{\cdot}6666$

$$\text{Second tie or strut} \quad \frac{3\,w\,D}{\sin a \tan a} = w\,D \times 2{\cdot}0000$$

$$\text{Third tie or strut} \quad \frac{5\,w\,D}{\sin a \tan a} = w\,D \times 3{\cdot}3333$$

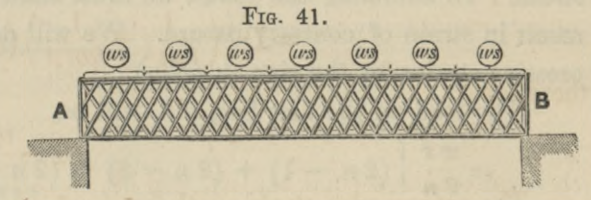
$$(n-1)^{\text{th}} \text{ tie or strut} \quad \frac{l\sqrt{3}-2\,D}{D} \cdot \frac{2\,w\,D}{3} = \frac{w\,D \times 1{\cdot}1547\,l - 1{\cdot}33\,D}{D}$$

$$(n)^{\text{th}} \text{ strut} \quad \frac{2\,l\sqrt{3}-D}{D} \cdot \frac{2\,w\,D}{3} = \frac{w\,D \times 2{\cdot}3095\,l - 0{\cdot}666\,D}{D}$$

The method of calculating the horizontal strains in these cases requires no explanation, for, from what we have already said, it appears that the strain at any joint may be calculated by the formula applied to solid girders, but calling the length of the flange equal to the span of the girder, then the moment of strain divided by the depth of the girder, will give the strain on the flanges.

It will frequently occur, that two or more series of triangulations may exist in the girder, as shown in Fig. 41, but this will not cause any inconvenience, for all that will be necessary, is to divide w by the number of series of triangulations, or we may calculate the sectional area that would be required for each bar in one series, and then divide this area among the number of bars by which one is replaced, which is by the number of series of triangulations. The strains on the flanges will be the same at the centre of the span, but they will increase from the abutments towards the centre by more frequent additions of less magnitude.

Fig. 41.



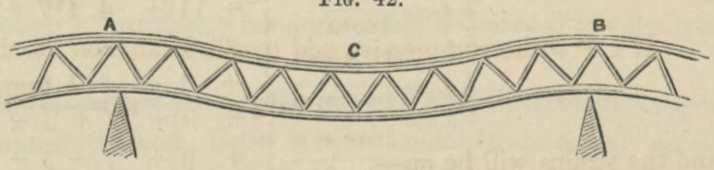
It is evident from the above, that if we wish to calculate the strains upon a beam fixed at one end and free at the other, that we may treat the diagonals as if the beam were half of a beam of twice the length supported at each end, but the strain on the flanges will be different.

A few remarks on continuous lattice girders may be desirable in this place.

The strains on the flanges may be found by dividing the moments of strain calculated by the method shown in the chapter on continuous girders by the depth of the beam, hence all we have to consider is the distribution of the strains on the diagonal bars.

Let A B C, Fig. 42, represent any span of a continuous lattice girder, and let C be the point of maximum strain in the central part of the girder, we may then, as far as the lattice web is concerned, divide the span into two parts A C, C B, and treat each as a beam fixed at one end (that over the pier) and free at the other. This distribution of strain appears natural when we consider that it is the web of the girder which actually transmits the vertical force produced by the load from the various parts of the girder to the piers, whereas the flanges seem to have as their duty the maintenance of the web in its proper form.

Fig. 42.

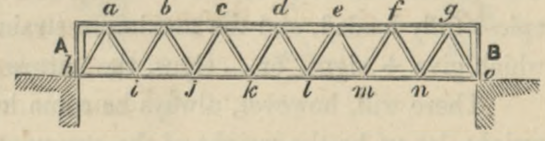


These flanges have also been termed booms, on the supposition that they keep the web of the girder extended in the same manner as a boom in nautical structures keeps a sail stretched.

We will again investigate the case of a lattice girder, subject to an uniformly distributed load, but by a method differing slightly from the foregoing one.

Let A B, Fig 43, represent the girder loaded as before, the same notations being used, $w\,s$ equal the weight on each apex on this occasion; we will also adopt this for the value of the load upon the apices of the extreme triangles.

Fig. 43.



The load on a will act on the piers A B in proportion to its distance from each, thus on A there will be a weight equal to $\frac{w\,s}{l}\left\{ l - \frac{s}{2} \right\}$ or calling n the number of triangles that part of $w\,s$ which is supported by the bar $a\,h$ $= \frac{w\,(2\,n-1)}{2\,n}$ and the load supported by the bar $a\,i = \frac{w}{2\,n}$

This last load will also be supported by all the other bars which will be acted upon by it alternately in tension and compression.

Similarly, the load on b will be divided into two parts, the one acting on $b\,i$ and $a\,h$

$$= \frac{w\,(2\,w-3)}{2\,n}$$

and the other acting on the bars $b\,j$, $c\,k$, etc.

$$= \frac{3\,w}{2\,n}.$$

Thus we see that each bar is acted upon by a number of loads, some of which act in tension and others in compression, and these may be thus distinguished: all strains entering at the bottom of any diagonal produces tension on that diagonal, but if it enters at the top of the bar it produces a compressive strain. In summing the loads, we must subtract those producing one kind of strain from those which result in strain of contrary nature. We will designate tensional strains by the sign — (*minus*), and compressive strains by the sign + (*plus*).

The load on the first bar, $a\,h$, will be,

$$= \frac{w\,s}{2\,n}\left\{(2\,n-1) + (2\,n-3) + (2\,n-5) + (2\,n-7) + \ldots\ldots\ldots (2\,n - [2\,n-1\,])\right\}$$

the load on the second bar, $a\,i$, will be,

$$= \frac{w\,s}{2\,n}\left\{1 - [\,(2\,n-3) + (2\,n-5) + (2\,n-7) + \ldots\ldots\ldots\ldots (2\,n - [2\,n-1]\,)\,]\right\}$$

and for all the others it may be found in like manner. We will work this calculation out, for the case shown in the Fig. 43, where $n = 7$.

The strains will be, if $a\,h\,i = \alpha$, $a\,i\,h = \beta$.

$$\text{On } a\,h = \frac{w\,s}{14.\,\sin\alpha}\left\{+13 + 11 + 9 + 7 + 5 + 3 + 1\right\} = + \frac{7\,w\,s}{2\,\sin\alpha}$$

$$a\,i = \frac{w\,s}{14.\,\sin\beta}\left\{+1 - 11 - 9 - 7 - 5 - 3 - 1\right\} = - \frac{5\,w\,s}{2\,\sin\beta}$$

$$b\,i = \frac{w\,s}{14.\,\sin\alpha}\left\{+11 + 9 + 7 + 5 + 3 + 1 - 1\right\} = + \frac{5\,w\,s}{2\,\sin\alpha}$$

$$b\,j = \frac{w\,s}{14.\,\sin\beta}\left\{+1 + 3 - 9 - 7 - 5 - 3 - 1\right\} = - \frac{3\,w\,s}{2\,\sin\beta}$$

$$j\,c = \frac{w\,s}{14.\,\sin\alpha}\left\{+9 + 7 + 5 + 3 + 1 - 3 - 1\right\} = + \frac{3\,w\,s}{2\,\sin\alpha}$$

$$c\,k = \frac{w\,s}{14.\,\sin\beta}\left\{+1 + 3 + 5 - 7 - 5 - 3 - 1\right\} = - \frac{w\,s}{2\,\sin\beta}$$

$$k\,d = \frac{w\,s}{14.\,\sin\alpha}\left\{+7 + 5 + 3 + 1 - 5 - 3 - 1\right\} = + \frac{w\,s}{2\,\sin\alpha}$$

And so on for all the bars in the girder, but if the girder be symmetrical, the positions of α and β will change at the centre, so that $\bar{d}\,k\,l = d\,l\,k = \alpha$, $e\,l\,m = \beta$. The strains on the flanges may be calculated by the same formula as before, but replacing

$$\frac{w}{2}\left\{s + \frac{D}{\tan\alpha}\right\} \text{by } w\,s.$$

In the above series, we have considered the girder as loaded all over, but there may be only certain apices fully loaded, and the maximum strain on any bar will exist when only those apices are fully loaded, which give + signs, or — signs, the nature of the sign depending upon whether the bar is a tie or strut.

There will, however, always be some load on every apex, because the structure must support its own weight, let w' be the weight of the structure per lineal unit, then the maximum strains will be,

$$\text{On } a\,h = \frac{w\,s}{14.\,\sin\alpha}\left\{+13 + 11 + 9 + 7 + 5 + 3 + 1\right\} = + \frac{7\,w\,s}{2\,\sin\alpha}$$

$$a\,i = \frac{w\,s}{14.\,sin\,\beta}\left\{ -11 - 9 - 7 - 5 - 3 - 1 \right\} + \frac{w'\,s}{14.\,sin} = -\frac{18\,w\,s}{7.\,sin\,\beta} + \frac{w'\,s}{14.\,sin\,\beta}$$

$$b\,i = \frac{w\,s}{14.\,sin\,a}\left\{ +11 + 9 + 7 + 5 + 3 + 1 \right\} - \frac{w'\,s}{14.\,sin\,\beta} = \frac{18\,w\,s}{7.\,sin\,\beta} - \frac{w'\,s}{14.\,sin\,\beta}$$

$$b\,j = \frac{w\,s}{14.\,sin\,\beta}\left\{ -9 - 7 - 5 - 3 - 1 \right\} + = \frac{w'\,s}{14.\,sin\,\beta}\left\{ 1 + 3 \right\} = \frac{4\,w'\,s - 25\,w\,s}{14.\,sin\,\beta}$$

and so on for all the diagonals in the girder.

The maximum strain on the flanges will be obtained when the girder is fully loaded. If there are many series of triangles, the value of w in the foregoing calculations must be replaced by that of $\frac{w}{\text{No. of series of triangles.}}$

On the girders, of which we are now speaking, there is no shearing strain, as the load is carried by the bars constituting the web.

CHAPTER VIII.

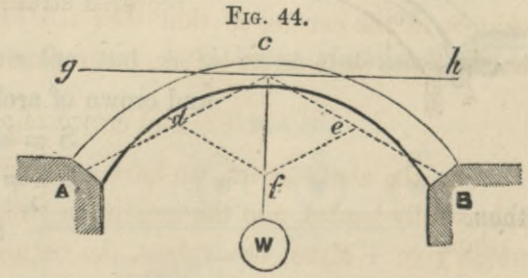
THEORY OF ARCHES AND BOWSTRING GIRDERS.

Although, at first sight, there appears to be a very considerable distinction between the arch and the bowstring girder, such is not really the case, and we may generally consider them identical in principle, though different in detail, the thrust on the abutment plates being in the one case withstood by masses of masonry, and in the other by an iron tie uniting the abutments. The theory of the arch is an expression which has usually been applied to investigations into the conditions of equilibrium of linear arches, but, although in the first instances, the construction of cast-iron arches was made to depend upon the same hypothesis, this has long since been abandoned with regard to metal structures, and the process contained in the following pages has for its object the determination of formulæ, whereby we may be enabled to calculate the direct crushing strain on any part of an arch.

The metal arch appears to combine in itself the properties both of the web and of the horizontal flanges in the plate and lattice girders, thus, at the summit of the arch, there is a horizontal strain exactly equal to that which would exist in a straight girder of equal span and load, having a depth equal to the versine of the arch, and as we progress from the crown of the arch to the abutment plate, the rib assumes more and more the offices of the web in the plate or lattice girder, and if at the abutment a tangent to the arch be perpendicular, the rib at that place assumes completely the conditions of the standard, or strut at the end of a straight girder.

Let A c B, Fig. 44, represent an arch, from the crown of which is suspended a weight W. This load must be sustained by the abutments, draw, therefore, the straight lines, A c, c B, from the point at which the load is applied to the abutments, complete the parallelogram $c\,d\,f\,e$, then if $c\,f$ = W, $c\,d$ and $c\,e$ will represent the thrusts upon the abutments resolved in those directions, and as the load is central these will be equal.

Fig. 44.



As the stains will be similar on both halves of the arch, we will consider one only.

Let $g\,h$ be a tangent to the curve at the crown of the arch, and therefore horizontal, and let a the angle $h\,c$ B. Then the weight supported by B, resolved in the direction B c, gives a reactive strain

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

which may be regarded as counteracting the pressure produced at c in the direction $g\,h$ by the semi-arch $A\,c$, hence, by resolving the above strain in the direction of the tangent, we shall find the horizontal strain at the crown of the arch; it will be

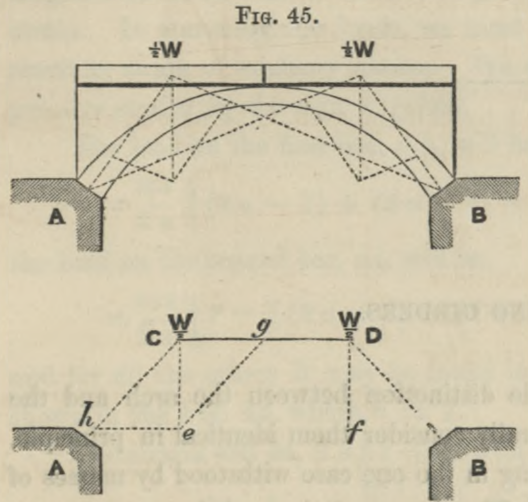
$$= \frac{W}{2 \sin a} \cdot \cos a - \frac{W}{2 \tan a}$$

but if $V =$ the versine and l the span of the arch, then will $\tan a = \frac{V}{\frac{1}{2} l}$ and the horizontal strains

$$= \frac{W\,l}{4\,V}.$$

If the strain at any other part of the rib be required, it may be found by resolving the above strain in the direction of a tangent to the curve at that point.

Fig. 45.



Let A B, Fig. 45, represent an arch loaded with a weight W, spread uniformly over its length, then, to find the horizontal strain at the centre, we may consider the load as divided into two parts, one being placed at the centre of each semi-arch. The effect of each load in the direction of either piers may be found by representing the weights $\frac{W}{2}$ by vertical lines and completing the parallelograms as shown. As the strains will be the same on both sides of the arch, we need only consider those on one semi-arch. To find the horizontal strain, we regard $\frac{W}{2}$ as acting along the lines $C\,h$ $C\,g$ and resolve it horizontally.

Let $C\,e$ represent $\frac{W}{2}$ and complete the parallelogram, $h\,C\,g\,e$ then is $C\,g$ equal to the horizontal strain.

Let $C\,h\,e = a$, then

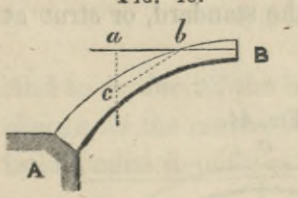
$$g\,e = \frac{W}{2 \sin a}, \quad C\,g = \frac{W}{2 \sin a}, \quad \cos a = \frac{W}{2 \tan a},$$

$\tan a = \frac{C\,e}{h\,e}$, but because $\frac{W}{2}$ is at the centre of the semi-arch, therefore $h\,e = \frac{l}{4}$ and $\tan a = \frac{V}{\frac{1}{4} l}$ and the horizontal strain then becomes,

$$= \frac{W\,l}{8\,V}.$$

This being known, we can now readily find the strain on any other part of the arch, thus—

Fig. 46.



Let A B, Fig. 46, represent the semi-arch, it is required to find the strain normal to any section C. Let $a\,b$ equal the horizontal pressure and $a\,c$ the load included between a and B, then will the resultant $b\,c$ of these forces be equal required strain, therefore, if we make

$W =$ total load on whole arch,

$W_1 =$ load between any section,

and crown of arch.

$S =$ strain normal to any section,

$S\,b =$ strain normal at abutments,

then,

$$S^2 = \left(\frac{W\,l}{8\,V}\right)^2 + W_1{}^2$$

$$\therefore S = \sqrt{\left(\frac{w\,l}{8\,V}\right)^2 + W_1^2}$$

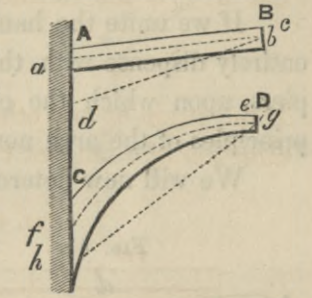
$$S\,b = \sqrt{\frac{W^2}{4} + \frac{W^2\,l^2}{64\,V}}$$

$$= \frac{W}{2}\sqrt{1 + \frac{l^2}{16\,V^2}}.$$

We now proceed to examine into the nature of the curve indicated by theory as suitable for the construction of metal arches, which is the curve of the line of pressures. It is always desirable that the line of pressures should, in such cases as that under consideration, be contained within the depth of the rib, for reasons which we are about to illustrate.

Let A B, Fig. 47, represent a straight beam fixed by one in a wall, and let a force be applied at the other end, then, if this force act in the direction of a......b, the strain will be direct; but if it act in some other, such as c......d, a bending moment will result. Again, let C D be a curved beam, similarly fixed, and strain if the force follows the curve $f\,e$, and strain at any section will be direct, but if it only passes through the ends, as shown at h and g, there will be a bending moment.

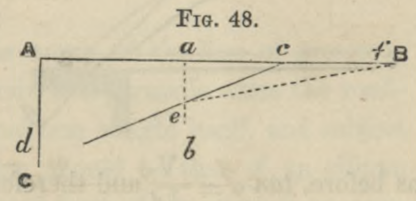
Fig. 47.



We have calculated our arch on the hypothesis that every section is subject to a direct strain only, therefore it is very necessary to make the rib of such form and depth, that the line of pressures does not at any point pass outside the section.

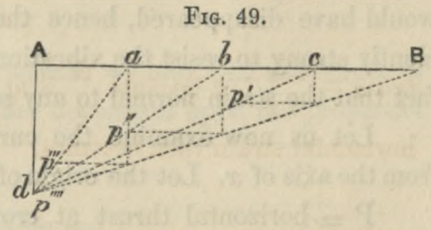
Let A B, Fig. 48, represent the loaded roadway supported by half the arch, C being one abutment. Take any section, $a\,b$, make $a\,f$ equal to the horizontal strain, and $a\,e$ equal to the weight between a and B or $w \times \overline{a\,B}$, then e B will be the resultant, and the line of pressures will pass through the point e. We must next find in what direction it passes e, to do this we will consider the load $w \times a$ B as concentrated at its centre of gravity c, which, if the load be uniformly distributed, bisects a B; then the proper direction for the strain between c and e is evidently a straight line, therefore $c\,e$ is a tangent to the line of pressures at e; but because $c\,e$ bisects a B, in c, therefore it is a tangent to a parabola, and the required curve is a parabola. We will now examine the curve proper for sustaining a rolling load, supposing the arch itself to be devoid of weight.

Fig. 48.



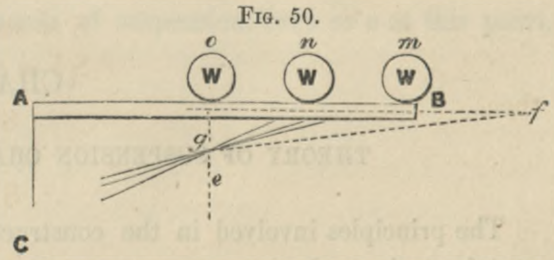
Let A B, Fig. 49, be half the length of the roadway, and let a load W roll from B to A. When the load is at c let $c\,e$ equal that portion borne by the abutment at d, then to find what is borne by the same pier as the load passes each of the points b a, etc, make A d = W, and join d B, draw $a\,g$ $b\,f$ perpendicular to A B and parallel to A D, and $c\,e$ to meet d B in $g\,f$, then $b\,f$, $a\,g$ are the required quantities. At each of these points the direction or the strain will be $c\,d$, $b\,d$, $a\,d$, resolving the strain in these directions, the strains become $c\,'p$, $b\,p''$, $a\,p'''\,'$ etc., and p' p'' p''' are points in the curve. From the construction in the curve it is at once evident that it is elliptical. This case cannot possibly occur in practice, but we thought it desirable to inquire into the nature of the strain produced by a rolling load, as this is frequently combined with a distributed load. The curve to be adopted in practice should be something between a parabolic and an elliptical curve, inclining most to the former or latter as the distributed or rolling load preponderates. The curve may, however, be laid out as follows:—

Fig. 49.



Let A B, Fig. 50, represent half the roadway as before, the part A B being the distributed load, equal to w per lineal unit; let the arch also support a rolling load W. First, to find the point in any section $d\,e$, through which the line of strain passes. Let d B equal x, $d\,f$ equal horizontal strain at the crown, and $d\,g = w\,x$ + W, then the resultant $g\,f$ equals the strain at g, and the line of pressures passes through g. Second, to find the direction of the line of pressures at g. Let W be at m just upon d B, find the centre of gravity of $w\,x$ and W, and draw a straight line from the centre of gravity through g, this will be one tangent; the same may be done when the weight is at n and

Fig. 50.

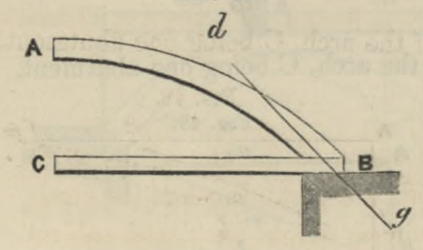


o to find other tangents; by means of these tangents various curves may be drawn, and the rib should be deep enough to contain all of them. It will only be necessary to find two sets of tangents, viz., one for the entry, and the other for the exit of the rolling load, from each part of the girder; for it is evident that all the others will be contained between these; and, therefore, if the rib be deep enough to contain the curves drawn from these two series, it must be deep enough to contain all the others that can be drawn. The thrust on the abutment of plate will of course be equal to the strain on the last section of the rib, and by resolving this strain in a horizontal direction the horizontal thrust upon the abutment will be obtained; whence the tendency of this thrust to upset the abutment-masonry may be determined.

If we unite the haunches, or tie abutment plates of the arch, by means of tie bars, we may then entirely dispense with the masonry required to resist the thrust of the arch, and replace it with slight piers, upon which the ends of the rib may be rested; and in so doing we do not in any way alter the principles of the arch nor the conditions of strain.

We will now determine the amount of tensional strain upon the tie bar.

Fig. 51.



Let A B, B C, Fig. 51, be the semi-arch and semi-tie, let the arch be loaded with an uniformly distributed load equal to W. Let $d\,g$ be the direction of the thrust on the abutment plate and the angle $d\,h\,c = a$, then the thrust on the abutment $= \frac{W}{2 \sin a}$ which resolved to give the tension on C B.

$$= \frac{W}{2 \sin a} \cdot \cos a = \frac{W}{2 \tan a}$$

as before, $\tan a = \frac{V}{\frac{1}{4} l}$ and therefore the strain on the tie becomes,

$$= \frac{W\,l}{8\,V}.$$

In bowstring girders the roadway is usually suspended from the arch by standards, the lower extremities of which are retained at their proper distances by pins passing through them and the tie bars. When the arch and its tie are braced together by diagonals, these may, at first sight, be supposed to be subject to the same conditions as the diagonals of a lattice girder, but if such were the case, the principle of the arch would have disappeared, hence the arched rib being properly designed, the bracing need only be sufficiently strong to resist the vibration of the roadway. The most striking property of the arch consists in the fact that the strain normal to any section is least at the crown of the arch.

Let us now examine the curve produced by laying off the strains at various sections, as ordinates from the axis of x. Let the origin of co-ordinates be at the crown of the arch. The thrust on any section $= y$.

P = horizontal thrust at crown, w = weight of load per lineal unit. Let the angle contained between the resultant of the vertical and horizontal strains and the horizon $= a$ then,

$$y^2 = (w\,x)^2 + p^2$$

$$y = \sqrt{(w\,x)^2 + p'^2}$$

also,

$$y = \frac{w\,x}{\sin a}.$$

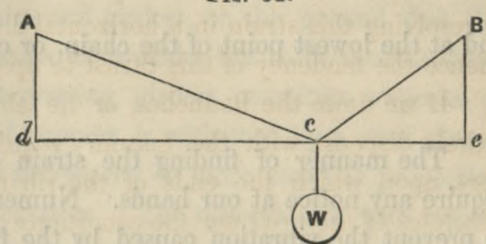
CHAPTER IX.

THEORY OF SUSPENSION CHAINS AND INVERTED BOWSTRING GIRDERS.

The principles involved in the construction of the suspension chain are very similar to those which appertain to the arch.

The suspension chain is not like the arch, a true curve, but is composed of straight bars, which are tangents to the curve indicated by theory. If, however, the load be not continuous on the chain, it will be more proper to use the system of straight bars than the true curve. In examining the form suitable for a suspension chain, we must remember that the chain is capable of altering its form, in order to establish equilibrium under any load.

Fig. 52.



Let A c B, Fig. 52, be supposed to represent a perfectly flexible chord devoid of weight, and let it be subject to the action of a load W concentrated at one point, then will the chord A c B assume the form of two straight lines A c, c B. The tension on either part of the chord may thus be found. Draw the horizontal line $d\,e$, and from A B let fall the perpendiculars A d B e upon $d\,e$, let the angle $\mathrm{B}\,c\,e = \alpha$, $\mathrm{A}\,c\,d = \beta$, then the tension on c B, if $l =$ the span A B.

$$= \frac{\mathrm{W}\,\overline{d\,c}}{l.\,sin\ \alpha}$$

and that on c A,

$$= \frac{\mathrm{W}\,\overline{c\,e}}{l.\,sin\ \beta}.$$

By employing the same method which we adopted in examining the curve of the line of pressures in the arch, we should find, that for a continuous load, the curve of the chord is a parabola when the roadway A B or $d\,e$ is loaded, but a catenary when we regard the chord as of uniform weight itself, and subject to no other load when the chain is subject to a rolling load only, its form should be that of an ellipse. When we consider that all these loads act upon the chain, the complicated form of the curve is at once evident, as it would seem to be a combination of the parabola ellipse and catenary. It will, however, be generally found most convenient to assume the parabola as the true form of the curve. The tension on the chain may be determined by the same formulæ which we applied to the arch, or, if the angle included between any bar of the chain and the horizon be represented by α, and that part of the load which is supported by that bar $= \mathrm{W}'$, then the tension on such bar will be,

$$= \frac{\mathrm{W}'}{sin\ \alpha}.$$

Assuming the curve to be a parabola, and the roadway to be horizontal, we may find the length of the suspending rods and of the chain by the following formulæ, which are dependent upon the properties of that curve.

Let $l =$ length of any suspending rod,
$l' =$ length of any shortest suspending rod,
$d =$ deflection of chain,
$s =$ semi-span,
$x =$ distance of suspending rod from centre of chord,

$$l = l_{,} + \frac{d\,x^2}{s^2}.$$

For the length of the chain, we have,

$$\lambda = \sqrt{s^2 + \tfrac{4}{3}\,d^2}$$

where $\lambda =$ half the length of the chain.

Let us now find the tension on the extreme links or points of suspension, let $\alpha = \theta$ at this point, then, the tension

$$= \frac{\mathrm{W}}{2\ sin\ \theta}.$$

At the lowest point in the chain the tension is

$$= \frac{\mathrm{W}}{2\ tan\ \theta}$$

But $\tan\theta \qquad = \frac{2d}{S}$

and $\sin\theta \qquad = \frac{2d}{\sqrt{(2d)^2 + S^2}}$

And the tension at the points of support,

$$= \frac{W\sqrt{S^2 + 4d^2}}{2d}$$

and at the lowest point of the chain, or centre,

$$= \frac{Wl}{8d}.$$

The manner of finding the strain on the backstays is sufficiently evident, and does not, therefore, require any notice at our hands. Numerous designs have, from time to time, been produced with a view to prevent the vibration caused by the form of the chain, adapting itself to every variety of load as it occurs, but by far the greater number appear to be based upon erroneous principles.

We shall not encumber our space while treating on this subject with formulæ, as our readers will readily perceive the method of resolving the strain on any element by the general expression

$$\frac{W'}{\sin\alpha}.$$

We will first consider the effect of load upon a rigid suspension chain. In this case the theoretical conditions of strain will exist only while the line of tension is contained within the depth of the chain, hence we gain no advantage over the arch, and in some cases a bending action will necessarily exist.

In the next case, the chain is constructed in two parts, each of which is rigid, and extends from one point of support to the centre of the span, where the two are united by a pin. The object of this joint at the centre is to allow for expansion and contraction of the chain, the effect of which in this case will be to raise and lower the platform at the centre, alternately, as the temperature increases or decreases. This arrangement also possesses the disadvantages belonging to the foregoing, but in a somewhat greater degree. Another method which has been proposed, consists in the employment of a number of supplementary chains, the object of which appears to be the prevention of an alteration of level in the platform by inequality of load; but if this end be effected, it must be accomplished by the load being carried by one of the supplementary chains. This arrangement appears in principle to be identical with the use of several chains, each of which is of the form most suitable for some particular load, and all of them being almost incapable of alteration of form. We will now take leave of structures which do not fulfil all the conditions required in a rigid suspension-bridge, and make a few remarks upon the principles which should constitute the basis of such designs.

There can be no doubt that the suspension-bridge is the lightest which can be constructed, as we here employ our metal in the most advantageous way, but the very principle of the structure would seem to exclude its use for railway purposes. This principle is that the chain shall be sufficiently flexible to adapt its form to any load that may occur, whereby we insure it against the occurrence of any but a direct strain. If we attempt to render the chain itself rigid we must evidently vitiate this principle of its action, and, in fact, we reduce it to the condition of an inverted arch, hence it follows that we must leave the chain perfectly flexible. From this it appears that we have no alternative but to regulate the manner in which the load acts upon the chain, so that it may always be distributed over some considerable length of the chain, whereby any extreme alteration of form will be avoided, without in any way affecting the principle of the structure. We may thus describe the difference between the action of the load as it ordinarily occurs, and its effect when regulated as stated above. Let us suppose a chain, devoid of weight, and a roadway, also devoid of weight and rigidity:—If a rolling load pass over such a structure, whenever it passed a suspension rod, the chain would assume the form of two straight lines diverging from the top of the suspension rods to the points of support, hence great vibration would be produced by the motion of such a load; if, however, the roadway were made sufficiently rigid to distribute the load over several suspension rods instead of allowing it to act solely on one, the changes in the form of the chain would be much more gradual, and they would not be of so great magnitude. Hence, we conclude, that some combination of the suspension chain with a roadway stiffened by a girder would fulfil all the required conditions, and it only remains to solve the problem with regard to the extent to which such stiffening of the platform can be economically carried; Mr. Barlow has carefully examined the principles of these combinations, and now proposes to construct suspension-bridges of very great span by the adoption of these principles.

Some objections have been raised to the combination of chains and girders, on the ground that the girders must be strong enough to carry the load without the assistance of a chain, but a due consideration of the matter will show that such is not the case. Inverted bowstring girders constitute a species of rigid suspension-bridge, but in this case the pull on the points of support is withstood by a strut placed between them, to which the chain is braced. These bridges must necessarily be more heavy in proportion to their strength than those in which the strain is withstood by backstays. The observations with regard to intensity of strain on ordinary bowstrings, apply equally to inverted bowstrings, but, in the latter case, the bow is in tension and the horizontal member is in compression.

CHAPTER X.

ON THE STRENGTH OF MATERIALS.

In the present chapter we purpose to investigate the nature of the strains to which such structures or elements of structures as have not been included in the foregoing pages, are subject, and also the behaviour of materials under direct strain. Of the laws of elasticity we shall say nothing in this place, as we have treated that subject in the first chapter. We shall hereinafter employ three factors of strength, each of which represents the ratio of an adopted stress to the ultimate strength, the first factor, which we shall designate $f\,u$, has reference to the ultimate or breaking strength, and is therefore equal to unity, the second, $f\,p$, is the ratio between the proof stress and the ultimate strength, and the former may be thus defined:— The proof strength is represented by a strain, which, being repeatedly applied to any given structure, does not produce an increasing deflection or set. The permanent set, is that portion of the deflection from which the structure does not recover after the removal of the load, and it is probable that such set is produced, although not appreciable even by exceedingly small loads, it is caused by the imperfect elasticity of the material. Any load greater than the proof load, will, on repeated application, increase the permanent set, and ultimately destroy the cohesion of the material.

The third factor, $f\,n$, expresses the ratio existing between the stress allowed in actual practice and ultimate strength of the material, this stress is usually taken considerably below the proof strength in order to allow ample resistance in case of any extraordinary load.

Let S = the ultimate strain, $S\,p$ = proof strain, $S\,w$ = working strain, then the following values are those which we shall adopt throughout this work.

	$S \div S\,p$	$S \div S\,w$	$S\,p \div S\,w$
Steel	2·5	5·0	2·00
Wrought Iron	2·5	5·0	2·00
Cast Iron	2·0	6·0	3·0

Let us now examine the mode of action of direct tensile strain. The only effect of this kind of force is elongation, for there can be no tendency to produce any other alteration of form, provided always that the element acted upon was not originally curved in the direction of the strain.

Suppose it is required to determine the dimensions of any tie bar subject to a direct tensile strain.

Then S being the breaking strain per square inch of sectional area, W the load, and A the sectional area for wrought-iron,

$$A = \frac{5\,W}{S}.$$

To find the area of a suspension rod of uniform strength, loaded with a weight W at the extremity, and with its own weight, A = the area at any section distant x from the lower extremity of the rod, and w = weight of the rod per lineal unit, then,

$$A = \frac{5\,W}{S} + \frac{5\,w\,x}{S} = \frac{5}{S}\left\{ W + w\,x \right\}.$$

Compressive strain gives rise to phenomena differing widely from those produced by tension. The modes of crushing are various, as splitting, shearing, bulging, buckling, cross-breaking. Those materials, which are of hard, vitreous texture, such as vitrified bricks, usually crush by splitting into a number of fragments, separated by smooth surfaces lying in directions nearly parallel to that in which the crushing force is caused to act. Granular materials, such as cast-iron and some kinds of stone, crush by shearing, or the sliding of portions of the element over one another; sometimes the sliding takes place on one plane surface, at others, two cones or pyramids are formed, which, being urged towards each other, split the block, which is subject to the compressive strain, into a number of wedge-shaped pieces. Occasionally, the block splits into four wedges, whose backs are represented by the top, bottom, and two sides of the block. Crushing by bulging or lateral swelling, and spreading of the block, is indicative of toughness; and materials possessed of this property, break gradually under a crushing strain. Crushing by buckling occurs in thin fibrous materials, when subjected to compressive force in the direction of the fibres. Crushing by cross-breaking occurs in columns whose lengths greatly exceed their diameters, in which case they first bend or deflect, and finally break in the manner of a beam subject to transverse strain. Whenever the material breaks under a compressive strain without bending, we may consider that the strength of the column or strut varies directly as the sectional area and the formulæ given for tensile strains will then apply in the present case, but the load W must be supposed to be placed upon the upper extremity of the column, and x must be measured from that extremity. We will now determine the conditions of strain on long flexible pillars or struts. Let W be the load which is placed upon the pillar, and A the sectional area of the pillar, then one part of the intensity of the strain on the pillar is simply due to the uniform distribution of the load over the sectional area, and may be represented by

$$S_{I} = \frac{W}{A}.$$

Another part of the greatest strain is that which arises from the lateral bending, which takes place in the direction of the least diameter; if the diameters be equal, let h be the least diameter, and b that perpendicular to it, l the length of the pillar, and d the greatest deflection of the axis of the pillar from its original position. The greatest bending moment will be W d, and the greatest strain produced by that moment is directly as the moment, and inversely as the breadth and square of the thickness of the pillar, (see resistance of materials to transverse strain), or calling this part of the greatest strain S_{II}

$$S_{II} \propto \frac{W\,d}{b\,h^2}$$

but the greatest deflection, consistent with safety, is directly as the square of the length, and inversely as the thickness,

$$d \propto \frac{l^2}{h}$$

also the product, $b\,h^2$, varies as the sectional area A, and the thickness h, therefore,

$$S_{II} \propto \frac{W\,l^2}{A\,h^2} \propto S_{I} \cdot \frac{l^2}{h^2}$$

and calling S the total intensity of strain on the material of the pillar,

$$S = S_{I} + S_{II} = \frac{W}{A}\left\{ 1 + a\,\frac{l^2}{h^2} \right\}$$

where a is a coefficient to be be determined by experiment. The strength of a long pillar will be,

$$W = \frac{S\,A}{1 + a\,\frac{l^2}{h^2}}$$

The values of the constants have been determined for the ultimate strength of cast and wrought-iron, by Mr. Gordon from the experiments of Mr. Hodgkinson.

Wrought Iron . . . $S = 36{,}000$ lbs. $a\ \frac{1}{3000}$

Cast Iron . . . . . . $S = 80{,}000$ lbs. $a\ \frac{1}{400}$

A pillar rounded at both ends is as flexible as a pillar of the same diameter fixed at both ends, and of twice its length, therefore in this case,

$$W = \frac{S\,A}{1 + 4\,a\,\frac{l^2}{h^2}}.$$

The strength of a pillar fixed at one extremity, and rounded at the other, is a mean between the strengths of two pillars of the same length and diameter, one of which is fixed, and the other rounded at both ends.

The formulæ deduced by Mr. Hodgkinson from his own experiments are as follows:—When the length is not less than thirty times the diameter. For solid cylindrical pillars h being the diameter in inches, and l the length in feet,

$$W = c\,\frac{h^{3\cdot6}}{l^{1\cdot7}}$$

for hollow cylindrical pillars h' being the internal diameter,

$$W = c\,\frac{h^{3\cdot6} - h'^{3\cdot6}}{l^{1\cdot7}}$$

the values of the coefficient, c, are as follows:—

Solid pillars with	round	ends	$c = 14{\cdot}90$	Tons.
,, ,,	flat	,,	$c = 44{\cdot}16$	,,
Hollow ,,	round	,,	$c = 13{\cdot}00$	,,
,, ,,	flat	,,	$c = 44{\cdot}30$	,,

If the pillar has a length less than thirty times its diameter, let b = breaking load, composed by the foregoing formula, c = crushing load of a short block of the same sectional area,

$$c = 49 \text{ tons} \times A \text{ square inches},$$

the crushing load of the pillar will be

$$W = \frac{b\,c}{b + \frac{3\,c}{4}}.$$

Shearing strain is that to which we shall now direct our attention. It may be regarded as acting in two distinct manners, the first of which is direct, the second by torsion or twisting of the material strained. In the first case, let S represent the shearing strain per unit of sectional area, then if A be the area of any element subject to a shearing strain produced by a weight W, then A,

$$= \frac{W}{S};$$

thus, if we wish to find the diameter of a bolt or rivet in a lap joint to sustain a weight W, we have

$$d = \sqrt{\frac{W}{\cdot7854\,S}}$$

and for a butt joint, with cover plates, top and bottom,

$$d = \sqrt{\frac{W}{1{\cdot}5708.\,S}}.$$

Suppose the pin to be used for the attachment of a suspension rod to the chain of a suspension-bridge, then the last formula will apply, but if it be intended to connect the links of the chain, it will not require to be so large in proportion to the strain, for if there be n bars, side by side, and $(n-1)$ bars alternately, then at every joint, the pin must be sheared at $2\,(n-1)$ sections before it can fail, and the above formula will become

$$d = \sqrt{\frac{W}{1{\cdot}5708\,(n-1)\,S}}$$

where W equals the tension on the entire sectional area of the chain at the point at which the pin is to be inserted.

The effects of a shearing strain acting in such a manner as to produce torsion, may be calculated as follows. Let us suppose a cylindrical axle to be subject to a tangential strain, then, if we consider the conditions of two sections taken very close together, one of them will have a tendency to rotate relatively to the other. Let s be the limit of shearing strain to which the material is to be exposed. Let r, be the external radius of the axle. Then if we call s' the shearing strain at any radius, s will be the value of s', for the radius r' at any other radius r

$$s' = s \frac{r}{r'}.$$

Let the cross section be supposed to be divided into an infinite number of infinitely narrow concentric rings, the breadth of each of which is $d\,r$. Let r be the mean radius of one of these rings, then its area is $2\,\pi\,r\,d\,r$, consequently the moment of shearing stress for the ring in question, is

$$= \frac{2\,\pi\,s}{r'}\,r^3\,d\,r$$

which being integrated, for all the rings from the centre to the circumference of the section gives for the moment of resistance to torsion,

$$M = \frac{2\,\pi\,s}{r'} \cdot \int_0^{r'} r^3\,d\,r\,\frac{\pi\,s\,r'^3}{2}$$

but $\frac{\pi}{2} = 1{\cdot}5708$, therefore,

$$M = 1{\cdot}5708.\,s\,r'^3.$$

Before concluding the present chapter, it may be desirable to offer a few remarks upon the relations existing between strain, strength, and resistance. The fundamental principle of statics (viz.) the equality of action and reaction in any body which preserves a state of equilibrium, requires that whatever strain is placed upon any structure, the material of which that structure is composed, must offer a resistance exactly equal to such strain, otherwise, the conditions of equilibrium will not be fulfilled, and work will be done in one direction or the other by which we mean, that a force will be exerted through a space. Let us suppose, a perfectly elastic prism to be subject to a compressive strain produced by a load, W, then the weight W, will continue to compress the prism until the resistance of the latter is equal to the intensity of the former, after which, the forces will be in equilibrio, but it is evident that a certain amount of work is done in compressing the material, and when the load is removed, the prism will regain its original size, in doing which, work will again be performed.

So long as the load is not sufficiently great to destroy the elasticity, or cohesion of the material, the prism will always exert an amount of resistance, exactly equal to the intensity of the load to which it is subject, without any regard to its ultimate strength.

All that is necessary to be considered in designing any kind of structure, is that it must be made of sufficient strength to be capable, without injury to its elastic power, of exerting an amount of resistance equal to the greatest strain that will at any time be brought to bear upon it, when it will adjust the amount of resistance actually exerted, at any time, to the intensity of the load which calls that resistance into action.

TABLE No. I.

THE STRENGTH OF CAST IRON.

The tensile strength is in pounds per square inch of sectional area. The transverse strength is in pounds on a semibeam, one inch long, and one inch square, the weight being applied at the end. The torsional strength is in pounds per square inch, acting with a leverage of one inch, upon a bar one inch in diameter. The compressive strength is in pounds per square inch.

Manufacturers.	Description.	Tensile.	Transverse	Torsion.	Crushing.
Hæmatite Foundry, Whitehaven	No. 1. Foundry Pig	14233	4644	3724	52136
Hæmatite Foundry, Whitehaven	No. 3. Foundry Pig	17751	5105	4182	82265
Hæmatite Foundry, Whitehaven	No. 4. Foundry Pig	17566	6100	4977	82583
Weardale Iron Company	No. 1. Pig Iron	18080	5600	5510	59771
Weardale Iron Company	No. 3. Pig Iron	21859	7374	5968	90046
Weardale Iron Company	No. 4. Pig Iron	23513	7672	6350	109286
Weardale Iron Company	Remelted	30333	8948	6277	122216
South Bank Furnaces	No. 2. Toughened Pig	18425	6260	6405	86886
South Bank Furnaces	No. 2. Foundry Pig	15835	5563	5626	77926
Stockton Iron Works	No. 1. Hot Blast	25810	7159	5872	99524
Stockton Iron Works	No. 3. Hot Blast	22271	6932	6305	87063
Butterley Iron Works	No. 1. Foundry Pig	23388	7106	7342	88488
Butterley Iron Works	No. 2. Foundry Pig	18970	6077	6011	74743
Butterley Iron Works	No. 3. Foundry Pig	23265	6692	6940	91661
Butterley Iron Works	No. 3. Blue Rake	24126	7364	6371	97041
West Hallam Iron Works	No. 1. Melting Pig	22107	7587	7312	82229
West Hallam Iron Works	No. 2. Melting Pig	29840	9033	8113	119483
West Hallam Iron Works	No. 3. Melting Pig	30115	9399	7973	120321
West Hallam Iron Works	Grey Forge	25380	7751	7711	99193
West Hallam Iron Works	No. 4. Strong Forge	19847	7173	6403	75318
Goldendale Iron Works...	Pig Iron	25430	7591	6837	113459
Netherton Iron Works	No. 1. Melting Foundry Pig ...	20517	6954	6130	73977
Netherton Iron Works	No. 2. Melting Foundry Pig ...	26012	8139	7186	86473
Netherton Iron Works	No. 3. Grey Forge..	24222	7634	6204	82406
Netherton Iron Works	No. 4. and 5 Forge	30344	9619	7570	108285
Netherton Iron Works	No. 5. Strong Forge		8129		
Park Head Furnaces	Foundry Melting Pig	27033	7778	7226	95787
Park Head Furnaces	Grey Forge	18831	5998	5969	69773
Park Head Furnaces	Forge	30554	8840	7697	107223
Old Hill Furnaces	No. 2. Foundry Pig	14593	5192	4642	50499
Old Hill Furnaces..	No. 3. Grey Forge...	23815	7478	7147	80991
Old Hill Furnaces	No. 4. Forge	22918	6926	5757	80213
Old Hill Furnaces	Strong Forge	27676	7793	5861	119807
Lay's Iron Works	Hæmatite 1st Melting	31480	8142	5071	119631
Level Iron Works	Hot Blast Pig Iron...	26198	7254	5879	99795
Level Iron Works	Cold Blast Pig Iron	25872	6903	5654	86650
East End Iron Works	No. 1. Grey Foundry Pig	20105	5744	5424	105395
East End Iron Works	No. 2. Mottled Iron		4869		
Heyford Iron Works	Pig Iron 1st Melting	10866	3075	3905	77690
Park End Furnaces	Foundry Pig	12593	5220	4478	56116
Park End Furnaces	Grey Forge	17019	6781	6232	73400

Manufacturers.	Description.	Tensile.	Transverse.	Torsion.	Crushing.
Ystalyfera Iron Works	No. 1. Foundry Pig ...	25172	7848	6704	87475
Ystalyfera Iron Works	No. 2. Foundry Pig ... 2nd Melting	26758	7944	6176	90894
Ystalyfera Iron Works	No. 3. Foundry Pig ...	24533	7228	5719	88772
Ystalyfera Iron Works	No. 1. Foundry Pig ...	25311	7656	6265	87570
Ystalyfera Iron Works	No. 2. Foundry Pig ... 1st Melting	21661	7206	6559	77573
Ystalyfera Iron Works	No. 3. Foundry Pig ...	23707	7785	6894	85094
Blaenavon Iron Works	No. 1, and C. Forge	26766	7947	5487	105203
Blaenavon Iron Works	No. 1. Cold Blast	25456	7493	5034	91897
Blaenavon Iron Works	Common Forge	28637	7805	5953	130860
Blaenavon Iron Works	No. 3. Cold Blast	23903	7600	5674	87358
Blaenavon Iron Works	Bright	23139	6594	3882	126086
Pontypool Iron Works	Cold Blast	26302	6669	5235	99618
Messrs. Begbie	Swedish Grey Iron		5673		
Acadian Iron Company	Nova Scotia Pig Iron	23194	8092	7726	86933
Messrs. Townsend, New York	Sterling Pig Iron	23613	8210	5734	84410
Indian Iron Company	Charcoal Pig Iron		4120		
Indian Iron Company	Elba Iron		4249		
Indian Iron Company	Tuscan Iron		3934		
Butterley Iron Company	Low's Patent Iron	25185	7914	4761	125198
Messrs. Price and Nicholson	Patent Iron	28960	9120		
Ulverstone Iron Company	Charcoal Iron	17592	5540		
Bowling Iron Company	Bowling Iron	24004	8309		
Low-Moor Iron Company	Low-Moor Iron, No. 1....	12694			60489
Low-Moor Iron Company	Low-Moor Iron, No. 2....	15458			95928
	Clyde Iron, No. 1....	16125			90805
	Clyde Iron, No. 2....	17807	6709		106011
	Clyde Iron, No. 3	23468	7654		106039
	Calder Iron, No. 1.	13735			74088
	Coltness Iron, No. 3.	15278	6766		101005
	Brymbo Iron, No. 1.	14426			75246
	Brymbo Iron, No. 3.	15508			76545
	Ynis-cedwyn Iron, No. 1.	13952			81084
	Ynis-cedwyn Iron, No. 2.	13348			76246
Mr. M. Stirling	Second quality	25764			122395
Mr. M. Stirling	Third quality	23461			144264
Butterley Iron Works	"Kishy Iron"		6552		
Butterley Iron Works	No. 2. Grey Iron ... ... Cold Blast		7812		
	No. 2 and 3, Mottled ...		8820		
Butterley Iron Works	Mottled Hot Blast		9198		
	No. 1. Carnbroe Hot Blast		8370		
Messrs. Grissell	Mixed		9504		
	Scotch Hot Blast		6975		
	No. 3. Langloan Hot Blast		6543		
	No. 3. Omoa Hot Blast...		8154		
	No. 1. Omoa Hot Blast...		7250		
	No. 3. Redsdale Hot Blast		9026		
	No. 1. Redsdale Hot Blast		7146		
	No. 3. Tow-lan Hot Blast		6372		
	No. 1. Crawshay Cold Blast... ...		7857		
	No. 1. Coalbrook Vale ... Cold Blast		7884		
	No. 3. Coalbrook Vale ...		8073		
	No. 1. Dundyvan Iron		5085		
	No. 2. Dundyvan Iron		6250		
	No. 3. Dundyvan Iron		5971		
	No. 3. Pontypool Iron		9157		
	No. 3, Crawshay Iron		8216		

Manufacturers.	Description.	Tensile.	Transverse.	Torsion.	Crushing.
	No. 2. Porthcawl Iron		8216		
	No. 3. Porthcawl Iron		7100		
	No. 3. Beaufort Iron		6975		
	No. 3. Maesteg Iron		6525		
	No. 3. Nantyglo Iron		6300		
	No. 1. Nantyglo Iron		5508		
	Gartsherrie Hot Blast		5364		
	Carron Cold Blast		5316		
	Muirkirk Hot Blast		5016		
	Ponkey Hot Blast		6000		
	Ponkey Cold Blast		6192		

TABLE No. 1.—REMARKS.

The results given in this table for the first sixty-three specimens of cast-iron, are taken from the recent experiments conducted at the Royal Arsenal, Woolwich, and those for the remaining specimens, are from the older experiments of Messrs. Stephenson, Hodgkinson, Morris, Bury, Curtis and Kennedy, etc., etc. We will now briefly examine these results, with a view to ascertain the conditions of varying strength.

In comparing the tensile and compressive resistances, we notice that there is not a constant ratio between these quantities, but, on the contrary, this ratio varies widely, and we find that with some specimens, the resistance to crushing force is three times the tensile resistance, and in others the crushing strength is even seven times as great as the tensile strength, and between these two values are the ratios of the strengths of the remaining specimens.

We have not deemed it advisable to encumber our space with chemical qualities of the various irons, but will now mention some of the characteristic constituents of some of the specimens exhibiting well-marked differences of strength.

As the tensile and compressive resistances do not bear any constant ratio to each other, we must consider the effects of chemical constitution upon each separately.

We will first consider tensile resistance. We here find, as a general rule, that the ingredients which deteriorate the strength in the greatest degree, are silicon, phosphorus, and sulphur, but there are some samples in the table, which are actually stronger than some others containing less of these constituents.

Of the effect of manganese we cannot certainly assure ourselves, as some experiments appear to be in its favour, whilst others give adverse indications, although the former are perhaps more marked than the latter.

Let us select a few specimens for special consideration.

Hæmatite iron, No. 1. F. P. has a strength of 14233 lbs., and the important foreign constituents are, manganese 0·11 per cent., silicon 3·02, graphite 3·22, phosphorus 0·06, and, again, the Netherton iron, Nos. 4 and 5, exhibits a strength of 30344 lbs., and contains, manganese 0·27, silicon 0.83, graphite 3·03, sulphur 0·04, phosphorus 0·31, we may expect here an increase of strength on account of the diminution of silicon and graphite, the former, most particularly; but, on the other hand, this iron contains a larger proportion of phosphorus than the preceding sample, and it also contains sulphur. The manganese is in excess, and it is a question which way this operates; if favourably, then the great strength of this iron is more readily accounted for.

If we examine Nos. 3 and 4 of the hæmatite iron, we find that their respective strengths are nearly equal, while their constituents are as follows: the manganese is the same in both samples, the silicon is least in the second, which is a little the weaker of the two, the total amounts of carbon are very nearly equal, but in the second sample a small portion of it is combined; there is ten times as much sulphur in the second as in the first, but there is less phosphorus—three-fifths of the quantity; from this it would appear, that sulphur does not exhibit its effects in so great a proportion as phosphorus, for the difference of silicon is but small, and the sulphur in the first specimen, is 00·1, and in the second 0·10, and the phosphorus in the first 0·05, and in the second 0·03.

We observe another case of two irons, one having a tensile strength of 30115 lbs., and the other 30334 lbs.; though the former contains twice as much phosphorus, one and a-half times the silicon and more sulphur than the latter, which disadvantages *appear* to be compensated by a great excess of manganese, and a slight diminution of graphite.

With regard to the compressive strength, it will be advisable to compare the constitution of specimens of equal tensile resistances, or nearly so.

In the case of two of the specimens, we find that one has a tensile resistance of 10886 lbs., the other 12593 lbs., while the compressive strengths are respectively 77690 lbs. and 56119 lbs. The first specimen contains only a trace of manganese, the second a considerable quantity, the first has twice as much silicon, less graphite, more sulphur, and eleven times as much phosphorus as the latter.

From this example, and also from three or four others where the tensile resistances are not widely different, but varying inversely to the compressive, we are lead to conclude that deterioration due to silicon and phosphorus is not so great for compressive resistance as for tensile, though a general examination of the table shows that an effect similar in kind, though different in degree, is certainly exhibited; we also find, that for compressive strength, the presence of manganese is not advantageous.

TABLE No. II.

THE STRENGTH OF CAST-IRON COLUMNS.

Solid cylindrical columns, with flat ends. Low-Moor Iron. No. 3.

Length.	Diameter.	Length ÷ diameter.	Strength per square inch.
60·500 inches	0·510 inches	118·00	2384 pounds
60·500 inches	0·770 inches	78·50	5339 pounds
60·500 inches	0·997 inches	60·50	7900 pounds
30·250 inches	0·500 inches	60·50	8480 pounds
60·500 inches	1·290 inches	47·00	19210 pounds
20·160 inches	0·510 inches	39·40	19150 pounds
30·250 inches	0·770 inches	39.20	18900 pounds
60·500 inches	1·560 inches	38·80	25860 pounds
30·250 inches	1·010 inches	30·00	25870 pounds
15·125 inches	0·510 inches	29·60	33820 pounds
20·160 inches	0·770 inches	26·10	33870 pounds
12·100 inches	0·500 inches	24·20	35975 pounds
10·080 inches	0·500 inches	20·16	45486 pounds
20·160 inches	1·020 inches	19·70	42500 pounds
15·125 inches	0·775 inches	19·50	46700 pounds
15·125 inches	1·000 inches	15·12	51602 pounds
7·560 inches	0·500 inches	15·12	56275 pounds
10·080 inches	0·768 inches	13·10	53000 pounds
12·100 inches	0·780 inches	12·90	51460 pounds
7·560 inches	0·777 inches	9·74	69000 pounds
3·780 inches	0·500 inches	7·56	87340 pounds
2·000 inches	0·520 inches	3·84	108000 pounds
1·000 inches	0·520 inches	1·90	119500 pounds

TABLE No. II, *Continued.*

THE STRENGTH OF CAST-IRON COLUMNS.

Hollow cylindrical columns. Low-Moor Iron. No. 3.

LENGTH.	Internal diameter.	External diameter.	Strength ÷ diameter.	Strength per square inch.
90·75 inches	1·110 inches	1·75 inches	51·8	14665 pounds
90·75 inches	1·180 inches	1·76 inches	51·5	12590 pounds
88·75 inches	1·880 inches	1·74 inches	51·0	13050 pounds
88·75 inches	1·210 inches	1·78 inches	49·8	13413 pounds
90·75 inches	1·360 inches	2·01 inches	45·1	18005 pounds
89·80 inches	1·310 inches	1·99 inches	45·1	15378 pounds
88·75 inches	1·415 inches	2·01 inches	44·1	17832 pounds
88·75 inches	1·460 inches	2·04 inches	43·5	20384 pounds
88·75 inches	1·540 inches	2·23 inches	39·7	19886 pounds
30·25 inches	0·767 inches	1·26 inches	24·0	43000 pounds
30·25 inches	0·781 inches	1·26 inches	24·0	42800 pounds
25·92 inches	0·752 inches	1·17 inches	22·1	49400 pounds
25·92 inches	0·768 inches	1·25 inches	20·7	46300 pounds
22·92 inches	0·770 inches	1·16 inches	19·7	51400 pounds
20·16 inches	0·805 inches	1·14 inches	17·6	53100 pounds
20·16 inches	0·770 inches	1·21 inches	15·7	61000 pounds
16·92 inches	0·910 inches	1·15 inches	14·1	65700 pounds
15·12 inches	0·770 inches	1·08 inches	14·0	60300 pounds
15·96 inches	0·920 inches	1·15 inches	13·8	67300 pounds
15·12 inches	0·932 inches	1·16 inches	13·0	71500 pounds
13·92 inches	0·792 inches	1·15 inches	12·1	68300 pounds
8·76 inches	0·910 inches	1·13 inches	7·7	105400 pounds

TABLE No. 2.—REMARKS.

This table was compiled and calculated from the experiments of Mr. Eaton Hodgkinson.

We have omitted the experiments upon columns with rounded ends, as we considered them inapplicable to the purposes of which we treat, those given will apply, whether the column acts as such or as a strut. The ratio of strength for columns is very complicated, and the formulæ which have been proposed for calculating the resistance of columns are numerous, but we will here give the expression which Mr. Hodgkinson, deduced from his experiments, and which is, perhaps, the most reliable, although from its construction, the aid of logarithms is required in its use.

B = breaking weight in tons, d = external, and d' = internal diameter in inches, and l = length.

For columns whose length is greater than thirty times their diameter:—

$$\text{Solid columns} \quad B = 44{\cdot}16\,\frac{d^{3\cdot6}}{l^{1\cdot7}}$$

$$\text{Hollow columns} \quad B = 43{\cdot}30\,\frac{d^{3\cdot6} - d'^{3\cdot6}}{l^{1\cdot7}}$$

which are logarithmically:—

$$\text{Solid columns} \quad log.\ B = 1{\cdot}645029 + 3{\cdot}6\ (log.\ d) - 1{\cdot}7\ (log.\ l).$$

$$\text{Hollow columns} \quad log.\ B = 1{\cdot}636488 + log.\ \{d^{3\cdot6} - d'^{3\cdot6}\} - 1{\cdot}7\ (log.\ l).$$

in which $d^{3\cdot6}$ is equal to the natural number, corresponding to 3·6 (*log. d*).

The great advantage of hollow columns is at once perceived.

For columns whose length is less than thirty times their diameter, b = result of foregoing formula, c = crushing strength of pillar,

$$B = \frac{b.\ c.}{b + \frac{3c}{4}}.$$

TABLE No. III.

THE TENSILE STRENGTH OF WROUGHT IRON.

Manufacturer.	Description.	Sectional Area.	Strength per square inch.
Low Moor Iron Works	1 inch square bars	1·0377 square inches	60364 pounds
Low Moor Iron Works	1 inch round	0·7854 square inches	61798 pounds
Low Moor Iron Works	$\frac{11}{16}$ inch rivet	0·3712 square inches	60078 pounds
Low Moor Iron Works	1 inch square, planed	0·6016 square inches	60245 pounds
Low Moor Iron Works	1½ inch round, forged	0·4779 square inches	66392 pounds
Bowling Iron Works	1 inch round, bars	0·7854 square inches	62404 pounds
Bowling Iron Works	1¼ inch round, turned	0·7854 square inches	61477 pounds
Farnley Iron Works	1 inch round bars	0·7854 square inches	62886 pounds
Bradley, Charcoal	1 inch round bars	0·7854 square inches	57216 pounds
Bradley, B. B. Scrap	1 inch round bars	0·7854 square inches	59370 pounds
Bradley, S. C.	¾ inch rivet bars	0·4417 square inches	56715 pounds
Bradley, S. C.	⅞ inch round bars	0·6220 square inches	62231 pounds
Thorneycroft and Co....	$\frac{13}{16}$ inch rivet bars	0·5153 square inches	59278 pounds
Lord Ward	$\frac{11}{16}$ inch rivet bars	0·3848 square inches	59753 pounds
Govan. Ex. B. Best	¾ inch square bars	0·5929 square inches	56655 pounds
Govan. Ex. B. Best	¾ inch round bars	0·4334 square inches	57591 pounds
Govan. Ex. B. Best	1⅛ inch round bars	1·0432 square inches	58358 pounds
Govan. Ex. B. Best	1 inch round bars	0·8171 square inches	59109 pounds
Govan. Ex. B. Best	⅞ inch round bars	0·6362 square inches	58169 pounds
Govan. Ex. B. Best	¾ inch round bars	0·4657 square inches	57400 pounds
Govan. B. Best	1⅛ inch round bars	0·9984 square inches	60879 pounds
Govan. B. Best	1 inch round bars	0·7933 square inches	62849 pounds
Govan. B. Best	⅞ inch round bars	0·6291 square inches	61341 pounds
Govan. B. Best	¾ inch round bars	0·4657 square inches	64795 pounds
Govan. B. Best	1⅛ inch round bars	0·0387 square inches	58326 pounds
Govan. B. Best	1 inch round bars	0·8334 square inches	59424 pounds
Govan. B. Best	⅞ inch round bars	0.6362 square inches	63956 pounds
Govan. B. Best	¾ inch round bars	0·4756 square inches	61887 pounds
Glasgow. B. Best	1 inch bars	0·7760 square inches	58885 pounds
Glasgow. B. Best	$\frac{15}{16}$ inch round bars	0·6650 square inches	58910 pounds
Glasgow. Rivet	⅞ inch round bars	0·5945 square inches	57092 pounds
Glasgow. B. Best	1 inch bars, forged	0·4613 square inches	59045 pounds
Govan. B. Best	¾ inch round bars	0·4506 square inches	59548 pounds
Coatbridge. B. Best Rivet	¾ inch round bars	0·4417 square inches	61723 pounds
Blockairn. B. Best Rivet	¾ inch round bars	0·4417 square inches	59219 pounds
St. Rollox. B. Best Rivet	$\frac{13}{16}$ inch round bars	0·5185 square inches	56981 pounds
R. Sollock, B. Best	$\frac{11}{16}$ inch round bars	0·3848 square inches	57425 pounds
Ulverstone Rivet	¾ inch round bars	0·4417 square inches	53775 pounds
Mersey Co. Best...	¾ inch round bars, forged	0·3986 square inches	60110 pounds
Per Eckman & Co. R. F.	Tilted bars	0·5772 square inches	47855 pounds
Per Eckman & Co. R. F.	Tilted bars	0·6581 square inches	48232 pounds
Prince Demidoff, C. C. N.	Tilted bars	0·4885 square inches	49564 pounds
Prince Demidoff, C. C. N.	Tilted bars	0·6723 square inches	56805 pounds
	Hammered Scrap	0·4537 square inches	53420 pounds
	Bushelled Turnings...	0·4443 square inches	55878 pounds
	Piece of Crank Shaft	0·6582 square inches	47582 pounds
	Piece of Crank Shaft	0·7854 square inches	44758 pounds
Low Moor Iron Works	Plate with grain	0·5998 square inches	52000 pounds
Low Moor Iron Works	Plate across grain	0·5964 square inches	50515 pounds

Manufacturer.	Description.	Sectional Area.	Strength per square inch.
Bowling Iron Works	Plate with grain	0·7246 square inches	52235 pounds
Bowling Iron Works	Plate across grain	0·7290 square inches	46441 pounds
Farnley Iron Works	Plate with grain	0·7400 square inches	56005 pounds
Farnley Iron Works	Plate across grain	0·7400 square inches	46221 pounds
Farnley Iron Works	Plate with grain	0·4893 square inches	58487 pounds
Farnley Iron Works	Plate across grain	0·4966 square inches	54098 pounds
Farnley Iron Works	Plate with grain	1·1250 square inches	58437 pounds
Farnley Iron Works	Plate across grain	1·1150 square inches	55033 pounds
Consett Iron Works	Plate with grain	1·2040 square inches	51245 pounds
Consett Iron Works	Plate across grain	1·0270 square inches	46712 pounds
Bradley. S. S....	Plate with grain	0·9833 square inches	55831 pounds
Bradley. S. S....	Plate across grain	0·9856 square inches	50550 pounds
Bradley. L. F., S. S.	Plate with grain	0·7473 square inches	56996 pounds
Bradley. L. F., S. S.	Plate across grain	0·7651 square inches	51251 pounds
Bradley. S. S....	Plate with grain	0·8503 square inches	55708 pounds
Bradley. S. S....	Plate across grain	0·8716 square inches	49425 pounds
Wells. B. Best...	Plate with grain	0·8275 square inches	47410 pounds
Wells. B. Best	Plate across grain	0·8887 square inches	46630 pounds
Wells. K. B. M.	Plate with grain	0·5780 square inches	46404 pounds
Wells. K. B. M.	Plate across grain	0·5780 square inches	44764 pounds
Mossend. B. Best	Plate with grain	0·8066 square inches	43433 pounds
Mossend. B. Best	Plate across grain	0·8066 square inches	41456 pounds
Glasgow. Boiler	Plate with grain	0·8490 square inches	53849 pounds
Glasgow. Boiler	Plate across grain	0·8053 square inches	48848 pounds
Glasgow. Ship...	Plate with grain	0·8056 square inches	47773 pounds
Glasgow. Ship...	Plate across grain	0·8300 square inches	44355 pounds
	Plate with grain	0·9710 square inches	45598 pounds
	Plate across grain	0·7451 square inches	39544 pounds
Govan. Best	Plate with grain	0·6893 square inches	43942 pounds
Govan. Best	Plate across grain	0·6845 square inches	39544 pounds
Glasgow	Ship Beam	0·8345 square inches	55937 pounds
Dundyvan	Ship Strap	0·9855 square inches	55285 pounds
Mossend	Ship Strap	0·9397 square inches	45439 pounds
Thorneycroft	Ship Strap	0·9323 square inches	52789 pounds
Consett...	Ship Angle	0·8750 square inches	50807 pounds
Dowlais	Ship Beam	0·9657 square inches	41386 pounds

TABLE No. 3.—REMARKS.

All the results given in this table, are taken from the experiments of Messrs. Napier and Son, of Glasgow.

We at once observe on glancing at the table, that no such extreme variations of strength as occur to different qualities of cast-iron, are found in wrought-iron, and this may be accounted for by the comparitive purity of the latter, regardless of its source.

Unfortunately, we are not possessed of the analyses of the irons experimented upon, wherefore our remarks upon the effects of certain ingredients, must be based upon data obtained from other sources.

We shall not here speak of the effects of admixtures, or combinations of carbon and nitrogen, as when these occur, the material becomes steel, of which we shall speak further hereafter.

It is not to be supposed that the purest wrought-iron is in every case the strongest, for it has been experimentally demonstrated that the addition of some substances is actually beneficial, and perhaps the most prominent of these is titanium, which is generally believed to be capable of improving the quality of iron, and it exists in notable quantity in the celebrated Dannemora iron. We may also mention nickel as being among those substances which improve the strength of wrought-iron, but it appears to occur in scarcely any iron except the meteoric, which exhibits great similarity to wrought-iron, although it would seem most natural that it should be crude or cast-iron.

If, however, we accept the theory of these meteoric specimens, the contained iron in passing at a high temperature through our atmosphere may have a portion of its impurities removed by oxidation in the form of slag.

We notice, that generally speaking, large bars are less strong in proportion than smaller ones, thus we find that of two bars of Bowling iron, one being wrought, and the other being turned from a larger wrought bar, so that both have the same sectional area, the former exhibits a resistance of 62404 pounds per square inch, whereas the latter gives a resistance of only 61477 pounds per square inch. This does not hold good in every case, but it appears to be the general rule, and it is at the same time only what we should reasonably expect.

It appears somewhat strange that the strength of the bars prepared from hammered scrap should be so low, nor can we account for it, except the iron was of very inferior quality. The Low Moor iron has long been considered of high quality, and the highest resistance exhibited in the table, is by a specimen of that manufacture, but the Farnley and Bowling irons are not far below it in strength.

The strength of plates is decidedly below that of bars, but in the generality of cases, it appears to matter little, whether the strain be in the direction of the fibre or across it, so this is an element which may in practice be disregarded.

We might find an average for the strength of wrought-iron generally, but do not consider it desirable so to do, for if we use it, we are as likely to get weaker as stronger iron, wherefore if we do not know the quality of the metal supplied, it will certainly be most safe to adopt the lowest strength.

TABLE No. IV.
THE RESISTANCE OF WROUGHT-IRON TO COMPRESSION.

Resistance of Rectangular Tubes, 10 *feet long.*

Dimensions of Tube.	Thickness of Plates.	Section of Tube.	Strength per square inch.
4·1 inches × 4·1 inc es	0·03 inches, nearly	Square	10980 pounds
4·1 inches × 4·1 inches	0·06 inches, nearly	Square	19259 pounds
4·25 inches × 4·25 inches	0·083 inches	Square	25171 pounds
4·25 inches × 4·25 inches	0·134 inches	Square	21585 pounds
8·175 inches × 4·1 inches	0·061 inches	Rectangle	15105 pounds
8·5 inches × 4·75 inches	0·264 inches	Rectangle	26914 pounds
8·4 inches × 4·25 inches	0·26 inches	Rectangle	29980 pounds
8·1 inches × 4·1 inches	0·059 inches	Double square	22125 pounds
8·5 inches × 4·75 inches	0·25 inches, nearly	Double square	†24640 pounds
8·1 inches × 8·1 inches	0·06 inches, nearly	Square	13274 pounds
8·37 inches × 8·37 inches	1·139 inches	Square	20379 pounds
8·5 inches × 8·375 inches	0·219 inches	Square	25421 pounds
8·5 inches × 8·4 inches	0·245 inches	Square	†24640 pounds
8·1 inches × 8·1 inches	0·0365 inches	Quadruple	21585 pounds
1·495 inches diameter	0·1015 inches	Circular	14672 pounds
1·964 inches diameter	0·1045 inches	Circular	23184 pounds
2·49 inches diameter	0·1075 inches	Circular	29770 pounds
2·35 inches diameter	0·2425 inches	Circular	21504 pounds
2·34 inches diameter	0·21 inches	Circular	22176 pounds
2·995 inches diameter	0·151 inches	Circular	27691 pounds
4·05 inches diameter	0·136 inches	Circular	27642 pounds
4·06 inches diameter	0·150 inches	Circular	26230 pounds
6·366 inches diameter	0·1298 inches	Circular	35885 pounds
6·187 inches diameter	0·0939 inches	Circular	33394 pounds

TABLE No. 4.—REMARKS.

This table is extracted from the results of experiments, published in Mr. Edwin Clark's work on the Britannia and Conway Tubular Bridges.

There is some difficulty in arriving at general conclusions with regard to the resistance of wrought-iron to compression, principally because that material in its *simple* forms, is not well adapted to resist compressive strain, by reason of its tendency to flexure.

We consider that long struts should be well braced, in order to prevent the element of length from complicating the calculation of strength, for, by this precaution, the strength may be rendered tolerably uniform.

We cannot, from these experiments, arrive at any certain rule for the resistance of plates to compression, but those of which the experimental tubes are formed, are generally very thin, compared to those used in practice, wherefore, we think, that for well-braced elements of wrought-iron in compression, and of moderate thickness, that from three to four tons per sectional square inch, is not too great a working load.

The results marked † did not break at the strain given.

TABLE No. V.
THE TENSILE STRENGTH OF STEEL.

Manufacturer.	Description.	Area.	Strength per square inch.
Turton and Sons	Forged from bars	0·2480 square inches	132909 pounds
T. Jowitt	Forged from bars	0·2552 square inches	132402 pounds
T. Jowitt	Forged from bars	0·2630 square inches	124852 pounds
T. Jowitt	Forged from bars	0·2552 square inches	115882 pounds
T. Jowitt	Forged from bars	0·2529 square inches	118486 pounds
Bessemer	Forged from bars	0·4150 square inches	111460 pounds
Wilkinson	Forged from bars	0·2665 square inches	104298 pounds
T. Jowitt	Forged from bars	0·2597 square inches	101151 pounds
Moss and Gamble	$\frac{3}{4}$ inch round bars	0.4417 square inches	107286 pounds
Naylor, Vickers, and Co.	$\frac{3}{4}$ inch round bars	0·4387 square inches	106615 pounds
Krupp...	Rolled bars	0·6614 square inches	92015 pounds
Shortridge and Co.	$\frac{9}{16}$ rivet rods	0·2485 square inches	90647 pounds
Shortridge and Co.	Forged bars	0·4477 square inches	89724 pounds
T. Jowitt	Forged $\frac{3}{4}$-inch bars	0·2441 square inches	72529 pounds
Mushet	Forged bolts	0·2704 square inches	75119 pounds
Mersey	Forged bars	0·4487 square inches	71486 pounds
Blockairn	Rolled bars	0·4873 square inches	70166 pounds
Blockairn	Forged from slabs	0·4496 square inches	65255 pounds
Blockairn	Forged from bars	0·4626 square inches	62769 pounds
Turton and Sons	Plate with the grain	0·5190 square inches	94289 pounds
Turton and Sons	Plate across the grain	0·5290 square inches	96308 pounds
Naylor and Co.	Plate with the grain	0·5000 square inches	81719 pounds
Naylor and Co.	Plate across the grain	0·5000 square inches	87150 pounds
Moss and Gamble	Plate with the grain	0·4500 square inches	75594 pounds
Moss and Gamble	Plate across the grain	0·5230 square inches	69082 pounds
Shortridge and Co.	Plate with the grain	0·3700 square inches	96280 pounds
Shortridge and Co.	Plate across the grain	0·3800 square inches	97150 pounds
Shortridge and Co.	Plate across the grain	0·7800 square inches	96989 pounds
Shortridge and Co.	Plate with the grain	0·491 square inches	72408 pounds
Shortridge and Co.	Plate across the grain	0·493 square inches	73580 pounds
Mersey Co., Puddled Steel	Plate with the grain	0·278 square inches	101450 pounds
Mersey Co., Puddled Steel	Plate across the grain	0·272 square inches	84968 pounds
Mersey Co., Puddled Steel	Plate with the grain	0·491 square inches	102593 pounds
Mersey Co., Puddled Steel	Plate across the grain	0·494 square inches	85365 pounds
Mersey Co., Puddled Steel	Plate with the grain	0·494 square inches	77046 pounds
Mersey Co., Puddled Steel	Plate across the grain	0·494 square inches	67686 pounds
Mersey Co., Puddled Steel	Plate with the grain	0·492 square inches	71532 pounds
Blockairn, Puddled Steel	Plate with the grain	0·376 square inches	102234 pounds
Blockairn, Puddled Steel	Plate across the grain	0·380 square inches	84398 pounds
Blockairn, Boiler Plate	Plate with the grain	0·585 square inches	96320 pounds
Blockairn, Boiler Plate	Plate across the grain	0·624 square inches	73699 pounds

TABLE No. 5.—REMARKS.

Steel being a combination, is, like cast-iron, liable to considerable variations of strength; thus we observe from the table, that it varies from 62769 pounds per square inch to 132909 pounds, which is more than twice as great.

It is highly probable that so long as cast-steel is used, these variations will occur, but by the employment of puddled-steel, now so successfully manufactured at the Mersey Iron Works, we may expect to obtain a greater uniformity of strength.

We have already made some observations on the chemical constitution of steel, but we may here observe, that M. Fremy has proved, by recent research, that steel may be formed by causing iron to combine with carbon and nitrogen.

TABLE No. VI.

DIMENSIONS, WEIGHT, STRAIN, AND COST OF VARIOUS CAST-IRON BRIDGES.

Name.	Description.	Length.	No. of Spans.	No. of Ribs.	Greatest Span.	Depth of Rib.	Rise or Versine.	Distance between Ribs.	Width of Platform.	Weight of Iron.	Strain per Sq. Inch.	Reputed Cost.
		Feet. In.			Feet. In.	Feet. In.	Feet. In.	Feet. In.	Feet. In.	Tons.	Tons.	£
Reading	Straight girder	62 10	1	2	55 10	4 0			12 4	49		
Austerlitz	Arched rib		5	7	106 0	4 4	10 7	6 5	45 0	623	2·78	
Carrousal	Do. do.		3	4	154 2	2 10	16 1	9 2	36 8	546	1·46	
St. Denis	Do. do.		1	4	102 5	2 10	11 4	{ 6 10 / 4 3 }	22 2	246	1·37	
Villeneuve	Do. do.		3	7	49 3	2 3	4 11	4 5	31 0	363	1·98	
Du Mèe	Do. do.		3	7	131 3	5 9	16 5	4 5	31 0	824	1·65	
Goude Choren ...	Do. do.		2	7	115 0	3 3	13 1	4 5	31 0	700		
Bernière	Do. do.		3	6	72 2	1 8	8 0	{ 3 9 / 5 0 }		213	1·47	
Montereau... ...	Do. do.		4	6	80 8	1 8	10 2	{ 3 9 / 5 0 }		240		
Nevers	Do. do.		7	7	137 9	3 9	15 0	4 3	29 9	800	1·90	
Rhone	Do. do.		7	8	197 10	5 7	16 5	4 1	32 8	1800	2·37	
Mulatiere	Do. do.		7	9	137 8		14 9	{ 4 1 / 5 7 }		600	1·91	
Westminster ...	Do. do.	1160 0	7	15	120 0	{ 2 7 / 1 10 }	20 0	{ 5 2¾ / 7 9 }	83 0	3100	3·00	235000
Southwark... ...	Do. do.	800 0	3	8	240 0			4 0	42 6	4585		384000
Vauxhall	Do. do.	830 0	9		80 0	6 0	24 0		36 2			300000
Stamford	Do. do.		1	6	90 0	2 4	12 0	{ 6 0 / 5 0 / 4 6 }	25 0	231	1·00	
Sunderland ..	Do. do.		1	6	200 0	5 10	30 0	6 0	30 0	260		26000
Buildwas	Do. do.		1	3	130 0	3 10	27 0	9 0	18 0			
Coalbrookdale ...	Do. do.		1	5	100 0		45 0					
Rochester	Do. do.	485 0	3	8	170 0	4 6	17 0	5 10½	40 0	2500		
Tewkesbury ...	Do. do.	196 0	3		57 0		5 2			520		10192
Boston	Do. do.		1		100 0		4 0					
Standish	Do. do.	133 6	1	5	83 4	1 6	11 0	8 10	35 2			

TABLE No. VII.

DIMENSIONS, WEIGHT, STRAIN, AND COST OF VARIOUS WROUGHT-IRON BRIDGES.

Name.	Description.	Length.	No. of Spans.	No. of Ribs.	Greatest Span.	Depth of Rib.	Rise or Versine.	Distance between Ribs.	Width of Platform.	Weight of Iron.	Strain per Sq. Inch.	Reputed Cost.
		Feet. In.			Feet. In.	Feet. In.	Feet. In.	Feet. In.	Feet. In.	Tons.	Tons.	£
Britannia	Tubular	1511 0	4	2	460 0	30 0			14 0	9360	5·62	601865
Conway	Do.	424 0	1	2	400 0	25 5			14 0	2892	4.50	145190
Moissac	Do.	1031 0	5	2	221 0	18 0				2020	3·80	
Aiguillon	Do.	559 0	3	2	208 0	18 0				994	3·80	
Victoria, Montreal	Do.	9500 0	25	1	330 0	22 0			16 0	10400		1350000
Asnieres	Tubular Girder	551 0	5	5	101 0	7 6			40 0	1036	3·17	
Victoria, Australia	Do. do.	216 0	1	3	200 0	14 3		19 3	40 0	490	5·29	
Trent Lane ...	Do. do.	264 0	3	2	96 0	8 0		26 0	29 0			
Langon	Plate do.	673 2	3	2	245 0	18 0			27 2	954	3·80	
Allier	Do. do.	1107 0	9	2	131 0	9 3			28 2	592	4·06	46000
Ciron	Do. do.	108 0	1	3	98 5	{ 4 7 6 6 }			28 9	74	3·80	
Staines	Do. do.	278 0	3	3	88 6	7 6			30 6	364		10000
Clichy	Do. do.	110 6	1	2	67 9	6 6			46 0	72	3·17	
Via del Mar ...	Do. do.	300 0	6	4	50 0	3 0		{ 8 0 4 0 }	29 6	72	2·70	
Kaffre Azzayat...	Do. do.	1400 0	11		104 0				42 0			
Newark Dyke ...	Triangular Girder	259 0	1	4	240 6	16 0			15 2	485	5·32	11003
Jumna	Do. do.	3224 0	15	4	219 6	16 5		{ 13 0 7 0 }	35 0	12015	5·14	
Sursuttee	Do. do.	364 0	3	2	88 10	6 9			14 2			3400
Londonderry ...	Do. do.	66 9	1	2	60 0	6 0			13 3		3·00	1320
Crumlin	Do. do.	1800 0	10	4	150 0	15 6		{ 9 0 6 0 }	26 0			27200
Taptee	Do. do.	1891 0	30	2	60 0					727		42000
Boyne	Lattice Girder	550 0	3	2	264 0	22 6			24 8	792	5·00	140000
Prescott St. ...	Do. do.			2	73 3	17 2½			26 0		5·00	
Lerida	Do. do.	712 0	5	2	131 3	10 6		16 0	16 0	359		
Deepdale	Trellis do.	660 0			58 4	6 4				900		
Beelah	Do. do.	1000 0	16	3	60 0	5 10½		11 0	26 0			
Charing Cross ...	Do. do.	1365 0	8	2	154 0	13 6		49 4	67 4			160000
Londonderry ...	Do. do.	1161 0	8	2	119 0	17 0		24 0	{ 24 0 30 0 }			
Leven...	Do. do.	1562 6	48	4	36 0					395		18604
Kent	Do. do.	1566 0	50	4	36 0					406		15056
Harper's Ferry...	Truss do.	128 0	1		124 0					44		
Swale...	Do. do.	120 0	2	5	60 0	6 0		{ 6 4½ 7 11 }	28 7½			
Windsor	Bowstring do.	213 5	1	3	182 10		25 0		35 0	393	1·92	
Shannon	Do. do.		2	2	165 0		20 2		28 0	311		25000
Chepstow	Do. do.	305 0			304 10		50 0		41 2			
Saltash ...	Do. do.	2240 0	19		445 0				17 0	4000		225000
Pimlico	Arch do.	920 0	6	4	175 0	3 6	17 6	{ 13 2½ 2 0 }	30 9	1521		90000
Hungerford ...	Suspension	1353 0	3		676 6		50 0		14 0			98760
Union	Do.				449 0		30 0		18 0			
Menai	Do.				579 11		43 0		28 0			
Brighton Pier ...	Do.	1336 0	4		255 0		18 0		12 8			
Hammersmith ...	Do.		3		422 3		29 6		30 0			
Broughton... ...	Do.		1		145 6		12 6		18 3			
Paris	Do.		1		557 6		37 2		28 6			40000
Des Invalides ...	Do.		3		236 6		26 4		25 8			
De Tournon ...	Do.	557 6	2		278 9		25 4		13 9			
Geneva	Do.	268 9	2		109 0				7 6			
Argental	Do.		1		350 5		25 9		13 9			
Vienna	Do.		1		334 0		21 3		30 0			
Fribourg	Do.		1		820 0		63 6		21 0			24000
Chelsea	Do.	704 0	3		348 0		29 0		47 0			88000
Niagara	Do.		1		800 0				24 0			80000
Twerton	Do.	230 0	3		120 0				14 0			

PART II.

PRACTICAL.

CHAPTER XI.

PRACTICAL APPLICATION OF THE FORMULÆ.

In the present chapter we purpose showing which of the formulæ obtained from the foregoing investigations are of practical utility, and also those methods of applying them which are most convenient and expeditious; we shall, therefore, prefer such expressions as may be most readily remembered and applied, provided they approach the truth sufficiently nearly, to more complicated ones, even though these latter have the advantage of absolute mathematical accuracy.

Although the effects of the greater number of strains, and of all the most important may be ascertained by calculation, yet there are many forces to the action of which structures are constantly liable, which we are totally unable to estimate with any degree of accuracy, even with the assistance of the most refined methods of analysis, and these forces frequently result from the peculiar nature of the material of which the structure is composed.

The course which we must pursue when determining the dimensions of any structure will be as follows:—

First, determine the number and nature of calculable strains to which the structure will be subject, and also their intensity; sufficient material must be provided to withstand the action of these strains without any possibility of its elasticity being impaired by them.

Secondly, we must provision for those strains which do not admit of calculation, and in this operation we must be guided almost entirely by experience, and if any case occur having no satisfactory precedent, we must allow ample material and dispose it in such manner as appears most likely to meet the requirements of the case; and we shall, in the following pages, point out as far as possible the most satisfactory methods of guarding against such contingencies.

We will consider the application of the formulæ in the same order which is adopted in those investigations from which they were deduced, commencing, however, with the treatment of direct tensile compressive and shearing strain, as we shall have to use the data requisite for the determination of those elements which resist direct strain only throughout the remainder of the present work.

The value of the ratio of the working strain to the ultimate stress, as stated in a previous chapter, is

$$f\,w = \tfrac{1}{5} \text{ for wrought-iron and steel}$$
$$= \tfrac{1}{6} \text{ for cast-iron.}$$

The values of the ultimate strength, etc., for various kinds of steel and wrought and cast-iron, may be obtained by inspection of our tables of strength of materials, but we shall here adopt the mean values for ordinary specimens. The following tabulated forms show these quantities in tons per square inch of sectional area:

	Tension.	Compression.	Shearing.	Modulus Elasticity.
Steel... ...	40 ...	— ...	—	29,000,000 lbs.
Iron Plates ...	25 ...	17 ...	20	——
Iron Bars ...	28 ...	17 ...	20	29,000,000 lbs.

	Tension.	Compression.	Shearing.	Modulus Elasticity.
Iron Wire Cable.	40	—	—	15,000,000 lbs.
Cast Iron ...	9	49	14	16,000,000 lbs.

The modulus of elasticity is given in pounds per square inch.

The working strain will be for steel in tension 8 tons per square inch, wrought-iron in tension 5 tons per square inch, in compression 3·5 tons, and under shearing strain 4 tons per square inch, wire cable in tension 8 tons per square inch, cast-iron in tension 1·5 tons, in compression 8 tons, and under shearing 2·33 tons per square inch. It will often happen in cast-iron, especially when the casting is very large, that it will not be of uniform texture and density, throughout we must, therefore, make some allowance for such imperfections, according to the size of the work and the position in which it is cast.

Let it be required to construct suspension chains subject to a strain of 1000 tons at the centre, then the effective area in square inches.

$$= \frac{1000}{5} = 200$$

if there be two chains, the area of one will be 100 square inches at the centre; we will supply this by 10 bars 1 inch thick and 10 inches deep, alternated by 9 bars $1\frac{1}{9}$ thick and 10 inches deep, then to find the diameter of the connecting pin, we have (see p. 57)

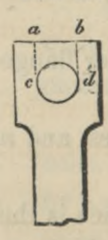
$$d = \sqrt{\frac{W}{1{\cdot}5708\,(n-i)\,S}}$$

$$= \sqrt{\frac{500}{1{\cdot}5708\,(9)\,4}}$$

$$= \sqrt{\frac{500}{56{\cdot}55}} = 3 \text{ inches nearly.}$$

But the apertures through which the pins pass will cause a diminution of area, and if no excess of strain has been allowed, the ends of the bars must be made 3 inches wider than the central parts, and this should always be done, otherwise the chain, if strong enough to bear the load, will possess a certain amount of material, which may be dispensed with.

FIG. 53.



Let Fig. 53 represent one end of one of the central bars, then the strain on the pin will also produce a shearing strain on the sections *a c*, *b d*, (see p. 40), therefore the sum of the areas of all these sections for one set of bars must be sufficient to resist the whole tension of one chain as shearing force, and must therefore,

$$= \frac{500}{4} = 125 \text{ square inches.}$$

There are 10 bars, therefore 20 sections, each 1 inch wide, or 9 bars, 18 sections, each $1\frac{1}{9}$ wide, which amount to the same, the total gives a section 20 inches, therefore the overlap or length of each section will be,

$$\frac{125}{20} = 6{\cdot}25 \text{ inches.}$$

The dimensions of the chain and pins at every other joint may be similarly calculated, as also the suspension rods and pins by which they are attached to the main chain.

We will now take an example of compressive strain, in which case the effective area will not be diminished by rivet or bolt holes, as the thrust will be transmitted through the rivet or bolt to the material on the opposite side of it. Let it be required to provide three cast-iron arched ribs to sustain a compressive strain at the centre or crown equal to 1200 tons, then will each rib be subject to a compressive force equal to 400 tons. We have called the compressive resistance of cast-iron, 8 tons per square inch for nett area, we will allow for imperfect texture, etc., 2 tons, which will reduce the working strain to 6 tons per square inch. The area of each rib will be at the centre,

$$\frac{400}{6} = 66{\cdot}66 \text{ square inches.}$$

If the rib be of wrought-iron,

$$\frac{400}{3 \cdot 5} = 114 \cdot 28 \text{ square inches.}$$

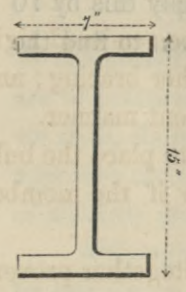
The strain will pass direct from one element to another. We shall therefore neglect the joints, as regards the dimensions of rivets, etc., which will be fully explained in the chapter devoted to the consideration of rivetted and other joints. Any other part of the arched rib may be calculated in like manner when the direct force to which it will be subject is known.

Having given these examples of the method of dealing with direct strain, we will pass on to the calculation of direct strain produced on the various elements of bridge structures by the loads to which they may be subject.

Our first subject is the resistance of materials to transverse strain.

In calculating the strength of girders of small depth and considerable thickness of material, it will be necessary to determine the moment of inertia by means of a formula suitable to the section, which will be found in Part I., Chapter II., and the method of proceeding will be as follows:—

FIG. 54.



It is required to find the moment of resistance of a cast-iron girder of the section shown at Fig. 54, the depth being 15 inches, the breadth 7 inches, and the thickness $1\frac{1}{2}$ inches, we have for the moment of inertia,

$$I = \frac{b\,d^3 - b'\,d'^3}{12} = 1178 \cdot 58$$

And for the moment of resistance,

$$M = \frac{2\,I\,s}{d} = 157 \cdot 14\ s.$$

Where s equal the direct resistance of the material, but s has two values, one for tension, and the other for compression. The one flange of the girder is in tension, and the other in compression, hence one of them will yield to the strain before the full resistance of the other is brought into action.

If we make the girder with equal flanges, we must adopt the lowest value of s, but in this case, there will evidently be a very considerable excess of material in one flange; it will, therefore, be most advisable to proportion the flanges so that they may be of equal strength.

The ratio between the compressive and tensile working strains $= \frac{15}{80} = \frac{3}{16}$, and we must so proportion the girder that the flanges will be subject to strains whose relative intensities are 3 and 16.

If we neglect the web of the girder, which is usually done, the moment of resistance of either flange will be about $n\,a\,s$.

When the flange is thin, a being the sectional area in inches, s the working strain in inches, and n the mean distance of the flange from the neutral axis or centre of gravity of the two flanges.

We have previously stated, that the neutral axis is at the centre of gravity when the resistance is the same for tension and compression, but such is not the case with cast-iron; therefore the foregoing formula do not apply to it, and we must make the flanges of equal strength, in order that the neutral axis may be at the centre, therefore, the areas of the flanges must be in the proportion of

$$1 \text{ to } 5\,\tfrac{1}{3}.$$

Mr. Hodgkinson found, from his own experiments, that this was the proportion which gave equal strength; that is to say, that the result of those experiments gave proportions very nearly approaching the above.

Mr. Hodgkinson neglected the webs of the girders in his experiments, which in some cases differed, and upon this difference calculations have been based which give as the ratio of the flanges of a cast-iron girder of maximum strength.

$$1 \text{ to } 3 \cdot 5, \text{ to } 1 \text{ to } 4 \cdot 5.$$

If we neglect the web, and call A the area of either flange, s its resistance, and d the distance cetween the centres of gravity of the two flanges, the moment of resistance will become for either flange,

$$M = \frac{s\,A\,d}{2}$$

which being equated with the moment of strain, will give an equation from which the area of the flange may be found when the load is known.

We shall hereinafter combine this expression with those for the moment of strain under various different conditions of load.

For wrought-iron girders the same formulæ will obtain, but in this case the ratio of resistance to tension and compression, is $\frac{35}{50} = \frac{7}{10}$.

In calculating wrought-iron girders, we must consider the effect of rivet-holes, and it will frequently be found convenient to take both the resistances as 3·5 tons per square inch, and consider the gross area of each flange as the effective area, when we shall not be far off from the truth, as the rivet-holes do diminish the strength of the tension member, whereas they have no such effect on that member which sustains the compressive force.

If any long pieces of metal be subject to compressive strain, whether they are elements of girders or otherwise, they will necessarily assume the same condition as pillars, but we proportion the area of struts as if they were only subject to a crushing strain, therefore means must be taken to prevent them from bending. In short struts this may be done by making them of an + section, in which case the feathers will impart the required amount of rigidity. If the struts be too long to be rendered rigid by such construction they must be braced to other parts of the structure.

The compression member of a girder is in the condition of a long strut, and it may be rendered rigid in various ways;—first, by making it in the form of a tube or box; secondly, by plate or other bracing; and thirdly, by the attachment of feathers formed of angle- or T-iron, or in any other convenient manner.

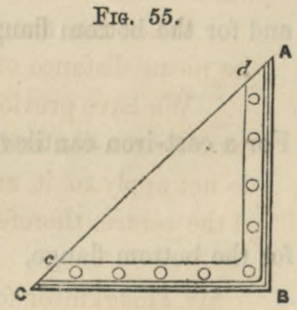
In designing the compression member, it should be borne in mind that it is desirable to place the bulk of the material of which it is composed as far from the neutral axis as possible, therefore if the member be a box or tube, such box or tube should be made as flat as is consistent with safety.

If the girder have a plate web, this will greatly stiffen the flanges, but will not altogether prevent their buckling at the edges, especially if the flange be very wide; it will, therefore, be necessary to attach feathers, as above described, at these places.

The resistance to crumpling in wrought-iron increases as some power of thickness between the square and the cube very nearly as 2·2 power. There will also be a considerable tendency of the flanges to approach each other, thereby crumpling the web which connects them, and this force must be withstood by rigid standards, or stiffening plates placed at frequent intervals along the girder.

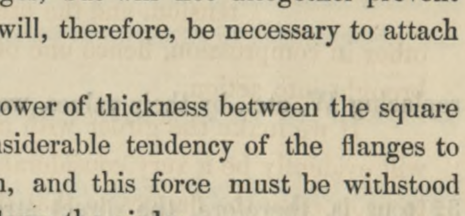
Such strain as tends to produce an alteration in the relative angular positions of any two elements; such, for instance, as the side and top of a rectangular tube, may be provided for by the attachment of gusset plates.

Fig. 55.



Let A B, Fig. 55, represent the side, and B C the bottom of a rectangular tube, and sppose the tendency to alteration of angular position to be withstood by a gusset plate *a* B *c*, then there will be a moment of strain about the point B as an axis, which will probably produce compressive and tensile strains alternately, and the gusset plate will be in condition of half a girder, subject to a transverse load, the direct strain on any fibre being equal to nothing at B, and increasing in proportion to the distance from B measured on a line perpendicular to *a c*, hence the greatest strain is on the fibres at the edge *a c* of the gusset; it would, therefore seem desirable to strengthen this part of the plate by angle-iron, or it might be sufficient to substitute the gusset plate by an open triangle made of T-iron.

Fig. 56.



Let A B C D, Fig. 56, represent two main girders connected by cross girders A C B D, then any change of value of the angles may be prevented by the use of light tie bars A D, C B, attached to the system at or near the angles; then it is evident that no alteration of form can occur without throwing an amount of tension upon one or the other of the diagonal ties. These tie bars should not be united at the point *e*, as they are intended to resist tension only.

The brace, if intended to resist compression, should be made with feathers, similar to those upon struts, and it may frequently be convenient to make the brace of cast-iron, where it has to resist a compressive force only; but, in any case, where there is the possibility of compressive strain, the diagonals should be united at their intersection. Let it be required to construct a semi-beam or cantilever, fixed at one end to support a weight of 10 tons at the extremity, its length being 4 feet, and its depth 15 inches at the fixed extremity.

The formula for the moment of resistance, is

$$M = (R' + R'')\frac{d}{2}$$

where R′ and R″ are the resistances of the top and bottom flanges, but these being equal,

$$M = R\,d$$

and for the moment of strain at any point, we have,

$$M = W\,x$$

and at the point of support,

$$M = W\,l$$

but the moments of strain and resistance must be equal, hence we can find the direct strain or resistance of either flange, thus:—

$$R\,d = W\,x$$

$$\therefore R = \frac{W\,x}{d}$$

$$\text{also } R = \frac{W\,l}{d}$$

at the point of support, but if s equal the working strain per square inch, and a the area of either flange,

$$R = s\,a$$

$$\therefore A = \frac{W\,l}{s\,d}$$

replacing W, etc., by their values, we find,

$$A = \frac{10 \times 4}{s \times 1{\cdot}25} = \frac{32}{s}$$

32 tons is, therefore, the direct strain on either flange. We will replace s for wrought- and cast-iron, observing that the top flange in tension, and the lower one in compression, then for a rolled girder with no rivetted joint, for the top flange,

$$A = \frac{32}{5} = 6{\cdot}4 \text{ square inches,}$$

and for the bottom flange,

$$A = \frac{32}{3{\cdot}5} = 9{\cdot}14 \text{ square inches.}$$

For a cast-iron cantilever, for the top flange,

$$A = \frac{32}{1{\cdot}5} = 21{\cdot}33 \text{ square inches;}$$

for the bottom flange,

$$A = \frac{32}{8} = 4 \text{ square inches.}$$

Let a semibeam be required 6 feet long and 1 foot deep, to support an uniformly distributed load of 1·25 tons per foot-run, then the moment of strain at any point, is

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

whence

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

$$\therefore A = \frac{w\,x^2}{2\,s\,d}.$$

Inserting the above values,

$$A = \frac{1 \cdot 25\, x^2}{2 \,.\, s}.$$

And at the point of support, where $x = l$

$$A = \frac{1 \cdot 25 \times 36}{2\, s} = \frac{22 \cdot 5}{s}.$$

For wrought-iron, top flange,

$$A = \frac{22 \cdot 5}{5} = 4 \cdot 5 \text{ square inches;}$$

bottom flange,

$$A = \frac{22 \cdot 5}{3 \cdot 5} = 6 \cdot 42 \text{ square inches.}$$

For cast-iron, top flange,

$$A = \frac{22 \cdot 5}{1 \cdot 5} = 15 \text{ square inches;}$$

bottom flange,

$$A = \frac{22 \cdot 5}{8} = 2 \cdot 81 \text{ square inches.}$$

The dimensions of the flanges at any other point will be similarly determined. It now remains to calculate the thickness of the web, supposing it to sustain the shearing force. In the first case the shearing force is 10 tons, hence for wrought-iron the area of the web must be

$$\frac{10}{4} = 2 \cdot 5 \text{ square inches,}$$

and, as the web is 15 inches deep, the thickness must be 0·1666 inches; but this is less than could be conveniently given in practice, hence the web will always have an excess of strength in this respect; but greater strength will be required to prevent the web from crumpling.

We shall not in all future cases carry the calculation out, but we have done so in the last in order to exhibit the method.

We will now determine the formula for beams supported freely at both ends. Let L equal the span, d equal the depth of the girder, and W equal the concentrated load; also in other cases, let w equal distributed load per foot-run.

x being the distance of any point from one point of support, M the moment of strain, and S the total strain on either flange, and therefore the resistance of either flange

$$M = -\frac{W}{2} x \text{ for a central load,}$$

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

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

And at the centre, the point of maximum strain, where $x = \frac{l}{2}$

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

for a load the distance $x + z$, from the point of support,

$$M = -W \left\{ \frac{l - x - z}{l} \right\} x$$

$$\therefore S = W \left\{ \frac{l - x - z}{l} \right\} \frac{x}{d}$$

And for the point of maximum strain, where x is also the distance of the load from the point of support,

$$M = -W \left\{ \frac{l - x}{l} \right\} x$$

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

When the load is at a distance $x - z$ from the point of support.

$$M = -W\left\{\frac{x-z}{l}\right\}\{l-x\}$$

$$\therefore S = \frac{W}{d}\left\{x - z - \frac{x^2 - x\,z}{l}\right\}.$$

For an uniformly distributed load, the strain at any point is,

$$M = \frac{w\,x}{2}\{x-l\}$$

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

The direct strain we consider as positive, whatever may be the sign of the moment, but when the moment is + M the top flange is in tension, and the bottom one in compression, and when *vice versâ*—when the moment is — M, thus, in the semi-beam—we find the moment positive, but in the entire beam it is negative; for, although in the last value of M the first sign is positive, yet in the factor $(x - l)$ the negative quantity is the greater.

At the centre of the beam which is the point of greatest strain, $x = \frac{l}{2}$ and

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

By calculating the moments of strain for a number of points in the span of a girder, or bridge, and laying down these moments to an arbitrary scale, we are enabled to draw a curve which shows the strain at any point whatever, and the area of the curve will represent the sum of all the moments of strain; and as the sectional area depends upon the moment of strain, when the depth is constant, the quantity of material, or the weight of the bridge, will be theoretically represented by the area of the curve; the nature of the curve will, of course, depend upon the method of loading.

To find the direct strain on either flange at any point in the girder by means of the curve of moments, we measure the ordinate to the curve at that point by a proper scale of moments, and divide the value so found by the depth of the girder at that point from the centre of gravity of the upper flange to that of the lower. Thus we can find the proper area of the flanges at any point, and proportion the thickness of the plates accordingly.

We have calculated and plotted two curves of moments, shown on diagram No. 1, exhibiting the moments on every point of the bridge erected to carry Victoria Railway, in Australia, over the Saltwater River.

The span of this bridge is 200 feet, and its weight 2·268 tons per foot-run.

The small curve is constructed on the supposition that the bridge is subject to the action of a rolling load of 30 tons, in addition to its own weight, and the ordinate at any point represents the moment of strain at that point, when the moving load is over that point; hence this may be called a curve of maximum moments.

The larger curve shows the moments of strain when the girder is subject to a load of 2 tons per foot-run, as well as its own weight.

The second figure on the diagram shows the shearing strains, that line which refers to the rolling load is slightly curved, but the shearing strains for the continuous load are shown by two straight lines, meeting each other, and the axis of x at the point of maximnm horizontal strain, which is in this case the centre of the span.

The shearing strains are represented by their ordinates to the straight lines.

We will here show the method which we followed in the calculation of the curve. The ordinates were calculated for points at every 10 feet of the girder.

The general equation to the moment of strain for an uniform load is,

$$M = \frac{w\,x}{2}\{x-l\} = 2{\cdot}134\,x^2 - 426{\cdot}8\,x.$$

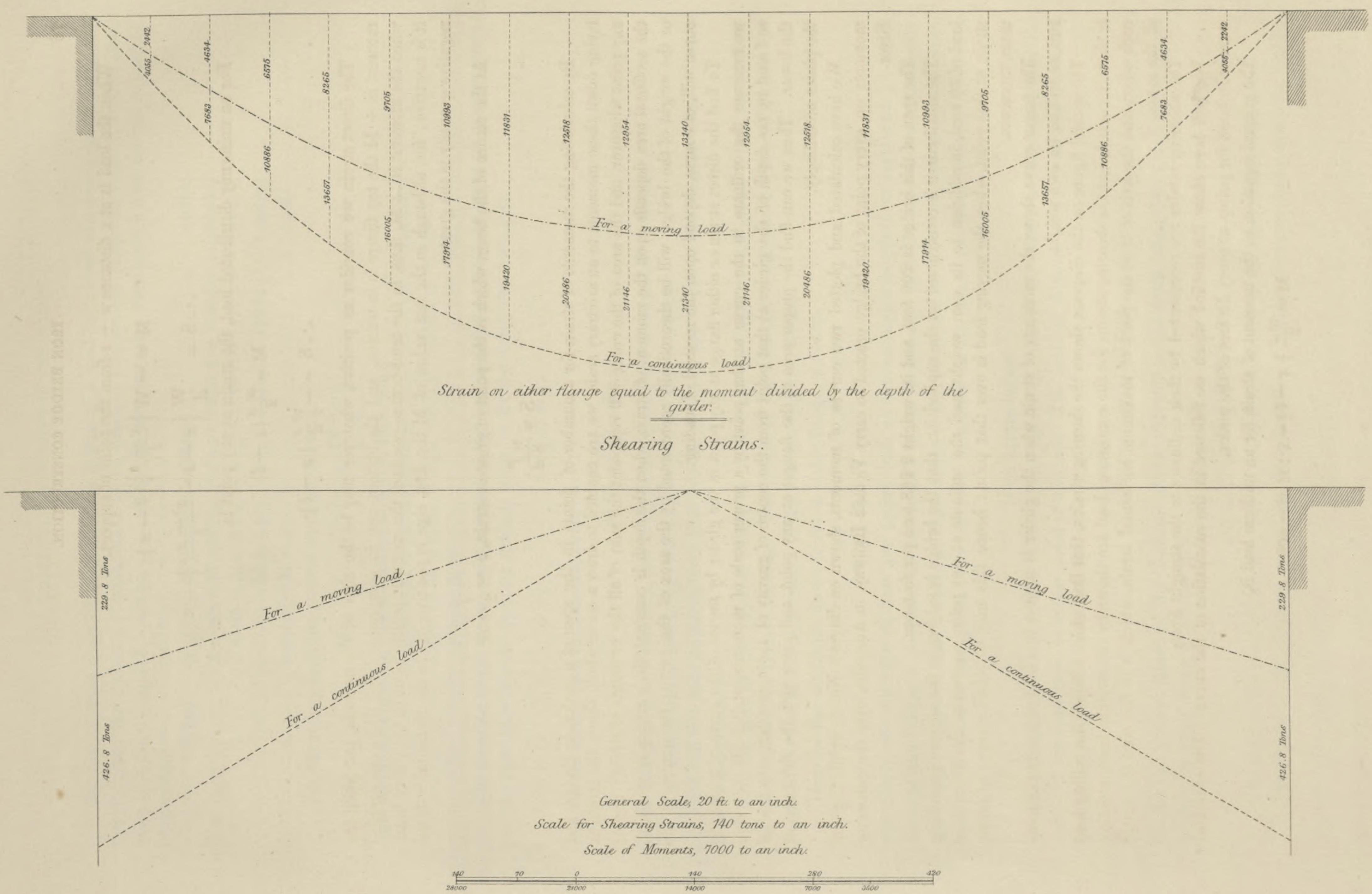
To face page 76.

BRIDGE OVER SALT WATER RIVER, AUSTRALIA.

Diagram Nº 1.

CURVE OF MOMENTS.

Span 200 feet. — Dead Weight 2.268 tons pr. foot. — Live Weight 2 tons pr. foot. — Moving Weight 30 tons.



Strain on either flange equal to the moment divided by the depth of the girder.

Shearing Strains.

General Scale, 20 ft. to an inch.

Scale for Shearing Strains, 140 tons to an inch.

Scale of Moments, 7000 to an inch.

W. Humber, dir.

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The first of these quantities varies as the square of x, and the second varies as x, we will call the first $P\,x$, and the second $q\,x$, replacing x by its value to indicate any particular value of P or q, thus, when $x = 10$, $P_{10} = 213{\cdot}4$ and $q_{10} = -4268$; the following table will show the most convenient method of procedure:—

$$\begin{array}{llll}
\text{When} & x = 10 = M = 213{\cdot}4 - 4268 = & & = -\ 4055{\cdot}6 \\
 & x = 20 = M = 4{\cdot}P_{10} - 2{\cdot}q_{10} = & 853{\cdot}6 - 8536 & = -\ 7683{\cdot}4 \\
 & x = 30 = M = 9{\cdot}P_{10} - 3{\cdot}q_{10} = & 1923{\cdot}6 - 12804 & = -\ 10880{\cdot}4 \\
 & x = 40 = M = 4{\cdot}P_{20} - 2{\cdot}q_{20} = & 3414{\cdot}4 - 17072 & = -\ 13657{\cdot}6 \\
 & x = \text{etc.} = M = & & \text{etc.}
\end{array}$$

For the rolling load, the general equation will be,

$$M = \frac{w\,x}{2}\left\{x - l\right\} - W\left\{\frac{l - x}{l}\right\}x = 1{\cdot}254\,x^2 - 256{\cdot}8\,x,$$

from which the curve may be calculated as before, thus, when

$$\begin{array}{lll}
x = 10\ M = 125{\cdot}4 - 2568 = & & = -\ 2442{\cdot}6 \\
x = 20\ M = 4\,P_{10} - 2{\cdot}q_{10} = & 501{\cdot}6 - 5136 & = -\ 4634{\cdot}4 \\
x = \text{etc.}\ M = & & = \ \text{etc.}
\end{array}$$

The shearing strain at any point is equal to the load included between the point of greatest strain, and the point at which the shearing strain is taken, or for the distributed load, it is

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

and for the rolling load the shearing strain is,

$$= W + \frac{w\,l}{2} - w\,x,$$

in the first case, $w = 4{\cdot}268$ tons, and in the latter, $w = 2{\cdot}268$ tons, and $W = 30$ tons.

The description of this bridge will be given in a subsequent chapter.

We will now take an example to illustrate the practical application of the formulæ when a bridge is to be constructed:—

Let the span of the required bridge be 75 feet, and the depth of each girder 7 feet, and let it be required to carry two lines of railway. The weight of the structure is taken at 1 ton per foot, and the weight of the load at 2 tons per foot. Total load per foot-run, 3 tons.

Then for the maximum direct strain on either flange, we have,

$$S = \frac{w\,l^2}{8\,d} = \frac{3 \times 75 \times 75}{8 \times 7} = \frac{16875}{56} = 301\tfrac{1}{2}$$

tons nearly, we will take $3\frac{1}{2}$ tons per square inch as the working strength of wrought-iron, (of which material the bridge is supposed to be required to be constructed) both for tension and compression, neglecting rivet-holes, then the area of either flange at the centre will be $A = \frac{301{\cdot}5}{3{\cdot}5} = 86$ square inches, nearly. If there be two girders this area must be halved, for the area of one flange which will therefore become,

$$= 43 \text{ square inches,}$$

we may calculate the strain at other sections, and reduce the area of the flanges as we proceed from the centre of the span towards either point of support. The greatest shearing strain, which is that at the point of support,

$$= \frac{w\,l}{2} = \frac{3 \times 75}{2} = 112{\cdot}5 \text{ tons.}$$

for the two girders, therefore, the thickness of one web will be its depth, being 7 feet, and the resistance to shearing strain being 4 tons per inch,

$$\frac{112{\cdot}5}{4 \times 84} = \frac{11}{32} \text{ of an inch, nearly.}$$

The areas of the cross-girders may now be calculated according to the rules already laid down.

The weight of the structure must then be calculated, a proper allowance being made for bracing, etc., and this being divided by the span will give us a near approach to the true weight of the bridge per foot-run; we may then substitute this weight for that taken for the first approximation, and from the data so found, finally calculate the true area of the girders.

The construction of the bed plates which support the end of the girders will be described in the Chapter on Foundations.

We will here make a few remarks with regard to the general laws by which the weights of bridges are regulated.

The weight of the main girders is a function of the area of the main girders and the length of the span, or calling W g this weight, A t the total area of the girders, $l =$ the span, and C a constant

$$\mathrm{W}\,g = \mathrm{C\,A}\,t$$

but the area is a function of the strain, and from the formulæ,

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

we find that the area varies directly as the weight per foot-run, and the square of the span, and, inversely, as the depth of the girder, for a roadway bridge the weight per foot-run varies as the width, therefore, calling the width of the bridge b, we find

$$\mathrm{W}\,g = \mathrm{C}\,\frac{b\,l^3}{d}$$

for a railway bridge w is constant, and

$$\mathrm{W}\,g = \mathrm{C}\,\frac{l^3}{d}$$

The weight of the stiffening-plates will be proportional to the weight of the girders, and the weight of the cross girders will be for a roadway bridge,

$$= \mathrm{C}'\,\frac{b^3\,l}{d'}.$$

Where d' equal depth of cross-girders, and for a railway bridge, the number of lines being constant,

$$= \mathrm{C}''\,\frac{l}{d'}\,.$$

The strength of the bracing should increase as the square of the span, therefore its weight will be

$$= \mathrm{C}'''\,l^3\,b$$

for any bridge, and

$$= \mathrm{C}'''\,l^3$$

for a railway bridge.

In these comparisons we suppose the railway bridges to be taken for the same number of rails in every case.

Calling $f'\ f''\ f'''$ the factors of the weights; of the main girder with stiffening plates; of the cross-girders and platform; and of the horizontal bracing, and W the weight of any bridge, W$'$ that of a railway bridge, we have

$$\mathrm{W} = f'\,.\,\frac{b\,l^3}{d} + \frac{f''\,b^3\,l}{d'} + f'''\,l^3\,b$$

$$= b\,l\,\left\{ f'\,.\,\frac{l^2}{d} + f''\,.\,\frac{b^2}{d'} + f'''\,l^2 \right\}$$

but $b\,l =$ the area of the platform, therefore the weight per square foot (if b and l are in feet), will be

$$= f'\,.\,\frac{l^2}{d} + f''\,.\,\frac{b^2}{d'} + f'''\,l^2.$$

$$\mathrm{W}' = f'\,.\,\frac{l^3}{d} + f''\,\frac{l}{d'} + f'''\,l^3$$

$$= l\,\left\{ f'\,.\,\frac{l^2}{d} + f''\,\frac{1}{d'} + f'''\,l^2 \right\}$$

hence, the weight per lineal foot, will be

$$= f'\,.\,\frac{l^2}{d} + f''\,.\,\frac{1}{d'} + f'''\,l^2$$

The values of the factors should be obtained from examples which have proved satisfactory in practice, in order to insure safe results.

From these formulæ we may readily estimate the weight of any bridge which may be required, as far as the iron-work is concerned. Other parts, such as timber, etc., will depend upon circumstances, for which no general rule can be laid down.

The factors will, of course, have different values for different forms of construction.

We will now regard the girder of a single span as fixed at one end; we shall find that the central part of a girder so circumstanced is nearly twice as strong as the same girder would be if its ends were left perfectly free.

The principal objection to this form of girder in bridge construction appears to consist in the great length of bearing and solidity of foundation indispensable at the fixed end. We shall not, however, neglect this case, as it may under some circumstances be desirable to fix one end of a small girder.

The general equation to the moment of strain is

$$M = \frac{w\,x^2}{2} + \frac{w\,l}{8}\{l - 5\,x\}$$

therefore the direct strain on either flange is

$$S = \pm\frac{w\,x^2}{2\,d} + \frac{w\,l}{8\,d}\{l - 5\,x\}$$

which at the point of greatest strain where $x = \frac{5}{8}l$, measured from the fixed extremity, becomes

$$S = \frac{w\,l^2}{14{\cdot}25\,d}\text{ nearly.}$$

At some parts of the span the moment of strain will have a *plus* sign, and at others a *minus* sign, hence there is a point of no strain, and this is the point of contrary flexure, in all girders fixed at one end, supported at the other, and loaded uniformly; it is at one-fourth of the span from the fixed extremity, and the upper flange is in tension, and the lower flange in compression from the fixed extremity to the point of contrary flexure; beyond this, the girder is in the same condition as one of three-quarters the span, simply supported at the ends, hence the girder is divided into two parts—a semi-beam, or beam fixed at one end and free at the other, but supporting at its free extremity one end of a girder, of which the other end is supported on a pier. The direct strain on either flange, immediately above the pier upon which the beam is fixed, is

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

The same as would exist at the centre of the span if both ends were free.

From this pier the strain diminishes to the point of contrary flexure, where it changes, whence it increases to the point of maximum strain, after which it diminishes to *o* at the pier beneath that extremity of the girder which is supported only. It is very important to notice the change of strain on the flanges, in order that it may be made of a suitable form.

A greater economy of material is obtained by the adoption of this method of construction than may at first sight appear, and it depends entirely upon a practical point, which we will here point out. In examining the curve of strain on a single girder, supported freely at both extremities, we find that it is greatest at the centre of the span, and that it diminishes to *o* at both points of support; hence, theoretically, those parts of the girder which bear the horizontal strain, namely, the flanges, should be gradually reduced, and should finally disappear at the points of support; but this cannot be carried out in practice; hence a certain amount of material at the point of support is actually wasted, because it does not assist in bearing the load. It is, therefore, evident that if we could relieve the flanges of a portion of the strain to which they are exposed at the centre, and cause it to be borne by the extremities, we should be able to reduce the sectional area at the centre, without giving rise to the necessity of any considerable increase at the ends. This is accomplished, with regard to one extremity, in the girder fixed at one end; but there is a somewhat considerable increase of strength over the pier; this, however, is more than neutralized by the reduction at the centre.

If the girder be of sufficient dimensions, it will be found desirable to reduce the area of the flanges slightly as the strain diminishes.

We will here insert a formula for the upward strain on the foundation at the fixed extremity :—

Let L = the length of bearing measured from the first line of contact with the pier to the most distant holding-down bolts. Then, if U = the mean lifting force when the bridge is fully loaded,

$$U = \frac{w l^2}{4 L}.$$

And this force must be withstood, either by bolting the end of the girder down to strong anchor-plates, carrying a large mass of masonry, or by imbedding the end of the girder in masonry.

This form of beam is most frequently to be seen employed to support brickwork or flooring, one end being firmly imbedded in the wall of a building, while the other extremity is supported either upon a beam or upon a pillar, as in the supports of galleries, etc.

Another, which frequently occurs in girders employed to support brickwork, is that of a beam fixed at both extremities. In this case the moment of strain is

$$M = \frac{w x^2}{2} + \frac{w l}{12} \{ l - 6 x \}.$$

Hence the direct strain on either flange is

$$S = \pm \frac{w x^2}{2 d} + \frac{w l}{12 d} \{ l - 6 x \}$$

which at the point of maximum strain, where $x = \frac{l}{2}$

becomes

$$S = \frac{w l^2}{24 d}$$

one-third of that which would exist if the ends of the girder were free, the strain on either flange over either pier will be

$$S = \frac{w l^2}{12 d}.$$

In this case we evidently obtain greater economy of material than in the last, for we make the metal in both ends of the girder useful. It will be observed in both these cases, that the strain over the pier is about twice that at the point of maximum strain at the central part of the girder. The lifting force on either pier, in the latter case, will be

$$U = \frac{w l^2}{6 L}$$

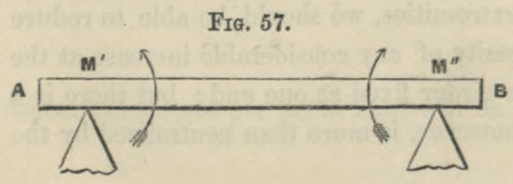
one-third less than in the case of the girder fixed at one extremity.

The beam fixed at both extremities cannot be applied to span any considerable obstacle, as the expansion and contraction of the metal, between summer and winter, would most certainly unsettle the foundation of the piers.

The shearing strains will be exactly the same in the last case as in a girder supported freely at both extremities, but not so in the former, for a greater part of the bridge will be borne on one point of support than upon the other. As we have before stated, the amount of shearing strain on the girder close to the pier, will be equal to the weight borne by that pier, and we will now try to make the method of finding this for fixed or continuous girders more evident. If a simple girder, loaded with an uniformly distributed weight, be supported at two points, half the total weight will be supported by each point of support, and will be represented by

$$\frac{W l}{2}.$$

Fig. 57.



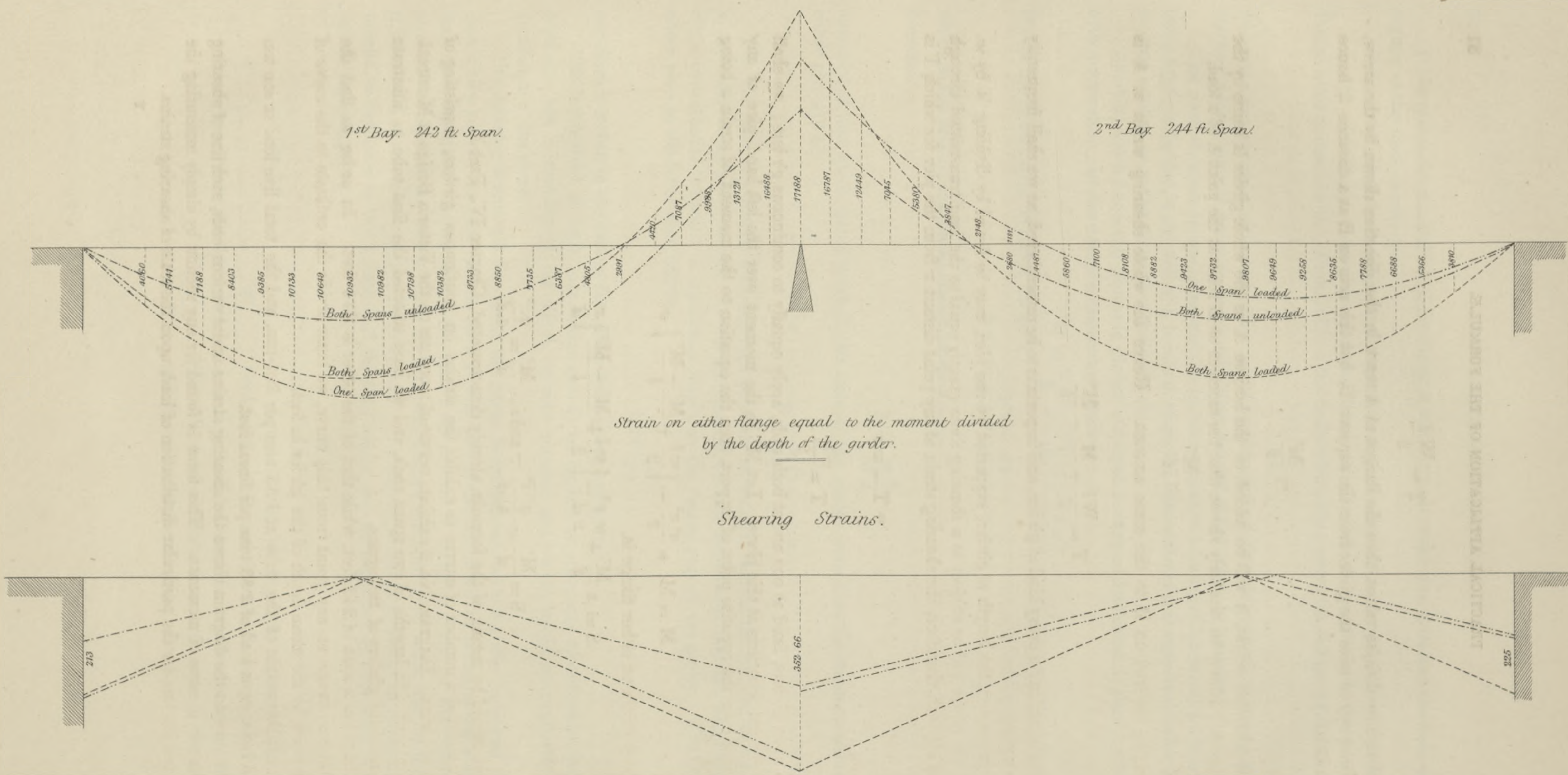
Let A B, Fig. 57, represent a beam, either supported or fixed at A and B, and if there be any moment of strain over A or B, or both, let them be represented by M′ M″, as shown. We will consider the shearing strain close to the support *a*, and call it, = T, if M′ = M″ = *o*,

To face page 81. Diagram No 2.

VICTORIA BRIDGE, MONTREAL.

CURVES OF MOMENTS, 1ST & 2ND BAYS.

Dead load 1·33 tons pr foot run. Live load 1 ton pr foot.



General Scale, 40 ft. to an inch.

Scale for Shearing Strains, 225 tons to an inch.

Scale of Moments, 9000 to an inch.

W. Humber, dir.

35. P. 238.

Standidge & Co. litho. Old Jewry, London.

London: E. & F. N. Spon, 16, Bucklersbury.

$$T = \frac{W\,l}{2}.$$

Suppose M′ M″ to have definite values, then the flanges at A react in the direction shown by the arrow, which shows a tendency to raise the girder from the support B, but this acts on B at a distance l, hence this tendency at B is

$$= \frac{M'}{l}.$$

But if b is relieved of this weight, it will be added to the load on A, as the whole girder is borne by the two points of support. It may be similarly shown that the moment at B relieves the point A of a load,

$$= \frac{M''}{l}$$

Therefore, increasing the load on B to the same amount. Hence the whole shearing strain at A is expressed by

$$T = \frac{W\,l}{2} + \frac{M'}{l} - \frac{M''}{l}.$$

this is the general equation for every kind of girder, and important to be remembered, as we shall frequently have occasion to use it.

From this expression, the length of girder, supported by one pier, may be found by dividing it by w. Each section of the girder will be subject to a shearing force exactly equal to the load transmitted through it to the point of support, therefore, the shearing strain at any point distant x from the pier for which T is taken, is

$$= T - w\,x$$

If $M' = M''$, then

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

All our remarks upon girders, fixed at one and at both ends, apply equally to continuous girders; we shall merely give the general equations at this place. Let M′ be the moment over the left hand pier of any span, M″ being that over the opposite point of support, then the equation to the moment of strain x being measured from M′ is,

$$M = M' + \frac{w\,x^2}{2} - \left\{ \frac{w\,l}{2} + \frac{M' - M''}{l} \right\} x$$

therefore, the direct strain on either flange is,

$$S = \pm \frac{M'}{d} + \frac{w\,x^2}{2\,d} - \left\{ \frac{w\,l}{2} + \frac{M' - M''}{l} \right\} \frac{x}{d}$$

And over the piers,

$$S = \frac{M'}{d} = \frac{q.'\,l^2}{4.\,d'} - \text{and } s = \frac{M''}{d} = \frac{q.''^2}{4.d}.$$

The q's being determined by means of the formulæ already demonstrated, Chapter IV., Part I.

We have calculated a number of curves to exhibit the strains on continuous girders, consisting of various numbers of spans. Diagram No. 2 exhibits curves of moments on the Victoria Bridge at Montreal. This bridge is constructed in lengths of two spans each, and we have selected the first tube to illustrate the strains on continuous girders of two spans.

One of the bays has a span of 242 feet, while that of the other is 244 feet. In order to find the direct strain on either flange at any point from this curve, we must divide the ordinate to the curve of moments at that point by the mean depth of the girder in feet.

We have taken the weight of the tube at 1·33 tons per foot-run, and the total live load at one ton per foot-run, making a gross load of 2·33 tons per lineal foot.

The second figure on the diagram shows the shearing strains under various loads, each line of shearing force corresponding to a curve of moments. This force is found for any section by simply measuring the ordinate to the straight line for the particular distribution of load, upon the scale of shearing strains.

The quantities marked upon the curves of moments show the maximum moments at various points in the girder.

Diagram No. 3 shows the curves of moments on the bridge at Staines, on the Woking and Wokingham Railway, which is also illustrated at Plate No. 30, 31, and 32. This bridge consists of three continuous girders, one in the centre and one on each side of the railway; it is in three spans, the centre being 88 feet 6 inches, and the outer ones being 85 feet 3 inches. The total weight of the structure alone, is 1·3 tons per foot-run, that of the live load two tons, making a gross load of 3·3 tons per lineal foot, a load which is by no means extravagant, when we consider that the permanent way is ballasted.

The method of finding the strain on the sum of either the top or bottom flanges, will be the same as before, the total shearing strains on the three girders are shown in the second figure.

Diagrams Nos. 4 and 5, represent the strains upon one tube of the Britannia Tubular Bridge; it consists of four spans; the two end openings have each a span of 230 feet, and the central bays are of 460 feet span. We have shown the curves on the two diagrams for the sake of clearness.

The weight of the tube is taken at 3·82 tons per foot-run; that of the load being one ton per foot, making a gross load of 4·82 per lineal foot.

Beneath the curves are shown the lines of shearing strain.

In designing a continuous girder of considerable dimensions, the various curves being plotted as in the diagrams, the girder must be made sufficiently strong at every point to bear the greatest strain which may be brought upon it.

Having shown which formulæ may be most conveniently applied practically, whereby just conclusions as to the amount of metal absolutely necessary to withstand such strains, as we perfectly comprehend, with regard both to their natures and intensities, may be arrived at; we will describe the form of girders intended to resist these and other forces to which they may be subject.

A plate-girder may be said to consist of three essential parts—a web and two flanges, which in theory would constitute a complete girder. These can under no circumstances be dispensed with, and of them we shall first speak.

Fig. 58 represents a section of the simplest form of plate-girder for works over 12 feet span, *a b*, with the angle-irons *e e*, constitutes the upper flange, and the lower flange consists of the plate *c d* and angle-irons *f f*, *g h* is the web. Even at the outset we see a necessary deviation in practice from the maxims of theory, for the latter would require the metal in the angle-irons to be spread out so as to be as far distant from the neutral axis as possible; but it is evident that the form given is absolutely necessary in order to allow of the junction of the web with the flanges. This apparent inconsistency of theory with practice forces itself upon our notice at every step, and renders it very desirable to determine how far the two may be combined. We may briefly state the connection between theory and practice, thus to obtain at the same time safe and economical results, practice must be based upon true theory, true theory being not the speculations of a mere mathematician, but definitions and explanations of the laws whereby the structures under consideration are ruled, the formulæ deduced therefrom being made to depend upon conditions determined by experiment; thus, for example, if we wish to know the strength of a solid beam to resist a transverse force, we should not, when we have determined the laws of resistance to flexure, fill in the constant quantities from data derived from experiments on direct resistance; but experiments should be made on transverse strain, from which the direct strength should be calculated, so that when this is again transformed we may have confidence in the result. Hence it appears, that we must derive our ultimate methods of designing structures from three different investigations:—First, pure theory; second, practical conditions and exigencies; third, theory modified to suit the conditions and exigencies of practice.

Fig. 58.

We will now return to our subject, observing that we have already considered the theory of the flanged-plate girder, and shown the form which must be adopted; we will, therefore, modify one of our theoretical axioms, viz., that the strength of the girder increases with its depth.

The problem assumes this form, how far may the depth of a girder be increased, the span being constant, to obtain the utmost practical economy? Let us suppose that a plate-girder, of the section shown

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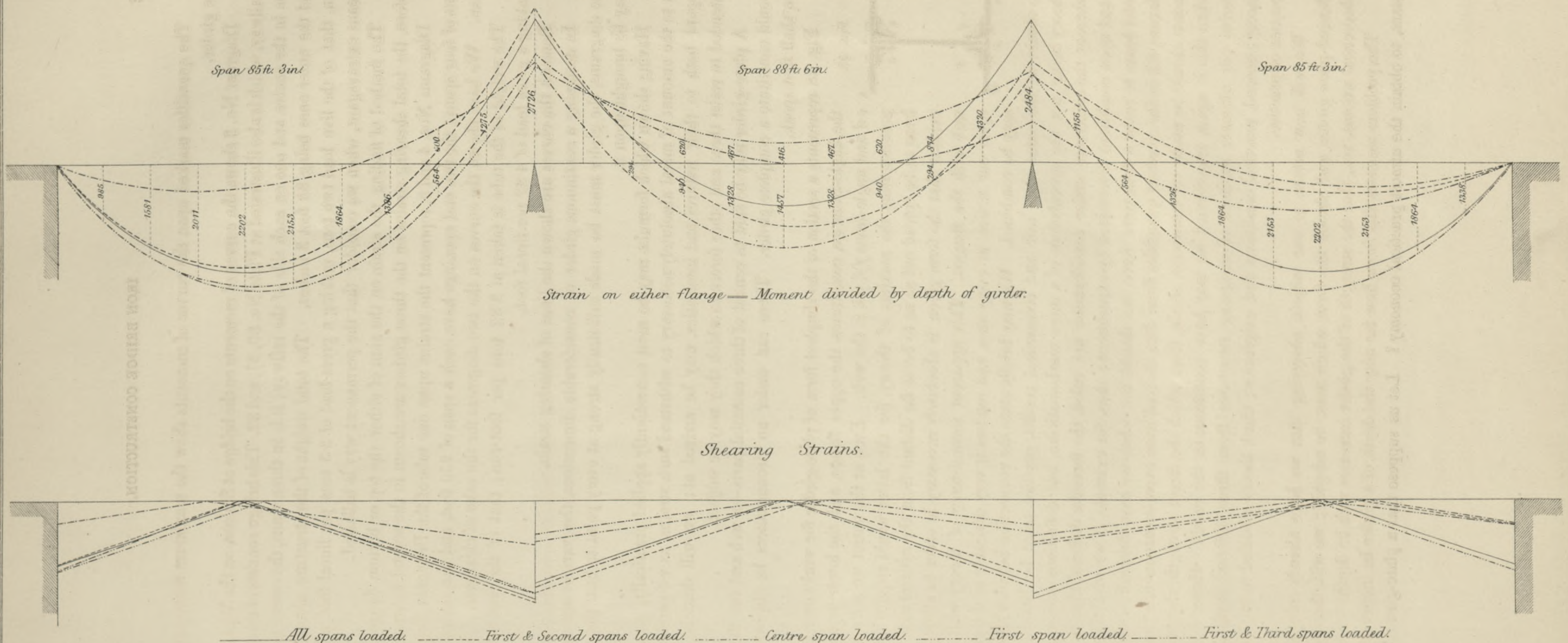
Diagram No 3.

STAINES BRIDGE.

CURVES OF MOMENTS.

Dead load 1·3 tons per foot. Live load 2 tons per foot.



W. Humber, dir.

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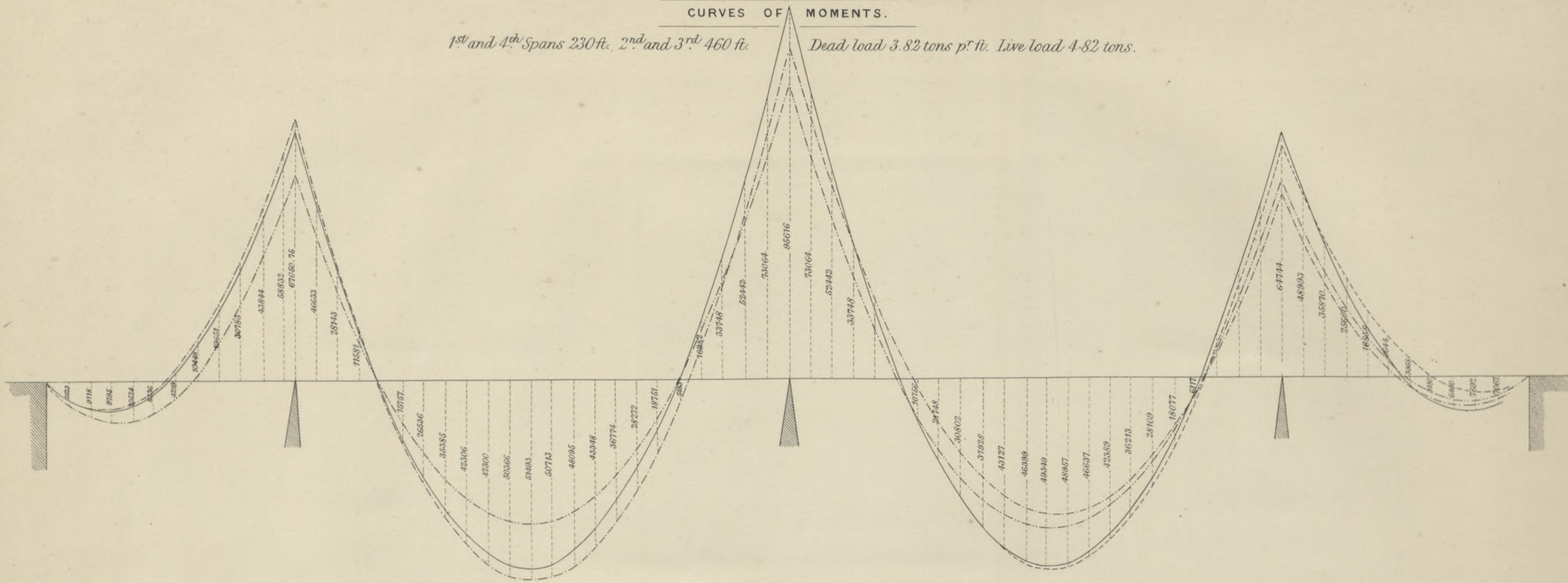
Diagram No. 4

BRITANNIA BRIDGE.

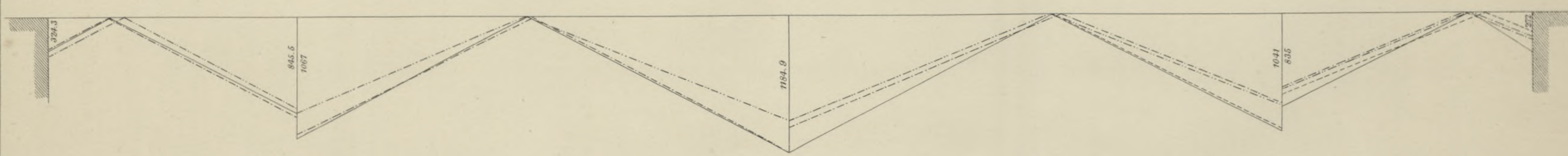
CURVES OF MOMENTS.

1st and 4th Spans 230 ft. 2nd and 3rd 460 ft. Dead load 3.82 tons pr ft. Live load 4.82 tons.



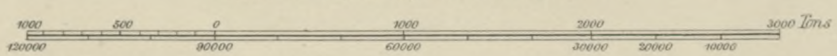
Strain on either flange equal to the moment divided by the depth of the girder.

Shearing Strains.



——— all spans loaded, ... 1st 2nd and 3rd spans loaded, —·—·— 1st and 2nd spans loaded, — — — 1st span loaded,

General Scale, 110 feet to an inch.
Scale for Shearing Strains, 1000 tons to an inch.
Scale of Moments, 30000 to an inch.



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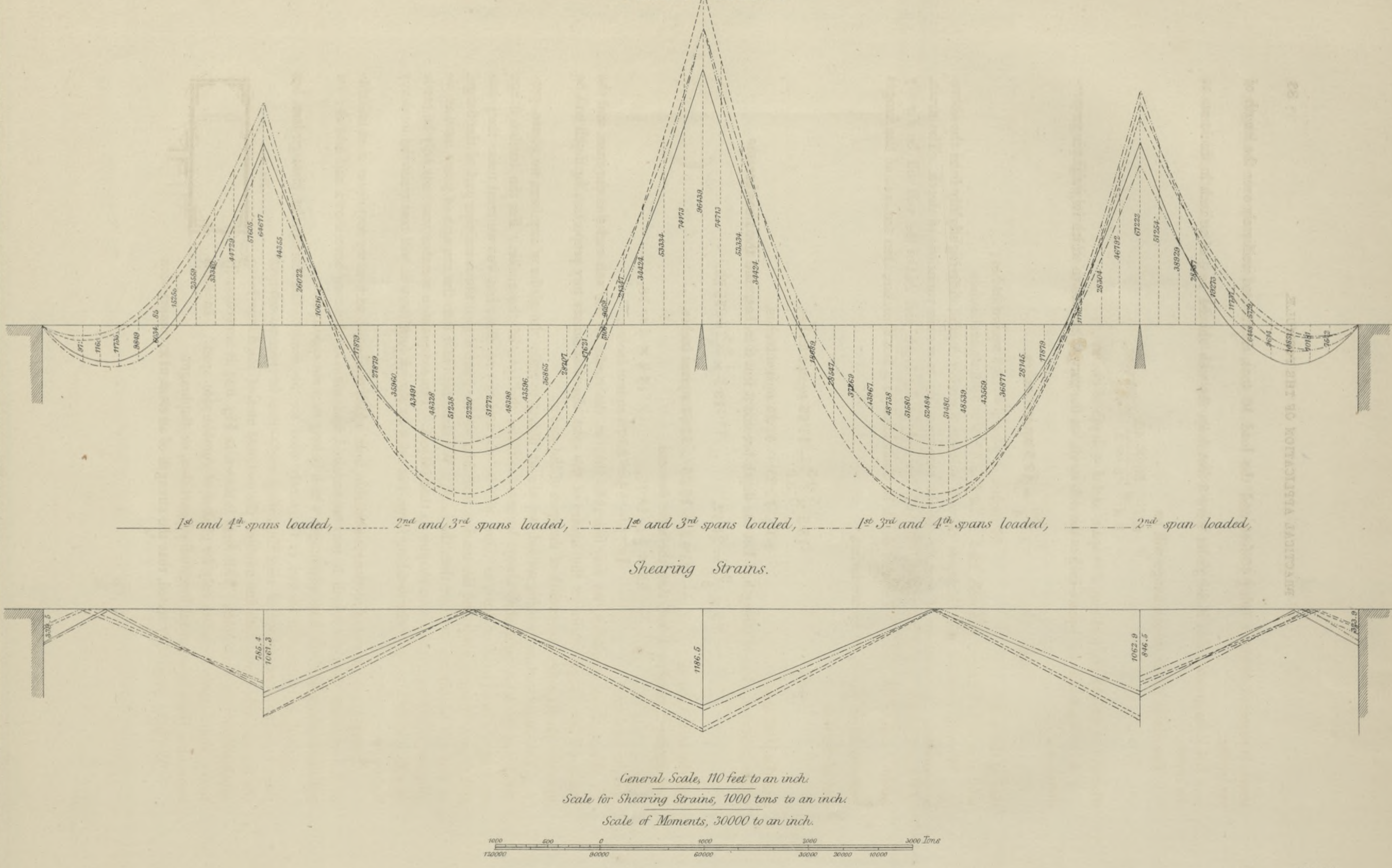
To face page Diagram Nº4. Diagram Nº 5.

BRITANNIA BRIDGE.

CURVES OF MOMENTS.



W. Humber, dir.

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above, is required, the span being 40 feet, and the load 30 tons, distributed uniformly over the length of one girder.

Let it be granted that if the girder were 3 feet deep, the web should be $\frac{3}{8}$ of an inch in thickness at the centre.

The direct strain on one flange will be

$$\frac{w\,l}{8\,d} = \frac{30 \times 40}{8 \times 3} = 50 \text{ tons,}$$

where w = weight per foot-run, l = span, and d = depth of girder.

Allowing 4 tons per square inch as the safe strain on the flange, its area to which its weight is proportional, will be

$$\frac{50}{4} = 12{\cdot}5 \text{ square inches.}$$

that of the web is $36 \text{ inches} \times \frac{3}{8} \text{ inch} = 13{\cdot}5$ square inches;

total area of two flanges and web, $12{\cdot}5 + 12{\cdot}5 + 13{\cdot}5 = 38{\cdot}5$ square inches.

We will consider this thickness of web suitable for other depths, differing slightly from the above, and ascertain the variations of area, or weight corresponding to certain variations of depth. The strain, and therefore the area of the flanges, will vary inversely as the depth, and that of the web will be directly as the depth of the girder. Let the depth be diminished to 2 feet 6 inches, then the area of the flanges becomes $25 \times \frac{3}{2{\cdot}5} = 30$ square inches,

and of the web

$$13{\cdot}5 \times \frac{2{\cdot}5}{3{\cdot}0} = 11{\cdot}25 \text{ square inches,}$$

and the total area $30 + 11{\cdot}25 = 41{\cdot}25$ square inches,

which indicates a loss of strength. Let the depth become 3 feet 6 inches, then the areas become

$$25 \times \frac{3}{3{\cdot}5} = 21{\cdot}42, \qquad 13{\cdot}5 \times \frac{3{\cdot}5}{3} = 16{\cdot}25,$$

and the total area $= 21{\cdot}42 + 16{\cdot}25 = 37{\cdot}67$ square inches.

Let the depth become 4 feet, then the areas become

$$25 \times \tfrac{3}{4} = 18{\cdot}75, \qquad 13{\cdot}5 \times \tfrac{4}{3} = 18,$$

and the total area $= 18{\cdot}75 + 18 = 36{\cdot}75$ square inches.

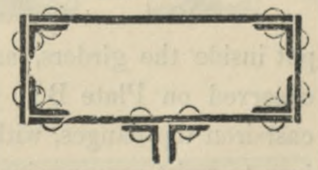
From these quantities, we may conclude that as the strain on the web varies as the span, and the strain on the flanges varies as the square of the span, the most satisfactory proportion for depth will be upon the data given—one-twelfth of the span of the girder.

This important point being settled, we are at liberty to make the girder of any form we please, provided that the least depth shall always be one-twelfth of the span of the girder. For the tension flange we may use flat plates, or bars, as there will be no tendency to distortion, but we must make the joints sufficiently strong to withstand the tension, and then we need not regard the remaining part of the flange, as that will be always stronger than the joint. We shall not enter into the strength of joints now, as we purpose to devote a subsequent chapter to that subject. The compression member must not only be made of sufficient area to resist the crushing force, but must be also of such form as will best resist the tendency to buckle.

A very great variety of sections is available from which to choose, such as we may deem most suitable to the peculiar case for which it may be required, and some of these we have shown at Plate A; we will, however, point out those advantages which belong to some special forms.

A hollow, rectangular section, is frequently used for the compression member of plate-girders, but these have the disadvantage of requiring many angle-irons in order to connect the top and side plates with one another, the sectional area of the flange being much reduced by rivet-holes. This form is shown at Fig. 59; it is necessarily heavy, and suitable therefore only for girders of considerable span. In some cases also, there is a central longitudinal partition, or diaphragm.

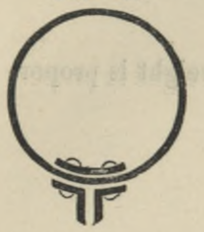
Fig. 59.



We have previously stated, that the strength of an element in compression is not reduced by rivet-holes, as the thrust is transmitted through the rivets, but this is upon the supposition, that the rivets bear on their whole surface; we must, therefore, if this is not the case, make some allowance for the defect, and it is evidently desirable, on many accounts, to avoid having more joints than are necessary.

For girders of small span, the compression member may conveniently be made of circular section, as shown at Fig. 60, in which case, no more joints would be required than are used when the flange consists of a single flat plate.

Fig. 60.



All the other sections are derived either from the rectangle, circle, or ellipse.

It is very important that the angle-irons should be of proper proportions, as it is upon these we impose the duty of uniting the various principal parts of the girder.

We may accept it as a general rule, that the thickness of the angle-iron should not be less than one-eighth of the length of one side, in order to insure satisfactory work.

There are many cases in which we shall find it necessary to deviate from any set of maxims which we may lay down, no matter how accurate or useful they may be generally, for it is absolutely impossible to adopt any course of reasoning, which will provide for all the emergencies which occur in practice; we, therefore, state everything with the understanding, that there is no peculiarity about the circumstance affecting the structure under consideration, which may render the application of such maxims useless or deleterious.

As examples of such circumstances, we may take excessive spans, peculiar manners of loading, &c.

In applying any of the foregoing considerations to girders, either continuous, or fixed at the extremities, it will be necessary to use, not the actual, but the effective span of the girder, which is the distance between the points of contrary flexure.

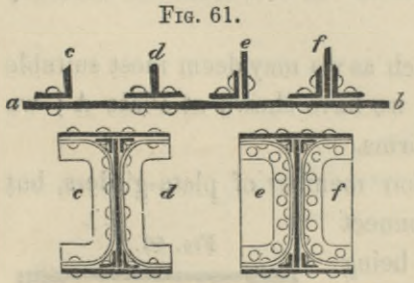
We will for the present conclude our remarks upon the flanges, and pass on to the web of the girder. In all girders there will be tendency of the flanges to approach each other; whether the load be applied on the top or bottom flange it matters not, and one of the duties of the web is to retain the flanges at their original distance apart.

We may regard the web, as a very wide, thin pillar, when we shall see that its strength should vary inversely, as the square of the depth of the girder, and directly, as some power of the thickness, which power is between the square and cube, but we shall, in order to be on the safe side, assume the square as the true ratio.

The thickness of the web, if it be intended to support this strain unassisted, must increase with the depth of the girder and with the load, but in that case, although the actual value of the buckling strain is not known, we are well aware that the web must be of much more considerable thickness than we have assumed it to be on a previous page; it will, however, be found more economical to leave the web itself to sustain only the shearing strain, and to serve as a means of connecting the various parts of the girder, which may be done by fixing struts or stiffening plates, as they are usually termed, at frequent intervals along the whole length of the girder; these struts should be placed at right angles to the flanges, and should be of such section as will best resist the action of compressive force.

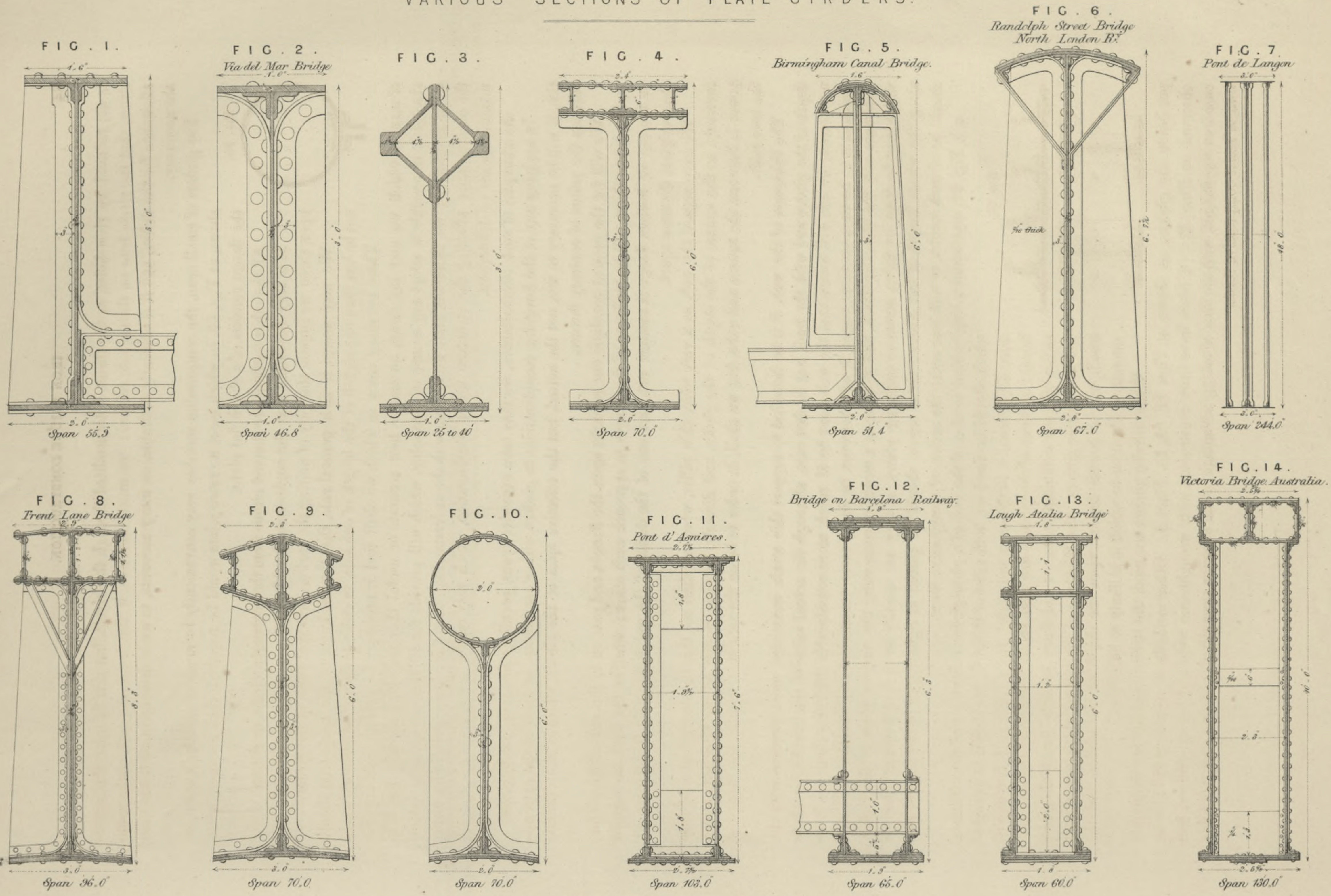
a b, Fig. 61, represents a longitudinal section of the web of a plate-girder. Sections of the stiffeners, which should be fixed on both sides of the web, are shown at *c* (angle-iron) *d* (T-iron) *e*, (angle-iron and plate), and *f* (plate and two angle-irons); sections of the girders are also shown, exhibiting the side elevation of these stiffening-plates. *c* and *d* are suitable for small, and *e* and *f* for larger girders; the thickness of the plates, which are always small, will of course depend upon the depth of the girder, and the amount of vibration to which it is likely to be subject.

Fig. 61.



When tubular girders are used, the stiffening-plates are frequently put inside the girders, as shown at Figs. 11, 13, 14, Plate A. Other methods of stiffening will also be observed on Plate B. Girders have been constructed with wrought-iron webs and bottom-flanges, and cast-iron top-flanges, with the idea of taking advantage of the superior strength of cast-iron over wrought-iron to resist compressive force.

VARIOUS SECTIONS OF PLATE GIRDERS.



N.B. These Girders are not all drawn to the same Scale.

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PLATE. B.

CROSS SECTIONS.

Showing various methods of attaching Platforms to Main Girders.

FIG. 4.

Pont du Ciron

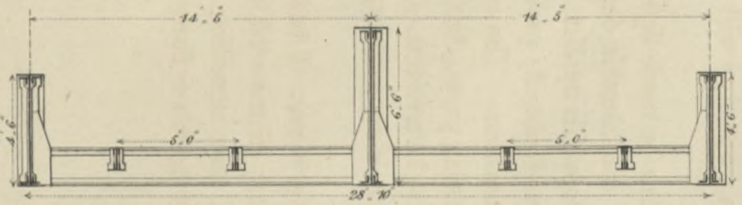


FIG. 5.

Sarsted Bridge.

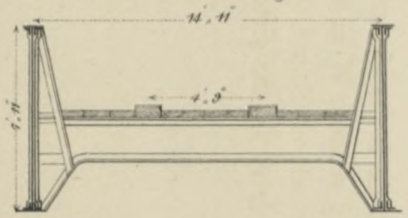
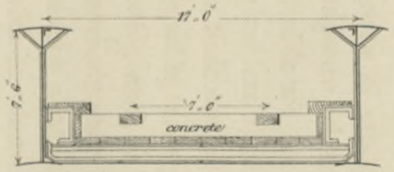


FIG. 6.

Chepstow Bridge.



Crumlin Viaduct.

FIG. 1.

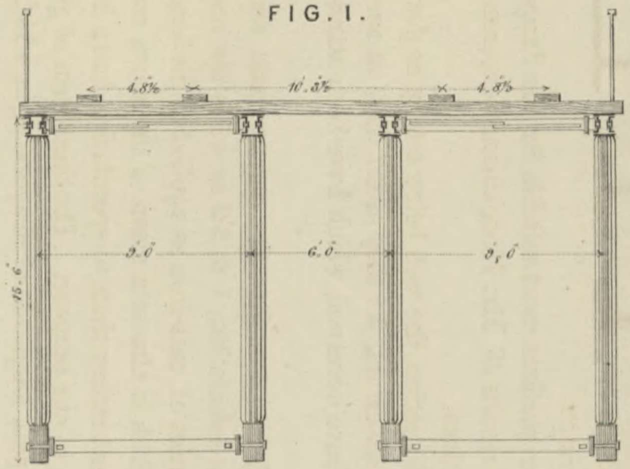


FIG. 9.

Via del Mar Bridge.

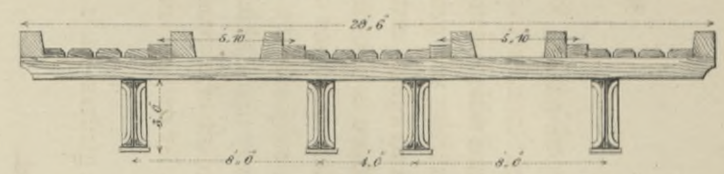


FIG. 10.

Pont de Langon.

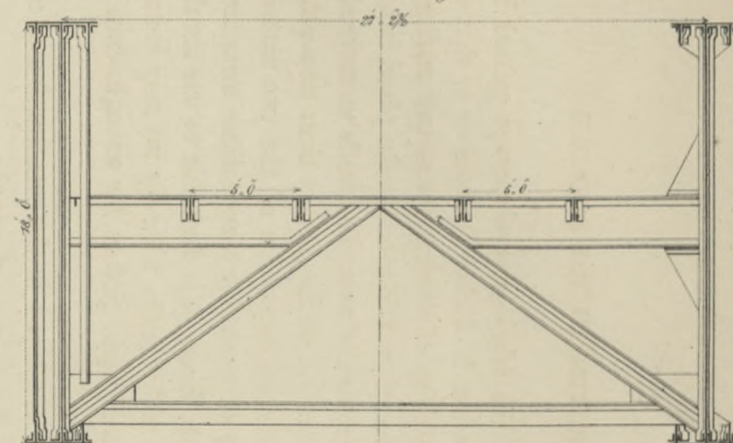
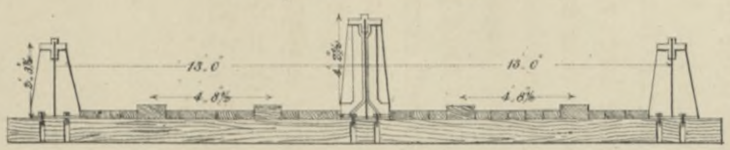


FIG. 7.

Bridge over the Stour.



Paddock Viaduct.

FIG. 8.

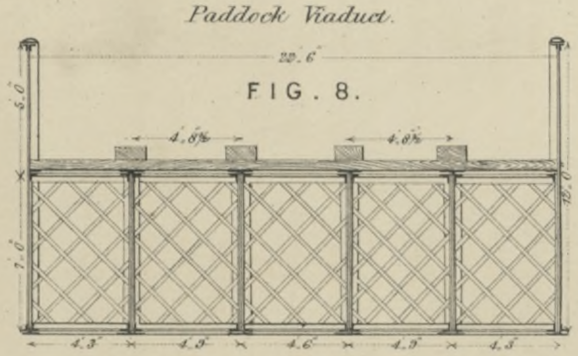


FIG. 2.

Mr. Fairbairn's proposed Cologne Bridge.

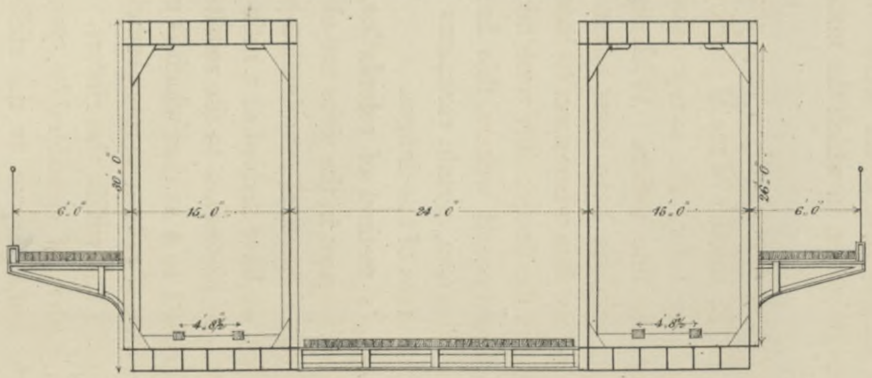


FIG. 3.

Pont d' Asnieres.

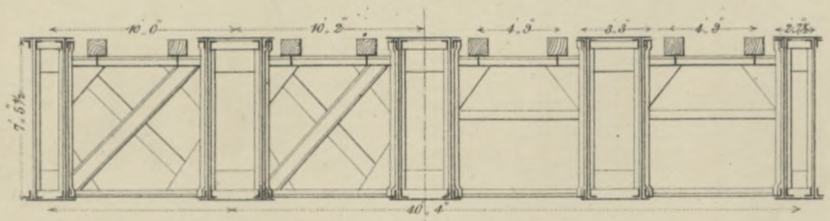
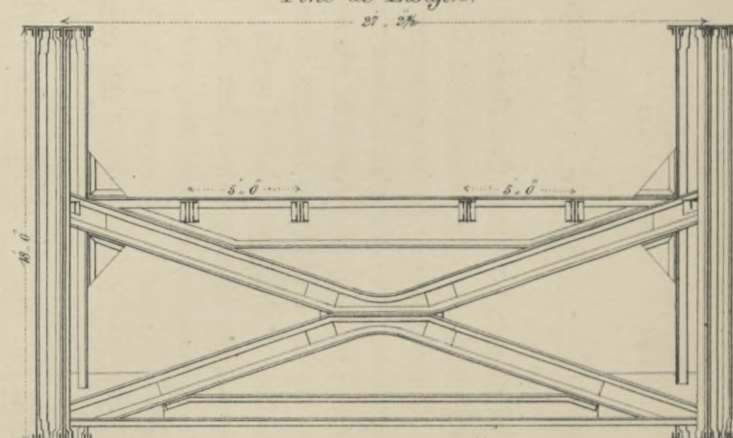


FIG. 11.

Pont de Langon.



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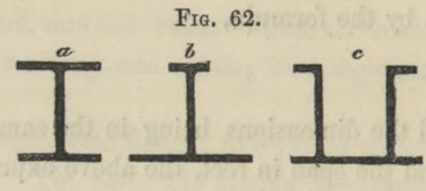
We think, however, that there is not on the whole any advantage gained by this combination, for although the strength of cast-iron in compression is decidedly superior to that of wrought-iron, we are exposed to the inconvenience attending the possibility of obtaining unsound castings, or metal which is not homogeneous; besides which, when the fracture of cast-iron does take place, it seldom exhibits any premonitory symptom of dislocation, whereas the rupture of wrought-iron is more gradual. And again, cast-iron is far more liable to rupture from the vibration of the girders than wrought-iron; there is also the probability of the production of strain by the unequal expansion of the two materials under variations of temperature. Wrought-iron has the property of assuming a crystalline structure when subjected to a rapid vibration, or to blows frequently repeated; hence it is desirable to use for the roadway upon which the load is carried some material which is tolerably elastic, whereby we may reduce the effect of concussions. Having now examined the construction of main girders for wrought-iron bridges, let us turn our attention to cast-iron girders.

Fig. 62 exhibits three sections, which are employed in the construction of cast-iron girders—*a* is a section in which the flanges have equal area, *b* a section of equal strength, and *c* is a trough girder, in which the material may be disposed either way.

Fig. 62.



In the first form there is evidently a very considerable loss of strength in proportion to the amount of metal employed, for the flange in tension will fracture long before the compression member brings its entire elastic resistance into action. With regard to the proportions most suitable for the flanges of cast-iron girders many opinions have been advanced, but the results of Mr. Hodgkinson's experiments furnish the most satisfactory data whereupon to base our calculations.

In these experiments the flanges only were taken into consideration, the web being totally neglected; and, according to the more recent writers, this has led to serious errors, for they assert that, on account of the webs varying in thickness, certain variations of strength have occurred, which have been attributed to different ratios of the areas of the flanges.

By Mr. Hodgkinson's method of calculation, he deduced 1 to 5 or 6 as the correct ratio for the flanges, but when the differences in the webs are elements in the calculation, 1 to 3·5 or 4·5 is the result.

By referring to our theoretical examination of the properties of cast-iron as applied to girders, our readers will observe that we have arrived at a ratio of 1 to 5, which is almost a mean of the above ratios; we must, however, give our preference to the results of experiment rather than to speculative data; hence we shall adopt the ratio of 1 to 4 as that which is most suitable to our purpose. The third form of girder is convenient to carry lines of railway across obstacles of small span, longitudinal sleepers, upon which the rails may be placed, being laid within the girders.

When cast-iron girders of considerable span are used, it may be desirable to construct them with stiffeners, to answer the same purpose as the stiffening-plates in wrought-iron girders; but they will not be so necessary, as the cast-iron web is far more rigid than the wrought-iron web.

Care must be taken that the girders be not too deep, for although they may thus be made lighter, they will be found very liable to fracture from blows or loads suddenly applied, if of considerable depth. Although cast-iron has been used for large ribs and girders, we shall not enter fully into this application, as the practical superiority of wrought over cast-iron for girders of large span is most undoubtedly proved. Cast and wrought-iron have been sometimes combined to form girders of large dimensions. The cast-iron being used for the compression elements, and the wrought-iron for the remaining parts; but the unequal expansion of the two metals must produce some strain which would otherwise be avoided. Before leaving the present subject, we will examine some formulæ in order to ascertain their applicability to the case of a cast-iron girder of uniform strength.

We will assume that the flange subject to tensile strain is the weakest, then let a = area of bottom flange at the centre of the span d = depth of girder, l = span, W = total uniformly-distributed load. We will take 1·5 tons per square inch as the working tensile resistance of cast-iron, then regarding half the length of the beam, and its depth as a bent lever, we have for the direct strain on the lower flange,

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

But the girder must be constructed to resist this strain, therefore,

$$S = 1{\cdot}5\,a.$$

From these two equations we may obtain by transposition as many as we may require; thus, for the area of the bottom flange, we have

$$1{\cdot}5\,a = \frac{W\,l}{8\,d}$$

$$\therefore a = \frac{W\,l}{12\,d}.$$

And for the area of the top flange, making it one-fourth of that of the bottom flange, we have

$$= \frac{W\,l}{48\,d}.$$

If the dimensions be given, and we are desirous of ascertaining the strength of the girder, we may do so by the formula,

$$W = \frac{12\,a\,d}{l},$$

all the dimensions being in the same name. If we take the areas in square inches, the depth in inches, and the span in feet, the above expression will become the area for the bottom flange,

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

and for the top flange,

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

the equation for the strength of the girder,

$$W = \frac{a\,d}{l}.$$

These expressions are exceedingly useful in practice, as their extremely simple forms render them very easy to be remembered.

Let it be required to construct cast-iron girders of spans 10 and 20 feet, to carry a railway over a narrow road, there being one girder to each rail; for small spans, such as are not longer, or very little longer, than a locomotive engine and tender, we must allow a load of 2 tons per lineal foot per line of rail for live load, and we will say for the structure per lineal foot 2 cwt.; then the total load on one line of rails will be, for the 10-foot girder 21 tons, and on each girder 10·5 tons, the depth being 10 inches, we shall find for the bottom flange,

$$A = \frac{W\,l}{d} = \frac{10{\cdot}5 \times 10}{10} = 10{\cdot}5 \text{ square inches},$$

and for the top flange, we have

$$\frac{10{\cdot}5}{4} = 2{\cdot}625 \text{ square inches.}$$

We may therefore make the bottom flange 7 inches wide, by $1\frac{1}{2}$ inches thick, and the top flange 2 inches wide, by $1\frac{1}{2}$ inches thick, and the web may be from $\frac{3}{8}$ to $\frac{1}{2}$ an inch in thickness, or if the girder is of a trough form, each web may be rather thinner.

For the 20 feet girders, we will allow a weight of 4 cwt. per lineal foot for the structure, the depth being 18 inches, the area of the bottom flange will be for one girder,

$$A = \frac{W\,l}{d} = \frac{24 \times 20}{18} = 26{\cdot}66 \text{ square inches},$$

and for the top flange, we have

$$\frac{26{\cdot}66}{4} = 6{\cdot}66 \text{ square inches},$$

the proportions of the section may now be determined as before.

If we require a beam of uniform strength at every section, we must diminish either the web, or the flanges, according to the ordinates of a parabola, representing the moment of strain at every section.

Having completed our observations on the main girders, we will pass on to the consideration of the platform, or roadway of straight girder bridges, commencing with the simplest form.

One of the most simple methods of constructing a bridge of small span, consists in using cast-iron main girders, with a roadway of flagstones, on which the ballast may be laid. The concussion produced by passing loads, may be much reduced, by using sawdust in the formation of the roadway. Another method which has been much used, consists in turning arches in brick between the girders, but on the whole it is not superior to the platform of flags.

For railway bridges when the rails are carried in, or on the girders, the intersticial spaces may be conveniently filled in with corrugated sheet-iron, and this is perhaps the most economical which can be adopted.

Cast-iron roadway plates, for foot and carriage bridges may also be used, and when employed, their thickness may be calculated in the following manner:—

The first point which demands our consideration, is the moving load, and the ballast which the roadway plates will have to support, we will take the maximum weight of roadway and moving load together, as 200 lbs. per square foot.

We will suppose the roadway plates to be 4 feet long, and 2 feet wide, with flanges to serve as means of attachment, which, however, will not be included in the calculation. The plate will assume the condition of a rectangular beam, four feet in span, and two in width, its thickness is to be determined; the total distributed load will be 1600 lbs. From the most reliable experiments, we find that the safe moment of resistance of a cast-iron bar, one inch square, when the length is in inches, and the load in lbs., is 1000 inch lbs. The moment of resistance of a rectangular section, is as the square of the depth, and as the breadth.

Let us replace the above roadway plate, by bars one inch wide, then we shall have on each a distributed load,

$$= \frac{1600}{24} = 66 \cdot 66 \text{ lbs.}$$

Which will give a maximum moment of strain,

$$= \frac{W\,l}{8} = \frac{66 \cdot 66 + 48}{8} = 399 \cdot 99, \text{ say, } 400 \text{ inch lbs.}$$

and to this the moment of resistance must be made equal. The width being one inch is the same as that of the section experimented upon, hence we need only regard the depth. By ordinary proportion we have the depth of the experimental section, being one inch,

$$1000 : 400 :: 1^2 : (\text{the required depth})^2$$

call the required depth $= d$, then

$$d = \sqrt{\frac{400}{1000}} = \sqrt{0 \cdot 4} = 0 \cdot 632$$

from which it is evident, that if we make the plates $\frac{3}{4}$ of an inch thick, they will be possessed of ample strength. Let us suppose that York Landings are to be used, the girders being two feet apart, the load on each piece 2 feet span, and one inch wide, will be

$$33 \cdot \dot{3}\dot{3} \text{ lbs.}$$

And the maximum moment of strain $=$

$$\frac{W\,l}{8} = \frac{33 \cdot \dot{3}\dot{3} \times 24}{8} = 100 \text{ inch lbs.}$$

The thickness will therefore be,

$$d = \sqrt{\frac{100}{50}} = \sqrt{2} = 1 \cdot 5 \text{ inches nearly,}$$

for the moment of resistance of one square inch of York Landings is 50 inch lbs.

The resistance of the same section for various other materials will be, for safe strain:—

Wrought-Iron ... ... 1200 inch lbs.

York Bluestone	... ...	50 inch lbs.
York Paving	... ...	24 „
Caithness Paving	... ...	140 „
Craigleith Sandstone	... ...	24 „
Kentish Rag	... ...	70 „
Cornish Granite	... ...	50 „
Portland Oolite	... ...	50 „
Bangor Slate	... ...	200 „
Langollen Slate	... ...	100 „

Planking may also be used, but it is not nearly so durable as the foregoing materials.

The moment of resistance of English Oak is 300 inch lbs. Let us suppose that the roadway of a bridge is to consist of these planks, supported at every 3 feet, the load on a piece 3 feet span and one inch wide, will be,

$$= 50 \text{ lbs.}$$

The maximum moment of which will be,

$$= \frac{W\,l}{8} = \frac{50 \times 36}{8} = 225 \text{ inch lbs.}$$

The thickness will, therefore, be,

$$d = \sqrt{\frac{225}{300}} = \sqrt{0{\cdot}75} = 0{\cdot}866.$$

One inch planking will, therefore, be sufficient. If pitch pine were used, the thickness would be,

$$d = \sqrt{\frac{225}{200}} = \sqrt{1{\cdot}125} = 1{\cdot}063 \text{ inches.}$$

We will assume that for roadways supported on girders, three feet apart, allowance being made for unsoundness, etc., that the planking should be two inches in thickness.

Let the girders be four feet apart, then the planking should be three inches thick, and if they be five feet apart, the planking should be four inches thick.

Generally, we may say, that the thickness of the planking should increase as the span of the planks; but if the girders upon which the planks rest be at considerable distances, the thickness of the timber need not increase in so great a ratio. We think, however, that it is not desirable to use timber platforms in such cases, on account of the great flexibility of that material. For the construction of platforms, a useful and ingenious invention, commonly known as the buckle-plate, is used with great advantage in the new Westminster and other bridges. The lightness of this method of construction is perhaps its greatest recommendation, as in large works it economises not only the material itself, but also that of the main girders by which it is supported. We will now describe these plates. They consist essentially of very thin wrought-iron plates, with flat edges, whereby they may be attached to the girders containing the central part, which is buckled, or dished, the whole being made in one piece. From this form it is evident to the most superficial observer, that a very great amount of strength and rigidity may be obtained, so that a platform constructed in this manner may be regarded also as a simple, but valuable method of horizontal bracing.

Let us now compare the relative weights of cast-iron roadway-plates and buckle-plates, supposing them to rest on girders 4 feet apart, to compose the platform of a bridge, of 200 feet span and 20 feet in width. The cast-iron plates must be $\frac{3}{4}$ths of an inch thick, but $\frac{3}{16}$ths of an inch will be sufficient for the buckle-plates. The weight of the cast-iron plates will be about 29 lbs. per square foot, while that of the buckle-plates is but 15 lbs. per square foot, giving total weight of,

116000 lbs. = 52 tons nearly for cast-iron,

60000 lbs. = 27 tons nearly for buckle-plates,

exhibiting a difference of 25 tons in the weight of the platform.

We will see what difference this will make in the weight of the main girders.

Suppose the bridge to be supported by two girders, each seventeen feet in depth, then the amount of area in both the top, or in both the bottom flanges, to support this difference of weight, will be, allowing 4 tons per sectional square inch as safe strain,

$$\frac{W\,l}{8\,D\,S} = \frac{25\cdot \times 200}{8 \times 4 \times 17} = 9\cdot 2 \text{ square inches nearly,}$$

which will be on both the flanges a total reduction of, say, 18 square inches.

If the total sectional area of the girders be reduced, as we approach the piers, this will also be reduced; we will, therefore, take 12 square inches, as the mean section, saved by the use of buckle-plates; then, for the present case, the total weight saved on the main girders will be,

12 square inches × 200 feet, × 3·33 lbs. = 8000 lbs.

Let us examine the relative cost of the two systems, taking this last quantity as 4 tons, to which we may consider it will amount when the cross girders, etc., are reduced.

In one case we have 52 tons of cast-iron which, taken at £8 per ton, would amount to £416 for the whole platform; in the other, we have 27 tons of buckle, which, at £15 10s. per ton, would be £418 10s., from which we must subtract the value of the metal saved in the main girder, which, at £17 10s. per ton, will be £70, leaving in this case a total cost of £348 10s., showing a balance of £67 10s. in favour of the buckle-plate system. If we assume the cost of such a bridge as the above, for light traffic, at £3000, and determine to use either cast-iron roadway-plates, or buckle-plates, we shall, by the adoption of the latter method, save $2\frac{1}{4}$ per cent. of the total cost.

It now remains for us to see how this difference varies with variations of span of the bridge.

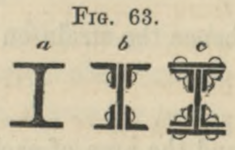
The weight of the platform will vary directly as the span, the width being constant, and the strain produced by that weight will also vary as the span. The weight saved on the main girders by using buckle-plates instead of cast-iron plates, will vary as the cube of the span, or as the weight of the bridge; and the weight saved in the platform will be nearly half the weight of the platform. We may, therefore, assume that generally in foot and roadway bridges, $2\frac{1}{4}$ per cent. of the total cost will be saved by using the buckle-plates, in preference to cast-iron plates, in the construction of the platform. We very much doubt whether anything is saved, in the first instance, by the adoption of timber, and we think it evident that there is a loss ultimately, as this material will require renewal every few years. For railway structures corrugated iron is evidently the lightest material to fill in the spaces between the girders, with and for the same reasons as above stated, is far superior to planking for that purpose, and has also been extensively used in the construction of platforms generally; and, from experiments expressly made, the most economical system to be adopted would be to make the corrugations 16 inches from centre to centre, and 4 inches deep, and to use plates 21 inches in width; these plates would overlap at the top of each corrugation, where they could be firmly rivetted.

Having now completed our remarks upon main girders, and the platforms used when they are placed at but short distances from each other, we will consider the formation of the platform when a greater amount of strength is required.

In these cases the platform is carried by a number of beams, extending from one girder to another; they are called cross girders, and may be made of wood, or of cast or wrought-iron, but we shall treat only of the last.

We must first examine the forms in which these girders may be constructed. Of these three are shown in Fig. 63. *a* is a rolled-iron girder, somewhat similar to a railway bar, but with wide flanges and a thin web; *b* shows a section of cross girder, which is composed of a plate-web, to which angle-irons are rivetted by way of flanges; *c* is a cross girder, in which the flange contains a plate as well as the angle-irons. Some other forms we may mention, such as T-irons, for very light work; box-girders, for very considerable spans; and lattice-girders, which latter, however, come more properly under the head of lattice-bridges.

FIG. 63.



The ordinary formulæ for straight girders may be applied to cross girders, and the method employed will be as follows:—We will first take the case of foot or carriage bridges, adopting, as before, 200 pounds per square foot as the maximum load. The load on each cross girder will be the width of the bridge multiplied by the distance between the cross girders and by the load per square foot. Let us suppose that the bridge is 20 feet wide, and the cross girders 10 feet apart, then the total load distributed on every cross girder will be,

$$200 \times 20 \times 10 = 40{,}000 \text{ lbs.} = 27{\cdot}85 \text{ tons.}$$

Let us make the cross girders 18 inches in depth, then the central strain on either flange will be, calling the load 28 tons,

$$\frac{W\,l}{8\,d} = \frac{28 \times 20}{8 \times 15} = 46{\cdot}66 \text{ tons.}$$

The area of each flange will therefore be,

$$\frac{46{\cdot}66}{4} = 11{\cdot}66 \text{ square inches,}$$

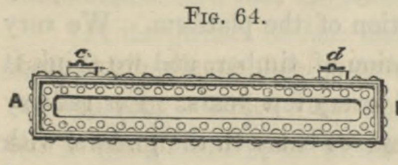
say, 12 square inches.

If the girders were placed five feet apart, we should require but 6 square inches in each flange, which might be supplied by angle-irons, 3 inches by 3 inches, by $\frac{5}{8}$ of an inch thick, hence we see that of the two, the latter arrangement will be the more preferable.

We may, in a similar manner, calculate the dimensions requisite for the cross girders of railway bridges, but the different application of the load must be taken into consideration. The cross girders may be made with the lower flange curved in proportion to the strain at various parts, whereby the amount of metal consumed in the webs will be reduced, causing, in large structures, a somewhat important saving.

If cross girders be used for a railway bridge, one of two methods may be adopted, the first consists in placing the girders about 3 feet apart, and placing the longitudinal sleepers directly upon them, without any other support, and the second requires the application of short longitudinal girders, placed under the rail, running from one cross girder to the next, and the relative economy of these two systems we are now about to examine.

Fig. 64.



Let a B, Fig. 64, represent a cross girder intended to carry a single line of railway, c d showing the positions of the lines of rails, then at c d will the load be applied, we will, however, for convenience'-sake, consider the load to be applied at the centre in any case.

The greatest strain that can come upon any cross girder will be that produced by a locomotive engine passing over it, and this we will take at 2 tons per foot-run. Let the girders be 12 feet long, 1 foot deep, and 3 feet apart, then the load on any girder will be 6 tons, if the timber platform be tolerably rigid, if not, the cross girder will be subject to a much greater strain, when the driving-wheel of a locomotive passes over it; let us, therefore, take the load at 12 tons, then will the strain on either flange be,

$$\frac{W\,l}{4\,d} = \frac{12 \times 12}{4 \times 1} = 36 \text{ tons.}$$

and the area of each flange will be,

$$\frac{36}{4} = 9 \text{ square inches.}$$

Now let the girders be 10 feet apart, with longitudinal girders between them.

The load on the centre of each cross girder will be, taking 2 tons per foot-run per load,

$$= 2 \times 10 = 20 \text{ tons,}$$

hence the strain on either flange,

$$= \frac{W\,l}{4\,d} = \frac{20 \times 12}{4 \times 1} = 60 \text{ tons.}$$

and the area of each flange be,

$$\frac{60}{4} = 15 \text{ square inches.}$$

There will be one longitudinal girder under each rail, therefore the maximum distributed load on each will be 10 tons; or we may take for the central load, produced by the passage of a locomotive, 7·5 tons on each girder; let the girders be 10 inches in depth, then the strain on one flange will be,

$$\frac{W\,l}{4\,d} = \frac{7{\cdot}5 \times 10}{4 \times {\cdot}833} = 22{\cdot}5 \text{ tons.}$$

The area of each flange being,

$$\frac{22\cdot5}{4} = 5\cdot625 \text{ square inches.}$$

It now remains for us to determine the relative weights of the two systems. Let us suppose that we wish to construct a platform upon one of the above systems for a bridge of 90 feet span by the adoption of the first, we shall require 30 cross girders, the weight of the whole being, taking the webs of the girders as $\frac{1}{4}$ of an inch thick,

21 square inches $\times$ 12 feet $\times$ 3·33 lbs. $\times$ 30 $=$ 25,000 lbs. $=$ 11·26 tons nearly.

By adopting the second method, we shall use 9 cross girders, the weight of which will be

33 square inches $\times$ 12 feet $\times$ 3·33 lbs. $\times$ 9 $=$ 11,880 lbs. $=$ 5·34 tons nearly.

We shall also require 18 longitudinal girders, whose weight will be

15 square inches $\times$ 10 feet $\times$ 3·33 lbs. $\times$ 18 $=$ 9,000 lbs. $=$ 4 tons nearly.

The total weight of metal required will be in the present case, say, 10 tons, whereas, in the former, the weight of material used amounted to $11\frac{1}{4}$ tons. The result of this calculation is evidently in favour of the system of distant cross girders, with longitudinal bearers between them; but notwithstanding this, we by no means decide in favour of this method, and to exhibit more clearly our reasons for coming to such a conclusion, we will carefully explain every point which is worthy of consideration in the construction of platforms.

In reading the observations we are about to make, it must be carefully remembered, that it is to railway structure, we principally refer, on account of the great amount of vibration caused by rapidly-moving loads, it becomes a point of great importance to secure the utmost amount of rigidity of platform and of bridge structures generally, and this may be obtained by constructing the platform as solidly as possible.

We must also consider that our girders are calculated to bear a continuous load, equally distributed. We should therefore endeavour to transmit the load on to the girders at as many points as may be convenient, in order that we may avoid action of very concentrated loads; and we may thus illustrate the means to be taken on this account, if we place cross girders five feet apart, to sustain a platform in one case, and ten feet apart in another place, to support an equally loaded platform, then, in the latter case, the pressures on the main girders will be twice as concentrated as in the former.

Sufficient attention must also be paid to the means by which the cross girders are to be attached to the main girders, so that there may not be more rivet-holes in the latter than are absolutely necessary. The undulation of the platform must also be counteracted as far as possible, and, to effect this in the highest degree, we should evidently adopt a great number of cross girders, which, on account of the proximity of the load to the points of support, will be far more rigid than the longitudinal bearers or girders.

From these considerations, we come to the conclusion that, upon the whole, the most satisfactory system which we can adopt is that in which the platform consists of a considerable number of cross girders placed moderately near to each other.

We may here observe, that in some cases which have occurred where we have made the platform as narrow as possible this system is actually the lightest, and therefore the most economical.

We cannot leave this subject without calling the reader's attention to the example which it presents of the necessity of modifying our theoretical arguments and deductions by means of practical observations, for if we relied solely upon our calculations in this matter, we should, in the present case, determine to use the open platform, whereas, a more just consideration of the office which the structure is destined to fulfil, clearly shows that we should adopt the other method.

A very important subject is that which we are now about to investigate,—viz., the method of attaching the cross to the main girders.

We may first indicate the conditions which we are desirous of satisfying in designing this portion of the structure. The first and most important condition is, that the load should be caused to act upon the girder in such a manner as to produce only a vertical force, all tendency to twist or throw the girder into winding, as it is called, being carefully obviated.

We must also so dispose our rivets that the holes may occur in those parts where the weakening effect will be a minimum.

There can be no doubt, that the most economical method of carrying a railway is by placing the main girders under the rails; but this cannot always be done, wherefore we must examine the most convenient method to substitute for it.

It immediately occurs to us, that the cross girders should, in a plate-girder bridge, be fixed either on the top of the upper flange, or to the bottom of the lower flange. In the former case, the bolts by which the cross girders are attached to the main girders will only require sufficient strength to ensure the stability of the platform; but, in the latter case, the bolts will have to resist a tensile strain equal to the total load upon the cross girder:—thus, if the load on any cross girder be twenty tons, the total area of all the bolts of rivets by which one girder is attached to the main girders must in no case be less than four square inches, and it will in most cases be found desirable to allow twice this area.

In calculating the strength of any bolt from its sectional area, we of course assume that the bolt itself will be drawn in two before the thread by which it is attached to the nut is stripped, the proportions to be given to the different parts of bolts and nuts, in order to insure this result, will be fully discussed in the Chapter upon Joints.

The cross girders of bridges have frequently been attached to the upper surface of the lower flange of the main girder, its length being terminated by the web of the main girders, but, by doing so, we produce a twisting strain, and if we continue the cross girder, as has sometimes been done through the web of the main girder, we thereby deteriorate the strength of the latter.

Another mode by which the cross girders may be connected with the main girders, consists in attaching them to the webs of the latter by means of angle-pieces or brackets, either attached to the webs of both girders, or to the web of the main girder and flanges of the cross girders, or to both; and one form of this method is exhibited by bolting the end-plates of the cross girders, which are continuations of the flanges, to the webs of the main girders. In many instances this method is exceedingly convenient. In Plate B, we have shown a variety of methods of effecting the junction between platforms and main girders of bridges; but the delineations there exhibited require no further explanation at our hands, as the foregoing remarks comprise the principles of all the systems.

We have now concluded our observations upon platforms, as far as cross girders, longitudinal bearers, and the covering or roadway plates are concerned; but we have yet to investigate the form and duties of the horizontal bracing within or below the platform. But we may here refer our reader to Plate C for various forms of parapets.

Tie-bracing will always be more economical than strut-bracing, and it may conveniently be formed of slight wrought-iron, extending in a diagonal direction from one extremity of one cross girder to the opposite extremity of the next cross girder,—whether they be attached to the top or bottom flange it matters not,—and they may be fixed by bolts passing through eyes in the extremities of the tie-rods, but much more conveniently by passing the ends, which should be screwed, through cylindrical apertures in shoes bolted to the cross girders, and drawing them up tight by means of nuts, we may thus adjust the amount of tension on each tie with a considerable degree of accuracy.

These methods of bracing, together with all that are similar, we shall include under the term "horizontal-plane," or "horizontal-superficial bracing," in contradistinction to other kinds now to be described.

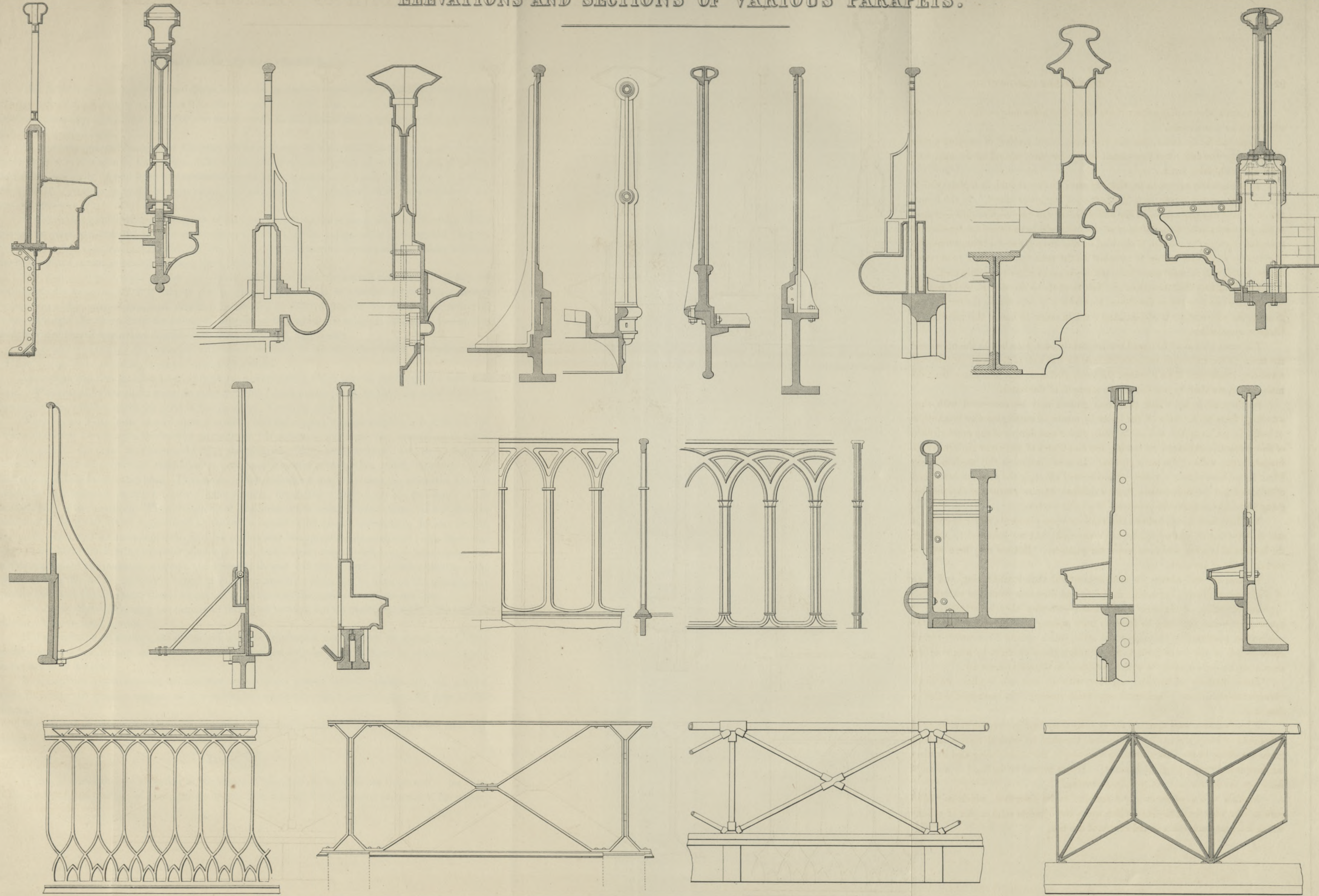
When we have two lines of railway carried by four girders distinct from each other, it will be found convenient to brace the two central girders together by vertical frames of bars, angle or T-irons, each frame consisting of four sides, forming a rectangle, the angles being tied by diagonals, this constitutes what we shall term "vertical-plane bracing." If the frames be inclined at an angle to the horizon, we shall designate the system as "oblique plane bracing."

Another system of bracing yet remains to be described, which is "cubic or box bracing." This consists of a number of tie-bars, which unite the opposite solid angles of a box or parallelopipedon, all of which intersect each other in the centre of the box.

To face Page 172.

ELEVATIONS AND SECTIONS OF VARIOUS PARAPETS.

PLATE . C.



N.B. These Parapets are not all drawn to the same scale.

W. Humber, dir.

London: E. & F. N. Spon, 16, Bucklersbury.

This latter method is applicable to bridges where the roadway is carried upon the upper flanges, the interior part of the structure being unused for the purposes of traffic, then one tie-bar will pass from the left-hand extremity of one cross girder of the lower series to the right-hand extremity of the next cross girder in the upper series.

In a previous part of the present Chapter, we have shown in what ratio the weight of the bracing should vary with the span of the bridge. The constant to determine the actual weight for any particular case must necessarily depend upon circumstances for which no general rule can be laid down, and should, whenever it is possible so to do, be determined from observation of structures which have withstood the test of actual use.

We have now completed our description of the form of plate girder bridges, with the exception of the bed-plates, piers, and foundations, the consideration of which will be postponed to a subsequent Chapter. We will therefore conclude our observations with a few remarks upon the means which are taken to ensure the action of the strains in accordance with the calculations.

The strains upon straight girders are always calculated upon the assumption that the girders are mathematically straight; but it is evident, that to obtain this condition, in practice under varying loads is utterly impossible; some means must therefore be adopted, in order to prevent the girder from deflecting below a straight line drawn from the points of support, so that when the bridge is subject to a maximum load, its conditions of strain may coincide with those calculated, very nearly.

We must first examine the obstacles with which we have to contend, after which, we shall be in a position to supply the necessary means of overcoming them.

That which will first manifest itself, is defective workmanship, which may be exhibited in a variety of ways. We must allow for slight errors in measurements for inaccurate joints, and also for want of perfect bearing on pins which connect the parts of the structure, if such be used.

When a girder has been erected upon a scaffold, and the support thus accorded to it is removed, the strains beginning to act upon the various elements of the structure, will cause the connections to adjust themselves until the parts bear accurately upon the pins, etc., which connect them; and, in so doing, a a certain amount of permanent subsidence is produced, a species of deflection which, however, cannot be properly included under that head. The amount of this subsidence may, of course, be estimated, but it is impossible to calculate it, as it is a matter of pure practice, depending entirely upon the excellence of the workmanship of the structure. When the effects of inaccurate work cease to become more evident, the true deflection of the girder, which is produced by the strain caused on its various elements by its own weight, becomes evident, and the amount of deflection due to this may be approximately calculated; this effect may be produced before the bearing upon the joints is perfect, for the strain may be sufficient to cause elongation and contraction of the material, although the intensity thereof will not compel a perfect condition of the joints. From this we are induced to expect a further permament alteration of form on the addition of a greater load, and this alteration should be carried to its limit by the application of the test load.

We do not intend discussing the manner of testing at present, but we merely enumerate the exigencies for which we must provide.

If the results be satisfactory, the girder will, after the application, and at every subsequent application of the maximum load, occupy the position of a line very slightly curved upwards from the points of support to the centre of the span.

To obtain this desirable result, we must not construct our girders actually straight, but with a curve upwards, giving the girder a camber, as it is called, the amount of which depends upon the span of the structure.

The amount of deflection upon a single girder, supported at each end, may be calculated from the formula.

$$D = \frac{5\,w\,l^4}{384 \cdot EI}$$

Where D represents the deflection at the centre of the span, w, the load per lineal unit, l, the span of the girder, E, the modulus of the elasticity of the material, and I, the moment of inertia of the girder.

In the application of the formulæ to practical cases, we have, by certain assumptions, avoided the complicated equations of the theoretical part, hence, it becomes necessary by similar means, to simplify the above expression as regards the moment of inertia, and, by so doing, we cannot materially injure the practical accuracy of the calculation, for it is certainly desirable that the deflection of a girder should be computed upon the same assumption as its strength. In a previous Chapter, it has been shown that, if

M = the moment of resistance of a section,

I = the moment of inertia,

h = the distance of extreme fibre from neutral axis, $= \frac{d}{2}$ for uniform sections,

d = the depth of the girder,

s = safe strain per sectional unit, then will

$$M = I\frac{s}{h} = 2\,I\frac{s}{d}$$

but by our assumptions in the present Chapter, we find for a deep girder with moderately thin flanges,

$$M = a.\,d.\,s,$$

Where a = the sectional area of one flange, therefore, in the present case,

$$2\,I\frac{s}{d} = a.\,d.\,s$$

from which we obtain the approximate formulæ,

$$I = \frac{a\,d^2}{2}.$$

Let us now apply this formula to the case of a girder bridge of 200 feet span, and 20 feet wide, carrying a carriage-way, the weight of the structure being taken at 130 tons, while that of the live load at 200 tons, the area of one pair of flanges will be, allowing 4 tons per square inch for safe strain, at the centre of the span, the depth being 20 feet, about 100 square inches. Taking this, therefore, for the total area of the top flanges of the girders, we have for the moment of inertia,

$$I = \frac{100 \times 57{,}600}{2} = 2{,}880{,}000.$$

The depth being necessarily taken in the same name as the area, viz., inches.

The modulus of elasticity for wrought-iron per square inch is about 24,900,000 lbs., the weight per unit lineal, or per inch, will be 308 lbs., the span in inches is 2400, the deflection in inches will therefore be,

$$D = \frac{5\,w\,l^4}{384\,E\,I} = \frac{5 \times 308 \times 33{,}177{,}600{,}000{,}000}{384 \times 24{,}900{,}000 \times 2{,}880{,}000} = 1{\cdot}86 \text{ inches.}$$

This formula has the great disadvantage of being tedious, let us therefore see if we cannot simplify it.

We find the deflection varies as the total load, as the cube of the span, and inversely, as the modulus of elasticity, the area of the flanges, and the square of the depth.

All dimensions being in inches, we have,

$$I = \frac{a\,d^2}{2},$$

but if a be in inches, and d in feet, we have,

$$I = \frac{a.144\,d^2}{2} = 72\,a\,d^2.$$

If d be in inches, and l in feet, W being total weight in tons, and E being in tons,

$$D = \frac{5\ W\,1728\,l^3}{384\ E72\,a\,d^2}$$

We may take E at 11·116 tons per square inch, then the expression for the central deflection in inches becomes

$$D = \frac{5.W\,1728\,l^3}{384 \times 11{\cdot}116 \times 72\,a\,d^2} = 0{\cdot}000028\frac{W\,l^3}{a\,d^2}.$$

Let us apply this expression to the same case as before, where a = 100 square inches, d = 20 feet, W = 330 tons, and l = 200 feet, we then find,

$$D = 0{\cdot}000028 \frac{W\, l^3}{a\, d^2} = 0{\cdot}000028 \frac{330 \times 8{,}000{,}000}{100 \times 400} = 1{\cdot}848 \text{ inches.}$$

a result not very different from that obtained above.

This formula, the form of which may readily be remembered, will apply to wrought-iron girders generally. The deflection of continuous girders may be calculated, first, by considering the parts between the points of support and contrary flexure, regarding them as semi-beams, subject to a distributed and to a concentrated load, as explained in a former Chapter; find the extreme deflection of these parts, which will give the positions vertically of the points of contrary flexure, the extreme deflection of the part of the girder between the points of contrary flexure may then be found by considering it as a simple girder supported at its extremities, and the deflections thus found is to be added to half the sum of the deflections at the points of contrary flexure, in order to obtain the total deflection of that bay of the girder.

The deflection of the central part will be found by the expression given above, but that of the end-parts by an equation which we will now give. The deflection of a beam, fixed at one end and loaded at the other, may be found from the expression, $D = \dfrac{W\, l^3}{3\, E\, I}$ inches,

all dimensions being in inches; but if we take our dimensions as in the last example, we shall find for wrought-iron semi-beams, the deflection in inches from the equation,

$$D = \frac{W\ 1728\ l^3}{3 \times 11{\cdot}116 \times 72\, a\, d^2} = 0{\cdot}000\,72 \,.\, \frac{W\, l^3}{a\, d^2} \text{ nearly.}$$

For an uniform load on the same beam, we have, at the maximum,

$$D = \frac{W\, l^3}{8\, E\, I} \text{ inches,}$$

which becomes, by taking out dimensions as above,

$$D = \frac{W\ 1728 \cdot l^3}{8 \times 11{\cdot}116 \cdot a\, d^2} = 0{\cdot}00027\ \frac{W\, l^3}{a\, d^2} \text{ nearly.}$$

The expression for the extreme deflection of the girder is, when there is one point of contrary flexure distant z from the point of support,

$$D = 0{\cdot}000028\ \frac{W}{a\, d^2} \left\{ l - z \right\}^3 + \tfrac{1}{2} 0{\cdot}00027 \frac{W\, z^3}{a'\, d'^2}.$$

When there are two points of contrary flexure, the other one being V distant from the nearest point of support, the deflection in inches will be,

$$D = 0{\cdot}000028\ \frac{W}{a\, d^2} \left\{ l - (z + V) \right\}^3 + \tfrac{1}{2} \left\{ 0{\cdot}00027 \left(\frac{W'\, z^3}{a'\, d'^2} + \frac{W''\, V^3}{a''\, d''^2} \right) \right\}$$

in which expressions, if $w =$ weight per lineal foot, $W = w\ \{l - z\}$, and $= w\ \{l - (z + V)\}$.

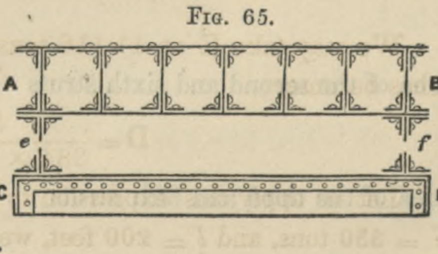
$W' = w\, z$, and $W'' = w\ V\ a =$ area of the flange, at the point of greatest strain, d being depth of girder at the same place, a', a'', and $d'\ d''$ being the areas and depths at the points of support.

We now pass on to notice another kind of structure, which, although in reality a plate-girder, has usually been known by another name—we speak of tubular bridges.

These structures may be described as consisting of two plate-girders, in which the top and bottom flanges are made so broad that they meet at the centre of the bridge, forming a rectangular tube, upon, or within, which the load is carried; in either case one of the flanges serves as a platform, or roadway, and must, therefore, be properly stiffened by cross girders, or some equally effective substitute. These girders can only be used economically for bridges of great span, such as require very considerable areas of flange, for their widths cannot be reduced below a certain limit, viz., that required for the roadway.

When the area of one flange is required to be exceedingly great, it may be made of the section shown at A B, Fig. 65, constituting a cellular platform; but when a small quantity of material will be used, it may be of the section C D, merely consisting of flat plates, stiffened by angle-iron, T-iron, or angle-iron and plate combined.

Fig. 65.



The webs are shown at e and f, and these, being very deep, must be well stiffened by struts; or standards, stout gusset-plates must also be used to retain the proper rectangular section of the tube.

The platform, as before stated, is formed by one of the flanges.

The remarks upon plate-girders also apply to tubular bridges, and the latter may be calculated in precisely the same manner as the former.

Continuous girders require no further remark, as the constructions already described will answer for single or continuous girders.

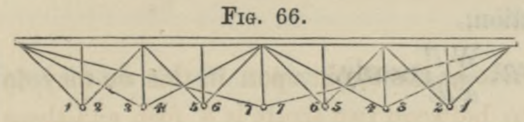
The next forms of bridge to which we shall direct the reader's attention, are those known as trussed, or lattice girder bridges, but these are so numerous, that to describe and exemplify the method to be adopted for every kind, would far exceed the space which is at our disposal; we must, therefore, restrict our observations to those forms which are most worthy of our attention.

In the Chapter on the Theory of Lattice Girders, we have given formulæ for simple trussed girders, such as are used for the support of travelling cranes, etc., which are sufficiently simple, and therefore require no further comment.

The first girder, the construction of which we shall proceed to consider, is that patented by Mr. S. Pittar, and is by principle a trussed girder.

This form of girder is shown at Fig. 66, the horizontal member and the upright standards are of cast-iron, and the ties being of bar-iron. We may consider a distributed load on this girder as consisting of a number of concentrated loads, one placed over each standard, then each of these loads will be equal to the weight per lineal unit multiplied by the distance between two standards.

Fig. 66.



Let us suppose that a girder of 64 feet span is to be constructed on this principle, the total load on the girder amounting to 2 tons per lineal foot. Let the girder be divided into 8 bays, by 7 standards or uprights, each of which is 6 feet long, the ties being arranged as shown in the above figure.

We must first determine the total load on each standard, after which it will be an easy matter to calculate the strain upon the various ties.

By examining the form of the structure, we at once perceive the distribution of the loads. The distance between two standards will be 8 feet, therefore the concentrated load over each standard will be 16 tons, and this will be borne by the two piers in the inverse ratio of their distances from it.

Also that portion of any load borne by a certain standard, must pass through certain ties and struts, those affected by such load depending upon the method of trussing.

The load on the first strut will be

$$= 16 \text{ tons.}$$

that on the second,

$$= \frac{16}{8} + 16 + \frac{5 \times 16}{8} = 2 \times 16 + 10 = 28 \text{ tons,}$$

on the third strut we have

$$= 16 \text{ tons,}$$

and on the fourth or centre strut,

$$\frac{2 \times 28}{8} + \frac{3 + 16}{8} + 16 + \frac{3 \times 16}{8} + \frac{2 \times 28}{8} =$$

$$7 + 6 + 16 + 6 + 7 = 42 \text{ tons.}$$

If we allow 5 tons per square inch as the safe strain upon the cast-iron standards, the areas of the standards will be as follows:—

Sectional area of first and seventh struts

$$= \frac{16}{5} = 3{\cdot}2 \text{ square inches;}$$

area of the second and sixth struts

$$= \frac{28}{5} = 5{\cdot}6 \text{ square inches;}$$

area of the third and fifth struts

$$= \frac{16}{5} = 3{\cdot}2 \text{ square inches;}$$

area of fourth or central strut

$$= \frac{42}{5} = 8{\cdot}4 \text{ square inches.}$$

We may now determine the areas of the ties.

The strain on any ties will be found by dividing the load which it carries, by the sine of the angle which it makes with the horizon, and the sine is equal to the length of the standard divided by the length of the tie.

We will first consider the ties proceeding from the first strut, marked 1 and 2, which will be in the same condition as those proceeding from the foot of the seventh strut, which are also marked 1 and 2.

By the law of the lever we find that if W is the load on the first strut, the portion on tie 1, is

$$\frac{7}{8} \text{W} = \frac{7}{8} \times 16 = 14 \text{ tons,}$$

the load on the tie 2, will be

$$\frac{\text{W}}{8} = \frac{16}{8} = 2 \text{ tons.}$$

We must now find the length of the tie-bar, in order to arrive at the strain upon it, this we may do from knowing that the length of the strut is 6 feet, the distance between two struts is 8 feet, and these lengths, together with the tie, form a right angled triangle, therefore the length of the tie

$$= \sqrt{8^2 + 6^2} = \sqrt{100} = 10 \text{ feet;}$$

hence the strain on these ties will be, on tie 1,

$$14. \frac{10}{6} = 23{\cdot}33 \text{ tons.}$$

And on tie 2, the strain is

$$2. \frac{10}{6} = 3{\cdot}33 \text{ tons.}$$

The areas of ties will be, for tie 1, allowing 5 tons per square inch as safe tensile strain,

$$\frac{23{\cdot}33}{5} = 4{\cdot}66 \text{ square inches,}$$

And on tie 2,

$$\frac{3.33}{5} = 0.66 \text{ square inches,}$$

but this should be made larger in practice.

If the ties be made of round rods, the first might consist of 3 rods, each 1·5 in diameter, and the second of one rod of the same dimensions; it may, however, be more convenient to use flat bars, in which case, tie 1, may consist of three, and being 2 inches deep, by $\frac{7}{8}$ of an inch thick, and tie 2 of two of the same depth, and $\frac{7}{16}$ of an inch thick.

We now pass on to the ties proceeding from the foot of the second or sixth strut, marked 3 and 4, the load on the tie 3 will be

$$= \tfrac{3}{4} \cdot 28 = 21 \text{ tons,}$$

that on tie 4, being

$$= \tfrac{1}{4}\, 28 = 7 \text{ tons.}$$

The length of each of these ties, will evidently be

$$\sqrt{= 6^2 + 16^2} = \sqrt{292} = 17 \text{ feet nearly.}$$

The strains will, therefore, be on tie 3,

$$21. \frac{17}{6} = 59{\cdot}5 \text{ tons.}$$

And that on tie 4, will be

$$7. \frac{17}{6} = 19{\cdot}83 \text{ tons.}$$

The areas of these ties must therefore be, for tie 3,

$$\frac{59\cdot5}{5} = 11\cdot9 \text{ square inches.}$$

The first of these bars may consist of four bars, 4 inches deep, and $\frac{3}{4}$ of an inch thick, and the second of three bars, 4 inches deep, and $\frac{5}{8}$ of an inch thick.

On the ties 5, 6, we have the loads thus on tie 5,

$$\frac{5}{8} \cdot 16 = 10 \text{ tons,}$$

and on tie 6,

$$\frac{3}{8} \cdot 16 = 6 \text{ tons.}$$

The length of ties is the same as that of the ties 1 and 2, hence the strain on tie 5, is

$$10. \frac{10}{6} = 16\cdot66 \text{ tons.}$$

And on tie 6,

$$6\cdot \frac{10}{6} = 10 \text{ tons.}$$

The areas will therefore be for tie 5,

$$\frac{16\cdot66}{5} = 3\cdot33 \text{ square inches,}$$

and for tie 6,

$$\frac{10}{5} = 2 \text{ square inches.}$$

These ties may be thus formed—tie 5 of these bars, $1\frac{3}{4}$ inches deep, by $\frac{5}{8}$ of an inch thick, and tie 6 of two bars of the same depth and thickness.

We now come to the last or central ties—No. 7, the load on tie 7,

$$= \tfrac{1}{2} \cdot 42 = 21 \text{ tons.}$$

The length of the tie 7, is

$$\sqrt{6^2 + 32^2} = \sqrt{1060} = 33 \text{ feet nearly.}$$

Hence the strain on tie 7,

$$= 21 \cdot \frac{33}{6} \; 115\cdot5 \text{ tons,}$$

and its sectional area must be,

$$\frac{115\cdot5}{5} = 23\cdot1 \text{ square inches.}$$

This tie may consist of 5 bars 6 inches deep, and $\frac{7}{8}$ of an inch thick.

We must now ascertain the dimensions requisite to the compression, or horizontal flange. At the point of support the compression member is subject to three strains, which are brought upon it by the strain passing along the ties 1, 3, and 7.

The strain thus produced by either tie will be equal to the strain on the tie, multiplied by the cosine of the angle which the tie makes with the horizon, which cosine is equal to the distance horizontally between the ends of the tie, divided by the length of the tie. The strains will, therefore, be as follows:— Strain by tie 1,

$$= 23\cdot33 \cdot \frac{8}{10} = 18\cdot66 \text{ tons;}$$

by tie 3,

$$= 59\cdot5 \cdot \frac{16}{17} = 56 \text{ tons;}$$

by tie 7,

$$= 115\cdot5 \cdot \frac{32}{33} = 112 \text{ tons;}$$

hence the total strain will be,

$$18\cdot66 + 56 + 112 = 186\cdot66 \text{ tons;}$$

the sectional area must, therefore be,

$$\frac{186 \cdot 66}{5} = 37 \cdot 33 \text{ square inches.}$$

The horizontal member may, therefore, consist of a girder-shaped piece, one foot deep, with flanges 8 inches wide and $1\frac{1}{2}$ inches thick.

The strain on the horizontal member will diminish towards the centre in this case, but we do not consider it desirable to make any reduction in its strength.

The platform, or road, to be carried by such girders as these may be supported by cross girders bolted to the horizontal members.

The vertical struts of cast-iron should be made with ribs or feathers, so as to present a + section, and the feathers should be wider towards the top of the standard, in order better to resist any bending strain that may come upon it. The ties should be of bar-iron, made deep in order to render them of sufficient rigidity; and those that are so long as to assume a curved form must be supported about the middle of their lengths by links attached to the cast-iron struts. The heads of the links must, of course, be of a greater breadth than the other part, in order to make up for the loss of material caused by the hole through which the connecting pin is to be passed. The platform of a bridge, supported by girders such as these, will require a greater amount of horizontal-plane bracing, because the girders themselves are not nearly so rigid, laterally, as the ordinary plate-girders; and they should also have vertical, plane, or otherwise, cubical bracing to maintain the girders in their proper relative positions.

Girders of this class are evidently only suited to bridges of small span, for if they were applied to large spans the vibration of the long tension-rods would be a great inconvenience.

Let us now suppose that a bridge is required to carry a single line of railway over an opening of 60 feet span, to be supported by two Warren girders.

In these girders the bars and flanges together form a series of equilateral triangles, the angle of any bar with the horizon being, therefore, 60°.

Let the girders consist of ten triangles, then the length of each lattice-bar will be 6 feet.

The depth of the girder will be,

$$\sin 60^\circ \times = \cdot 886 \times 6 = 5 \cdot 316 \text{ feet,}$$

which is the vertical distance between the joint pins.

Let the total load be 1·5 tons per foot-run; we shall neglect the difference in the end apices, and call the load on every joint 9 tons.

We will first take the maximum strains on the various lattice-bars.

The amount of any load passing through any lattice-bar will be determined by the law of the lever.

The load per joint, reduced to the unit of leverage, will be, as there are ten triangles,

$$\frac{9}{\text{span}} = \frac{9}{20} = 0 \cdot 45.$$

To obtain the strain on any lattice-bar, from the load which it supports, we must multiply the latter by,

$$\frac{1}{\sin 60^\circ} = \frac{1}{\cdot 886} = 1 \cdot 1286.$$

and,

$$0 \cdot 45 \times 1286 = 508 \text{ nearly.}$$

When the bridge is unloaded, we have 3 tons on each apex, and the above becomes,

$$0 \cdot 15 \times 1 \cdot 1286 = \cdot 169.$$

These quantities halved, for those appertaining to one girder become,

0·254, and 0·084.

We shall indicate a tensile strain by — (*minus*), and a compressive strain by + (*plus*), the strain will be as follows: —

1st bar $0 \cdot 254 \{ + 19 + 17 + 15 + 13 + 11 + \ldots\ldots\ldots + 1 \} = + 25 \cdot 4$ tons,

2nd bar $0 \cdot 254 \{ - 17 - 15 - 13 - \ldots - 1 \} + 0 \cdot 084 \{ + 1 \} = - 20 \cdot 49$ tons,

3rd bar $0 \cdot 254 \{ + 17 + 15 + 13 + \ldots + 1 \} + 0 \cdot 084 \{ - 1 \} = + 20 \cdot 49$ tons,

4th bar $0{\cdot}254\,\{-15-13 \ldots\ldots\ldots\ldots\ldots -1\}+0{\cdot}084\,\{+1+3\}=-15{\cdot}92$ tons,
5th bar $0{\cdot}254\,\{+15+13+11 \ldots\ldots\ldots +1\}+0{\cdot}084\,\{-1-3\}=+15{\cdot}92$ tons,
6th bar $0{\cdot}254\,\{-13-11- \ldots\ldots\ldots -1\}+0{\cdot}084\,\{+1+3+5\}=-11{\cdot}69$ tons,
7th bar $0{\cdot}254\,\{+13+11 \ldots\ldots\ldots\ldots\ldots +1\}+0{\cdot}084\,\{-1-3-5\}=+11{\cdot}69$ tons.

The 8th, 9th, and 10th bars should be regarded as struts, and made in every respect similar to the 7th. In this series we have considered that part of the load that would tend to counteract the greater strain on any bar, as a minimum, so that we might obtain the maximum strains.

The centre bars sometimes act as struts, and should therefore be made as such.

We will now calculate the areas of the diagonals, allowing 4 tons per sectional inch as the resistance of wrought-iron to compression, and 5 tons per inch as its resistance to tension, the nett area will then be,

$$\text{First diagonal area} = \frac{25{\cdot}4}{4} = 6{\cdot}35 \text{ square inches.}$$

$$\text{Second diagonal area} = \frac{20{\cdot}49}{5} = 4{\cdot}09 \text{ square inches.}$$

$$\text{Third diagonal area} = \frac{10{\cdot}49}{4} = 5{\cdot}12 \text{ square inches.}$$

$$\text{Fourth diagonal area} = \frac{15{\cdot}92}{5} = 3{\cdot}12 \text{ square inches.}$$

$$\text{Fifth diagonal area} = \frac{15{\cdot}92}{4} = 3{\cdot}98 \text{ square inches.}$$

$$\text{Sixth diagonal area} = \frac{11{\cdot}69}{5} = 2{\cdot}33 \text{ square inches.}$$

$$\text{Seventh diagonal area} = \frac{11{\cdot}69}{4} = 2{\cdot}92 \text{ square inches.}$$

These are the minimum theoretical areas. The struts should be made of angle-, T-, or channel-iron, with or without plate, and the ties may be of flat bars.

The first bar might be made of two T-irons, of the dimensions 4 inches by $2\frac{1}{2}$ inches by $\frac{1}{2}$-inch thick, rivetted back to back with a plate $4\frac{1}{2}$ by $\frac{3}{8}$ of an inch thick interposed. The second bar might consist of two flat bars, $4\frac{1}{2}$ inches wide and $\frac{5}{8}$ of an inch thick,—and so forth for all the others.

It is necessary to allow sectional areas somewhat greater than those determined theoretically, on account of slight deviations of angle, rivet-holes, etc.; and it is not worth while to reduce the sectional area of a small lattice bar at the centre.

We will next determine the strains upon the horizontal members. These will have their maximum values when the girder is loaded along its entire length.

First, let us determine the strains upon the top or compression flange, because the triangles affecting this flange are all equilateral; the strain produced upon this flange by any tie is equal to the strain on that tie.

We shall call that part of the flange between the apices of the first and second triangles the first section; that between the second and third apices the second section, and so on. All the strains on the compression-member will be produced by diagonals acting as ties. Thus, at the first joint, the second diagonal will produce a certain strain on the top flange, which will pass on to the second, where it will receive an increment produced by the strain on the fourth diagonal; and so on, increasing at every joint. The strains on each section will therefore be for either half of the flange, full load,—

On first section, $= +20{\cdot}31$ tons,
On second section, $20{\cdot}31 + 15{\cdot}23 = +35{\cdot}54$ tons,
On third section, $35{\cdot}54 + 10{\cdot}15 = +45{\cdot}69$ tons,
On fourth section, $45{\cdot}69 + 5{\cdot}08 = +50{\cdot}77$ tons.

The minimum areas of the various sections must therefore be as follows:—

$$\text{First section area} = \frac{20{\cdot}31}{4} = 5{\cdot}08 \text{ square inches.}$$

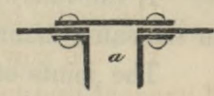
$$\text{Second section area,} = \frac{35{\cdot}54}{4} = 8{\cdot}88 \text{ square inches.}$$

$$\text{Third section area,} = \frac{45{\cdot}69}{4} = 11{\cdot}42 \text{ square inches.}$$

$$\text{Fourth section area} = \frac{50{\cdot}77}{4} = 12{\cdot}69 \text{ square inches.}$$

The fifth section may be made of a plate 9 inches wide, and $\frac{1}{2}$ an inch thick, two narrow plates each 5 inches wide, and $\frac{1}{2}$ an inch thick, and two angle-irons each of the dimensions 5 inches by 3 inches by $\frac{1}{2}$ an inch thick, the whole being arranged as shown in Fig. 67, the diagonals are fastened between the angle-irons at *a*.

Fig. 67.



This section is to be continued to the third section of the flange, where it may receive some diminution by reducing the narrow plates to four inches wide, by $\frac{3}{8}$ of an inch thick.

We next proceed to determine the strains on the tension flange.

All the strains upon the lower flange are produced by the strains upon the various struts, and they are equal to those upon the strut, except in the case of the end bars, by which a tension is produced equal to half the compressive force on those diagonals, on account of the strain being resolved between the pier and tension member, instead of between the tension member and a tie.

The strain on each section of the tension flange, will be

On the first section		$= - 12{\cdot}69$ tons.
„ second „	$- 12{\cdot}69 - 20{\cdot}31$	$= - 33{\cdot}00$ „
„ third „	$- 33{\cdot}00 - 15{\cdot}23$	$= - 48{\cdot}23$ „
„ fourth „	$- 48{\cdot}23 - 10{\cdot}15$	$= - 58{\cdot}38$ „
„ fifth „	$- 58{\cdot}38 - 5{\cdot}08$	$= - 63{\cdot}46$ „

These strains will require minimum areas, as follows:—

$$\text{First section area} = \frac{12{\cdot}69}{5} = 2{\cdot}54 \text{ square inches.}$$

$$\text{Second „} = \frac{33{\cdot}00}{5} = 6{\cdot}60 \text{ „}$$

$$\text{Third „} = \frac{48{\cdot}23}{5} = 9{\cdot}64 \text{ „}$$

$$\text{Fourth „} = \frac{58{\cdot}38}{5} = 11{\cdot}67 \text{ „}$$

$$\text{Fifth „} = \frac{63{\cdot}46}{5} = 12{\cdot}69 \text{ „}$$

The tension flange may consist of a number of bars, like the chain of a suspension-bridge, and the above areas represent theoretical minimum areas, in providing these, due allowance must be made for pin-holes.

The fifth section may consist of 6 bars, each 6 inches wide, by $\frac{9}{16}$ of an inch thick, the fourth section being the same.

At the third section the bars may be 6 in number, 6 inches wide, and $\frac{3}{8}$ of an inch thick, to be continued to the first section, where they are replaced by two bars, each of which are $4\frac{1}{2}$ inches deep, and $\frac{7}{16}$ of an inch thick.

The pins must be properly proportioned, if pins be used to sustain the various shearing strains to which the joints will be exposed; but we shall not consider that subject in this place, as a full account of it will be found in the Chapter upon Joints. It now remains for us to determine the construction of the platform, and the manner of connecting it with the main girders.

The platform should be carried upon cross girders placed 8 feet apart, so as to occur at the apex of every triangle, thereby avoiding the production of any bending strain upon the flanges, in addition to the direct strain already provided for.

There should be some kind of light longitudinal bearers between the cross girders, to support the permanent way.

The cross girders and platform generally may be calculated in the same manner as for a plate-girder bridge, but it should have stronger horizontal plane bracing, to make up for the want of rigidity of the main girders.

Light T-iron is a very suitable material for the bracing of the platforms of triangular girders, but round bars will sometimes be sufficient.

When T-iron bracing is used, the T-irons should be laid back to back, so that they may be rivetted together at the places where they cross each other.

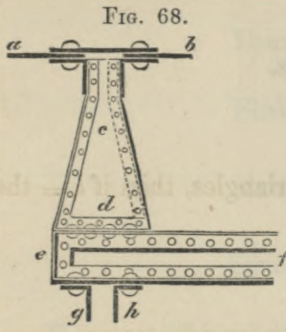
If the roadway be on the top of the girders, the cross girders may be attached to the upper flange in the same manner as to that of a plate girder.

The joints of the bottom flanges should then be connected transversely, by means of strong cylindrical distance pieces and light cubical bracing, or vertical plane bracing may be used with advantage.

In this case the cross girders and longitudinal bearers may be altogether dispensed with, by placing the main girders immediately beneath the lines of rails, the intervals being filled up by planking or corrugated iron, but with this arrangement, distance pieces must also be placed between the top flanges of the main girders.

The arrangement which we prefer, is that in which the roadway is placed nearly at the bottom of the main girders, and therefore between them, being suspended from the top flange by means of strong standards of a triangular form, as shown at Fig 68, which is a cross section of one girder, *a b* is the top flange, *c d* the standard composed of a triangular plate stiffened by angle-irons, as shown, and attached at its upper end to the top flange; in this case it must be midway between the apices of two triangles, the bottom of the standard is rivetted to the cross girder, the end of which is shown at *e f*, to the bottom of the cross girder are rivetted two brackets *g h*, between which pass the bars of the tension member, the pin connecting these also passing through the brackets.

Fig. 68.



These standards act as stiffening-plates, and as gusset-plates in a plate girder, thereby adding greatly to the rigidity of the main girders.

The effective minimum sectional area, viz., that at the top of the standard where it is attached to the top flange of the main girder, must be of sufficient extent to withstand the tensile effort of the load included between the two standards.

In the case which we have selected, this load will be 4·5 tons for one girder, the sectional area corresponding to which, would be about one square inch, but this is far less than could be practically applied, and the standard should be of the following dimensions. First, the plate half-an-inch thick, 3 feet 4 inches from the cross girder to the centre of the top flange, the width within the flange will be about $2\frac{1}{2}$ to 3 inches, which increases regularly to the bottom of the standard, where it may be from 12 to 15 inches. There should be two angle-irons, one passing down the outer edge and along the bottom of the standard on the one side, and another passing down the inner edge, and along the bottom of the standard on the other side; these angle-irons should be about 2 inches by $2\frac{1}{2}$ inches, by $\frac{5}{16}$ of an inch thick.

The dimensions of the cross girder may be calculated as before.

The brackets *g* and *h* should be about 10 inches deep, $6\frac{1}{2}$ inches wide, and 1 inch thick, and bolted or rivetted firmly to the bottom of the cross girder.

By adopting this arrangemeut of roadway, we dispense with the distance pieces and with all bracing, except the horizontal-plane bracing, which should be tolerably strong. When the girder is supported upon the piers by plates attached to the top flanges, there may be some alteration of form, which will, by the substitution of a tie for a strut, to terminate the girder, produce a variation in the intensity of the strain; but the method to be adopted in such a case, is sufficiently obvious to render any account of it in this work superfluous.

Before we proceed to more complicated examples, we will exhibit the method of treating a continuous girder. Let it be required to construct a continuous girder of two spans, each span being 80 feet. We will select the case in which both spans are fully loaded.

The point of contrary flexure will be 20 feet from the centre pier, leaving beyond it, a girder whose effective span is 60 feet. This girder may be treated as an ordinary lattice girder of 60 feet span, supported simply at each end. The other part of the girder, viz., that included between the points of contrary flexure and support, by the centre pier, may be treated as a lattice semi-beam fixed at one end, and loaded uniformly at the other end, the concentrated load at the extremity being equal to half the load on the other part of the girder. The girder may also as far as regards the lattice web, be treated as follows:—Bisect the central part, that which is in the same condition as a girder supported at both extremities, which is that part included between the points of contrary flexure, if there be two; but if there be but one, it is included between that point, and the point of support, upon which the girder freely rests; then treat the web between the point of bisection and the point of support as the web of a semi-beam similarly loaded.

The strains on the flanges may be calculated at the points of junction with the lattice bars, by same formulæ as were given for continuous plate girders.

Before proceeding further with the construction of triangular girders, it is necessary to determine what angle of lattice bar with the horizon is most satisfactory as regards economy of material.

The weight of the lattice web depends upon the length of the bars, their number, and sectional area. The sectional area varies, omitting the intensity of the load, as

$$\frac{1}{\sin a}$$

where a represents the angle included between the bar and the horizon.

The length of the bar also varies, as $\frac{1}{\sin a}$

Let n = number of bars, d = distance between the apices of the two successive triangles, then if l = the span of the girder,

$$n = \frac{2\,l}{d},$$

but $d = \frac{2 \text{ depth of girder}}{\tan a}$;

therefore the weight of the web varies, as

$$\frac{1}{\sin a} \times \frac{1}{\sin a} \times n$$

$$\propto \frac{1}{\sin a} \times \frac{1}{\sin a} \times \tan a$$

$$\propto \frac{1}{\sin a \,.\, \cos a}$$

hence the last expression should be a minimum, or what is the same thing,

$$\sin a \,.\, \cos a$$

should be a maximum, this quantity however varies as

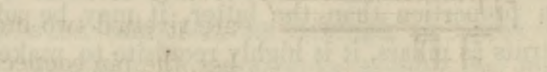
$$\sin 2.\, a.$$

which should therefore be a maximum. The largest sine which exists, is that which is equal to the radius, and this the sine of 90°, hence to secure the greatest economy of material, we must give a the following value, $\sin 2a$ = maximum, when $2a = 90°$

$$\therefore a = 45°$$

Let us suppose that a lattice girder is required of 60 feet span, having two series of triangulations, as shown in Fig. 69, the bars being set at an angle of 45° to the horizon. Let the load on one girder $\frac{3}{4}$ ton per foot-run. Then because there are two series of triangles, the load per foot-run on each series is $\frac{3}{8}$ of a ton.

FIG. 69.



Let the depth of the girder be 6 feet, then will there be 5 triangles in each series. The first step consists in finding $\sin a$, which in the present case,

$$\frac{1}{\sin a} = \frac{1}{\sin 45°} = \frac{1}{\cdot 7071}$$

The concentrated load on each apex will be,

$$0{\cdot}375 \times 12 = 4{\cdot}5 \text{ tons,}$$

the triangles are 5 in number, hence our coefficient of direct strain, is

$$\frac{4{\cdot}5}{10} \times \frac{1}{{\cdot}7071} = 0{\cdot}636.$$

The strains on the diagonals of the series, commencing with a strut, will be, the weight of structure being one-third total weight,

First bar, $0{\cdot}636 \{+9+7+5+3+1\} = +15{\cdot}90$ tons,
Second bar, $0{\cdot}636 \{-7-5-3-1\} + 0{\cdot}212 \{+1\} = -9{\cdot}96$ tons,
Third bar, $0{\cdot}636 \{+7+5+3+1\} + 0{\cdot}212 \{-1\} = +9{\cdot}96$ tons,
Fourth bar, $0{\cdot}636 \{-5-3-1\} + 0{\cdot}212 \{+1+3\} = -2{\cdot}87$ tons,
Fifth bar, $0{\cdot}636 \{+5+3+1\} + 0{\cdot}212 \{-1-3\} = +2{\cdot}87$ tons,

and the same for the other half of the girder; the areas, and also the strains on, and areas of, the top and bottom flanges may be determined as in the last case.

We have worked out this portion of the calculations for the present case only for the purpose of showing how we deal with a girder having more than one series of triangles, for one example is frequently worth many rules.

The Jumna bridge has been constructed on a principle wherein all the struts are upright, hence producing on those elements minimum strain, with minimum length. Let us see how far this may be desirable.

In the first place, the principle necessitates the use of longer ties, which on that account, from the diminution of *sin a*, suffer greater strains.

The struts will require greater sectional areas than the ties for equal strains; hence, being heavier in proportion than the latter, it may be advantageous to shorten them. And also, if we regard the struts as pillars, it is highly requisite to make them as short as possible, on account of the rapidity with which the strength of a pillar decreases as the length increases.

With respect to this last condition, we, however, do not consider that it bears upon the subject, for we presume that the struts are constructed of such section as to obviate their natural tendency to flexure under compressive strains.

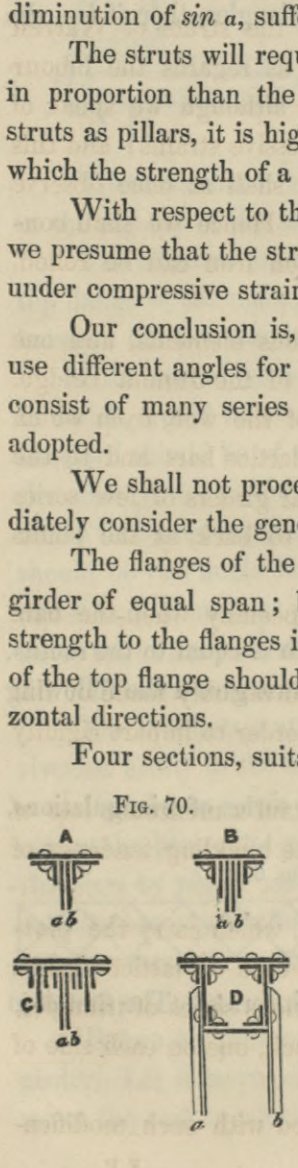
Our conclusion is, that, although for triangular girders of one or two series, we may with advantage use different angles for the struts and ties; that it is not desirable so to do in lattice girders, whose webs consist of many series of triangles, but that in this case the constant angle of 45° should always be adopted.

We shall not proceed further in the exemplification of the calculation of lattice girders, but immediately consider the general form and arrangement of the details.

The flanges of the main girders may be similar to those which may be used for a common plate girder of equal span; but as the web in the case of a lattice girder does not afford so great an amount of strength to the flanges in a vertical direction, as does the corresponding part of a plate girder, the form of the top flange should be such as will enable it to resist the buckling force, in both vertical and horizontal directions.

Four sections, suitable for the top flanges of lattice or triangular girders, are shown at Fig. 70. In each representation *a b* signify ties or struts.

Fig. 70.



The section shown at A is suitable for lattice girders of small span, or having a close web where the flange need not possess a very considerable amount of vertical rigidity, it consists of a flat plate and two angle-irons, between which the diagonal bars of the web are rivetted. When greater vertical stiffness is required, the section shown at B may be conveniently used; this consists of a horizontal plate, two angle-irons, and two vertical plates, between which the diagonals are rivetted. If this arrangement would require too much metal, one vertical plate may be used, placed between the diagonals.

The section C consists of one horizontal plate, two angle-irons, two vertical plates, and a channel-iron between, the ends of the diagonal bars being rivetted between the limbs of the channel-iron.

The vertical plates may, if necessary, be omitted, and then the diagonals may in some cases be more conveniently fixed by rivetting them up between the angle-iron and the channel-iron.

When a large sectional area is required, and the points of junction of the web and flanges are far apart, the section shown at D will be found very convenient. This last section is especially adapted to the construction of large triangular girders, having a web composed of one or two series of triangles.

In this method of construction the diagonals are attached to the outside of the box-shaped flange, by rivets or otherwise.

Of the bottom flange but little need be said, for as it is subject to tensile strains only, any form which will give the proper sectional area conveniently, and also afford suitable means for connecting the web and flanges, may be used.

For triangular girders, the bottom flange may most conveniently be formed of flat bars, extending from joint to joint, connected at those places by one or more pins, or bolts, the whole being very similar in construction to the chain of an ordinary suspension-bridge.

For lattice girders, in which the joints are close, the bottom flange should be of the form shown at A, Fig. 70: but if this will not give sufficient area conveniently, it may be made by the addition of vertical plates, as at B. For very small girders, the vertical plates may be used alone, or the angle-irons alone.

To the form of the diagonals we must now direct the reader's attention.

We have already spoken of those lattice bars which are formed of T-irons, rivetted back to back, with or without flat plates between them. These have been generally used for the struts of Warren girders, though it is evident that the number of rivets is a disadvantage, both as regards the labour expended upon the work and the strength of the diagonals when complete; for although we speak of rivet-holes as producing no deterioration in the resistance of materials to compressive strains; and this is true in theory, if we consider the manner in which these holes are made, we shall at once observe that the inherent strength or cohesion of the material must suffer some injury. Hence we shall conclude, that these sections may with great advantage be replaced by those in which iron can be rolled, such as H and channel-iron sections.

For very large struts we must necessarily use a section made up of various elements, and one convenient for this purpose will be seen in Plate No. 43, exhibiting the details of the Jumna Bridge. The H and channel-irons may be attached to the flanges, either by a portion of the web, from which the flanged parts have been cut away, or by plates rivetted to the webs of the lattice bars and to the flanges of the main girders. Channel-iron is very convenient for large triangular girders of few series of triangles; they should be so placed that they may be rivetted together, back to back, at the points of intersection.

Angle-irons are also occasionally used in the construction of lattice-webs, especially when the bars are long but of small area.

The diagonals of lattice girders having many series of triangles, are almost invariably made of flat bars, the struts and ties being rivetted together at their points of intersection, in order to impart rigidity to the structure.

The lattice bars in this case should be made as broad as possible, so as to obtain a minimum loss of area by the rivet-hole, but they must yet be of sufficient thickness to withstand the buckling tendency of the girders under a load.

It is desirable in all lattice and triangular girders to attach the cross girders, which carry the platform, to standards rather than to the web, and this is especially necessary in the case of lattice girders with a close web, in order that the load may be uniformly distributed upon the various series of triangles.

For small girders these standards may be made of T-irons, placed back to back, one on each side of the web, and rivetted together.

For larger girders, the triangular standards previously described, may be used with such modifications as may be necessary to render them applicable to the particular form of girder for which they are required.

These standards must both distribute the loads over the various systems of triangles, and also act as stiffening plates, and they should be used whether the platform be placed at the top or bottom of the girders.

But in the former case, the broad end of the stiffener will, of course, be attached to the platform.

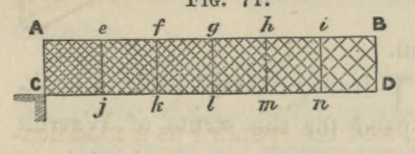
We have seen that the strains on the lattice bars increase as we proceed from the centre of the girder to the point of support, wherefore the strength of the lattice must be increased in the same direction, and the most convenient method of effecting this we will now proceed to consider.

In the construction of triangular girders, it is evident that the form, or size of one diagonal, can in no way materially cause it to interfere with any other, as it is only attached to others at its extremities, hence in these structures we may conveniently vary the areas of the ties and struts to suit the strain, the ties being as usual made of flat bars.

In lattice girders whose diagonals consist of flat bars, which are rivetted together at their intersections, it will be more convenient to increase the breadth than the thickness of the bars, for if the latter be done, the joints will not be so satisfactory as by the former method.

It may, however, be preferable in many cases, to increase the number of diagonals, retaining the same area for all. This method of arranging the lattice bars is shown in Fig. 71, where A B C D represents half a lattice girder, c being one point of support, and B D being the centre standard of the girder. The web of the girder is in this case divided into six equal bays, by the standards $e j$, $f k$, etc., and in each bay is a different number of lattice bars.

Fig. 71.



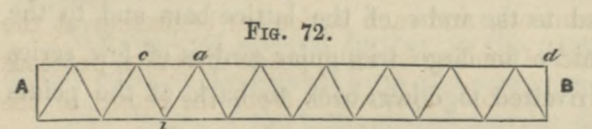
The numbers for each part of the girder, are calculated as follows:—

First, consider that each bay of the girder contains one triangle, calculate the strain on each bar, and the corresponding sectional area, then replace each of these bars by as many of those of which the web is to consist, as will make up the required sectional area.

The dimensions of plate girders may be calculated upon the supposition that the web consists of lattice bars, spread out very thin so as to form a continuous plate, but we shall not enter into this subject, as we consider that the methods already detailed for the calculation of plate girders are of greater practical value.

Before concluding our notice of lattice girders, it seems desirable to offer a few remarks upon the construction of Bridges intended to carry two lines of railway, with regard to the comparative advantages of carrying both lines on one bridge, or placing them on two bridges side by side. Let Fig. 72 represent a Warren girder supported upon piers A B, and let us first regard it as carrying a singe line of railway.

Fig. 72.



From our foregoing remarks it will be observed, that any strut $a b$, and its corresponding tie $b c$, will suffer maximum strains, when the part $a b$ included between such strut and the furthest extremity of the girder, is loaded.

Let the girder be of such span that a train will completely cover it, then when a train passes over the bridge, each lattice bar in turn will be subject to a maximum strain. Let the bridge carry two lines of railway, then it will very seldom happen that the part $a b$ will be subject to a full load and when such does occur, it can only bring a maximum strain on one strut in each girder, for to effect this, the trains moving in opposite directions must have progressed so far, that the front of the train entering upon the bridge arrives at a point midway between a and c at the same moment that the back of the departing train arrives at the same spot.

From this we conclude, that it is desirable to carry the two lines of rail on one bridge, especially when the structure is of small span, and, by so doing, we shall also gain other advantages, for when only one train passes over the bridge, the resistance of the various elements will be economised, and although we use factors of safety in designing the bridge, which are perfectly reliable, yet it is always desirable to save the structure any unnecessary strain.

Another point worthy of consideration is also gained by this method of construction, viz., greater width is obtained, and, therefore, greater lateral rigidity, which is highly desirable in railway structures.

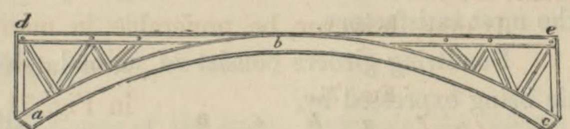
The remaining parts of lattice-girder bridges may be calculated in a manner similar to that proposed for the corresponding parts of plate girders.

We have now concluded our instructions with regard to straight girders, but we must remind our readers, that these being necessarily of a very general character, cannot possibly include one tithe of the emergencies which will arise in special cases, they must, therefore, be looked upon as explanatory of the most convenient method of procedure, which should, in every case, be followed as closely as the circumstances will allow.

We will now, in a similar manner, describe the construction of arch and bowstring bridges. The section usually adopted for arched ribs, whether of cast- or wrought-iron, is the simplest of those forms hitherto proposed for plate girders, viz., the I section.

We may regard each arched rib as consisting of three essential parts, the arch, *a b c*, in Fig. 73, the girder, *d e*, which carries the platform, and the fillings in of the spaces, *d a b*, *e c b*, which are called the spandrils of the arch.

Fig. 73.



A proper number of these ribs having been constructed, they are braced together, and the platform is placed upon the top.

It is evident that the part *d e*, may be treated as a continuous girder, and the spandril as a modification of a lattice girder, the platform being of course similar to that for a girder-bridge of equal dimensions, therefore the arch itself is the only part of which it is requisite here to treat, the bed-plates being described in a subsequent Chapter. Let us suppose that a cast-iron arch is required to carry a double line of rails over an obstacle of 100 feet in length.

Let the weight of the structure be 2·5 tons per lineal foot, that of the load 2 tons per lineal foot, and let the rise or versed sine of the arch be 8 feet 6 inches. The rib is supposed to be so braced that it will be subject to none but direct strains.

The arch will then be treated as if it consisted of an infinite number of infinitely short diagonal bars, each acting like those of a lattice girder, but having not only the shearing strain, but also the horizontal strain to bear, for the lattice bars (so to speak), terminate at the centre in a very short piece of top flange, as it were; the bottom flange being substituted by the abutments, and each lattice bar beginning at the crown of the arch transmits its load to the next, which, instead of tending in an opposite direction, proceeds in a similar direction, that is to say, all the diagonals are directed downwards from the centre of the arch to the point of support, therefore on every element the strain enters at the top, hence all the elements are struts, and the whole arch is subject to compressive strain.

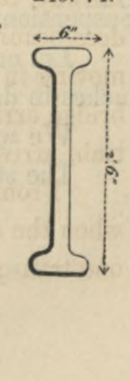
The crown forming part of a top flange, the strain at that point will be found full load, thus:—

$$S = \frac{W\,l}{8\,V} = \frac{(2 + 2{\cdot}5) \times 100 \times 100}{68} = 662 \text{ tons nearly,}$$

taking 5 tons per square inch as the resistance of cast-iron, the total area will be at the crown,

$$\frac{662}{5} = 132{\cdot}4 \text{ square inches.}$$

Fig. 74.



Let the bridge consist of three arched ribs, then we will take for the sectional area of each at the centre 50 square inches, in order to allow for defects.

This area may be provided by using such a section as that shown at Fig 74.

Using the formula for the strain at any section, given in the theoretical part on arches, we find for the strain at the abutments,

$$S = \sqrt{(662)^2 + \{4{\cdot}5 \times 50^2\}} = 695 \text{ tons nearly,}$$

requiring a sectional area, equal to

$$\frac{695}{5} = 139 \text{ square inches.}$$

It may be advisable to increase the sectional area at this place, by making the ribs 2 feet 9 inches deep, the other dimensions remaining the same.

Let us see what areas would be required if the bridge were of wrought-iron:—

Taking four tons per square inch as the resistance of wrought-iron, the area at crown,

$$= \frac{662}{4} = 165{\cdot}5 \text{ square inches,}$$

say 60 square inches in each rib, or it may be more convenient to increase the number of ribs; the sectional area at the springing must not be less than

$$\frac{695}{4} = 173{\cdot}75 \text{ square inches,}$$

say 65 square inches on each rib, if three be used.

The construction of these arches, being in all respects very similar to that of plate-girders, needs no further description.

Vertical, plane, or cubic bracing may be used for arched bridges, and the latter will perhaps be found the most satisfactory.

Bowstring girders consist of arched ribs, whose springings are connected by a tie, the strain in that tie being expressed by,

$$\frac{W\,l}{8\,V} = 662 \text{ tons.}$$

If the above case applied to a bowstring-girder, the area required for a tie of wrought-iron, being

$$\frac{662}{5} = 132{\cdot}4 \text{ square inches,}$$

or the same as that of the cast-iron arches at the crown.

The roadway may be suspended from the arch by standards similar to those used in lattice girders. The tie may be similar in form to the bottom flange of a lattice girder.

The arches should be of such section as will give the greatest amount of rigidity in all directions, well braced to the tie and to each other at the crown.

With regard to the suspension chains and inverted bowstring girders, the strains may be calculated in exactly the same manner as those upon arches and upright bowstring girders; but it must be borne in mind that the strains, although equal in intensity, are not all similar in description.

The strains are compressive on the arch, tensile on the chain, compressive on the upright bow, tensile on the inverted bow, and so on with other parts of the girders.

The trigonometrical formulæ may also, if preferred, be applied to suspension chains. The strength of materials requires no remark at this place, the formulæ given at the theoretical part being quite practical.

We will now examine the relative economy of various kinds of girders, considering them all to be constructed of wrought-iron, without joint, and of the least theoretical sectional area. We shall assume the resistance of wrought-iron per sectional square inch to be, for tensile strains 5 tons, for compressive strains 4 tons, and for shearing strains 4 tons.

The weight will be taken at 10 lbs. for 3 feet of wrought-iron by one square inch.

We shall include the following structures: the Plate-Girder, the Warren-Girder, the Arch, and the Suspension-Chain.

Let one girder be required to bear a load of one ton per lineal foot, of 60 feet span, and 5 feet 4 inches in depth.

We will commence with the plate-girder.

The strain on either flange at the centre of the span will be,

$$\frac{W\,l}{8\,d} = \frac{1 \times 60 \times 60}{8 \times 5{\cdot}33} = 84{\cdot}42 \text{ tons,}$$

The sectional area of the top flange at this point will be,

$$\frac{84 \cdot 42}{4} = 21 \cdot 105 \text{ square inches,}$$

diminishing as the ordinates of a parabola to *o* at the points of support.

Assume the flange of rectangular section 9 inches wide, then the weight of the top flange will be,

$$\frac{2}{3}\left\{2 \cdot 345 \times 60 \times 12\right\} \times 10 \times 0 \cdot 25 = 2813 \cdot 5 \text{ lbs.}$$

The sectional area of the bottom flange will be,

$$\frac{84 \cdot 42}{5} = 16 \cdot 485 \text{ square inches.}$$

The weight of this flange will, therefore, be

$$\frac{2}{3}\left\{1 \cdot 831 \times 60 \times 12\right\} \times 10 \times 0 \cdot 25 = 2107 \cdot 2 \text{ lbs.}$$

The shearing-strain on the web at the points of support will be,

$$\frac{W}{2} = \frac{60}{2} = 30 \text{ tons,}$$

and the sectional area at those points,

$$\frac{30}{4} = 7 \cdot 5 \text{ square inches,}$$

which diminishes as the ordinates of a straight line to *o* at the centre of the span; therefore, the weight of the web will be,

$$7 \cdot 5 \times 30 \times \tfrac{1}{3} \times 10 = 750 \text{ lbs.}$$

Summing these weights, we find the total theoretical weight of the Plate girder, thus—

$$2813 \cdot 5 \times 2197 \cdot 2 \times 750 = 576 \cdot 7 \text{ lbs.}$$

We will now calculate the weight of the Warren girder. It will contain 10 triangles, each lattice bar being 6 feet in length, the load in each triangle will be 6 tons, and the strains must be calculated for a total load as in the last case.

We will take half the girder, beginning at the centre, and double the weight so obtained for the total weight of the girder. The strains will be as follows, the load being on the top.

First bar from centre is a tie—no strain.

Second bar from centre is a strut	$\frac{6}{sin\ 60}$	=	6·772 tons.
Third bar from centre is a tie	$\frac{6}{sin\ 60^\circ}$	=	6·727 do.
Forth bar from centre is a strut	$\frac{12}{sin\ 60^\circ}$	=	13·544 do.
Fifth bar from centre is a tie	$\frac{12}{sin\ 60^\circ}$	=	13·544 do.
Sixth bar from centre is a strut	$\frac{18}{sin\ 60^\circ}$	=	20·316 do.
Seventh bar from centre is a tie	$\frac{18}{sin\ 60^\circ}$	=	20·316 do.
Eighth bar from centre is a strut	$\frac{24}{sin\ 60^\circ}$	=	27·088 do.
Ninth bar from centre is a tie	$\frac{24}{sin\ 60^\circ}$	=	27·088 do.
Tenth bar from centre is a strut	$\frac{30}{sin\ 60^\circ}$	=	33·860 do.

Each square inch 6 feet long weighs 20 lbs.

The areas and weights of the lattice bars, will be

	Area, Square Inches, nearly.	Weight of Bars.
Second bar ...	$\frac{6\cdot772}{4} = 1\cdot693$	$1\cdot693 \times 20 =$ 33·86 lbs.
Third bar ...	$\frac{6.772}{5} = 1\cdot354$	$1\cdot354 \times 20 =$ 27·08 do.
Fourth bar ...	$\frac{13\cdot544}{4} = 3\cdot386$	$3\cdot386 \times 20 =$ 67·62 do.
Fifth bar ...	$\frac{13\cdot544}{5} = 2\cdot709$	$2\cdot709 \times 20 =$ 54·18 do.
Sixth bar ...	$\frac{20\cdot316}{4} = 5\cdot079$	$5\cdot079 \times 20 =$ 101·58 do.
Seventh bar ...	$\frac{20\cdot316}{5} = 4\cdot063$	$4\cdot063 \times 0 =$ 81·26 do.
Eighth bar ...	$\frac{27\cdot088}{5} = 6\cdot772$	$6\cdot772 \times 20 =$ 135·44 do.
Ninth bar ...	$\frac{27\cdot088}{5} = 5\cdot418$	$5\cdot418 \times 20 =$ 108·36 do.
Tenth bar ...	$\frac{33\cdot860}{4} = 8\cdot465$	$8\cdot465 \times 20 =$ 196·30 do.
	Total weight of half the web	778·68 do.

The half of the top flange, will consist of six sections, the strains on which will be as follows:

On the first section from pier...		no strain.
On the second section from pier		27·088 tons.
On the third section from pier	$27\cdot088 + 20\cdot316 =$	47·404 do.
On the fourth section from pier	$47\cdot404 + 13\cdot554 =$	60·958 do.
On the fifth section from pier...	$60\cdot958 + 6772 =$	67·730 do.
On the sixth section from pier		67·730 do.

The areas and weights of these sections, will be

	Area, Square Inches, nearly	Weight of Bars.
First section	none	none.
Second section... ...	$\frac{27\cdot088}{4} = 6\cdot772$	$6\cdot772 \times 20 =$ 135·44 lbs.
Third section	$\frac{47.404}{4} = 11\cdot851$	$11\cdot851 \times 20 =$ 237·02 do.
Fourth section ...	$\frac{60\cdot958}{4} = 15\cdot239$	$15\cdot239 \times 20 =$ 304·78 do.
Fifth section	$\frac{67\cdot730}{4} = 16\cdot932$	$16\cdot932 \times 20 =$ 338·64 do.
Sixth section	$\frac{67\cdot730}{4} = 16\cdot932$	$16\cdot932 \times 10 =$ 169·32 do.
	Total weight of half top flange	1185·20 do.

The strains on half the bottom flange will be,

On the first section from pier		16·93 tons.
On the second section from pier	$16\cdot93 + 37\cdot088 =$	44·018 do.
On the third section from pier...	$44\cdot018 + 20\cdot316 =$	64·334 do.
On the fourth section from pier	$64\cdot334 + 13\cdot554 =$	77·888 do.
On the fifth section from pier...	$77\cdot888 + 6\cdot772 =$	84·66 do.

The areas and weights of these sections will be,

	Areas, Square Inches, nearly.	Weight of Bars.
First section	$\frac{16\cdot93}{5} = 3\cdot386$	$3\cdot386 \times 20 =$ 67·72 lbs.

	Areas, Square Inches, nearly.	Weight of Bars.
Second section	$\frac{44 \cdot 018}{5} = 8 \cdot 204$	$8 \cdot 204 \times 20 = 164 \cdot 08$ lbs.
Third section	$\frac{64 \cdot 334}{5} = 12 \cdot 867$	$12 \cdot 867 \times 20 = 257 \cdot 34$ do.
Fourth section	$\frac{77 \cdot 888}{5} = 15 \cdot 578$	$15 \cdot 578 \times 20 = 311 \cdot 56$ do.
Fifth section	$\frac{84 \cdot 666}{5} = 16 \cdot 233$	$16 \cdot 233 \times 20 = 324 \cdot 66$ do.
Total weight of half bottom flange		$1125 \cdot 36$ do.

The total weight of the complete Warren girder will be,

$$2 \left\{ 778 \cdot 68 + 1185 \cdot 20 + 1125 \cdot 36 \right\} = 6178 \cdot 48 \text{ lbs.}$$

The arch next demands our attention.

The strain at the crown is,

$$\frac{W\,l}{8\,V} = \frac{1 \times 60 \times 60}{8 \times 5 \cdot 33} = 84 \cdot 42 \text{ tons.}$$

And therefore the sectional area is,

$$\frac{84 \cdot 42}{4} = 21 \cdot 105 \text{ square inches.}$$

The strain at the springings will be,

$$= \sqrt{(84 \cdot 42)^2 + (30)^2} = 8932 \text{ tons.}$$

And therefore the sectional area is,

$$\frac{89 \cdot 32}{4} = 22 \cdot 33 \text{ square inches.}$$

The weight of the arch will be,

$$5187 \cdot 6 \text{ lbs.}$$

We will consider the spandril as a continuous plate on which the load is rested. Its horizontal area will be,

$$\frac{60}{4} = 15 \text{ square inches.}$$

The weight of the spandril plates will therefore be,

$$132 \cdot 5 \text{ lbs.}$$

The total weight of the arch will be,

$$5187 \cdot 6 + 132 \cdot 5 = 5320 \cdot 1 \text{ lbs.}$$

For a suspension chain the weight will evidently be $\frac{4}{5}$ of this, the ratio of the compressive and tensile strengths, therefore the weight

$$= \tfrac{4}{5} \times 5320 \cdot 1 = 4256 \cdot 08 \text{ lbs.}$$

Let us now collect together the different weights:—

The plate girder weighs - - - -	5760·70 lbs.
The Warren „ „ - - - -	6178·48 lbs.
The arched rib „ - - - -	5320·10 lbs.
The chain „ - - - -	4256·08 lbs.

These are the *theoretical* numbers, and, looking at them, we observe the following order of economy, beginning with the most economical:—

Chain Bridge; Arched Rib; Plate Girder; Warren Girder.

Let us now examine the subject *practically*. First, the plate girder cannot be made with either the web or the flanges, reduced as specified above; we will, therefore, make an addition to the theoretical weight, but omitting stiffening plates, etc., which will be required by all girders.

We will add one-half to the weight of the web, and one quarter to that of the flanges. The weight of the girder will then become

$$2813 \cdot 5 + 703 \cdot 8 + 2197 \cdot 2 + 549 \cdot 3 + 750 + 375 = 7388 \cdot 8 \text{ lbs.}$$

With the Warren girder we can approach theoretical accuracy much nearer than with the plate girder, because the web being aggregated together in various places, becomes of sufficient rigidity to succeed practically, and it is only the central lattice bars that we need augment.

We will, in this case, add one-third to the weight of the web, and one-sixth to the weight of the flanges, the total weight of the Warren girder will then become,

$$2 \left\{ 778 \cdot 68 + 259 \cdot 56 + 1185 \cdot 2 + 197 \cdot 5 + 1125 \cdot 36 + 189 \cdot 56 \right\} = 7471 \cdot 7 \text{ lbs.}$$

This does not differ very widely from the weight of the plate girders, but it is certainly in excess, this disadvantage may, however, be entirely concealed by greater convenience of manufacture, carriage, etc.

It will generally occur, that there is less work in Warren girders than in plate girders, which will, of course, materially reduce the price. It may also be made of bar- instead of plate-iron, but otherwise we think the plate girders are preferable on account of their greater solidity.

The arch is theoretically lighter than either of the above, but we have to consider the longitudinal girder that will be required upon the top of the spandril to carry the roadway. Let this be supposed to be connected with the platform at every 6 feet; then, because the sectional area of either is as the square of the span, and inversely as the depth, we may reckon the weight of this from that of the plate girder, thus:—taking the depth at 1 foot,

$$5760 \cdot 7 \times \frac{1}{100} \times 5 \cdot 33 = 307 \cdot 045 \text{ lbs.}$$

We must also make an addition to the weight of the spandril; let this be about one-third, or say 50 lbs., then the total weight of the arch will be,

$$5320 + 50 + 307 \cdot 0 = 5677 \text{ lbs.,}$$

leaving a slight advantage to this system. A longitudinal girder will also be required to carry the load in the case of the suspension-bridge, and it must be somewhat stronger than the last, in order to prevent great vibration. Let this girder be 3 feet deep, and let it be required to distribute the load over a considerable length of chain, so that we may consider its effective span as 20 feet. Its weight will be,

$$5760 \cdot 7 \times \frac{1}{9} \times \frac{5 \cdot 33}{3} = 1137 \cdot 2 \text{ lbs.}$$

The total weight of the chain bridge will then become,

$$4256 \cdot 08 + 1137 \cdot 2 = 5393 \cdot 28 \text{ lbs.}$$

On examining these structures, we see that these longitudinal girders may, if we please, be constructed as continuous girders, if so, we may reduce their strength, that in the case of the arch becoming, approximately,

$$\tfrac{2}{3} \times 307 = 204 \cdot 66 \text{ lbs.,}$$

and that for the suspension-bridge,

$$\tfrac{2}{3} \times 1137 \cdot 4 = 758 \cdot 13 \text{ lbs.}$$

This last resisting nine times the strain that the former is required to do, in order to prevent considerable alterations in the form of the chain.

We will once more place this number together for a final comparison.

Weight of the plate girder - - - -	7388·80 lbs.
Weight of the Warren „ - - - -	7471·72 lbs.
Weight of the arched rib - - - -	5574·61 lbs.
Weight of the chain bridge - - - -	5014·21 lbs.

We may regard this statement as being very near the truth, for it is but reasonable that the arch should require less material than the girder bridge, for we relieve the structure entirely of one flange in this case, the lightest certainly, but in the inverted arch or chain we remove the heaviest elements of the structure.

The only disadvantage with these arched structures is, that we cannot avail ourselves of the principles of continuous girders if we use them where many spans are required.

When we increase the span the great difference between the chain and the girder systems will, of course, become more marked, and we shall arrive at spans to which suspension-bridges only can be at all economically applied; for the saving is equal to one-half the total weight of the chain bridge. By going more closely into the matter, we should find more reason to give our preference to the chain bridge, on account of the saving in standards, etc.

We have plenty of margin also, to make the longitudinal girders stronger if we deem it necessary, or to employ more bracing. When we carefully consider all these facts, although they may at first appear somewhat overstrained, we are led to examine more closely into the disposal of materials in the suspension-bridge, and, in so doing, we cannot fail to be struck with the excellent aptitude of each of the more important elements to play the part for which it is designed. Hence, we conclude, that from its lightness, its applicability to large spans, and its convenience of adjustment, the suspension chain acting in conjunction with a straight girder is that form of bridge which seems destined, at some future period, to supersede the others for carrying all kinds of traffic over considerable obstacles.

For some purposes, and, especially, for bridges of small span, Warren, and other lattice girders, are frequently very convenient, and, no doubt, quite as economical as plate girders; they are, however, both excelled in point of lightness and beauty by arch bridges, but where a continuous girder of many small spans can be applied, the straight girder is preferable to the arch.

We think we cannot conclude the present Chapter better than with a recapitulation of such formulæ as are practically useful in designing bridges, so that any of them may be convenient for reference.

The following notations will be used throughout this recapitulation. The areas are for wrought-iron:—

l = total length of clear span.
x = distance of any point from one pier.
y = distance of any point from centre of span.
d = depth of girder.
s = direct strain on either flange.
W = total load on any girder.
w = load per lineal foot.
M = moment of strain at any point.
M′ = moment of strain over first pier.
M″ = moment of strain over second pier.
M c = moment of strain at centre.
C = thrust or compressive force.
T = pull or tensile force.
A = sectional area in square inches.
R′ = reaction on first pier = weight on ditto.
R″ = reaction on second pier = weight on ditto.

All lineal measurements are in feet; all forces in tons. The strength of iron is taken at 5 tons per square inch in tension, 4 in compression and shearing. Other notations will be explained in progress.

(1.) For a beam fixed at one end, and loaded at the other:—

$$M = W\{l - x\} \quad S = \frac{W}{d}\{l - x\}.$$

For strain at point of support:—

$$M = W\,l \quad S = \frac{W\,l}{d}.$$

Area of top flange at any point:—

$$A = \frac{W}{5\,d}\{l - x\}.$$

Area of bottom flange at any point:—

$$A = \frac{W}{4\,d}\{l - x\}.$$

(2.) For a beam fixed at one end, and loaded uniformly:—

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

For strain at point of support:—

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

Area of top flange at any point:—

$$A = \frac{w}{10\,d}\,\{l-x\}^2.$$

Area of bottom flange at any point:—

$$A = \frac{w}{8\,d}\,\{l-x\}^2.$$

(3). For a beam, (plate girder) supported at both ends freely, and loaded at the centre,

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

$$M\,c = \frac{W\,l}{4} \quad S\,c = \frac{W\,l}{4\,d}.$$

Area of top flange at any point,

$$A = \frac{W\,x}{8\,d}.$$

Area of bottom flange at any point,

$$A = \frac{W\,x}{10\,d}.$$

(4). For a beam, (plate girder) supported at both ends freely, and loaded uniformly,

$$M = \frac{w\,x}{2}\left\{ x - l \right\} \quad S = \frac{w\,x}{2\,d}\left\{ x - l \right\}$$

$$M\,c = \frac{w\,l^2}{8} \quad S\,c = \frac{w\,l^2}{8\,d}.$$

Area of top flange at any point,

$$A = \frac{w\,x}{8\,d}\left\{ x - l \right\}.$$

Area of bottom flange at any point,

$$A = \frac{w\,x}{10\,d}\left\{ x - l \right\}.$$

(5). For a beam, (plate girder) supported at both ends, one being free and the other fixed, and loaded uniformly,

$$M = \frac{w\,x^2}{2} + \frac{w\,l}{8}\left\{ l - 5\,x \right\}$$

$$S = \frac{w}{2\,d}\left\{ x^2 + \frac{l}{4}(l - 5\,x) \right\}.$$

$$M' = \frac{w\,l^2}{8} \text{ over pier, } S = \frac{w\,l^2}{8\,d}.$$

Area of top flange at any point,

$$A = \frac{w}{8\,d}\left\{ x^2 + \frac{l}{4}(l - 5\,x) \right\}$$

Area of bottom flange at any point,

$$A = \frac{w}{10\,d}\left\{ x^2 + \frac{l}{4}(l - 5\,x \right\}$$

Note.—x is measured from the pier upon which the girder is fixed.

(6.) For a beam (plate girder) fixed at both ends, and loaded uniformly,

$$M = \frac{w x^2}{2} + \frac{w l}{12} \{l - 6 x\}$$

$$S = \frac{w}{2 d} \left\{ x^2 + \frac{l}{6} (l - 6 x) \right\}$$

$$M' = \frac{w l^2}{12} \text{ over pier, } S = \frac{w l^2}{12 d}.$$

Area of top flange at any point,

$$A = \frac{w}{8 d} \left\{ x^2 + \frac{l}{6} (l - 6 x) \right\}.$$

Area of bottom flange at any point,

$$A = \frac{w}{10 d} \left\{ x^2 + \frac{l}{6} (l - 6 x) \right\}.$$

(7.) For a lattice web fixed at one end, and loaded at the other.

λ = length of any lattice bar,

Σ = strain on any lattice bar,

$$\Sigma = W \frac{\lambda}{d}.$$

The area of any diagonal directed down towards the point of support,

$$A = \frac{W \lambda}{4 d}.$$

Area of any bar directed upwards towards the point of support,

$$A = \frac{W \lambda}{5 d}.$$

(8.) For a lattice web fixed at one end and loaded uniformly. (Note.—If the load be on the top, x must terminate at the foot of the bars, but if the load come first upon the bottom flange, x must terminate at the top of the bar.)

$$\Sigma = \frac{w \lambda}{d} \{l - x\}.$$

Area of any bar directed downwards to the point of support,

$$A = \frac{w \lambda}{4 d} \{l - x\}.$$

Area of any bar directed upwards to the point of support,

$$A = \frac{w \lambda}{5 d} \{l - x\}.$$

(9.) For a lattice web supported at both extremities and loaded at the centre,

$$\Sigma = \frac{W \lambda}{2 d}.$$

The area of any diagonal directed down from the load towards either point of support,

$$A = \frac{W \lambda}{8 d}.$$

Area of any diagonal directed in the opposite course,

$$A = \frac{W \lambda}{10 d}.$$

(10.) For a lattice web supported at both ends, and loaded uniformly. (Note.—The former note in (8.) applying to x, applies here to y.)

$$\Sigma = \frac{w y \lambda}{d}.$$

Area of any bar directed downwards from the centre of the span towards the abutment,

$$A = \frac{w y \lambda}{4 d}.$$

Area of any bar placed in the opposite direction,

$$A = \frac{w y \lambda}{5 d}.$$

(11.) For an arch loaded uniformly at crown, (d = versine.)

$$C = \frac{w\,l^2}{8\,d}$$

$$A = \frac{w\,l^2}{32\,d}.$$

At any other point,

$$C = \sqrt{\left(\frac{w\,l^2}{8\,d}\right)^2 + (w\,y)^2}$$

$$A = \frac{\sqrt{\left(\frac{w\,l^2}{8\,d}\right)^2 + (w\,y)^2}}{4}$$

(12.) For a suspension chain at centre of span,

$$T = \frac{w\,l^2}{8\,d}.$$

$$A = \frac{w\,l^2}{40\,d}.$$

At any other point,

$$T = \sqrt{\left(\frac{w\,l^2}{8\,d}\right)^2 + (w\,y)^2}$$

$$A = \frac{\sqrt{\left(\frac{w\,l^2}{8\,d}\right)^2 + (w\,y)^2}}{5}.$$

For continuous girders, see Chapter IV., Part I. The formulæ are too bulky for recapitulation. The expressions given above for strains, will, of course, apply to any bridge whatever; but those for the sectional areas, apply only to wrought-iron.

CHAPTER XII.

ON THE FORM OF IRON, AND ON THE PROCESSES OF MANUFACTURE.

In commencing this important Chapter it is highly desirable that we should define the limits with which we purpose to bound our observations.

Our description will commence with the material as it is placed in the workman's hands for the construction of girders; we shall briefly notice the various manipulations which it undergoes, and conclude when we arrive at the condition of the structure as it leaves the workshop.

Cast-iron, wrought-iron, and steel, are the materials upon which we work, the former being the first form of iron from which the others are derived.

We have not sufficient space at our command to enable us to enter into the subject of conversion of cast-iron into wrought-iron and steel, nor would it be necessary so to do if such were the case, as this is certainly not a part of our subject, although very nearly related to it; we will, however, describe the physical and chemical properties of these materials in order that their natures and, therefore, the proper mode of treating them may be known.

We will first describe the nature and aspect of cast-iron. This substance is a complicated mixture, consisting chiefly of iron, but also containing greater or less proportion of the following substances: sulphur, carbon, free, and in the form of graphite; silicon, manganese, phosphorus, combined carbon; and, finally, minute proportions of various foreign metals.

The manner in which each of these elements affects the nature of cast-iron, is not by any means determined, though recent experiments have shown the action of a few.

Many have attributed the superiority of Swedish iron to the titanic acid which it contains, but this seems to have been done, rather from the discovery of titanic acid in comparatively large quantity in that iron, than from experimental research.

It has also been stated, that an admixture of certain metals, especially nickel, produces a great improvement in the quality of iron, and that the conclusions arrived at, are the results of some year's experience.

The manner in which the carbon in the various forms of iron is combined, has been a matter of much doubt; but it seems very probable from recent experiments, principally synthetical, that it exists in combination with nitrogen gas, in the form of cyanogen, so that steel must be regarded as a cyanide rather than as a carburet of iron, though it is quite probable that the cyanogen is not combined with the iron, or, at all events, with but a small portion of it. The solution of this interesting problem would, in all probability, throw some considerable light upon the cause of the superiority of steel to iron, but it is a question which comes within the scope of the Metallurgical Chemist, rather than within that of the Engineer.

These observations are quite general, and apply to the various forms of iron, but we will now return to the description of cast-iron.

The physical properties of this material vary necessarily with the chemical, and consequently there is an endless variety. The most general are, however, crystalline texture, sometimes approaching in appearance to a granulated form; brittleness, rigidity. White cast-iron is of crystalline structure, hard, and brittle; its fracture is whitish and radiated. Grey cast-iron is softer and much tougher than the white; its fracture is grey, and presents a granular appearance with some metallic lustre.

Between these there are various qualities tending to one or the other of these extreme descriptions.

The tensile resistance of cast-iron varies from 13,000 lbs. per square inch of sectional area to 30,000 lbs., the average being 16,500 lbs.

The resistance per square inch of cast-iron to compressive force varies from 82,000 lbs. to 145,000, the average being 112,000 lbs.

From this we find that the resistance of cast-iron to compression is nearly seven times its resistance to tension, which fact indicates a peculiar molecular condition; pointing to the conclusion that the inherent repulsive forces co-existing between the molecules are superior to the mutual attractive forces between the same; and as this condition does not exist in wrought-iron, it would seem to be attributable to some ingredients which possess great affinity for iron, but greater repulsion to each other; thus, if a a' be two atoms of iron, having equal attraction and repulsion, and b c be two atoms of different substances, but with similar properties, repelling each other, but combining with iron; when the atoms are brought together, so as to combine, the effects noticed in cast-iron would result. We will not now enter farther into these speculations, but proceed with the more practical part of our remarks.

Wrought-iron is produced from cast-iron by depriving the latter of most of its foreign ingredients; hence it is the most pure of commercial irons. It is of fibrous texture; very tough, flexible, ductile, and malleable. Its tensile strength per square inch averages 60,000 lbs., and its compressive strength about 45,000 lbs. Iron chemically pure, obtained at a high temperature, is white, and shines like silver.

Steel, the last kind of iron which we have to notice, may be produced by depriving cast-iron of nearly all its foreign ingredients, with the exception of carbon, and adding nitrogen; or, it may be formed by exposing wrought-iron to a high temperature, in contact with pulverized carbon in the presence of nitrogen. It is probable that in either case the carbon combines with nitrogen, as before stated, to form canogen; and we feel inclined to lay considerable stress upon this point.

The tensile resistance of steel per square inch varies from 100,000 lbs. to 130,000 lbs., and it is capable of assuming different degrees of hardness, according to the manner in which it was cooled when last heated; but, however hard it is rendered by this process, it may be softened by annealing. A coating of steel is imparted to wrought-iron articles, by heating them in contact with any body capable of yielding nitrogen and carbon, as leather, horn, ferrocyanide of potassium, etc., whereby we obtain articles possessed of the toughness and strength of wrought-iron with the hardness of highly-tempered steel. This process is called case-hardening. The process of hardening and tempering is conducted as follows:—

The steel to be hardened is heated to redness and plunged in salt-water, being moved about; when cold it is withdrawn, and the part to be tempered is made bright by rubbing with a grindstone; it is then heated to the temperature which reduces the hardness to the proper degree, when it is allowed to cool. The temperature is judged of by the colour exhibited by the coating of oxide formed upon the bright part of the steel. The colour changes with the thickness of this coating, as the interference of the rays of light will cause the extinction of certain rays depending upon the thickness and refractive power of the medium upon which the rays strike.

Cutting tools are usually tempered to a pale, straw-yellow, slight springs to a blue, and heavy springs to a dark blue tint.

If one part only is to be hardened, that part only should be cooled at first, then quickly brightened with the stone; the heat being conducted from the other part of the article will raise it to the proper temperature, when the whole should be cooled.

Large articles to be tempered all over, may be reheated over a fire. Slight articles may be tempered by turning them about upon a piece of hot iron until the proper tint appears.

Springs, after hardening, may be tempered either by dipping them in grease or oil, and heating until it is nearly all burnt off, or by immersing them in a bath of melted lead, in which, however, a portion of lead remains unmelted, in order that the temperature may not rise above the melting point of that metal, about 612° Fahrenheit's scale.

When long articles are hardened, care should be taken that they are immersed vertically in the water-trough, otherwise they will exhibit a great tendency to warp or twist.

We have explained this property of hardening in this place, on account of its being peculiar in its extent to steel.

The workman with whom we first have to deal for one particular purpose, receives the iron, either as pig-iron, or scrap, or a mixture, that is to say, in this form he receives iron from which castings are to be made. The wrought-iron and steel is provided in the shape of plates, and bars of various forms, and also as wire.

We will commence with the production of cast articles, which are manufactured by pouring the metal in a molten state into a cavity into which it is allowed to solidify. The formation of this cavity is a manipulation requiring considerable care and skill, but the first step consists in the construction of a representation of the article to be cast, about which the above-mentioned cavity may be formed.

This representation is called a pattern, and must be somewhat larger than the size of the finished article, in order to allow for the contraction in cooling, and also to provide sufficient material to allow of a finished surface being imparted to the casting, if such be required. The pattern must also be slightly tapered, to allow of its removal from the cavity which is formed in the sand, and called a mould. The contraction of iron castings in cooling is about $\frac{1}{10}$ of an inch for every lineal foot, this must, therefore, be allowed in the formation of the pattern.

Before we can proceed further with pattern-making, it will be necessary to explain the next manipulation partially, and then we must finish the description of the two together, otherwise we cannot render the matter sufficiently clear, except we occupy an exorbitant amount of the reader's time.

The manipulation of which we speak, consists in the formation of the above-mentioned cavity, and is known by the name of moulding. Three kinds of materials are used for the construction of moulds; we will, therefore, arrange the moulding of articles under three heads, viz.:—moulding in green sand; moulding in baked sand; and moulding in loam.

The mould must possess three essential properties, it must in every respect resemble the article to be cast, it must be sufficiently solid to resist the weight of the molten metal poured into it, and it must, moreover, be possessed of sufficient porosity to allow of the escape of air from the cavity, and also of the gases generated by the contact of the heated metal with carbonaceous matter in the sand.

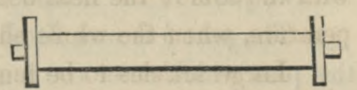
If the texture of the mould be not sufficiently open, the air or gas will probably rise to within a short distance of the surface of the metal, and there settle, rendering the casting defective; this is called blowing. It may even occur to a very dangerous extent, forcing the liquid metal out of the mould to a considerable elevation.

If the binding properties of the sand be insufficient, fine details of form cannot be preserved, and the mould will be apt to scab; that is, the part of the bottom of the mould will peel off and settle on the top of the casting.

Let us now suppose that a short flange cylinder is to be cast, to serve as a brace or distance piece, the casting being made in green sand. The pattern will be, externally, of the same form as the required distance piece, but somewhat larger, and instead of having a cylindrical hole through its entire length, a cylindrical protuberance of the same diameter as the required hole will be attached to each end of the pattern, as shown in Fig. 75.

These projections are called prints, and they are intended to form recesses in the sand, wherein cylindrical pieces called cores, may be rested to produce the cylindrical aperture in the casting.

Fig. 75.



If there is to be a recess in the casting, which cannot be produced in the formation of the mould, a core must also be used in this case, and if it be too long or heavy to be supported by the recesses produced by the core prints on the pattern, it may be supported by nails of suitable size, which will, of course, remain in the thickness of the casting; it is necessary to employ these nails to support the cores of long pipes.

If the casting be very complicated, it will be necessary to make it in such manner that it may be taken to pieces, otherwise it would be impossible to remove it from the sand without destroying the mould. The patterns should also be provided with slight rods, which being attached to them, may be tapped with a hammer, in order to detach them from the sides of the mould.

The pattern being completed, the next step will consist in the formation of the mould.

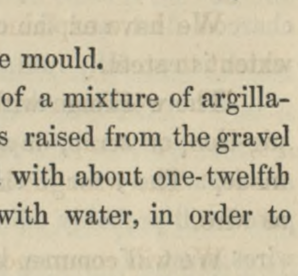
Green sand, the material of which the mould is to be constructed,—consists of a mixture of argillaceous sand, and pulverized coal; the former is employed in the state in which it is raised from the gravel pit, but it should first be sifted through a fine wire-sieve, and subsequently mixed with about one-twelfth of its volume of finely-pounded coal, and it should then be slightly moistened with water, in order to enable it to retain any form which may be impressed upon it.

When this sand has been once used, and consequently subjected to a high temperature, it is rendered unfit for the formation of fresh moulds, and should then only be used for filling up parts of other moulds which are below the level of the cavity intended for the reception of the melted metal. It is true, that by mixing this used sand with some fresh, and a considerable quantity of water, that moulds may be formed of it, but there is great danger of parts of the casting being rendered excessively hard by the unusual dampness of the sand, which would make the subsequent working of the metal very troublesome, and cause considerable loss of time and breakage of tools.

It must be borne in mind, that a casting should have no sharp, re-entering angles; for if these be allowed, the shrinkage of the metal is liable to produce cracks at such places, or, at all events, to render the casting weak.

Besides the pattern, the moulder uses a cast-iron box of peculiar form, which admits of being readily separated into two parts; it is shown in Fig. 76.

Fig. 76.



This box, technically called a flask, has neither top nor bottom, but it is so constructed that its two parts may be accurately fitted together. This is accomplished by forming one part *a* of the flask with projections, which are called lugs, having holes drilled through them, into which pins attached to lugs similarly placed on the other part *b* of the flask may be inserted.

When the dimensions of these flasks are considerable, they are also provided with cross-bars, intended to afford support to the sand of which the mould is composed.

The pins in the part *b* also afford means of firmly joining the two parts, which may be effected by forming eyes in the ends of the pins, through which keys are driven after the adjustment of the two parts to each other.

We now come to the production of the mould; to effect this, the part *a* of the flask is placed on a platform, in the position shown in Fig. 76, so that the cross-bars, if there be any, may be uppermost, the moulder then fills it with sand, which he forces down between the bars by aid of a rammer.

This having been done, the half-flask is inverted, the sand being retained in its place by its cohesion and its friction against the sides of the box and the cross-bars. The upper surface having been formed in contact with a smooth plane, will, itself, present a similar appearance. The workman now takes the pattern from which the casting is to be made, and having scooped a cavity corresponding roughly to its form in the sand, imbeds it to nearly half its thickness. The whole is now slightly sprinkled with charcoal-dust or fine sand. This may be done by shaking a bag of coarse canvass, containing one of these substances in a finely-divided state over the mould, the powder thus deposited will prevent the adhesion of the two parts of the mould during the next operation.

The part *b* is now lifted into its place, being retained in its proper position relatively to the part *a* by the pins which enter the lugs of the latter part. The upper frame is then filled with sand, which is well and carefully rammed down, after which the two parts are separated by cautiously lifting the upper one in a vertical direction. The lower part of the flask will now contain the pattern imbedded to half its thickness in the sand, which is generally found to fit tolerably closely all round its sides, but as perfect contact in every part is seldom obtained, the mould is consolidated by first slightly wetting it, and subsequently pressing the sand down around the pattern with a small trowel.

These manipulations having been completed, the support of sand first formed is now destroyed, leaving the part *a* of the flask empty. The surface of the sand in *b*, is now well dusted with sand or charcoal-dust, and the upper part of the flask is placed upon the lower, after which it is filled with sand, which is carefully rammed down as above described.

Before filling this frame, however, it will be necessary to insert the above-mentioned rods or wires into the pattern, by tapping which it may be detached from the mould, before the two parts of the flask are separated; for if this precaution be not attended to, a portion of the sand of which the mould is composed will probably adhere to the pattern, thereby rendering the impression ragged and unsightly; these wires must, of course, be long enough to project completely through the sand in the upper part of the flask.

The pattern having been thus detached from the sand, the wires are withdrawn, and the two parts separated, care being taken to perform this operation without inclining the flask, the upper part of the flask when removed, is placed with its moulded surface upwards, and the pattern having been removed from the lower part of the flask, any imperfections in the mould are repaired with moistened sand, applied by means of trowels of various shapes and dimensions.

The cores, if there be any, are now inserted, they should be made of loam, or some other tenacious sand, mixed with some filamentary material, and well dried over a suitable furnace; when large they may be supported on a bar of iron, with a covering of hay-bands wound round it.

If the two parts of the flasks be now joined together, they will evidently contain a cavity, corresponding exactly to the form of the casting required, but before the metal can be cast, it will be necessary to prepare apertures for that purpose; these are called gits, or gates, and the mould must be furnished with at least two of them, one to admit the liquid metal, and the other to allow the air and gases contained in the mould to escape, and if the casting be large, or its thickness small, a greater number will be required.

The sand may be rendered more porous by piercing it with a wire, producing thereby numerous apertures reaching to within about an inch of the cavity intended for the reception of the metal.

After preparing these gates, or openings, and carefully making good with his trowel any imperfections in the mould, the moulder finely dusts with sand or charcoal, by the method already described, the whole interior of the cavity, and smoothes it with a polished trowel.

The two parts of the flask may now be joined, after which the casting can be proceeded with.

For complicated forms, it is frequently necessary to join together a number of flasks, as well as making the pattern in several pieces, or using a large number of cores.

When the pieces to be cast are heavy, the flasks are generally moved by a crane, and if the patterns be large, they are withdrawn from the sand by the united efforts of several men, who lift it with one hand, whilst with the other they tap it, in order to detach any sand which might otherwise adhere to it.

The patterns are usually constructed in wood, but it is preferable, especially when many castings are to be made from one pattern, to use metal, whereby all danger of twisting, or warping by contact with the damp sand will be avoided.

For small patterns in wood a waterproof varnish may be used with advantage, and an economical black varnish of this character, may be prepared by dissolving a quantity of common shellac in methylated spirit by the assistance of moderate heat, and adding vegetable-black to colour it; coatings of this varnish should be applied until the pattern presents an uniform and somewhat bright surface, it will occupy but little time thus to varnish the patterns, as this material dries almost as quickly as it is laid on.

If the varnishing produce any roughness, this may be removed after the varnish is perfectly dry, by the use of fine sand-paper, after which another coating may be applied.

This process is very useful for thin, or otherwise delicate patterns, as by protecting them from moisture, it effectually prevents shrinking or warping.

The occurrence of well-defined internal angles may be prevented, by rounding-off the projecting angles of the mould, or, it may be provided for in the construction of the pattern, or the corners of the pattern may be filled up with cement.

We will now speak of moulding in used or baked sand. The mechanical processes employed in the preparation of moulds of baked sand are precisely similar to those adopted, when green sand is the material employed.

When, however, a great weight of metal is to be cast into sand of this description, it is usual to close the bottom of the flask loosely, by means of iron plates attached by clamps to the cross-bars. This is necessary, because, from the inferior tenacity of baked sand, the mould would otherwise be very liable to destruction, by the great pressure of the metal employed for the casting.

The plates used, admit of being readily removed at the close of the operations, and are perforated with numerous small holes to allow of the escape of the gases evolved. Baked sand is generally used without any admixture of coal-dust, and after the completion of the mould, it is at once removed to a kind of oven or drying-kiln, where it remains until the moisture is entirely removed, and the metal is poured into the mould, while the sand is yet hot from the kiln.

Castings produced in this way, are not so liable to imperfections and air-holes as those produced in ordinary green sand, for the greater porosity of material used for the mould allows easier egress to the air and gases generated; and, as the mould is kiln-dried before the introduction of the metal, there will be less chance of the formation of aqueous vapour; and also, the metal will cool more gradually; whereby it will obtain more uniform texture, and it will also be free from hard places, produced by the chilling effect of damp sand.

The third kind of moulding, viz., that in which the moulds are formed of loam, now demands our attention.

Moulds of this kind are prepared direct from drawings of the required castings, as the nature of the material employed in their construction, renders the use of patterns unnecessary.

The mould is formed of a mixture of clay, water, sand, cow-hair, which, after having been reduced to the state of a hard paste, and thoroughly kneaded together in a loam-mill, or pug-tub, is, by suitable manipulation, and the use of proper instruments, made to assume the form required. The proportions of the ingredients are varied, so that the loam may be made suitable to the purpose for which it is required; and when a very light quality of loam is desirable, a little chopped straw or horsedung is usually added. This method of casting is chiefly used for the production of large cylinders, sugar-pans and lead-pots in which the thickness of the metal is very inconsiderable, when compared to the other dimensions of the casting.

The workman has frequently, when moulding in loam, no other guide upon which to rely, and is, therefore dependent upon his own correctness of eye for the regulation of his tools.

When, however, the form of the required casting is bounded by surfaces of revolution, they may be produced by the revolution of scrapers suitably formed, and the work becomes thereby much simplified.

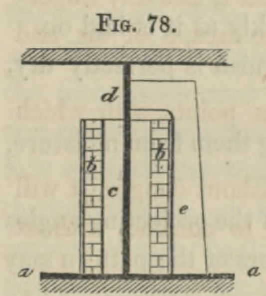
We will illustrate this method of moulding by an example. Let it be required to produce a casting having the form of a flanged cylinder, as shown at Fig. 77 in section, the black part representing the thickness of the metal.

Fig. 77.

The workman having been provided with drawings of the required cylinder, and loam properly mixed, causes suitable scrapers to be prepared for the formation of the external and internal surfaces, and proceeds as follows ;—

A flat cast-iron ring, *a a*, Fig. 78, is first laid on the floor of the casting-house. On this, the workman builds a hollow brick cylinder, *b b*, approaching to the form of the required casting, but smaller. On the surface of this brick cylinder, a thick coating of prepared loam is subsequently spread by a trowel. When this has been completed, its surface is again covered with another coating of loam an inch thick ; the loam employed on this latter case is of much finer quality, than that used to cover the brickwork, having been passed through a fine sieve before mixing.

Fig. 78.



In the centre of this mould is placed a bar *d*, resting in a recess in a piece of iron at the bottom and also retained in position by a similar bearing at the top, and this bar is so placed that it is capable of revolving in a truly vertical position·

To the bar *d*, is attached a board *e*, of such a form, that its edge which rests againsts the moulds, exactly represents the form of the internal surface of the required casting shown in section ; then it is evident, that if this board attached to the axis *d*, be caused to revolve around that axis, a surface will be produced upon the mould corresponding exactly to the internal surface of the article which is to be cast. By this means, then, the superfluous clay applied to the surface of the cylinder, is scraped off, and a mould of the interior of the vessel readily formed.

At this stage of the operation a quantity of smallcoal, which had previously been introduced into the cavity *c* in the brick cylinder, is ignited by means of openings left under the cast iron-ring *a a*, which serve the purpose of air-holes, and, if the top of the mould be closed, of chimneys for the exit of smoke also. The combustion of the coal must, however, be conducted very slowly, otherwise the loam will be very liable to crack, thereby rendering the mould useless.

The activity of the combustion is regulated by opening or closing the draught-holes left under the cast-iron ring, which, being formed in sand, may be very readily adjusted to admit exactly the right quantity of air to ensure a suitable activity of combustion.

As the surface of the mould begins to dry, it is thickly painted over with a mixture of charcoal-dust and clay, so mixed with water as to admit of its being readily applied with a brush. This is done to prevent the adhesion of the second layer of loam which is now to be applied, and corresponds therefore to the dusting of green sand or baked sand moulds with charcoal, or sand before filling the upper part of the flask. As soon as the surface has been properly coated with the above-mentioned mixture, a second layer of loam is applied over the whole surface of the mould, the thickness of which stratum is regulated by that of the article to be cast.

The board *e* is now replaced by one which in form corresponds to the external surface of the required cylinder, and by again causing the spindle *d* to revolve together with the attached scraper a second surface of revolution is obtained, and as by this process the excess of loam will be removed, a form exactly similar to that of the external surface of the article to be produced will be obtained. The vertical spindle, *d* and the scraper are now removed.

The combustion of the coal is still maintained within the mould, and as soon as the new surface begins to dry, it is, like the former, thickly painted over with a mixture of charcoal-dust, clay, and water, in order to prevent its adherence to the third and last coating of loam which is now about to be applied.

Another cast-iron ring is now laid down around the former, and adjusted to it by means of pin points. The surface of revolution obtained by means of the last scraper, is now covered with a stratum, about two inches thick, of fine loam laid on by hand, which, after being well kneaded together and smoothed over, is covered by another brickwork cylinder erected on the outer cast-iron ring.

After this has been completed, the fire is kept up until the entire mould has become perfectly dry, when chains are adapted to bolt-holes perforated in the second annular plate, which, together with the cylinder of masonry which it supports, is carefully lifted off.

It will be found that the outer stratum of loam has, on account of the coating of charcoal-dust and clay over which it was applied, separated readily from the second layer, which represents the thickness of metal in the casting, and as there is a similar coating between this layer and the first, it follows that these two may be separated without danger of injuring the latter.

The stratum representing the thickness of the metal is now carefully broken away, so as to expose the convex surface, corresponding to the interior of the required casting, and this surface, as also that which represents the external surface of the article to be produced is carefully examined, and any imperfections made good. Holes must be left in the brickwork in this case to allow of the escape of air and the evolved gases.

The outer cylinder is now replaced, its proper position being insured by the pin points, with which the two rings are adjusted to each other, when the metal may be cast.

As soon as the casting has become sufficiently solid to support its own weight without danger, it will be desirable to loosen or remove the internal brickwork in order to allow the casting to contract without restraint as it cools.

When very hard surfaces are required to iron castings, a process termed chilling is adopted, which consists of casting the metal in a mould formed partially or entirely, as the case may require, of iron, which by reason of the great power of conducting heat cools the molten metal with great rapidity, producing surfaces on the casting possessed of considerable hardness.

Having explained those processes in moulding which are most common, we will now describe the casting of metal properly termed founding.

We shall assume that the required articles are to be cast from pig-iron remelted, and not from the blast furnace itself, as the former most commonly occurs. The iron employed for casting by second fusion should contain a considerable amount of carbon, as from the circumstance of a portion of this being oxidised during each operation, the metal would otherwise become very difficult to fuse, and be also liable to obstruct the furnace in which the second fusion is conducted.

For melting cast-iron, a small blast furnace called a cupola is most generally employed. These cupolas vary in size according to the quantity of metal which they are intended to fuse at one time, but the same principles of construction are in all cases invariably observed.

The furnace consists of a wrought-iron casing internally lined with refractory argillaceous sand, and supplied with air by one or more nozzles or tuyeres connected with a rapidly-revolving fan. The external cases of these cupolas consist of stout sheet-iron rivetted together in a manner similar to that employed iu the construction of steam-boilers.

The annexed figures show a convenient form of this kind of furnace. The mass of brickwork *a*, is elevated about two feet from the floor of the casting-shop, and on the top of this is firmly bedded a strong cast-iron plate B, which serves for the foundation of the furnace. The cylinder C forming the body of the cupola is thickly lined on the inside with fireclay, and is surmounted by a hood D of thinner iron, which is connected with an external chimney to carry off the smoke, evolved from the apparatus when in action. The hole E is used for drawing off the fused metal into moulds prepared for its reception, and is provided with a small iron gutter for the purpose of guiding the stream of molten iron in a proper direction. The aperture in the hood is used for the introduction of coke and pig-iron, with which the furnace is at intervals supplied.

Fig. 79.

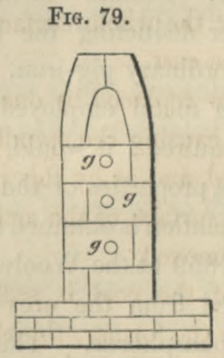
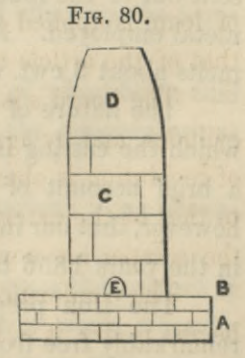


Fig. 80.



The three holes at g are for the reception of the tuyeres, by which the blast is supplied. The lining of the furnace is conducted in the following manner:—After having covered the bottom of the furnace to the depth of six or eight inches with a layer of argillaceous sand, the surface of which is slightly inclined towards the orifice of discharge, a wooden cylinder of the whole length of the cupola, and of a diameter a little less than the opening of the throat is set upright in the axis of the metallic casing. Fine sand containing a certain proportion of argillaceous matter, is now firmly rammed into the annular space existing between the wooden core and the metallic casing of the cupola, the core is now removed, and the sandy lining is cut away, so as to make the diameter of the furnace a little larger at the bottom than at the top.

When the fire is to be lighted, a little wood is placed in the bottom of the furnace, and ignited at the aperture E, which is left open for that purpose, and when the coke, which is subsequently added, has become sufficiently inflamed, the blast is gradually applied.

At this period the flame escapes both by the aperture at the top of the furnace and at the opening E, which, being lined afresh after each day's work, becomes much consolidated, by this means. At the expiration of about a quarter of an hour, the aperture E is closed by a plug of moist clay, which is applied by a wooden rod, provided with an iron disc at one of its extremities, upon which the lump of soft clay is stuck.

When the furness is required to contain a large quantity of melted metal the clay plug alone would not have sufficient strength to prevent its egress, wherefore in this case it is supported by an iron plate fastened against the aperture E.

When the furnace is lighted the apertures g are all kept open, the blast is first admitted through that opening which is nearest the ground, but as the melted metal accumulates at the bottom the lower holes are successively closed, and the blast is, towards the end of the operation, only admitted through that placed highest in the series.

Besides the holes shown in the back of the furnace, the same cupola has frequently two corresponding sets in the opposite sides, so that three distinct jets of air are often in the larger furnaces employed at the same time.

The lining of a furnace lasts from five to six weeks, during which about thirty meltings may be made.

When the castings to be made are large, they are usually sunk in the floor of the foundry near the cupola, and the molten metal is then conducted into them by suitable channels; but when they are small, the metal is drawn off into large ladles lined with a siliceous lute, and carried to the mould into which it is to be poured.

When a large casting is to be made at a distance from the cupola, larger ladles similarly lined are used, and they are moved about by cranes or other suitable machinery.

Every casting requires considerably more metal than is necessary to fill the mould; this goes to form the gates, false seams, etc., which are removed after the cooling of the iron, and before the castings are sent out of the foundry. Besides which, there is always an actual waste of about 6 per cent. of the whole metal employed. After deducting the losses, it is found that each cwt. of coke thrown into the furnace melts about 3 cwt. of ordinary pig-iron.

The nature of the metal employed will, of course, depend in a great measure upon the purpose for which the casting is required; it would, therefore, be useless to enter into this subject, but we will give a brief account of the properties of the irons made in various parts of the country, previously stating, however, that our information is obtained from the published report of the experiments on cast-iron conducted in the years 1856 to 1859 at the Woolwich Arsenal.

The iron prepared from the ores of Whitehaven, Weardale, and those of the Forest of Dean, are remarkably free from phosphorus. This is due to the very high quality of these ores, which are almost entirely free from phosphoric acid; the considerable percentage of silicon contained in a great number of the ores is not so easy of explanation, but may, perhaps, be due to the use of hot blast in the reduction of the iron.

The products of the ores of South Staffordshire and South Wales are, for the most part, of excellent chemical quality. Out of twenty-six samples examined, only three occur in which the silicon amounts to 2 per cent.

Out of thirteen samples from the Netherton and Old Hill Works, only seven cases occurred in which the silicon exceeded 1·15 per cent.

The proportion of phosphorus and sulphur in irons from the ores of these districts were also inconsiderable in the samples of iron which were examined, in only three out of twenty-six samples did the sulphur amount to 1-10th per cent., and the phosphorus amounted to less than 0·5 per cent. in twenty samples; of the remaining six samples three contained less than 0·6 per cent; one from the Old Hill Works contained 0·63 per cent., and two samples from the Brierly Hill Works contained 0·64 and 0·72 per cent. The ores employed at the latter works contained somewhat higher percentages of phosphoric acid, the effects of which on the composition of the iron produced are therefore clearly traced. The percentages of the phosphoric acid in the ores used at the Old Hill Works are also higher than in those used at the remaining South Staffordshire Works, and its influence on the quality of the product is shown by a reference to the composition of some of the iron from those works.

The oolitic ore and clay iron-stones used at the works in the North Midland and North Staffordshire districts, as also the ochrey-brown iron-stones used at the Northamptonshire Works contained proportions of phosphoric acid, which are more considerable than those existing in the greater number of the other ores examined. Thus the ore employed at the South Bank Furnaces contains nearly 2 per cent. of phosphoric acid; that used at the Stockton Works contains upwards of 1·5 per cent,, and very nearly the same amount exists in one of the ores in the Butterly Works, and in that employed at the Goldendale Works. The proportions of phosphorus in the various samples of iron from these sources exceed 1 per cent., except one sample from the Butterly Works, in which, however, it amounts to 0·72 per cent.

The samples of iron produced from the Northamptonshire ores also contain more than 1 per cent. of phosphorus, the proportions of phosphoric acid in the ores are, however, not quite so high as those in the last alluded to, amounting to 0·84 and 1·03 per cent. The samples from the East End Works are stated to have been produced by the cold blast.

The North Staffordshire ore used at the Goldendale Works contains nearly 1·5 per cent. of phosphoric acid, and the proportion of phosphorus in the iron produced from it just exceeds 1 per cent.

A general inspection of the results obtained from the examination of the ores and of the irons manufactured from them appears to justify the following conclusions:—

The proportion of silicon in the iron is much less influenced by the constitution of the ores than by the conditions of smelting under some circumstances, among which may be included a deficiency of alumina in the ore of flux employed, an ore containing much silica is very liable to give rise to a highly siliceous iron.

The proportion of sulphur existing even in light-grey pig-iron is never so considerable as to exert even appreciable influence on the properties of the metal, and none of the descriptions of British ores which have been examined contain an amount of sulphur sufficient to exercise any prejudicial effect on the irons produced from them.

The proportion of phosphorus in iron is in a great measure due to the percentage of phosphoric acid in the ore employed, while at the same time it is probably to some extent regulated by the temperature at which the iron is reduced.

A reference to our tables will show the mechanical properties belonging to the various irons above-mentioned.

Although these remarks bear more, perhaps, upon metallurgical chemistry than upon the subject in hand, yet it is desirable to know the characteristics of the various materials which we may work, whereby we shall be enabled to select such qualities as may be most suitable for our work, and it is on this account that we have inserted the above extract.

We must not consider differences in the strengths of various irons as altogether due to the variations of composition, but due allowance must be made for uniformity of texture, etc.; for it is evident that a sample of grey iron, uniform and compact in structure, may resist the application of a much greater tensile strain than one which is of a higher chemical quality (*i. e.*, contains less silicon phosphorus), but of which the structure is comparatively open and irregular.

We shall now for the present take leave of cast-iron, and pass on to the consideration of wrought-iron and steel; but we shall subsequently have occasion again to refer to the former, in order to explain certain methods of manipulation to be described hereafter. The principal difference in constitution between cast- and wrought-iron consists in the absence of appreciable quantities of carbon, silicon, phosphorus, and sulphur; and this difference entirely changes the physical properties of the material; and in wrought-iron we have a substance of tolerable purity, with resistances to tension and compression not differing very widely, ductile and malleable.

This material will necessarily require a more complete description than we have bestowed on the manufacture of cast-iron, as the processes employed are more varied.

We must commence our description of the manipulations to which wrought-iron is subjected, with its condition as it leaves the hammers, which compress and squeeze out the vitreous slag from the puddled balls.

The metal, after it is removed from the hammer, is compressed by rollers of two distinct kinds.

The first to the action of which it is subjected are called puddling-rolls, and they compress and weld together the balls of metal as soon as they leave the hammer, although in some localities, especially Wales, they are passed through the rollers directly they leave the furnace.

The second kind is used for the purpose of extending into bars the masses of puddled-iron after they have been cut into short lengths, and so welded in order to improve their quality; these are known by the name of rollers, and are differently grooved according to the pattern of the bars they are intended to produce.

These cylinders are fixed in pairs, in strong frames of cast-iron, and are connected by powerful wheel-work so as to turn in opposite directions.

The shafts of these cylinders are driven either by steam- or water-power, and the distance between the rollers admits of being readily adjusted by screws acting upon the bearings in which the journals of the rollers rest.

A narrow oblong trough is placed beneath the frame by which the rollers are supported, through which a stream of water is caused to flow, and into which the scale detached from the hot iron during the process of rolling falls. The sides of this pit are usually composed of blocks of stone resting on a solid mass of masonry, into which are built strong beams, to which the uprights supporting the rollers are firmly secured.

A small stream of water is occasionally conducted by a pipe to each pair of rollers, which are thereby kept cool and prevented from receiving any injury from the hot iron which passes between them.

The roughing rolls are usually five feet in the clear between the bearers, and eighteen inches in diameter; the trunnions are turned out of the same piece of metal, and are each about one foot in length. By means of these rolls the metal is drawn out into elliptical bars, after which the bars are flattened out, or reduced to any required form in the finishing rollers.

The first three or four grooves of the roughing rolls are also commonly provided with asperities resembling the teeth of a file, which take a firm hold of the mass of metal presented before them, and draw it through without danger of slipping, which would occur if smooth surfaces only were employed.

To support the balls and masses of metal which are to be passed through the rollers, a thick plate of cast-iron, notched on the edge so as to be closely applied to the cylinder, is employed, and it is situated on a level with the bottom of the notches on the lower cylinder. The plate of cast-iron is termed the apron, and is supported by iron bars stretched between two consecutive standards of the frame by which the standards are supported.

Puddled-iron which has been rolled into bars, immediately after its removal from under the hammer, is always of very inferior quality, being extremely hard and brittle, besides being subject to numerous flaws and defects not noticed in ordinary bar-iron.

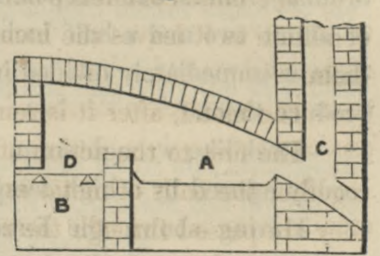
This hardness is, however, in some instances, not found disadvantageous, such as for the manufacture of railway bars, etc.

When the iron is required for purposes such as those of which we more particularly treat, it should be possessed of maximum tenacity and moderate rigidity, and its texture should also be as homogeneous as possible; and to obtain these important ends the rough bars are cut into short lengths, and after welding them together in bundles they are again drawn out into bars by passing them between cylinders, on the surfaces of which grooves of the proper forms and sizes are cut.

To effect this the bars of puddled-iron are by powerful shears cut into pieces about one foot long, and subsequently heated to the welding point in a furnace adapted to that purpose, called a mill furnace.

When the iron has been cut into short lengths, it is made into heaps or piles proportionate to the dimensions of the bars to be manufactured, and then heated in a reverberatory furnace, of which a section is shown at Fig. 81.

Fig. 81.



A represents the hearth of the furnace, and B the fire-grate, D is the door by which the fuel is introduced into the furnace, and there is another door under the chimney by which the bundles of bar-iron are introduced, and removed when sufficiently heated to be passed through the rollers.

It is of the greatest importance in furnaces of this description, that no air be allowed to come over the hearth unless it has previously lost all or the greater part of its oxygen, otherwise there will be considerable danger of oxidising a quantity of the iron which is being heated. It is therefore essential that the doors should remain closed during the whole of this operation.

When the piles of puddled bars placed on the hearth have acquired the temperature at which they are to be welded together, they are removed from the furnace and passed between the finishing rollers until they have been reduced to the required form and dimensions.

These rollers are turned with greater precision and more accurately adjusted than the roughing rollers. As it is necessary that the finished bars should have regularity of form and smoothness of surface, they are also caused to revolve with greater rapidity in order that the iron may be completely formed before it has had time to lose so much of its heat as to render it difficult or unsafe to roll it.

The actual velocity will of course depend upon the thickness of the required bars, for it is evident that the thinner the metal is, the sooner it will become cold. For this reason, where very small iron is manufactured, it is usual to employ a series of three rollers placed one above the other, so that after the passing, say between the top and middle rollers, it may be returned through middle and bottom rollers, and so forth.

For the manufacture of sheet-iron, two sets of rollers are also used, similar in arrangement to those employed for the production of bar-iron.

By the first set, the metal is roughed out into something approaching the required dimensions; and by the second, which only differs from the first by being more accurately constructed and adjusted, the sheets are finished off and given an even and polished surface.

The metal employed for making sheet-iron should be very soft and tough, and when very thin sheets are required, such as those from which tin plates are produced, no iron, except the best prepared by charcoal, should be used.

To give the metal the form of sheets, it is first drawn out into flattened bars of greater or less thickness, according to the dimensions of the required sheets or plates. These are subsequently cut by powerful shears into lengths corresponding to the width of the sheets to be prepared: this is usually done immediately after the metal has passed through the preparing rollers, and while it is still hot. The prepared masses are then introduced into a reverberatory furnace, where they are heated to redness, and when sufficiently hot, they are passed through the roughing rolls in such a manner that the length of the bars may be parallel to the axes of the cylinders between which they are compressed.

The plates are in this way acted on by the rollers two or three times, bringing them closer together between each operation in order to reduce the thickness of the plates at each passage between the cylinders.

The roughed plates are again heated in the reverberatory furnace, great care being taken to exclude atmospheric air which has not passed through the fire-grate. The plates are now passed through the finishing rollers, and are subsequently beaten with wooden mallets to remove the scale, of which a greater or less quantity always adheres to them.

When the iron bars have been rolled into very thin sheets, such as are employed for the manufacture of tin plate, the smoothing of the surface is effected by a distinct and separate operation. For this purpose, the thinned sheets are reheated in order to restore their softness, after which they are laid in heaps and powerfully compressed in an hydraulic press.

Very small square or rectangular iron bars, such as nail rods, are formed by rollers which are termed slitters. The ridges of these instead of being obtuse and exactly meeting each other, as in the case of ordinary rollers, are composed of sharp steel discs of an annular form, which enter into opposite grooves of about two and a-half inches in depth, so that any piece of sheet-iron which may be passed between them is immediately divided into a series of strips corresponding in number to the circular cutters which produce them.

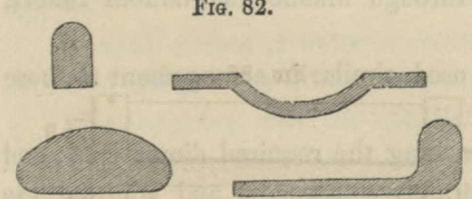
Steel, like wrought-iron, admits of being rolled into bars and plates, but on account of its being more readily injured by a high temperature it must be worked at a lower heat.

Having shown the forms into which wrought-iron is generally converted, we will now proceed to specify certain particular forms required for special purposes; observing, however, that as the method of procuring rolled forms is similar for all sections, we shall only notice those slight differences of detail which may appear essential.

We will first give an account of those forms which are generally required for bridge construction.

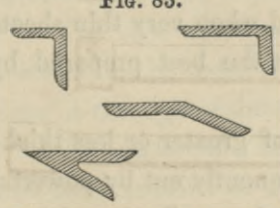
Bars of any section can of course be rolled if required, provided the section is not too large, but if they be not of sections commonly known in commerce, they will have to be purchased by contract; in the following pages we shall only describe those sections of iron which are usually manufactured. We will first speak of simple bars. These are manufactured in almost endless variety, the rollers being so arranged that their dimensions may in most cases be readily adjusted as may be required. The forms of these bars are circular, semi-circular, segmental, segmental-annular, square, rectangular, triangular, trapezoidal, sash-bar form, elliptical, semi-elliptical, etc. Also various modifications of these forms, as sections, which are double convex, plane convex, three plane and one convex or concave surface, also the sections shown in Fig. 82, etc., etc.

Fig. 82.



The square rods of iron and steel may be obtained of dimensions as small as one-quarter of an inch, increasing up to several inches. The round bars may also be obtained of similar dimensions. The width and thickness of rectangular bars vary still more, so that we can lay down no definite dimensions, but take it for granted that we can obtain any required size.

Fig. 83.



Angle-irons of sections shown at Fig. 83, are also manufactured in very great variety, the rolls by which they are produced being in most cases so constructed that the thickness of angle-iron having a constant breadth may be varied at pleasure; but we do not consider it advisable to employ angle-iron in bridge structures, whose thickness is less than about one-eighth of the least breadth, that is to say, the breadth of the narrowest web or side.

It is always desirable to have the angle-irons tolerably thin, in order to reduce the loss of material caused by rivet-holes.

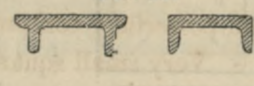
Angle-irons are mostly manufactured with a right angle, but they are also to be had made with an obtuse or an acute angle.

The next form which we notice is that called T-iron, which is shown in Fig. 84.

FIG. 84.

T-iron may be obtained with breadth and depth as small as three-quarters of an inch, and it is, like angle-iron, manufactured of various dimensions and proportions. This material is extremely useful; it may be employed in some instances in place of a pair of angle-irons, thereby saving the rivetting; it is also extremely suitable for the formation of bracing struts of Warren and other triangular girders, stiffening plates and standards, and it may occasionally be used in the place of cross girders, and, in fact, that form of girder known as deck-beam iron is only a species of T-iron.

FIG. 85.



Channel-iron, the form of which is shown in Fig. 85, is also useful in bridge construction. Like T-iron, it is a very suitable material for the construction of struts and standards for triangular girders, and it may also be used for bracing. This form of iron may also be used in the construction of flanges, especially those required for lattice girders.

Bars of this section can also be obtained very small, such as one and five-eighth inches broad on the back, and three-quarters of an inch deep.

It is evident that as far as the adjustment of the rollers is concerned, we can only vary the thickness of the back of either T- or channel-iron.

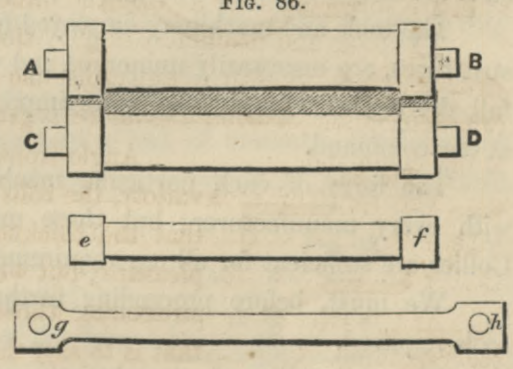
As it is very necessary when occupied in designing iron structures to know what forms and dimensions of iron are kept in stock, this is a matter which must not be neglected, but full particulars can always be obtained by referring to the sheets of sections of iron issued by the various manufacturers.

We must, however, call the attention of our readers to the wrought-iron rolled girders, which are sold by Messrs. Mather, Ledward and Co., of Liverpool; and this we consider it important to do, because it is not very long since the manufacture of *rolled girders* twelve inches deep was commenced.

By using this form of iron for the cross girders of bridges, where the strength is sufficient, greater economy will necessarily be obtained than by the employment of the ordinary rivetted-plate and angle-iron cross girders. We may here mention another special form of iron manufactured by Messrs. Howard and Ravenhill, of the King and Queen Ironworks, Rotherhithe.

It is necessary, in the formation of links for suspension chains, to make the heads of every link of such dimensions that they may be equally strong with the other parts of the bar, after the hole is bored for the insertion of the connecting-pin. This was formerly accomplished by forming the links of round bars, the ends of which were bent round so as to form eyes which were completed by welding, but such welded joints are by no means to be compared with eyes which are formed in the rolling of the bar, and which is now accomplished by a method patented by the above firm. The principle upon which this process depends may be thus briefly described.

FIG. 86.

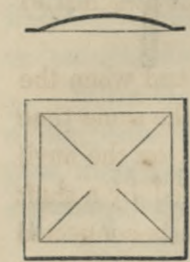


Let A B and C D, Fig. 86, represent two strong rollers, varying in diameter as shown; let a heated bar of iron, whose width is equal to that of the required link, be now passed between the cylinders in such a manner that its length is parallel to the axes of the rolls; then it is evident that the ends of the heated bar will be spread out, while the centre part remains unaltered; the bar is to be repeatedly passed between the rolls, which are brought nearer to each other before every passage, until the ends of the bar are reduced to the thickness of the heads of the required link, when the bar will present the appearance shown at *e f;* it may now be reheated and passed through an ordinary pair of bar rollers, in order to reduce the central part of the bar to its required thickness and to extend the link to the proper length; it will then appear as shown at *g h*, having the extremities of the same thickness as the central part, but of greater width. By this mode of construction we have the same sectional area of head of the link, after the hole for the connecting-pin has been bored, as exists at the central part of the bar.

Another very great improvement in bridge construction, consists in the invention of wrought-iron buckled roadway plates.

The buckled plate as commonly manufactured by the licensed makers, consist of square plates of wrought-iron, very thin, and slightly arched, with a very small rise, or curvature, from the springing at the edge of the plates in all directions to the centre, as shown at Fig. 87. Each plate is, therefore, a very thin and flat polygonal dome, or groined arch, the thrusts of an imposed load on which, in the direction of any two opposite sides, are borne by the tensile resistance of the outer portions of the adjacent sides of the plate.

Fig. 87.



The flat margin circumscribing the plate, is called the fillet.

The resistance of *square* buckled plates, varies directly as the thickness of the plate; and inversely, as the square of the line of bearing or distance between the supports.

A buckled plate bolted or rivetted down all round the sides, gives double the resistance of the same plate merely supported all round, and if two opposite sides be wholly unsupported, its resistance is reduced in the ratio of eight to five.

Within the limits of safe load, the resistance is nearly the same, whether the load be equally diffused or collected near the crown. The stiffness at any point of the plate as against unequal loading, varies as the square of the thickness of the plate, and inversely, as the curvature. The curvature (unless for some special purpose) should never exceed that which will just prevent the crippling load from bringing the plate down flat by compression of the material; less than two inches for the versed sine of the the curvature has been found sufficient for a buckled plate four feet square and one-quarter of an inch thick.

The following remarks will give some idea of the actual resistance of buckled plates :—

A three-foot square buckled plate of ordinary Staffordshire iron, one quarter of an inch thick, with a fillet two inches wide, and the total curvature amounting to one and three-quarter inches, supported only all round, requires nine tons weight diffused over about half the surface at the crown to produce failure, and double this, or eighteen tons, to cripple it if it be firmly bolted or rivetted down all round its edge.

A similiar plate of soft puddled-steel will bear nearly double the weight, or thirty-five tons per square yard.

The buckled plates used in the construction of New Westminster Bridge, each averaging seven feet by three feet, and being a quarter of an inch thick, were proved by lowering upon the crown of each a block of granite weighing seventeen tons, which did not produce injury: the plates have a curvature of three and a-half inches. In bridge-flooring, the plates should only be subjected to one-sixth of the crippling load.

We must now pass on to the description of the manipulations to which cast- and wrought-iron and steel are subjected after leaving the hands of the moulder, if cast, or after leaving the rolls, if wrought.

The tools and machinery employed for the various operations required in the manufacture of iron structures, are necessarily numerous and varied, hence we can only afford them a very brief notice; for a full description of even the more important machines, would occupy a far greater space than we have at our command.

The form of each particular machine, and especially of the more complicated, will probably vary with every manufacturer; but those manufactured by Messrs. Whitworth, Messrs. Muir, and Messrs. Collier are sufficient for all our requirements.

We must, before proceeding further, make a few remarks on the processes of forging and on the tools required.

The malleability of wrought-iron and steel will, as we have already seen, allow of their being pressed or drawn when hot into various different forms, this being effected when the masses of metal to be dealt with are large, by rollers or steam-hammers, which latter will now require a slight description.

Nasmyth's steam-hammer consists of the following parts.—The head, which is formed of a massive cast-iron block, to which is fitted a steeled wrought-iron facing, the weight of the entire head sometimes amounting to four or five tons. To lift this weight, it is attached to a rod which is connected with a piston working in a steam-cylinder, similar in construction to those used in the manufacture of ordinary steam-engines. The whole apparatus is supported by a strong framework of cast-iron, the sides of which are grooved, and act as guides to the head of the hammer. The anvil is placed between the uprights which support the cylinder, and is provided with a steeled wrought-iron facing similar to that attached to the head of the hammer.

The hammer is raised by the pressure of steam acting on the under side of the piston, and when the communication with the boiler is cut off, and the steam allowed to escape, the air being at the same time allowed access to the other end of the cylinder, the hammer falls with its whole weight on the anvil beneath. The steam in this arrangement is alternately admitted and cut off by a slide worked by a shaft attached to an eccentric; but when intermittent blows are to be applied, the slide, which is connected with a proper lever, is worked by hand.

These machines may be worked with a degree of accuracy perfectly astonishing to those who are not practically acquainted with such apparatus.

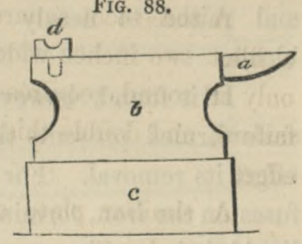
Condie's steam-hammer is similar to the above in principle, but the piston and piston-rod are fixed, whilst the cylinder to which the hammer-head is attached moves vertically. The steam is, of course, admitted through the piston-rod.

These steam-hammers are only required for very large forgings, the smaller ones being produced by hand- and sledge-hammers. The weight of the former may vary from about three to six pounds, while that of the latter may be fourteen, sixteen, or eighteen pounds.

During manipulations conducted with these hammers, the following tools are required:—

An anvil on which to lay the work, and which is shown at Fig. 88.

Fig. 88.



This consists of a heavy mass of iron *b* which is fixed on a block of wood *c* of suitable dimensions, the anvil being furnished with a hard steel facing. To one end of the anvil is attached a beak-iron *a*; it is of circular section, varying in diameter from the point to the base as shown; on this, various curved articles may be wrought.

The other end of the anvil is provided with a square hole, into which the tangs of certain tools, called swage tools, may be inserted, for purposes hereafter to be described.

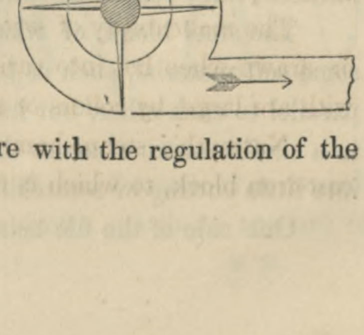
Tongs of various forms will also be necessary whereby to hold the work, and the jaws of these tongs must be of such a form as will conveniently retain any particular article to be operated upon; thus for round rods, the jaws must be so made that each of them is a sort of trough or hollow semi-cylinder; whereas, for wide flat bars, one jaw may be flat and wide, and the other rectangular and of moderate width. Different sized tongs will, of course, be requisite, varying to suit the dimensions of the materials to be worked, but these two forms may be accepted as types of the greater number of tongs.

The material to be operated upon being taken up in the tongs, it is retained there by passing a ring over the handles of that instrument.

A pair of slighter tongs, called smith's pliers, are also used for picking up small pieces of metal.

The metal to be wrought is heated in an open fire, supported on a bed of masonry, usually about three yards square, the fire is urged either by bellows, or by a revolving fan, a section of which is shown at Fig. 89.

Fig. 89.



When this fan is rotated, the air contained in the case is driven out in the direction of the arrow by centrifugal force, its place being supplied by other air which enters the case at apertures placed near the axis of the fan, and thus a constant draught is kept up. The fan must be caused to revolve at a very considerable speed.

The fire is usually covered by a hood at a little distance above it, placed near enough to carry off the smoke, but not sufficiently near to interfere with the regulation of the operations.

To obtain more regular surfaces than would be produced by the hammer alone, set hammers and swage tools are used. The former is a kind of hammer with a well-polished surface, which is laid upon the work, whilst the back of it is struck with a sledge-hammer, the set-hammer being moved between the blows.

For the production of surfaces not flat, swage tools are used; one of which, the bottom swage, is placed on the anvil, being retained there by its tang entering the square hole in the anvil; and the work being laid on this swage, the top swage is placed upon it and struck with a sledge, the work being moved between each blow.

The top swages are held by means of hazel-rods, to which they are fastened, whereby any jarring of the hand of the operator who holds them is avoided. These operations, when applied to small works, are usually conducted by two persons, the smith who holds the work in the tongs, and also the swages upon it, or if such are not being used, strikes it with a light hand-hammer, and who is responsible for the work; and the striker or hammer-man, who, when bellows are used, blows the fire and strikes with the sledge-hammer, as directed by the smith.

When a collar, or cylindrical projection, or increment of diameter is required around a rod, it is produced by heating the metal to redness up to a little beyond the point at which the collar is to be formed; after which the extremity is cooled in the water-trough, and the bar is struck endwise to swell out the metal, which is subsequently reduced to the desired form in suitable swages.

This process is termed upsetting the metal, and one similar to it is used for the formation of cranked pieces, etc.

In order to weld two pieces of metal together, they are slightly tapered off at their extremities, and raised to nearly a white heat in the fire, after which they are shut together by repeated blows.

It is found, however, that by exposure to the air the surfaces to be welded become oxidised, and the scale formed prevents their being properly united, wherefore it is necessary to use some kind of flux to effect its removal. For this purpose, a mixture of white sand and salt may conveniently be employed; it fuses on the iron, forming a slag with the oxide, and protecting the metal from further oxidation, but being liquid it is readily squeezed out when the joint is being made. Cast-steel and wrought-iron can scarcely be united by welding, but shear-steel may be thus united to iron. As, however, the steel will not bear so high a temperature as the iron, the welding together of these materials is thus conducted:—

The pieces to be united are heated in the fire, being plentifully supplied with the flux until the temperature of the iron slightly exceeds the welding heat, whilst that of the steel is not quite equal to it; then, when the two pieces are placed together, the steel acquires what temperature may be necessary by contact with the iron, and the two pieces may be shut together in the ordinary manner. By adopting this method, the danger of burning the steel is avoided.

A process for welding plates was invented and patented by Mr. Bertram, of Woolwich, but it has not come into general use; though there can be but little doubt that if plates could be conveniently shut together in this manner, it would be far superior to the joint formed by rivetting; for, by this latter method, with a single rivetted joint the strength of the joint is only about 0·56 of that of the plate itself, whereas a good welded joint should be of equal strength with the remainder of the work.

We will now take leave of the forge, and pass on to the other varieties of manipulations. We must commence with the cutting-tools used for metal, first considering the construction of files. These tools, which are too well-known to require an illustration, are manufactured of various forms and degrees of fineness; they consist essentially of numerous small cutting-points thus produced:—

The steel blank, of which the file is to be made, having been forged and rendered bright, is laid on some soft material, such as lead, and a number of cuts made over its surface with a chisel, these cuts being parallel to each other, but inclined to the sides of the file; when they are completed, other cuts are made in a similar manner across the former, so that the ridges formed by the first set of chisel-marks are divided into little cutting or scratching-points by the second.

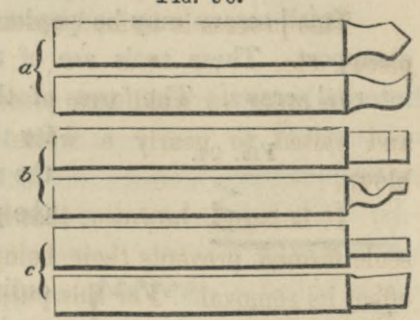
One side of the file being completed, it is turned over and teeth are similarly formed on the other side. Both sides of the file being completed, it is hardened and tempered by a process previously described.

When the first series of cuts only is made, the instrument is called a float, and when the surface is only provided with a number of isolated points raised by a narrow chisel, or by a punch, it is called a rasp. Files are sometimes made with one edge or side uncut, which is called a safe edge, as it may be rested against the works without danger of injuring it.

The files in common use are included under square, triangular or three-square, circular segmental, called semi-circular, the same, but convex on both sides, called crossing files; flat files both taper and parallel in their widths. These files are made of various degrees of fineness, included between rough and dead smooth, intermediate between which, are second-cut, bastard and smooth files. There are many other forms, but they are not required for our purposes. Fine saws are also used for metal, and their teeth may first be formed by punching, and subsequently finished with the file, after which they may be tempered. We will now describe what are commonly known as cutting-tools, being used for paring or cutting the metal either by hand, or with a planing, turning, or shaping engine.

We may divide these tools into two classes, machine-tools and hand-tools. The class of hand-tools includes cold chisels, scrapers, etc. The machine-tools are usually of the forms shown in Fig 90; *a* represents what is called a point-tool, it is used for the production of surfaces required to be accurate; in connection with the machines above mentioned, this tool may be considered to produce a number of parallel grooves placed close to each other when used in the planing or shaping machine, and one continuous spiral groove when used with the turning-lathe. *b*, is a spring-tool, having a broad, slightly-curved edge, it produces a finer surface than the point-tool, but is not quite so accurate: work may be reduced to nearly the required size with the point-tool, and then where absolute accuracy is not requisite, it may be finished-off with the spring-tool. *c*, represents a narrow-pointed tool with a square edge, it is used for dividing pieces of metal in the machines and is called a parting-tool.

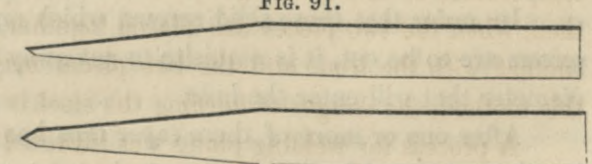
Fig. 90.



A similar form of tool is used for producing the threads of large screws. It is very important that the edges of these tools be properly ground, as the accuracy of the work necessarily depends upon them; and, moreover, if the cutting angles be not properly defined, a rough and unsightly surface will be produced.

Hand-chisels are made of the forms shown in Fig. 91. The surfaces which by meeting, form the cutting edges of the various tools should for the present purposes be so ground as to make with each other angles varying from 75° to 90°, the most obtuse being those used as scrapers.

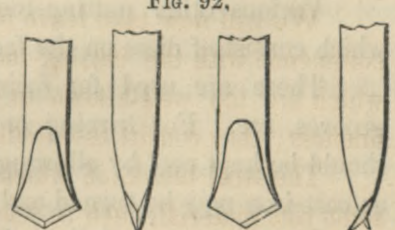
Fig. 91.



We will next speak of tools used for the purposes of drilling and boring.

Drills are of two sorts, viz., those which are worked with a rotatory reciprocating motion, and those which are worked by a continuous rotating movement; the former are of the shape shown at *a*, Fig. 92, and the latter are as shown at *b*, the difference being, that the former being ground on both sides scrape, but scrape equally well in both directions, whereas the latter are formed to cut in one direction only.

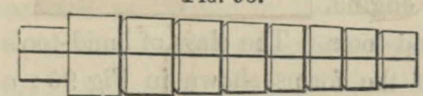
Fig. 92.



Drills are also made of various other forms for other purposes, such as flattening the bottoms of holes for forming conical or countersunk holes, etc. Holes or cylinders may also be bored out by means of short cutters attached to boring-bars passed through the axes of the cylinders, and supported on suitable bearings, the cutter being caused to move along the cylinder as the work of boring is progressed.

Holes may also be produced by the use of punches, thus conical recesses are formed by a punch with a conical point driven by blows from a hammer; the punch is called a centre punch on account of its being used to mark the centres in the ends of cylinders to be turned, or to indicate the centres of holes to be drilled. For punching cylindrical holes, a hard cylindrical punch is used, generally driven by a powerful machine; and it is by this method that the rivet-holes in plates intended for the construction of bridges, tanks, boilers, &c., are produced. It is sometimes necessary after holes have been punched or drilled, to render them more accurate by means of broachers or rhymers. These broachers consist of slightly-tapered pieces of metal, usually of a square or polygonal section; then, by causing these to revolve within the hole which it is required to broach out, the various irregularities and protuberances are scraped off and an uniform appearance is imparted to the whole of the surface. Rhymers may be also used when two holes in different plates, which are required to correspond with each other, do not exactly do so, but approximate tolerably near. A rhymer being passed through the two holes is caused to revolve until the sizes and positions of the two holes correspond. For clearing out square holes, tools called drifts are employed. These tools are made of hard steel, and in form are slightly tapered, as shown in Fig. 93: they are inserted into the aperture to be cleared, and driven through with blows of a hammer. We next come to the tools used for the production of screws and nuts.

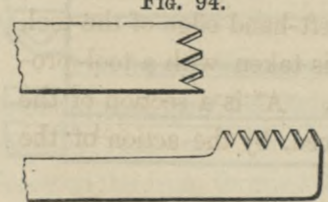
Fig. 93.



Screws may be best formed in a lathe, if large the thread may be cut by some modification of the point or parting-tool, the cutting-tool be caused to move along the work as the latter revolves at a proportionate speed.

Small screws may be produced in the lathe by means of screw-tools held in the hand, but rested on a support. These tools are of two sorts, those for producing external screws, and those for forming internal screws. The forms of these tools are shown at Fig. 94, the upper sketch representing the form for the external, and the lower that for the internal screw. The obliquity of the threads following the cutting edges, carries the tool along the work at the proper speed to produce a screw of the required pitch.

Fig. 94.



External screws may also be cut by means of dies, which are similar to ordinary nuts in general form; but they are of hardened steel, and usually made in two or three segments, in order that the divisions may give rise to cutting edges, and also that the dies may be gradually closed upon the cylinder, these dies are fitted into stocks provided with long handles, by which they are handed round when cutting the screw.

The nuts are produced by hard steel screws, the cutting edges to which are formed by planing or filing out grooves down the sides of the screw, thereby removing portions of the thread.

In order that these solid screws, which are called taps, may readily enter the holes within which the screws are to be cut, it is requisite to cut away a portion of the thread so as to taper the top down to a diameter that will enter the hole.

After one or more of these taper taps has been passed through the nut, which is effected by wrenching the tap round by means of a stock or tap wrench into a socket in which the head of the top fits, it is necessary to make it true and parallel throughout, which must be accomplished by passing a parallel or plug tap through the hole. A number of taps are usually kept for the formation of screw-tools, etc., these are called master-taps, and the others are distinguished as working-taps.

Various other cutting-tools not mentioned above are also frequently used, such as rotatory cutters, which consist of discs on the face or periphery of which cutting edges are formed.

These are used for facing-up hexagonal and octagonal nuts, and also occasionally for cutting grooves, etc. For turning or planing tough fibrous materials such as wrought-iron, the cutting-tool should be kept cool by allowing a solution of soft soap to drip continually upon it; but materials such as cast-iron may be turned and planed dry, as there will not be so much friction in this case.

In using taps and dies, oil may be employed to prevent them from becoming heated by the friction.

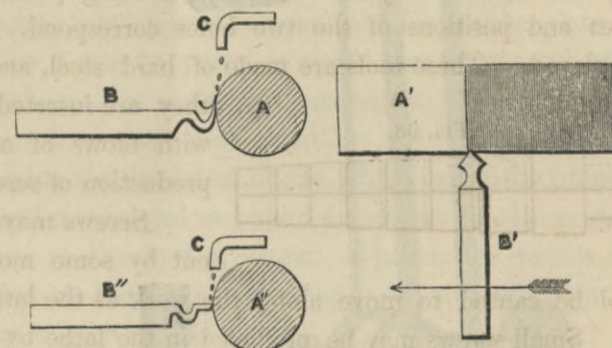
In drilling wrought-iron, the soap solution may be used with advantage.

Before cast-iron is turned, planed, or otherwise wrought upon by edged-tools, a portion of the siliceous coat adhering to it should be rubbed off by a piece of hard oven-coke. The cutting-tool should also take its cut below the hard skin, as otherwise the keen edge will be very rapidly vitiated and destroyed.

We will now illustrate and describe such of the various machines employed by the mechanical engineer, as are most generally required for the completion of the various parts of iron bridges.

Let us first examine the construction of the turning-lathe. A cylindrical surface whether convex or concave, is produced by causing the materials from which the cylinder is to be manufactured to revolve on a fixed axis, during which time a cutting-tool, of one of the foregoing forms, is held at a certain distance from the axis of the cylinder in such a manner as to remove all the material which extends from a greater distance to the axis, and this is effected along the entire length of the work, by causing the tool to progress gradually in a direction parallel to the axis of the required cylinder. If the tool be moved in a direction inclined to the axis, a cone, or the frustrum of a cone will be produced. The action of the cutting-tool is shown in Fig. 95, A is a section of cylinder which is being turned, B is the cutting-tool which is paring off the material, C is a tube or spout by which a cooling liquid may, when such is necessary, be supplied to the cutting edge of the tool. A′ is a plan of the same cylinder partly turned, the reduced portion being represented by the shaded part, the shading lines may be considered as representing the minute and close grooves which constitute the surface of the work. The tool is caused to progress slowly in the direction of the arrow, as the work revolves. An inspection of the sketch shows, that the left-hand edge of the tool, and its point, are the only parts which are cutting, and frequently the first cut is taken with a tool provided with these parts only, a large quantity of material being at once removed. A″ is a section of the same cylinder, on which a brighter and more polished surface is being produced, by the action of the spring tool B″.

Fig. 95.



This tool by reason of its elasticity, yields to any hard point in the work, instead of tearing it out after the manner of the point-tool, and as the tool has a very broad point, it obliterates the grooves which remain after the action of the point-tool. It is necessary, however, to round off the corners of the spring tool, in order that they may not penetrate too deep into the work.

The tools for light-work may be held by hand, being supported on a rest; but for heavy-work, and all such as is required to be exceedingly accurate, the tools must be firmly fixed in a rest attached to and moved by the machinery connected with the turning-lathe, and it is also to be remarked that the spring-tool should not be used for work which is required to be extremely true in its contour.

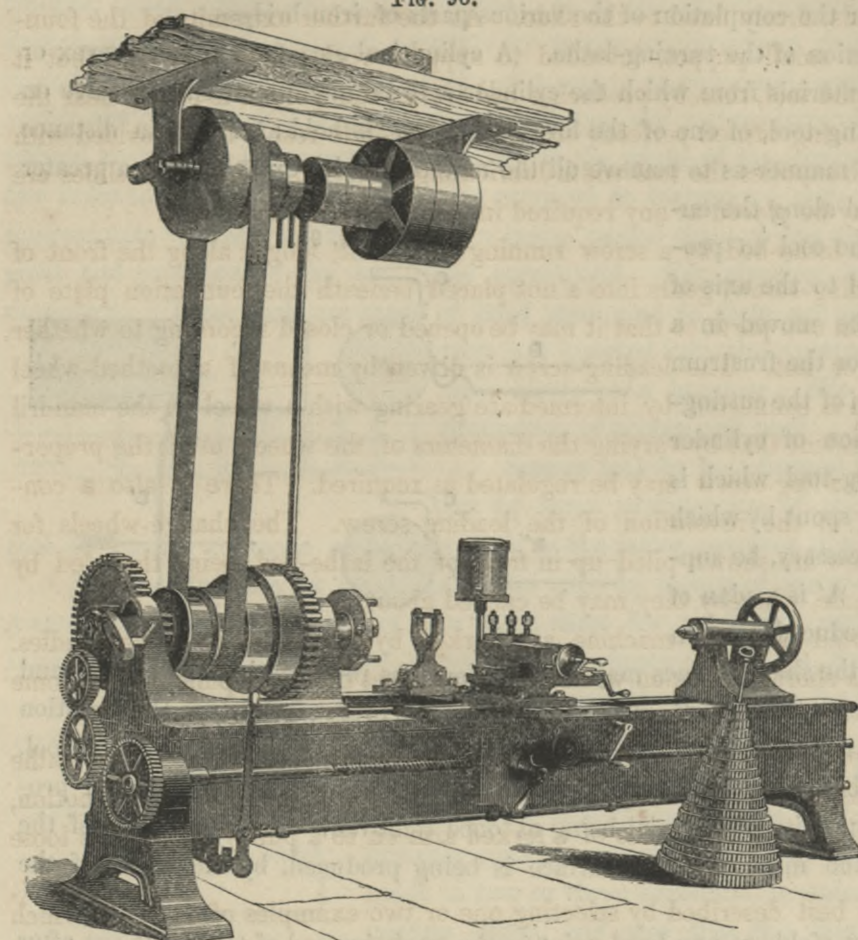
Turning-lathes may be driven by a treddle worked by the foot, or by hand or steam power, ,but the latter should always be preferred for this and other similar purposes, when it is to be had; we shall, therefore, only describe this form of lathe, observing, however, that the details of the lathe itself vary but little for different methods of driving, and in many cases they are exactly similar.

The accompanying figure (Fig. 96) represents a power-lathe, as constructed by Messrs. Muir and Co., of Strangeways, Manchester. The bed of the lathe consists of a strong cast-iron frame, as shown, the top of which is accurately reduced to a true plane by first planing, and subsequently scraping the surface, until it is true, which is ascertained by the planometer, which we shall presently describe.

At the left-hand extremity of the lathe-bed is fixed a strong frame carrying a cylindrical rod called a mandril; this mandril revolves in hard conical bearings, and the iron of it, that part looking towards the lathe, is fitted with a screwed nose, upon which chucks are fixed to support the work or ensure its revolution; there is also a recess to receive a conical-pointed centre, which supports one end of the work by entering a countersunk recess in that end.

Upon the mandril are placed various speed-pulleys, connected by a strap with similar ones attached to a short shaft supported by brackets or hangers fastened to the wall or roof of the factory.

Fig. 96.



There is also another shaft placed beside the mandril for the purpose of obtaining a greater variation of speed than the pulleys alone would afford, upon this shaft are two toothed-wheels, one large and one small, and they are thus used. This short shaft is slid endwise in its bearings, so that the toothed-wheels upon it gear with those upon the mandril; the speed-pulleys are then, by loosening a screw, so adjusted that they can revolve upon the mandril independent of it; these being set in motion, carry the small-tooth wheel attached to their smallest extremity round with them, which, gearing with the large wheel of the short shaft, causes the latter to revolve, while the small wheel at its further extremity, gearing with the large wheel attached to the mandril, drives the latter. Some small lathes are not provided with this arrangement.

By using this side shaft we obtain a much slower motion than by driving direct from the speed-pulleys, but whichever way we work, the latter affords the means of adjusting, within certain limits, the speed of the mandril, by shifting the driving-band to any pulley or rigger which may be most convenient for our purpose. The mandrils, together with the shaft at its side and the frame supporting it, etc., constitute the front poppet-head of the lathe; it is carefully fitted to the lathe-bed, so that the mandril is exactly parallel to the latter, and it is firmly fastened down to the bed by a clamp adjusted with a screw-bolt.

The short shaft beside the mandril may be retained either in or out of gear by a pin, the end of which falls into a groove in the journal or bearing of the shaft, or in some cases the bearing itself is fixed in an eccentric by the revolution of which its distance from the wheels on the mandril is regulated.

At the other end of the lathe-bed is seen the back poppet-head, the frame of which was accurately fitted to the lathe-bed in such a manner that it may slide longitudinally thereupon without losing its parallelism to the same. This poppet-head is retained in any desired part of the bed by a clamp drawn up tight by a screw-bolt. At the top of this poppet-frame is a cylinder within which is a mandril which admits of adjustment endwise by means of a screw working within it, and revolved by the hand-wheel at the back of the head; the mandril is prevented from revolving with the screw by a small stop or key which works in a groove or shot in the bottom of the mandril. In one end of the mandril is inserted a conical-pointed centre. The centre is retained at any distance from the poppet-frame by a small set-screw, which may be screwed down tight upon the mandril.

In the centre of the lathe-bed is the apparatus by which the tool is carried, it is called the slide-rest, and consists essentially of the following parts: a bottom piece accurately got-up to slide truly upon the lathe-bed, and to fit it exactly, so that all shakiness is avoided; another piece placed upon and bolted to this by bolts whose heads are retained in undercut grooves, the top of this slide is accurately planed and then scraped to make it true, it is also fitted with a screw running its whole length and placed parallel to the lathe-bed, this screw works a smaller slide which is placed above it, upon which is the tool-holder, also capable of being adjusted by a screw, but this one is placed at right angles to the lathe-bed, being intended to regulate the depth of the cut. The tool is fastened under a plate which is screwed down upon it by means of nuts working on bolts or studs, firmly fixed to the slide. At the further extremity of the foundation piece of the slide-rest is seen a small poppet, intended to support work which is so long that it vibrates or bends under the action of the tool, thereby producing bad work; this poppet is placed near the tool and retains the cylinder which is being turned perfectly straight. The slide-rest is also provided with a light standard to support a can which supplies the tool width lubricating material. The upper slides are frequently so constructed that they may be placed at any required inclination to the lathe-bed.

The slide-rest is moved along the lathe-bed by a screw running its whole length along the front of the same. This screw, called the leading-screw, gears into a nut placed beneath the foundation plate of the slide-rest, and which nut is made in two parts, so that it may be opened or closed according to whether the slide is to be moved by the screw or not. The leading-screw is driven by means of a toothed-wheel fixed to its extremity, as shown, which is connected by intermediate gearing with a wheel on the mandril of the front poppet-head; then it is evident that by varying the diameters of the wheels used, the proportion of the speed of the mandril and leading-screw may be regulated as required. There is also a contrivance for reversing the direction of the revolution of the leading-screw. The change-wheels for adjusting the speed of the leading-screw are shown piled up in front of the lathe-bed, being threaded by a rod furnished at the top with a handle, whereby they may be carried about.

The various hand-screws connected with the machine, are worked by means of moveable handles.

The overhead motion consists of a short shaft upon which are fixed a set of speed-pulleys with some fast and loose pulleys.

The shaft is driven by a band coming from one of the main shafts of the factory; when the lathe is to be put in motion this band is retained upon a pulley fixed firmly to the shaft of the overhead motion, but when the contrary is desired, it is moved by means of a forked arm on to a pulley which runs loose upon the shaft.

The use of this machine may be best described by selecting one or two examples of the work which may be done by it.

The lathe illustrated, is intended for turning plain cylinders and screws; also it may be used for facing the ends of cylinders, flanges, etc., but if a plate is to be faced of very large diameter, a lathe should be used, having a discontinuous lathe-bed, so that there may be room for the periphery below the mandril.

The figure represents what is termed a self-acting power-lathe, with 12-inch centres, and a 12-foot bed, so that any article whose radius does not exceed 12 inches may be turned in it.

Let us suppose that a plain cylinder is to be produced. Material nearly approaching the required form is selected, and the centres of the extremities being found, conical recesses or countersunk holes are bored, into which the lathe centres may fit. A ring of iron, of such size that it will press over the end of the bar, is now selected, having on one side a projecting arm, this is passed on to the end of the rough cylinder, and firmly fixed there by means of a set-screw. This end of the work is now placed against the front head of the lathe, so that the conical centre enters it; then the back poppet having been brought up to the further extremity of the work, and there secured; the mandril contained in it, together with the centre, is worked forward by the screw until the work is firmly retained between the two centres.

A suitable tool is now fixed in the slide-rest, which has previously been brought close up to the back poppet, and the lathe is started. One of the bars projecting from the small chuck, attached to the nose of the mandril, coming in contact with the arm projecting from the ring, which is fastened to the work, causes the latter to revolve. This ring is called a carrier. The workman now throws the leading-screw into gear with the slide-rest, and taking the handles of the two adjusting screws of the slide-rest, regulates the depth of the cut; this being done, the lathe need not be touched until one cut has been taken along the whole length of the cylinder, or of that part of it which is to be turned, the slide-rest is then thrown out of gear with the leading-screw, and brought back to its former position, when another cut may be taken, either by the same or by another tool, as the case may require, and these operations are repeated until the work is complete.

N N

To face a plate, it is attached to a large flat chuck, pierced with holes, and called a face-plate, by means of clamps and screw-bolts.

The slide-rest is then brought close up to the front head of the lathe, the tool being fixed, the tool-holder is worked across the face of the plate by the proper screw. The ends of cylinders may be faced by a side-tool.

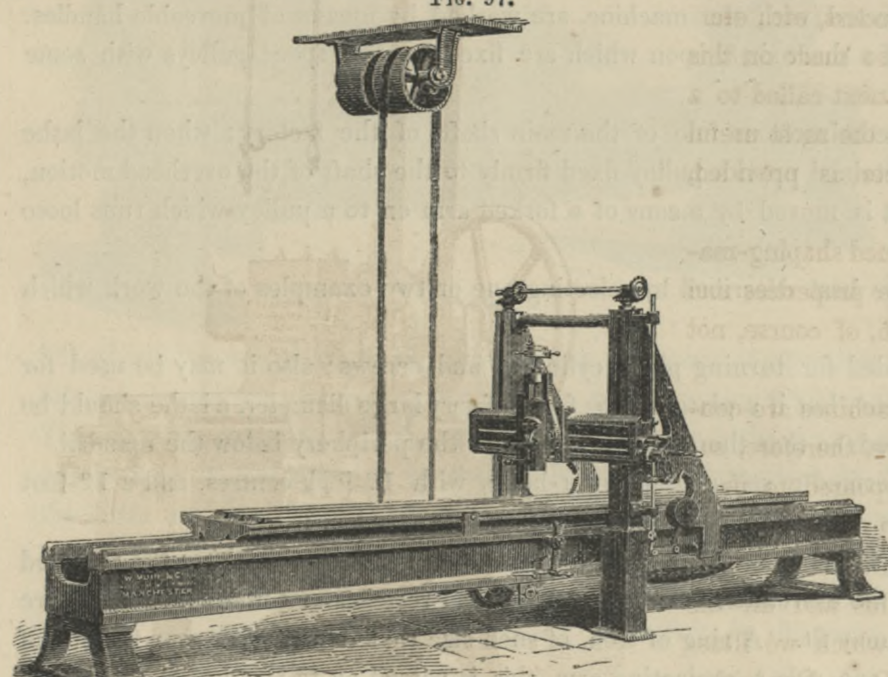
The interiors of short cylinders may be bored by a side-tool, but those of larger ones are bored in the following manner. The cylinder to be bored is attached to the lathe-bed, and a boring-bar is placed between the lathe centres. Upon this bar is a boring-head, which carries the requisite cutters, and which is traversed along the bar by a screw let into a slot running the entire length of the boring-bar. Of the other chucks employed besides those already described, the most important are the cup-chuck and the dog-chuck.

The former consists of an iron cup furnished with set-screws passing through its periphery, by which work is held and adjusted.

This chuck may be used for short work without the assistance of the back centre.

The dog-chuck is a species of face-plate furnished with L-shaped pieces of iron called dogs, whose distances from the centre of the chuck are regulated by screws working in slots in the latter, and gearing into nuts attached to the backs of the dogs. Solid work may be held in this chuck by screwing the dogs down upon it, and hollow work may be retained whilst its exterior is being turned by screwing the dogs up against its internal periphery.

Fig. 97.



Let us now pass on to the description of a form of planing-machine which, like the foregoing lathe, is constructed by Messrs. Muir. This machine as will be seen on inspecting Fig. 97, is not nearly so complicated as the turning-lathe.

This apparatus is provided with a strong bed, having on the two edges V-grooves planed and subsequently reduced to an accurate form by scraping. Upon this bed is fitted a sliding table, having on its under surface two V-pieces which are constructed to fit accurately into the groove of the bed. To this table is communicated a reciprocating rectilinear motion by means of suitable rack and wheel gearing placed beneath it, the motion being reversed by stops fixed to the sides of the table which are brought in contact with machinery, whereby the driving-strap is shifted. Along the length of the table undercut grooves are formed, into which the heads of bolts may be slid by which clamps may be screwed down tight upon the work to be planed.

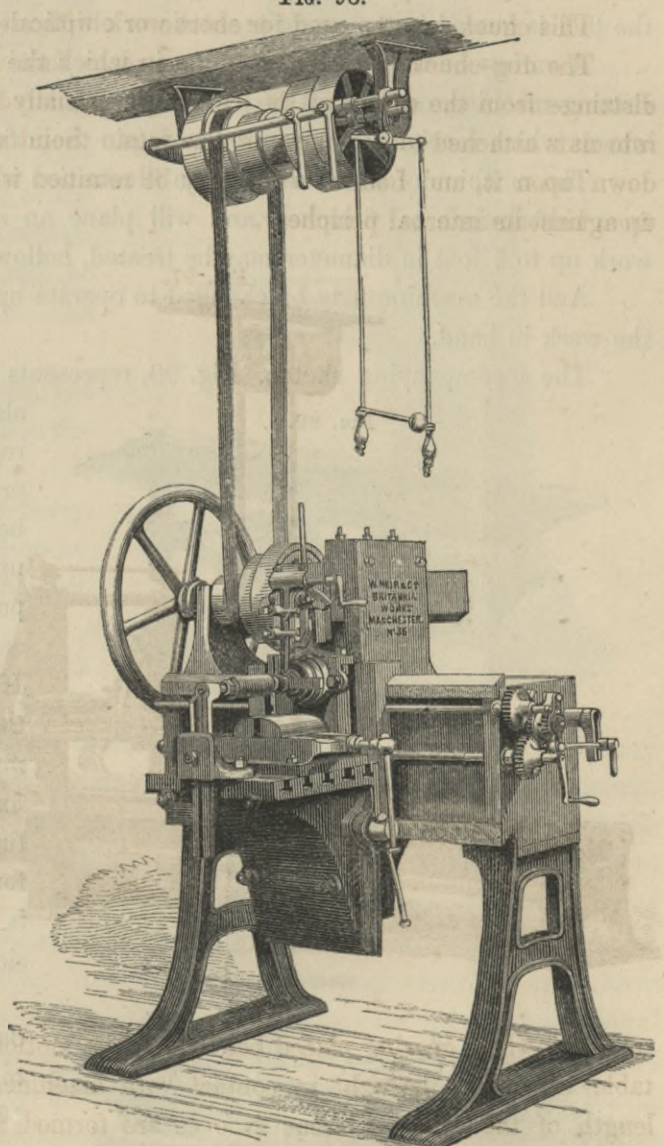
On each side of the bed is placed, as shown, a strong upright: these uprights carry a transverse slide, which may be raised or lowered according to the thickness of the work by means of two screws worked by the bevelled or mitre gearing on the top of the frames. To this transverse slide is accurately fitted a smaller slide, which may be moved along the former by hand or by suitable apparatus attached to the machine, as shown. The small slide supports another which can be adjusted by means of set-screws at any angle to it; in the woodcut it is at right angles, the tool-holder carried by this slide may be moved along it through a small space by a screw which may either be worked by hand or caused to self-act. At every stroke the tool is advanced through a very small space in a direction parallel to the width of the surface being planed. The same tools may be used in this machine as are employed with the lathe. It will be observed that the tool in the planing-machine only acts while the bed is moving in one direction, being allowed a movement on a hinge to clear it of the work at the return stroke; hence all the time occupied by the return strokes is lost. In order to reduce the inconvenience of this loss of time as much as possible, the planing-machines are made so that the table returns after the cut at a much more considerable velocity than that at which it moves when taking a cut : this is very readily accomplished, as the driving-strap is after each stroke shifted on to a different set of gearing.

A planing-machine has been invented in which the tool cuts in both directions; but it has not come into general use.

This might also be accomplished by using two slides, one on each side of the transverse slide, each carry a separate tool, and both being adjusted to correspond with each other.

The machine illustrated, is called a self-acting planing-machine, for planing horizontal, vertical, and angular surfaces, 12 feet long, 2 feet 6 inches wide, and 2 feet 6 inches in height, with quick return motion.

Fig. 98.



Some planing-machines are fitted with a separate overhead motion, adapted to give a very slow motion, or feed to the bed of the machine, which being also furnished with suitable heads and other gearing, may be used for a great variety of purposes, such as grooving rollers, boring cylinders, etc., etc.

Small planing-machines are also made on this and other plans. Our attention is next called to a class of machines which are perhaps the most useful with which the mechanical engineer is provided, omitting however the turning-lathe.

These apparatus which are termed shaping-machines, include within themselves the properties and capabilities of many others, although, of course, not to so great an extent.

The plans upon which these machines are constructed are very numerous, we have, therefore, selected one for illustration, which appeared to us to contain all the requisites of such an apparatus. It is from plans furnished by the same firm as above, and we may here mention, that this and all the others by the same manufacturer, which we illustrate, procured a prize-medal at the Paris Exposition of 1855, and a medal was also awarded to the makers of this machine, by the Society of Arts in the same year.

An inspection of the woodcut shows that this machine is based upon a strong bed, supported by cast-iron A frames. To one side of the bed is attached a slide, capable of receiving a horizontal motion, either by hand, or by self-acting gear, and to this slide, which is vertical, is attached a table or chuck, as shown, supported by a bracket, and of which the vertical height may be adjusted to suit the thickness of the work operated upon; this table, like that of the planing-machine, is formed with undercut slots.

An apparatus shown in the sketch, may also be fixed to this table, to support round work whilst it is being turned, slotted, or otherwise operated upon. A pair of jaws for holding rectangular work is also shown.

We now come to the fittings fixed upon the top of the bed of the machine; the most important is a large strong standard, or head, which contains a stout sliding-bar accurately fitted to it, the end of which carries the tool-holder, with its slides and screws for adjusting, feeding, etc., the slide is driven by an eccentric, or crank fixed upon a shaft, which revolves in bearings formed in suitable supports.

This shaft is provided with speed-pulleys, a fly-wheel to regulate the speed, and prevent shocks, and also with means of working the self-acting feed gear. This machine is supplied with an overhead motion, for the purpose of connecting it with the prime mover, and in this illustration is shown a very ingenious contrivance to prevent the machine from being started or stopped by the jarring going on around; its mode of action is evident, as when either handle is pulled down, the heavy ball slides along the rod connecting the handles, and maintains the position thus given to it.

The action of this and other similar machines is exceedingly excellent, and constitutes a beautiful example of the effects which may be obtained by mechanical contrivances.

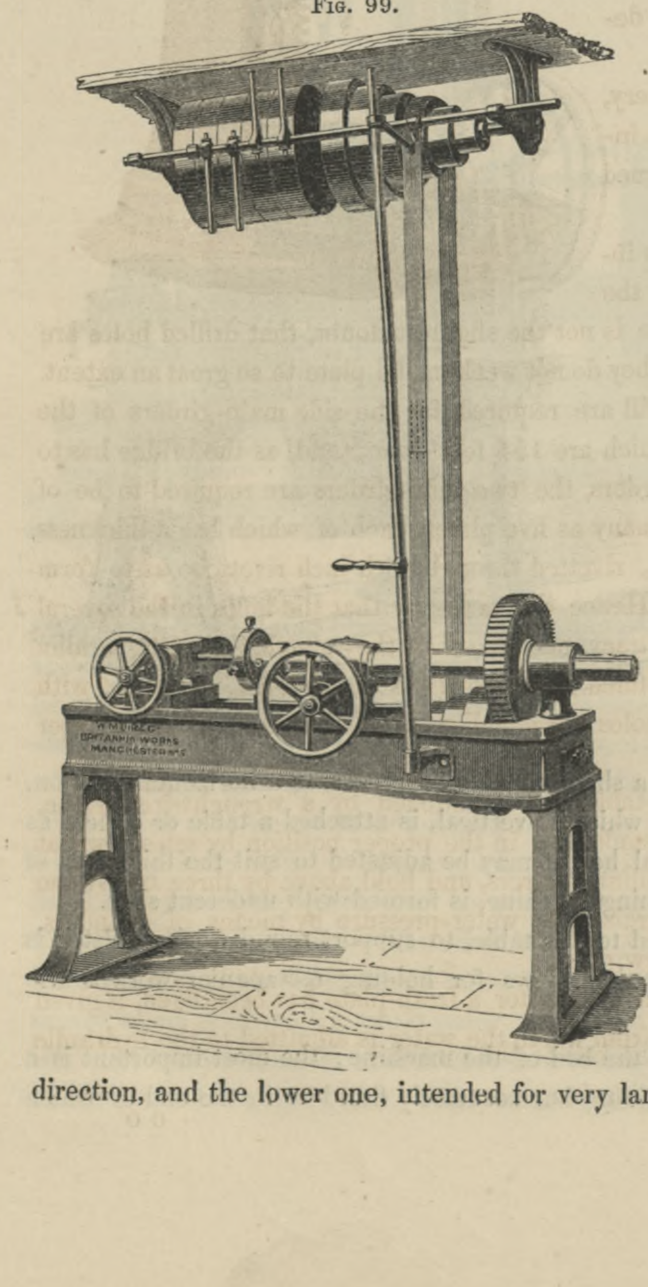
Some shaping-machines are so constructed, that the slide which carries the tool-holder has quick-return motion, this is effected by using an extra shaft to that which carries the eccentric or crank to drive the tool is geared by means of eccentric or elliptical-toothed wheels.

A variety of the shaping-machine in which the tool-holder moves in a vertical direction, is termed a slotting-machine, on account of its being especially adapted to the planing out of the grooves, or slots in wheels which receive the keys used to retain them firmly upon the shafts for which they are intended.

The machine of which we give an illustration is of the following dimensions. It has a variable stroke, from half-an-inch to 6 inches, and will plane an object 2 feet long, by 6 inches broad. In it, round work up to 1 foot in diameter may be treated, hollows from $\frac{1}{4}$ of an inch to 6 inches in radius.

And the machine may be changed to operate upon round, hollow, or flat surfaces without readjusting the work in hand.

Fig. 99.



The accompanying sketch, Fig. 99, represents Messrs. Muir's screwing-machine. This machine, also one of a class having numerous varieties, may be regarded as a modification of the turning-lathe. It is provided with speed-pulleys, in order, that the speed may be adjusted to suit the nature of the material operated upon and its size; to the shaft carrying these speed-pulleys a toothed-wheel is keyed, which gears into another attached to a stout shaft supported as shown. If a solid cylinder is to be screwed, it is attached to the chuck screwed on to the end of the main shaft or mandril, and the dies by which it is to be formed are fixed in the modification of the slide-rest, shown at the further end of the bed of the machine. If a nut is to formed, the blank from which it is to be manufactured is firmly fixed in the rest, and a tap of suitable dimensions is driven through it by the chuck.

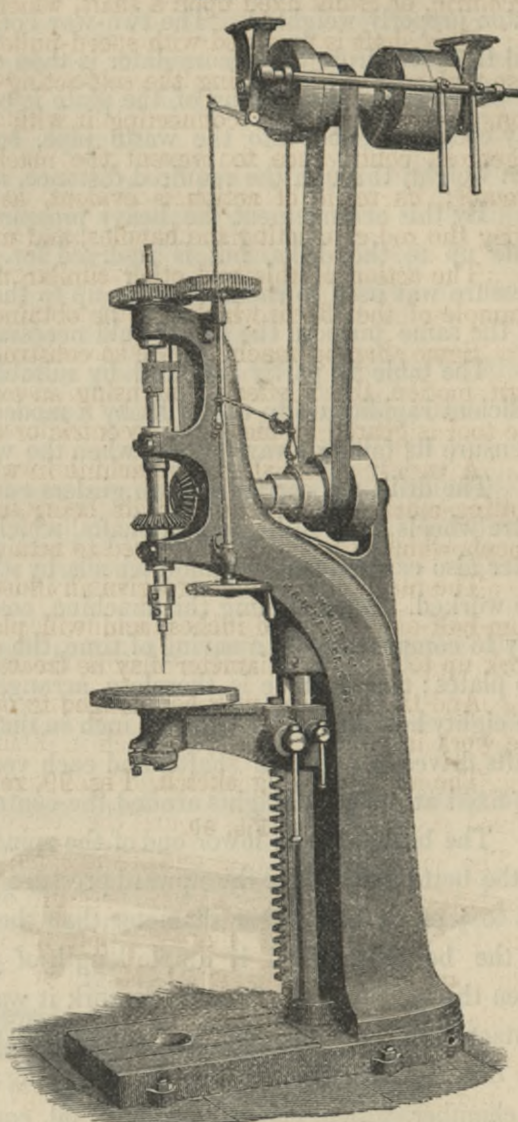
The machine here represented, is intended to produce bolts and nuts varying in diameter from $\frac{1}{2}$ an inch to $1\frac{1}{2}$ inches, it is supplied with 10 machine taps, and 10 master taps.

For boring holes in works which are too large to be effected by hand, the lathe is sometimes used, but more frequently, a machine specially designed for drilling holes is employed.

These are manufactured on several plans, but the most convenient form in which to construct them, is that shown at Fig. 100. This machine consists of a strong standard or frame, to which are attached two tables; the upper one, which is circular, is moveable in a vertical direction, and the lower one, intended for very large work is fixed, being on the level of the floor.

The drill is carried by a small chuck fixed to a vertical mandril, to which is keyed, a mitred-tooth wheel. There is also a short horizontal shaft, provided with speed-pulleys, and having at its extremity a mitre-wheel, which gears into that on the vertical mandril, as shown. This mandril is raised and lowered by a screw working in a thread fixed to the upper extremity of the mandril, the screw although connected with the mandril does not revolve with it. The screw is raised or lowered by causing a nut attached to the top of the machine to revolve. To effect this, the nut is formed in the centre of a toothed-wheel, which is geared with another toothed-wheel, keyed to the upper extremity of a vertical shaft, as shown; to the lower extremity of which is fixed a hand-wheel, by revolving which, the mandril and the drill which it carries is raised or lowered, as the case may be. For small work, where holes not very deep are to be drilled, this is, perhaps, the most convenient form that can be used; but for larger work, having deep holes, it is better to have the machine double-geared, and also provided with self-acting apparatus. The starting and stopping machinery is similar to that of other apparatus. These instruments may also be used for boring out apertures as well as drilling, but for this work they are not equal to the lathe.

Fig. 100.



Several kinds of portable drills have been invented, but they are represented in principle, by that which we have described above.

Before taking leave of the subject of drilling machinery, we think it desirable to describe an excellent machine intended for drilling a large number of holes at once, designed for the perforation of wrought-iron plates.

This apparatus by Mr. John Cochrane of Dudley, is intended for boring rivet-holes in wrought-iron plates, in the place of punching them as heretofore practised, and there is not the slightest doubt, that drilled holes are far more accurate than those produced by punching, and they do not weaken the plate to so great an extent.

The plates which this machine was designed to drill are required for the side main-girders of the bridge over the Thames, at Hungerford, the spans of which are 154 feet clear; and, as the bridge has to carry four lines of railway without any intermediate girders, the two side-girders are required to be of very great strength, and are, therefore, constructed of as many as five plates, each of which has a thickness of $\frac{5}{8}$ of an inch, besides four rows of 6-inch angle-irons, rivetted through with inch rivets, so as to form one compound plate of great strength and soundness. Hence the necessity that the holes in the several plates should correspond to each other with perfect accuracy, and also that they should be truly parallel through each plate, so as to insure their being completely filled when the rivets are rivetted, up which; with more than three thicknesses, is impracticable when the holes are punched, as they are then always larger on one side of the plate than on the other.

The plate to be drilled is placed upon the drilling-table and surrounded by a wrought-iron frame, securely bolted on the table within which the plate is made fast in the proper position by set-screws at each corner, being supported underneath by four longitudinal bearers, and held above by three transverse bars. The table is guided by two end-frames, and is raised by water-pressure by means of cylinders, similar to those of hydraulic presses, but fitted with hollow rams.

The pressure necessary for raising the table up to the drills after a fresh plate has been fixed, is given by a tank placed at a suitable height above the machine from which the water is admitted to the hydraulic cylinders by a two-way cock.

The pressure thus produced is not sufficient to make the drills act, but the necessary degree of force for this purpose is obtained by the use of an accumulator, consisting of an upright cylinder fitted with a piston properly weighted. The two-way cock from the tank is closed as soon as the table has been raised, and the valve from the accumulator is then opened, when the drilling immediately commences.

As soon as the drilling of the plate is completed, the valve of the accumulator is closed and the two-way cock is opened into the waste pipe, so that both pressures being removed, the table falls by its own weight, through the required distance, and the drilled plate is removed to make room for another.

By this arrangement the heavy pressure of water in the accumulator is not wasted in raising the table up to the drills, but is reserved for giving the required working-pressure when drilling. Water-pressure was used to raise the table up to the drills, as otherwise much time would be lost, as with gearing for the same purpose the feed would necessarily be much slower.

The table is partly balanced by suitable balance weights, in order to allow of its being raised with sufficient rapidity up to the drills by a moderate pressure of water, but a sufficient preponderance is left to ensure its falling away quickly when the water-pressure is removed.

The drills are carried by two girders running along the top of the machine, and are driven by toothed mitre wheels fixed to horizontal shafts, which gear into similar wheels attached to vertical shafts, which latter also carry common pinion wheels, by which the drill shafts, having pinions at their upper extremities, are worked. In designing this machine, cost had, of course, to be considered, and it was therefore necessary to compensate, by a saving of time, the much greater expense incurred, by drilling instead of punching the plates; the machine is, therefore, arranged so as to drill all the holes in one plate simultaneously; there are eighty holes in each plate, one inch in diameter, arranged in four holes each; each of the two horizontal shafts drives ten vertical shafts, and each vertical shaft drives four drilling shafts, the pinions of which are fixed at different heights around the centre verticle shaft, so as to run clear of each other.

The bushes at the lower end of the spindles are formed of wrought-iron, bevilled, and faced with steel at the bottom to receive the upward pressure of the drills when working. These bushes are carefully bored out to $\frac{1}{8}$ of an inch larger diameter than the drill spindles, with the exception of about $\frac{1}{2}$ an inch length at the bottom, which is ample length of guide to insure the drills running with sufficient steadiness; when the machine was first got to work it was found that the drill spindles became chafed in running by contact with the entire length of the bushes, and the latter were, therefore, bored out as described, without any diminution in the steadiness of working; the space thus left in the upper part of the bush forms an oil chamber, which, being kept full of oil, constantly maintains the lubrication of the shoulder of the drill spindle.

The drills have tapered shanks fitting into sockets in the lower extremities of the drilling shafts, and they are there firmly secured by means of set-screws, so as to admit of any drill being speedily removed in case of injury, and replaced by another.

The simultaneous lubrication of all the drills while in action, is effected by the plate being immersed in soap-suds, which are confined within the frame that surrounds the plate and drawn off through pipes at the sides when required.

In ordinary drilling-machines, the time when the drills are most frequently broken, is just when they are beginning to come through the plate, at which period there is not left a sufficient thickness of metal to withstand the working-pressure upon the drills, and in order to provide against injury to the drills used in the present machine at that stage of the operations, there are provided four spiral buffer-springs fixed to the upper frame, and pressing on the corners of the table which oppose an increasing resistance to the upward pressure from the rams; thus the pressure on the drills is gradually diminished as they progress through the plate, and the table comes in contact with a fixed stop as soon as the drills are completely through.

The springs are so constructed as to admit of adjustment in height according to the length of the drills, so as to allow for a variation occasioned by the wear of the drills by work.

By the employment of water-pressure, the feed is brought entirely under control, and may be regulated to be slow or fast, exactly as required. The use of water-pressure also affords the means of measuring with great accuracy the pressure requisite for the working of the drills, and it has been found by experiment that the most economical load on each drill is 5 cwt., making a total upward pressure of 20 tons on the table of the machine, which is obtained by loading the accumulator with cast-iron weights to the required extent. The drills are driven at a velocity of from 40 to 50 revolutions per minute, and at this speed the whole 80 holes of one inch diameter, each are drilled through a $\frac{5}{8}$ of an inch plate within 15 minutes in the most perfect manner, and without difficulty.

From this it appears that the maximum velocity of the extreme edges of the drills amounts to about 10 feet 6 inches per minute, and each drill removes at each revolution a layer of wrought-iron amounting to about $\frac{1}{1000}$ of an inch in thickness. The drills stand very well, being found on the average to last about 10 hours without grinding. There is no necessity for marking the plate before drilling it, but it is simply put into the frame and adjusted by the set-screws provided for that purpose.

The truth of the work is so complete, that a number of the plates thus drilled are put together indiscriminately, and 4 turned pins are passed through the corner holes, after which the plates are placed on the planing-machine and the ends planed to gauges.

The power required to drive one machine is about 10-horse power, or 1-horse power will work 8 drills 1 inch in diameter, the machine being driven by bands and pulleys on opposite ends of the horizontal driving shafts. The accumulator is kept charged by a pair of 1-inch pumps worked from the ordinary shafting by eccentrics.

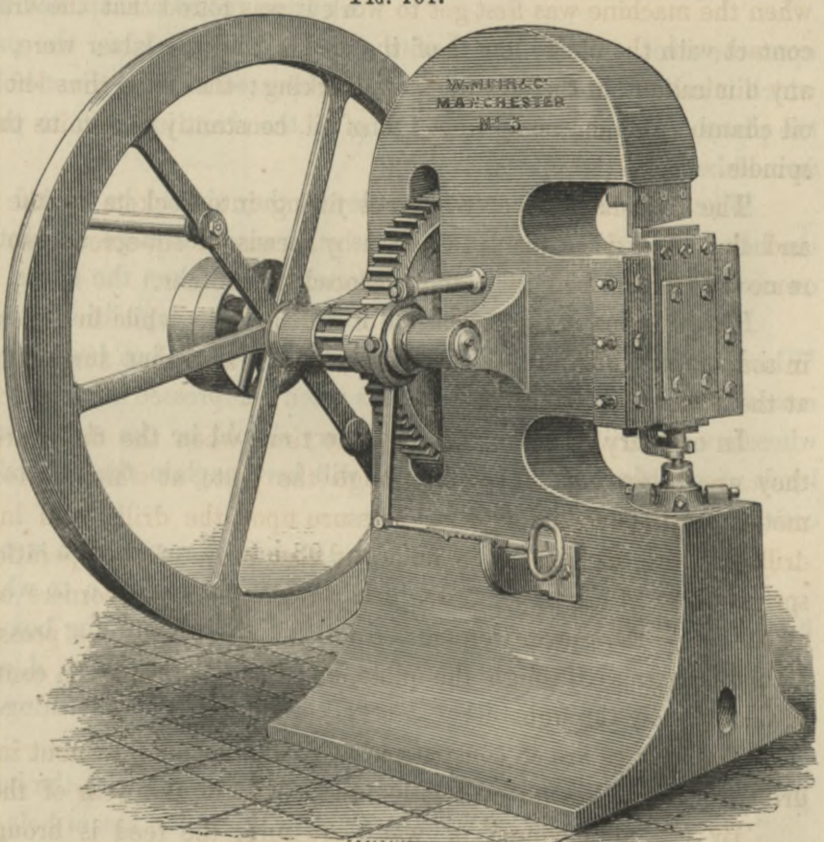
The present machine is constructed to drill the holes in each row at 4 inches apart, centre to centre, and the plates thus drilled are for the top flanges of the girder.

The holes in the bottom plates require to be 3·995 inches apart, and for this purpose another machine must be constructed.

This machine, together with others since constructed, has been found to work very satisfactorily, and to answer thoroughly the purpose for which it was designed.

It may be imagined that there is some difficulty in getting the drills to wear equally well in all parts of the machine, but such is not the case, as the workmen soon find out which drills are hard enough to work well, and then lay by the softer ones to be rehardened.

Fig. 101.



The drills cut very smoothly and uniformly, turning out long spiral shavings.

The principal difficulty experienced was produced by the want of uniformity in the hardness of the plates, hard parts sometimes occurring which would dull the edges of a few of the drills, which had then to be replaced; but a stock of from 20 to 30 spare drills was kept in hand, and any drill can be readily replaced in a few moments, by simply slacking the set-screw and tapping the drill which falls out, when another may be inserted.

We have extended this notice to a somewhat greater length than we at first intended, but the great importance of this machine, as well as the novelty of the purpose for which it was designed, and in the execution of which it has so admirably succeeded, appeared in our eyes to warrant a full and complete account of its construction and capabilities; but, as we could not spare sufficient space for this purpose, we have amply explained the principles upon which it is constructed, and have also given such statistics of its working as we thought might be practically useful.

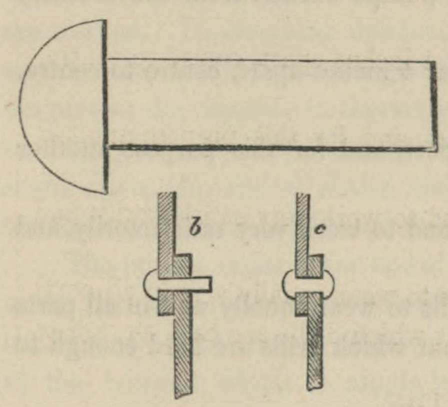
We must now, however, pass on to the description of the ordinary punching-machine, some forms of which are exhibited in Plate D.—The machine here illustrated (Fig. 101) is a portable punching- and shearing-machine, it may be driven either by hand- or by steam-power. It consists of a short shaft carrying a fly-wheel to regulate the speed, and a small-tooth wheel which may be caused to revolve with the shaft, or not; the connection being effected by means of a clutch.

This small-toothed wheel gears into a large one, as shown, which is keyed fast on to another short shaft, to the opposite end of which is fixed an eccentric; this eccentric moves a vertical slide, which is guided by grooves in the frame of the machine; to the bottom of the slide is fixed a punch, working into a bored bed or matrix, below this is for punching holes in plates.

To the top of the slide is fixed a shear edge, and another is attached, as shown, to the frame above, for shearing plates.

We must now describe the method of rivetting plates together as done in the construction of wrought-iron girders, &c.

Fig. 102.

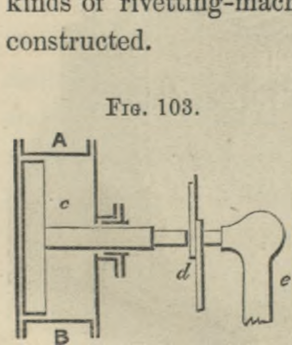


At Fig. 102. is shown a rivet formed of bar-iron, with a spherical head at one extremity, the other end being slightly tapered. In order to rivet two plates together, holes having been previously punched or drilled in the required positions, a rivet is placed in a fire and raised to a cherry-red, or to a white heat, it is than placed, by means of a pair of tongs, in a hole, passing through the two plates to be joined, as shown at *b*, a workman then holds a hammer against the head, and another, stationed on the opposite side of the work, first strikes the plate a few blows round the rivet-hole, in order to bring it close up to that to which it is to be united, and subsequently hammers the tapered end up so as to form a head, as shown at *c;* the head is rendered smooth and superfluous, metal about the edges is removed by placing a swage, called a snap, having a conical or spherical recess in the end, upon the rivet, and striking it with a sledge-hammer when the process will be complete. This operation, which is called hand-rivetting, must be repeated at every rivet-hole. It is considered necessary to hammer the plates together previous to closing up the rivet, in order to prevent any swelling or collar from being formed between the plates.

When very long rivets are employed, they should be cooled in the centre before inserting them into the holes intended to receive them, as otherwise the large amount of contraction which would take place on cooling, would be very liable to stretch and weaken the rivets, or might even draw the heads off.

Rivetting may also be effected in a far more expeditious manner by means of a rivetting-machine. Which method produces the most satisfactory work, has been a matter of much discussion, but it seems reasonable that the softer the metal is when compressed into form, the less will be the straining of the fibres; wherefore, the rivets closed by machinery should be the strongest. At Plate D. we have shown several kinds of rivetting-machines, but we will here explain the general principles upon which they are usually constructed.

Fig. 103.



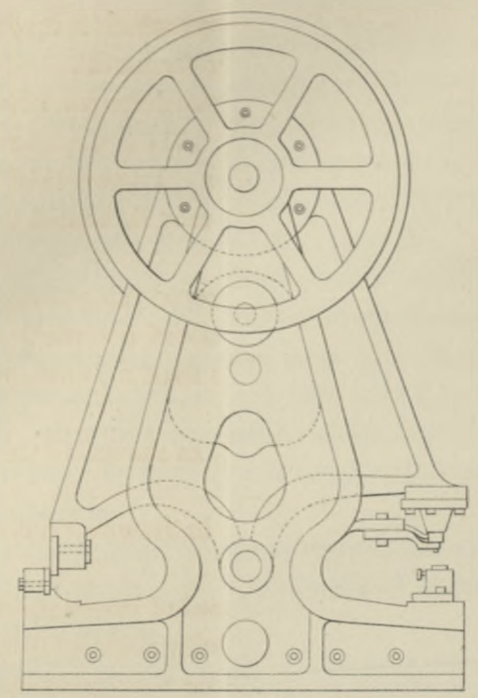
In Fig. 103 let *a b* represent a steam-cylinder of the ordinary construction, fitted with a steam-tight piston, *c*, to which is attached a stout piston-rod, working air and steam tight in a stuffing box in the cover of the cylinder, and carrying at its extremity a cup-shaped snap, *d*, as shown; opposite this snap is another fitted to a very solid standard, *e* forming part of a massive framing by which the machine is supported.

The plates to be united with the heated rivet in its place are put between the snaps, as shown, and the steam being admitted behind the piston *c*, its snap is driven forward, and the rivet-head is immediately formed. In some machines there is a contrivance to

PUNCHING, SHEARING & RIVETTING MACHINES.

FIG. 1.

DE BERGUE'S PUNCHING & SHEARING.



COOK'S PUNCHING SHEARING & RIVETTING.

FIG. 2.

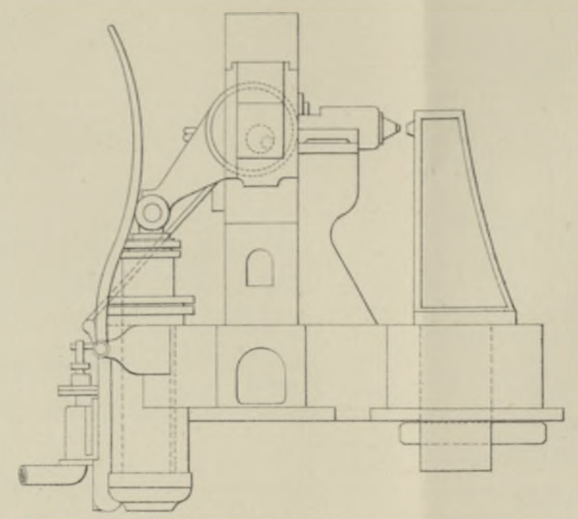


FIG. 3.

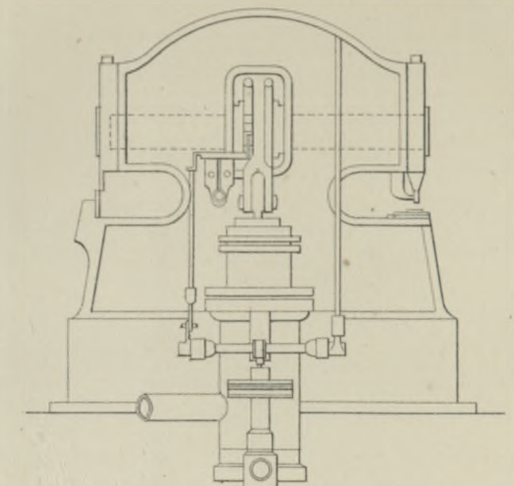


FIG. 4.

DE BERGUE'S RIVETTING.

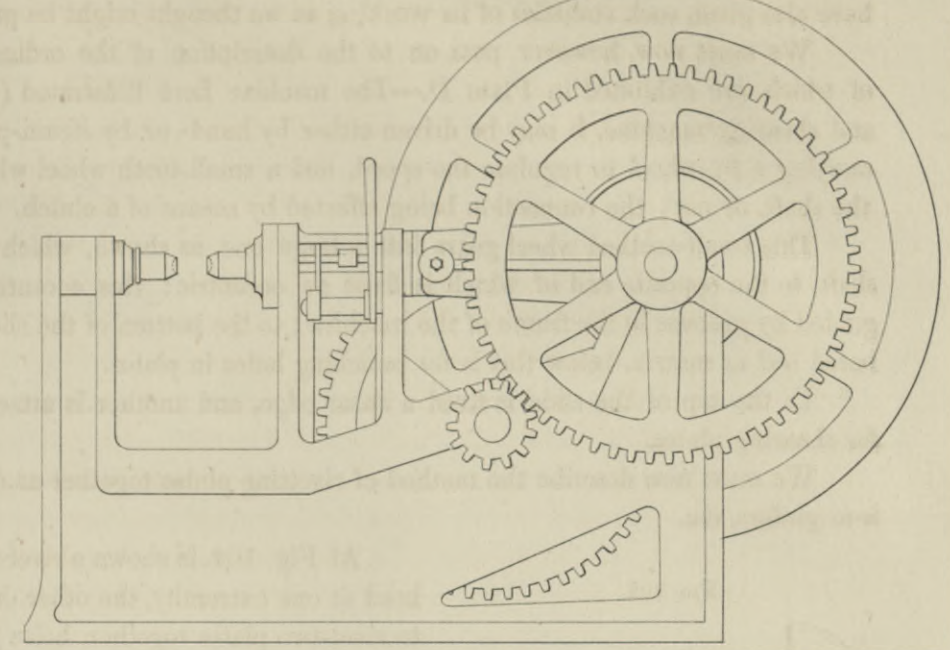


FIG. 5.

MORGAN'S SHEARING.

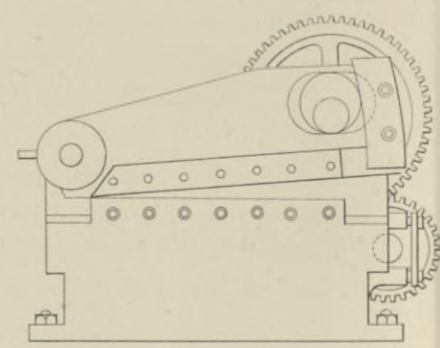


FIG. 6.

DONALD'S SHEARING & PUNCHING.

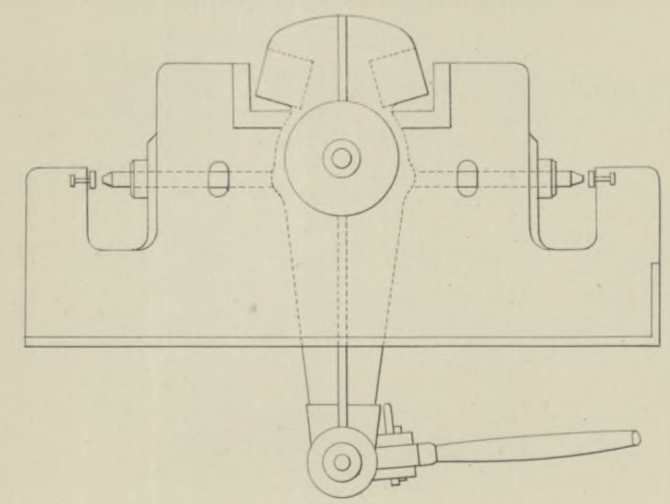


FIG. 7.

RIVETTING MACHINE
USED FOR THE
CONWAY BRIDGE.

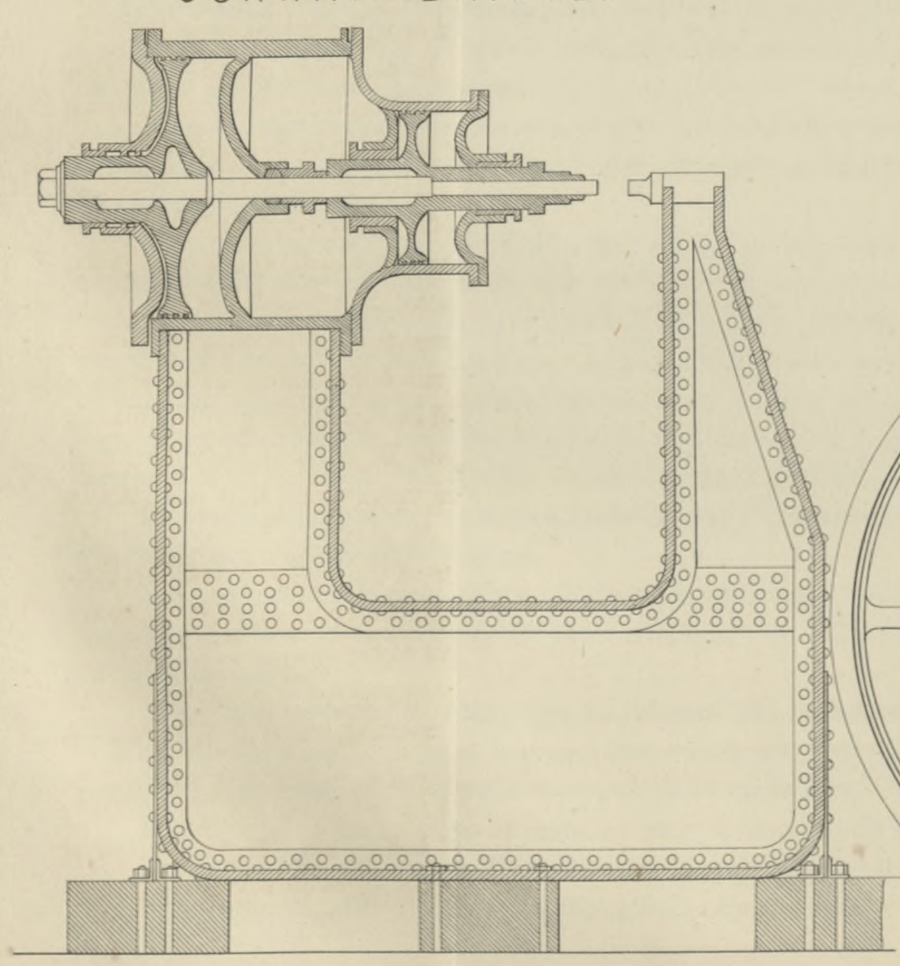


FIG. 8.

PUNCHING MACHINE
USED FOR THE
BRITANNIA BRIDGE.

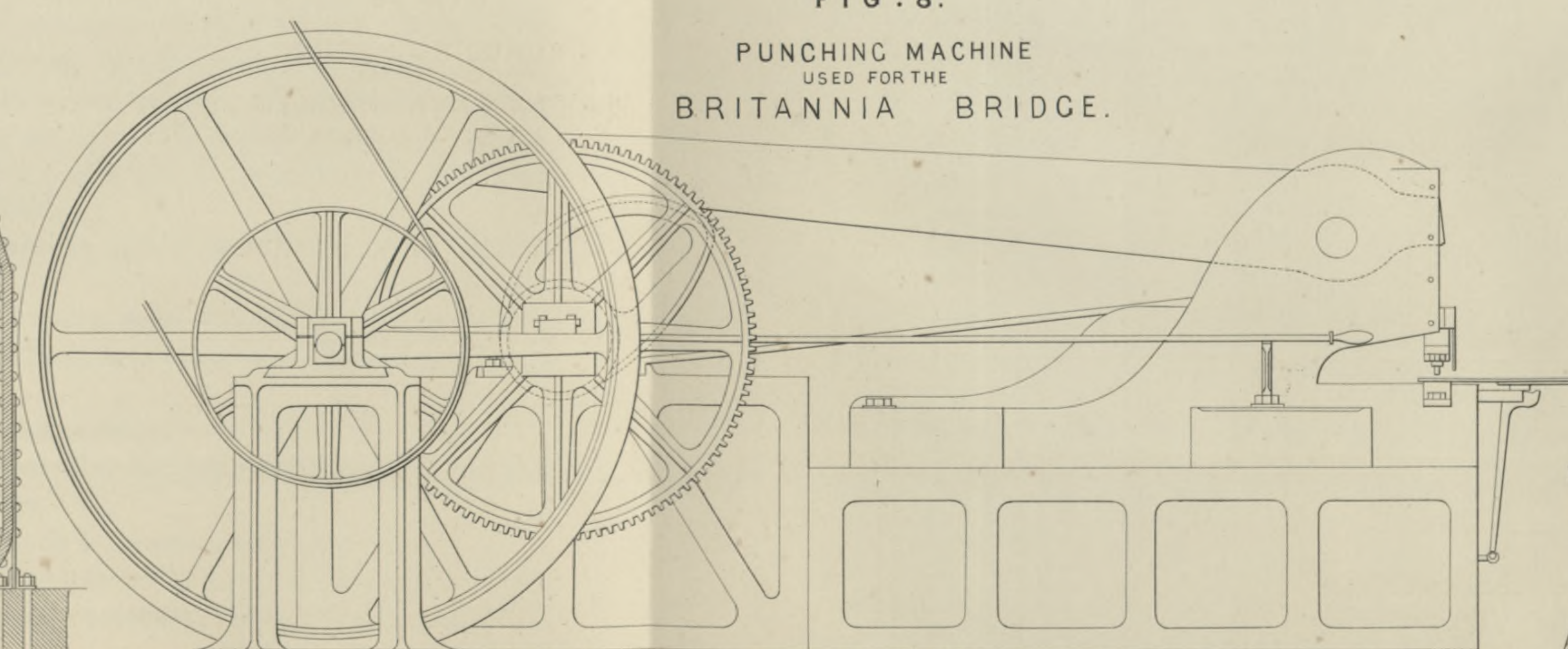
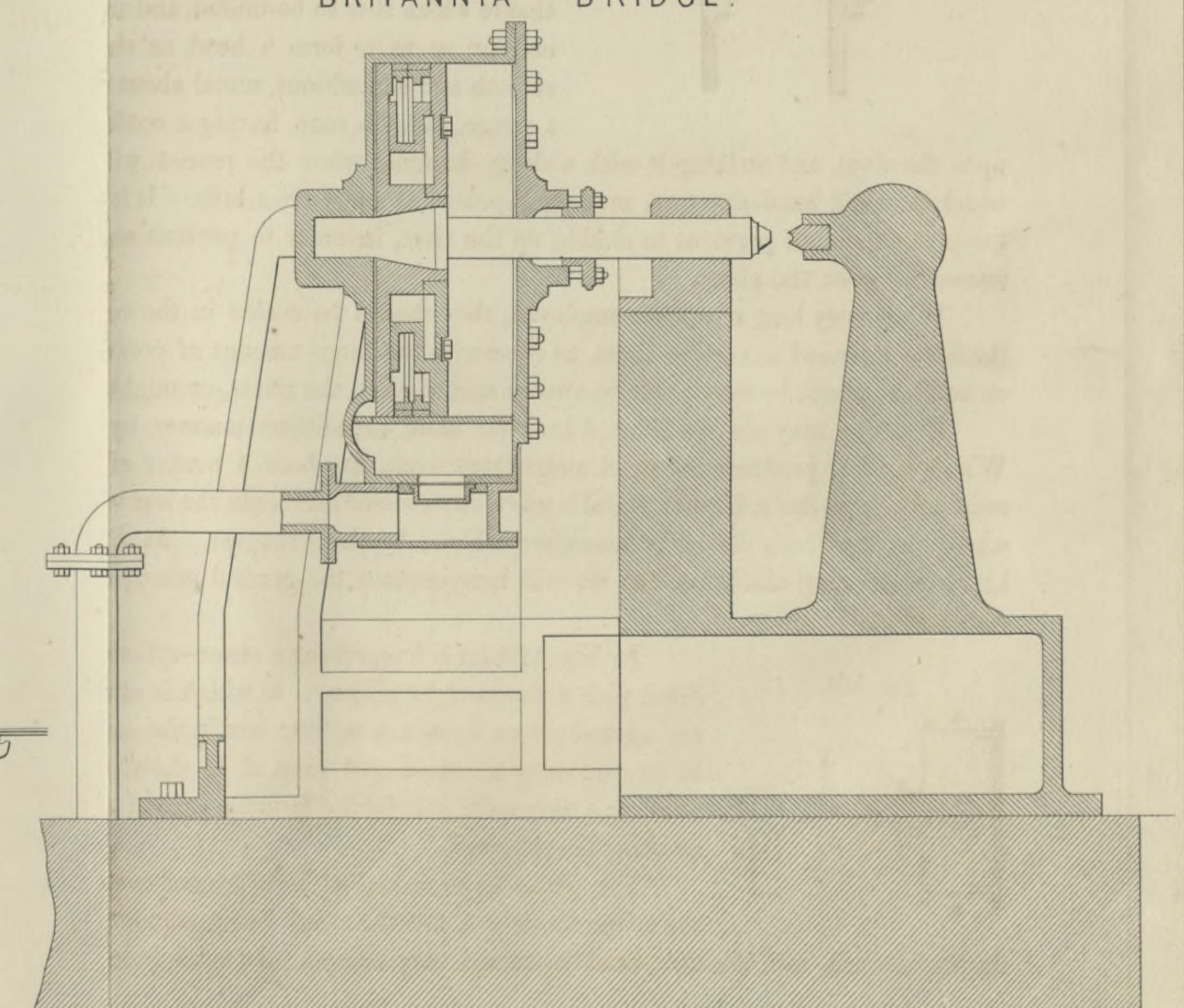


FIG. 9.

RIVETTING MACHINE
USED FOR THE
BRITANNIA BRIDGE.



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47 P 237

Standidge & Co. Litho. Old Jewry

London. E. & F. N. Spon, 16, Bucklersbury.

hold the plates in close contact whilst they are being rivetted together. Rivetting-machines may also be constructed so as to be capable of being driven by a strap or band working on a rigger.

A very excellent punching- and shearing-machine has been devised by Messrs. C. De Bergue and Co., an illustration of which will be found at Plate D, as also one of a very useful rivetting-machine by the same firm; this machine is driven by a band. Other rivetting-machines, including those used for Britannia and Conway Bridges are also shown at Plate D.

The rivetting-machine will head from 16 to 20 rivets per minute.

In order to indicate the position of the holes to be punched in wrought-iron plates, the following method is usually adopted, although it is by no means indispensable:—When a number of plates are to be similarly punched, a template of thin metal is carefully marked and drilled exactly as the plates are required to be; then this template is placed upon the plate to be drilled, and the positions of the holes marked on the latter by passing a brush of paint or other colouring-matter over it, the template forming, in fact, a stencil-plate.

There are some other processes required in the manufacture of iron structures which we have not yet described, and which are usually included under the head of fitting, the men accustomed to such processes being called fitters.

Fitting essentially consists in the finishing off of various elements of a machine or structure, so that when such machine or structure is finally erected, they must fit to each other with the required degree of accuracy.

The tools required by the fitter consist of a work-bench, a tail-vice, also called a smith's-vice, some common and centre punches, some drills, cold-metal chisels and scrapers, a large assortment of files, some hammers; and finally, for the purpose of measuring and testing the accuracy of his work, dividers, callipers, rules, straight-edges, squares, and surface-plates.

The dividers or compasses are too well known to require any description at our hands. Callipers are of two sorts, outside callipers and inside callipers. The former may be described as compasses, of which the legs are bowed, and they are used for measuring the diameters of cylinders and the thickness of various kinds of work. The inside callipers have their feet turned outwards away from each other, and may be formed by crossing the legs of outside callipers; they are used for measuring the internal diameters of cylinders, the width of recesses, etc.

Rules, straight-edges, and squares, require no description, and the surface-plate or planometer is a strong plate of cast-iron, whose surface is wrought up to as near a true plane as can possibly be obtained by the means at our disposal.

Files have been already described; they are of various sizes, and are usually held by the handle in one hand while the workman rests with his other hand upon the extremity of the file. When a flat surface is to be formed by filing, great care is requisite to prevent its edges from being rounded off, and the pressure exerted by each hand must be continually varied as each stroke is made, because the leverage of each, acting about the work as a fulcrum, is continually varying. The greatest pressure at the commencement of the stroke must be exerted by that hand which rests upon the point of the file, but at the termination of the stroke this hand must exert the least pressure.

Instructions for getting up surfaces truly by the file would be useless, as it is by practice only that excellence can be attained in this art.

The file may be prevented from clogging by rubbing it with a piece of chalk, and the tool should not be pressed heavily upon during the return stroke, as by so doing there would be created a great tendency to break the teeth.

When files are worn out, they may be softened and re-cut as before.

Machines for filing have been invented, but we are not aware that they are extensively used, nor is it possible that all kinds of filing could be executed by machinery.

Let us suppose that a piece of cast-iron is to be brought up to an accurately plane surface.

As much of the siliceous coating of sand as may be, is first removed by means of hard coke, and then by an old coarse file, after which it may be entirely removed by the cold-chisel and hand-hammer; or it may then be put into the planing-machine and planed with the point-tool. The surface should then, when tested by a straight-edge, present a tolerable approximation to a plane surface.

The marks left by the tool may now be obliterated by using a file, after which the work must be completed with a scraper, which may conveniently be made by grinding the teeth off an old three-square file.

Some red colouring-matter is now rubbed on the surface-plate, and the work with its face downwards is laid upon it and gently moved about; by so doing, the highest points on the surface of the work will be marked by the colour, these are to be reduced by scraping, and the process repeated, when it will be found that a great many more points will take a bearing upon the surface-plate; these are continually removed until the surface takes the colour uniformly all over, when it is as near the truth as it can be made.

When the work is too heavy to be moved, the surface-plate may be moved about upon the surface to be wrought.

When surfaces not plane are to be fitted accurately to one another, they are made as true as possible separately, and then some coloring-matter is rubbed upon one of them which is moved in contact with the other, whereby the points of bearing will be marked, which must be constantly reduced until the two surfaces bear equally all over.

These methods of producing true surfaces by scraping, are used for engine-slides, bearings, lathe-beds, planing-machine beds; and, in fact, for all kinds of work which is required to be accurate.

When the fitter requires to drill a hole by hand, he places the drill in a suitable stock, which may be so worked; but these stocks are so numerous, that we cannot afford space for their description.

The punches and cold-metal chisels are driven by hand-hammers.

Fitters and turners are usually skilled artisans, who receive the highest wages paid to that class of workmen, though, perhaps, the engine-smith is entitled to rank amongst them; but the other work, such as that performed by planing, slotting, and drilling-machines, etc., is usually conducted by machine-men or labourers, who are incompetent to the above offices, and who receive lower wages. The patterns for iron and other castings, are made to drawings by a superior class of joiners known as pattern makers, who may also be classed as skilled artisans, the drawings with which they, in common with other workmen, are supplied, show the dimensions of the machine or structure when finished. Smiths should be furnished with separate sketches of the forgings required, drawn upon pieces of plank full size for small works, so that they may be able to measure their work upon it without allowing it to cool; and, moreover, it often happens that if they are supplied with full-sized drawings upon paper, these are destroyed from the hot iron being placed too near them in order to test its accuracy.

Rivets, bolts, and nuts may be economically made by boys, or machinery.

We must now make some remarks upon the power employed to drive machinery such as we have described above.

In most cases uniformity of speed is an almost indispensable requisite, wherefore an excess of power should be available; and the prime mover, whether it be a steam-engine or a water-wheel, should be provided with a suitable apparatus to adjust the quantity of steam or water admitted to the amount of work to be done.

The common two-ball governors are in most general use for this purpose; but we are inclined to think that this is not the best form, and that one which is put into action by the strain upon the main shaft of the prime mover is preferable. We do not, however, intend to enter into that question here, although it might be shown that by using the ordinary governors, we do not ensure the engine working at an uniform speed for every pressure; but that on the contrary, for a heavier load, the engine must move slower to allow the throttle-valve to be sufficiently open.

We have stated above that it is desirable, in order to work our machinery satisfactorily, to have plenty of power always; having in hand more than can possibly be required in ordinary working; and this power must be partially employed, according to the number of machines being worked, by the same prime mover at once, but there are also other points to be attended to.

The first matter which we naturally consider is attended with difficulty of determination, and this is, What power do we require to drive any given machine? and this we cannot give an accurate answer to, for it varies under different circumstances; thus the speed at which the work is moving, the depth of the cut, the hardness of the material, etc.

The speed at which the surface which is being operated upon is moved, should always be kept within definite limits for the same material; thus, if we are turning a large cylinder, it should perform one revolution in four times as long as a cylinder of one-quarter its diameter, but the varieties of speed obtainable with lathes, etc., by means of the speed-pulleys, are not sufficient to meet the requirements of all cases, hence we shall be subject to variations of velocity which we cannot control.

Also we have to contend with different depths of cut; thus it is probable that in turning a cylinder a very much deeper cut will be taken at first than afterwards.

The hardness of the material, its homogenity or otherwise, are entirely out of our control. We may perhaps regard the 80 drill drilling-machine as a species of boring-lathe in its action, and this seems the more probable, when we remember that like the latter, the former removes the material in the form of spiral shavings; then, calculating from the results given above, we find that motive-power of one horse is capable of cutting away in this manner, economically, nearly four pounds and a-half of wrought-iron per hour, but it is most probable, that in machines of a different construction a much greater quantity could be removed, twice or three times as much at least. Before concluding this Chapter, we will offer a few remarks upon the preservation of iron surfaces from the action of the air.

The action to be resisted will, of course, depend upon the locality in which the structure is situated.

Thus, if there be a bridge, or other structure, placed in a situation where the air is pure, that is to say, in a normal condition, which it may be cousidered to assume at a distance from towns, we shall only have to provide against the oxidizing influence produced by the oxygen of the air. The most natural method of accomplishing this appears to present itself in the employment of paint, but the use of the generality of paints, is not unaccompanied by disadvantages.

There is, however, a species of paint, which has been extensively applied with success, manufactured by Messrs. Lewis Roughton and Co., of the Watergate Colour Mills, Deptford Green.

This paint is an oxide of iron, which accounts for its great practical efficiency, for two metals of the same name will not produce a galvanic action, whereas this always occurs in the case of a metal in contact with a different metal, provided that moisture be present.

Thus it is probable, that if a lead-paint be used, under some circumstances, a portion of the oxide of lead may be reduced at the expense of the iron it is intended to protect, but this will not occur to any great extent in ordinary cases.

The process known as galvanising, has been very largely employed, in order to preserve iron surfaces from the corrosive action of the atmosphere, and the value of this treatment necessarily depends upon the superior resistance of the metal with which the iron is coated, to the action of the constituents of the atmosphere. This method may be very successful when the structure is not exposed to the saline atmosphere of the parts at, or near the sea-coast; nor yet to the sulphurous atmosphere of any of our coal-burning cities; but when the metal is subject to either of these corrosive media, the galvanising process becomes almost useless, as the coating thus imparted to the metal is soon destroyed, and the iron itself attacked.

From a research of a very extended character (reports of which may be seen in the Transaction of the British Association, 1839 to 1849), it has been concluded, that iron may most effectually be protected, by raising it to a nearly red heat, and then quenching it in well-boiled coal-tar, mixed with powdered caustic lime. It is also stated, that the surface so obtained, takes whitewash or distemper, and then oil-paint well, afterwards; and if the surface of iron thus treated, be afterwards coated with boiled coal-tar, and caustic lime, every third or fourth year (according to the climate), no period can be assigned to the durability of the metal; it is further stated, that sheet-iron roofing erected in 1842, and thus protected, is still sound.

With regard to the action of fresh and salt water upon iron, the following remarks may afford some slight information:—

Of the various descriptions of cast-iron, the light-grey kinds are those which most perfectly resist the action of water.

Cast-iron, from which the deposit is occasionally removed, will last longer than if it be allowed to remain; in the same manner that railway bars which are in use, last longer than those which are inactive, which may be thus explained:—First, a coat of oxide is formed upon the bar, which gradually acts upon the metallic iron beneath it, to which it yields its oxygen, then it absorbs more oxygen from the air, and the action is continued until the bar is destroyed.

It is therefore desirable when such can be effected, to occasionally clean cast-iron surfaces exposed to the action of water.

The rate at which the corrosion progresses is a minimum when the metal is most uniform and free from graphite; it is also less in fresh than in salt water.

In clear sea-water cast-iron is not corroded much faster than when it is exposed to the action of the atmosphere.

With regard to wrought-iron it may be observed, that those irons of most uniform texture last for the greatest period, and also, that highly siliceous irons corrode with great rapidity.

Bars of Staffordshire scrap-iron faggottedwere found to be the most durable; next to which in excellence are the Low Moor boiler-plates.

Foul sea-water is exceedingly destructive to iron, and the more so if it evolves sulphuretted hydrogen gas; but putrid mud appears to exhibit an action perhaps even more rapid than the above.

Having now, as far as the space at our disposal allowed us to do, explained the various machinery and manipulations required for the construction of cast- and wrought-iron bridge structures, so far as the work is conducted in the factory; we must conclude the present Chapter, and pass on to the next.

CHAPTER XIII.

ON RIVETTED AND OTHER JOINTS.

We have deferred this Chapter, which treats of that part of bridge construction which is certainly inferior in importance to no other, until we should have explained the processes of manufacture, for the reason that we considered it desirable before entering upon the present subject, to examine carefully the methods by which the various joints used in bridge construction are produced.

In the formation of all descriptions of joints, the object which we seek after is the formation of the junctions in such a manner that the element, composed of many pieces, may be as strong as if it were made in one piece.

We are only acquainted in the manufacture of iron with one joint, which may be called perfect, and this is that form of joint which is produced by joining two or more pieces of metal at a high temperature; with cast metals one of the pieces at least must be fused, so that it is melted on to the other; this is called burning on; but in wrought iron so high a temperature is not required, and the method is called welding.

The process of welding we have already described, and a good welded joint should be equally strong with the other part of the work.

This method may be applied to bars, etc., for bridge construction; but the parts for the junction of which this method is not used, are unfortunately most important—viz., the flanges, which are usually formed of plate- and angle-iron.

There can be little doubt that if Bertram's method of plate-welding were but satisfactorily tested, it would give results perfectly applicable to plates not exceeding a certain width, and the adoption of this process would produce stronger and more economical joints than those now employed.

It is, however, useless to extend our remarks upon this subject; we will, therefore, turn our attention to matters of more practical utility.

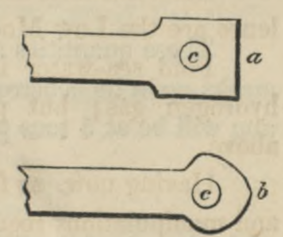
We shall first consider the simplest form of joint, which is similar to that used in suspension chains and the tension members of some triangular girders.

In all our calculations with regard to wrought-iron joints, we shall assume the working-resistances of the metal, as below: —

Tensile resistance = 5 tons per sectional inch.
Compressive do. = 4 „ do.
Shearing do. = 4 „ do.

The value of the coefficient for shearing-strain is deduced from experiments on the resistance of a pin to shearing, the pin being similarly circumstanced to that in the joint of which we are about to speak.

Fig. 104.



In Fig. 104, is shown one end of a link forming part of a suspension chain, *a* showing the form in which the head or eye-piece is most generally made, *b*, the form of the head as indicated by theory.

In the present case, the hole *c* is drilled or bored out carefully.

The dimensions of the various parts of this joint must be as follows:—

Let n = the number of sections at which the pin must be divided before the joint can fail.

S = the direct strain in tons to be borne by the joint.

d = diameter of the pin in inches.

a = area of the pin in square inches,

then will,

$$S = 4\, a\, n,$$

but,

$$a = 0{\cdot}7854\, d^2$$

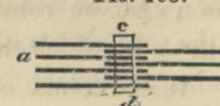
therefore,

$$S = 3{\cdot}1416\, d\, n^2$$

$$d = \sqrt{\frac{S}{3{\cdot}1416\, n}},$$

from which formula the diameter of any required pin may be found when the strain upon it and the number of places at which it will be sheared in case of failure are known.

Fig. 105.



Let us take a practical case to illustrate the use of this formula. Let *a b* represent one joint of a suspension chain of three and four links, alternately connected by the pin *c d*, and subject to a direct tensile strain of 60 tons. Then it is evident that before the links can separate, the connecting-pin must be cut through at six different sections, n being equal to twice the least number of links, then the diameter of the pin, supposing all the links to bear equally upon it, will be,

$$d = \sqrt{\frac{60}{6 \times 3{\cdot}1416}} = 1{\cdot}78 \text{ nearly;}$$

say $1\frac{7}{8}$ inches diameter.

This is the theoretical diameter, but in order to allow for imperfect bearing, the pin must be made larger; say 2 inches in diameter, then the strength of such a pin would be,

$$S = 6 \times 8{\cdot}1416 \times 4 = 75{\cdot}39 \text{ tons.}$$

In designing the joint, allowance must be made for the metal lost by the hole through which the pin passes, or the head of the link must be made wider than the body; as we have only to replace the metal removed by drilling, the depth of the head will be,—allowing the hole to be $\frac{1}{16}$ of an inch larger than the pin which passes through it, and calling the general depth of the link b,

$$b + d + \frac{1}{16} = \text{depth in inches.}$$

The whole strain upon the pin is brought to bear upon the metal immediately in front of it, so as to tend to force out the ends from the eyes by a shearing action; we must, therefore, allow sufficient overlap or length of head beyond the pinhole to withstand this, and we observe, that in each link the metal must be sheared at two sections, before the piece mentioned can be forced out, hence

if t = thickness of all the links taken together,

and l = length of overlap in inches,

the links of one series being taken, which, in the above case, would be either three or four,

$$S = 2.\ 4.\ l.\ t,$$

therefore,

$$l = \frac{S}{8.\ t}.$$

In the above example, let the thickness of all the links in one series amount to $2\frac{1}{2}$ inches, then must the least overlap be,

$$l = \frac{60}{2{\cdot}5 \times 8} = 3 \text{ inches.}$$

These quantities are all we require for the determination of the present joint, but we offer a few remarks upon its construction. If we take the modulus of elasticity at 11 115 tons, the amount of elongation will be at 5 tons per square inch,

$$= \frac{5}{11115} = \frac{1}{2223}$$

of the length of the link; and, therefore, if the link be 15 feet long, the extension under this strain would be,

$$\frac{180}{2223} = {\cdot}081 \text{ inches nearly,}$$

about $\frac{1}{12}$ of an inch.

From this it appears, that if there were an error of only $\frac{1}{32}$ of an inch in a hole in one of the bars, that is to say, if in one of the links, the holes at each extremity, were $\frac{1}{32}$ of an inch further apart than in the others when the chain is fully loaded, that bar will not be strained with more than $\frac{3}{8}$, the strain to which the other links are subject.

This circumstance renders it very necessary in specifying for suspension-bridges, only to allow very slight errors in this measurement, and to make due allowance for such errors. Suppose that an error of only $\frac{1}{64}$ of an inch was tolerated, and that half the links had this error, then the links being 15 feet long, two of the four bars would be $\frac{1}{64}$ of an inch longer than the other two, and, consequently, these two would bear $\frac{7}{16}$ of the strain nearly, whilst the other two would bear $\frac{9}{16}$ of the total strain; these quantities would in the case chosen above, amount to 26·25 tons on the longer bars, and 33·75 tons on the shorter bars, the strains per square inch on the bars becoming 4·375 and 5·625 tons. These apparently trivial errors produce much greater effects than might be imagined, unless the matter were carefully examined; and a due consideration of all these facts cannot fail to show us how utterly useless are mathematical theories, unless they be either modified or confirmed in every instance by practical observation.

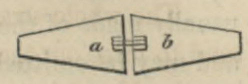
It appears to us, that a modification of Mr. Cochrane's drilling-machine, might, with advantage, be applied for drilling out the links of suspension-chains, whereby they would be obtained exceedingly accurate.

All that would be required for drilling the holes in various-sized links, would be a machine, consisting of a stout iron bed, upon which two drilling-frames might slide, so as to be adjustable within certain limits: the drill spindles might be driven by sliding bevel gear, or by straps, and the one machine would answer for various bridges.

We will now examine another simple form of joint. Let a and b be two frames, the strain on which tends to pull them asunder, but which are united by a rivet or bolt.

It is required to determine the dimensions of the various parts of the joint.

Fig. 106.



First, let us speak of the frames. The strain upon the rivet will tend to cut or draw out a round piece from the thickness of the frame, shearing through a sectional area equal to the thickness of the plate, multiplied by the circumference of the rivet-head or bolt-head, as the case may be. Using the same notations as before, but with the addition of D for the diameter of the head of the rivet or bolt, we find

$$S = 4.\,t.\,3{\cdot}1416\ D$$

$$\text{therefore } D = \frac{S}{12{\cdot}57\,t} \text{ nearly.}$$

Let us suppose that the arrangement is intended to support 4 tons, the plate being $\frac{3}{8}$ of an inch in thickness, then the diameter of the bolt-head must be,

$$D = \frac{4}{12{\cdot}57 \times \frac{3}{8}} = 0{\cdot}9 \text{ inches nearly,}$$

but the diameter of the bolt itself must evidently be,

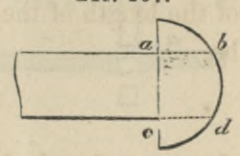
$$d = \sqrt{\frac{4}{5 \times 0{\cdot}7854}} = 0{\cdot}98 \text{ inches,}$$

wherefore the bolt-head must of necessity be made much larger than is requisite to withstand the strain, and for this reason the value of D does not usually require to be calculated.

Let us now consider the general proportions of rivets and bolts, that they may be equally strong in every direction.

First, we will determine the dimensions necessary for rivets.

Fig. 107.



At Fig. 107 is shown one head of a spherical headed rivet, and we have to consider the proportions which must exist between the diameter of the rivet and the length of the dotted lines *a b c d*, in order that the head of the rivet may not be drawn off by a shearing action produced by a force acting in the direction of the rivet, before the rivet itself is drawn in two by the tensile strain thus acting upon it.

We shall not, however, calculate our strains from the ultimate strength of the materials, but ascertain the requisite proportions in order to give the relative working strains which we have already laid down, viz.: 4 tons and 5 tons. The strain which the centre part of the rivet will safely bear is,

$$S = 5 \times 0{\cdot}7854\, d^2$$

and an examination of the figure shows us that the strength of the head to resist being drawn off is, calling h the length of either of the dotted lines,

$$S = 4\, h\, 3{\cdot}1416\, d$$

but we wish to make both parts of equal strength, therefore,

$$5 \times 0{\cdot}7854\, d^2 = 4\, h\, 3{\cdot}1416.\, d$$

wherefore,

$$h = \tfrac{1}{3}\, d \text{ nearly,}$$

hence the total height of the head should not be less than half the diameter of the rivet.

The same proportion will, of course, apply equally well to the heads of pins and bolts, but the latter are usually made equal in height to the nuts, which fit by screwing on to the other ends of the bolts.

Fig. 108.

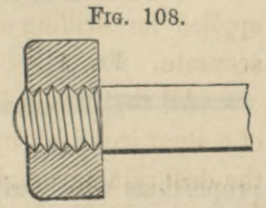


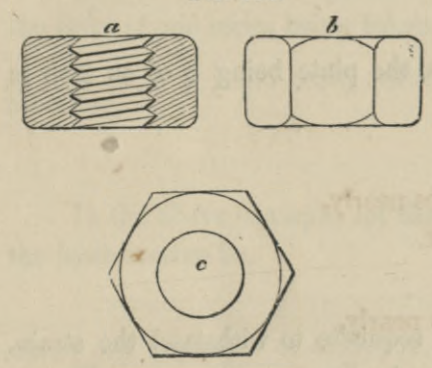
Fig. 108 shows one end of a screw-bolt with the nut, which is in section shaded. In this case the nut will be generally stripped of its thread when the bolt fails. To effect this, the thread must be completely sheared off.

The surface to be sheared usually amounts to nearly the whole internal periphery of the nut, but as the thread within it is cut by a tap, it follows that it is partly cut and partly squeezed up, whereby the strength of the material will be somewhat deteriorated. We will consider that the base of the thread, on account of this deterioration, and also on account of a portion of the material being removed, loses half its resistance per square inch to shearing-force.

Then making the necessary alteration in the formula for this diminution of strength, we find that for equal strength the height of the nut should be equal to the diameter of the bolt. The head of the bolt is usually made of the same dimensions for the sake of uniformity of appearance; but this is not necessary, and may be omitted in large foundation bolts, etc.

Sometimes a set-nut is also used to prevent the first nut from being started; if so, the set-nut is made of a height equal to half the diameter of the bolt, and it is screwed firmly down upon the ordinary nut.

Fig. 109.



An ordinary hexagonal nut is illustrated at Fig. 109, where *a* is a section, *b* an elevation, *c* a plan of the nut.

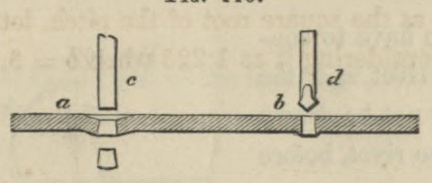
These proportions also apply to the depth of recesses screwed in work itself.

The diameter of the nut or rivet-head is usually about twice that of the bolt or rivet. We will now pass on to the consideration of rivetted joints generally. The first point to be considered is the effect of the rivet-holes in the plates to be joined.

These holes are made in one of two ways, they are either punched or drilled.

At Fig. 110 are shown, sketches somewhat exaggerated, illustrative of the effects of the punch and the drill upon plates of wrought-iron.

Fig. 110.



Let *a b* represent the plate to be perforated, *c* is a punch, and *d* a drill.

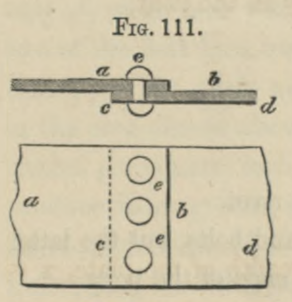
When a plate is punched, it is deflected, and the piece punched out of it is always larger at the lower surface than it is at the upper.

The average deterioration of strength by punching does not appear to have been accurately determined, but it is probable that it is not very considerable; and it may therefore be neglected in the calculation. When the hole is drilled, the material being cut away, the plate is not strained, and no deterioration of its strength results.

If the pieces to be rivetted together are subject to compressive-strain, their strength is not reduced by the rivet-holes, provided that the latter are completely filled up by the rivets; or if not quite filled up, the loss of strength is so little, that it may be neglected without danger of any appreciable error occurring.

When, on the other hand, the plate is subject to a tensile strain, its nett or effective area will be found by deducting the area of rivet-holes in any section of the plate from the total area of such section. Thus, if in 1-inch plates we have 1-inch rivet-holes, each hole causes a loss of 1 square inch of sectional area in the plate at the section which intersects its diameter.

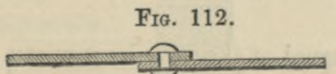
Fig. 111.



Let us determine the dimensions of ordinary lap-joints, such as are shown at Fig. 111, where *a b* and *c d* are plates which are to be united by means of the rivets *e*. Then the strength of the rivets must be equal to the strength of the plates.

We will first examine the case of a single rivetted lap-joint, as shown at Fig. 112.

Fig. 112.



Let b = the distance in inches between two consecutive rivets from centre to centre, t being the thickness of the plate, and the other notations remaining as before.

Let us take the joint as subject to a tensile strain and determine the proportions which will give equal strength to the rivets and plate at the line of rivets.

The strain to be sustained by one rivet will be equal to that sustained by a width of plate equal to b, and, in case of failure, the rivet will be sheared at one section. The resistance of the plate for a width, b making no allowance for rivet-holes, will be,

$$S = 5\,b\,t.$$

The resistance of the rivet will be,

$$S = 3{\cdot}1416\;d^2$$

we may then find the diameter for equal strength allowing for rivet-holes from the expression,

$$5\,b\,t = 3{\cdot}1416\;d^2 + 5\,d\,t$$

therefore,

$$d = \sqrt{1{\cdot}59.\,t.\,b + 0{\cdot}632\,t^2} - 0{\cdot}795\,t$$

we will resolve this equation for the pitches,

$$b = 3,\ b = 4,\ b \quad 5, \text{ and } b = 6.$$

Let $b = 3$ then

$$d = \sqrt{4{\cdot}77\,t + 0{\cdot}632\,t^2} - 0{\cdot}795\,t.$$

Let $t = \frac{1}{2}$ inch,

$$d = \sqrt{2{\cdot}385 + 0{\cdot}158} - 0{\cdot}365 = 1{\cdot}225 \text{ inches nearly,}$$

this is the approximation which comes within about $\frac{1}{15}$ of the true result.

If $b = 4$ we have t being $= \frac{1}{2}$

$$d = \sqrt{3{\cdot}18 + 0{\cdot}158} - 0{\cdot}365 = 1{\cdot}455 \text{ inches nearly,}$$

$b = 5$

$$d = \sqrt{3{\cdot}975 + 0{\cdot}158} - 0{\cdot}365 = 1{\cdot}665 \text{ inches nearly,}$$

$b = 6$

$$d = \sqrt{4{\cdot}77 + 0{\cdot}158} - 0{\cdot}365 = 1{\cdot}855 \text{ inches nearly.}$$

We shall probably find that the diameter of the rivet varies nearly as the square root of the pitch, let us see how nearly the diameters, so found, will approach to the above, considering it as $1{\cdot}225$ when $b = 3$, the results are as follows:

When $\qquad b = 3 \qquad d = 1{\cdot}225$ from above,

$$= 4,\ d\ 1{\cdot}225\sqrt{\frac{4}{3}} = 1{\cdot}410,\ d = 1{\cdot}455$$

$$b = 5,\ d\ 1{\cdot}225\sqrt{\frac{5}{3}} = 1{\cdot}581\ d = 1{\cdot}665$$

$$= 6\ d\ 1{\cdot}225\sqrt{\frac{6}{3}} = 1{\cdot}732\ d = 1{\cdot}855.$$

The foregoing calculations are, however, far too cumbersome to be practically useful, wherefore we recommend the following method of designing a joint, circumstanced as above.

Let Σ be the strain to which the joint will be subject, then will, for one rivet only in the joint,

$$\Sigma = 3{\cdot}1416\;d^2$$

therefore

$$d = \sqrt{\frac{\Sigma}{3{\cdot}1416}},$$

but if the joint contains m, rivets for one,

$$d = \sqrt{\frac{\Sigma}{3{\cdot}1416\;m}},$$

and the dimensions of the plate may then be found from the equation, if b = breadth of the plate, $\Sigma = 5\,b\,t - 5\,d\,t\,m = 5\,t\,\{b - m\,d\}$

$$= 5\,t\left\{b - m\sqrt{\frac{\Sigma}{3{\cdot}1416}}\right\}.$$

Let $\Sigma = 30$ tons, $b = 15$ inches, and $m = 6$.

Then will

$$a = \sqrt{\frac{30}{18{\cdot}8496}} = 1{\cdot}26 \text{ inches nearly.}$$

By a transformation of the above formula, we have

$$t = \frac{\Sigma}{5\,\{b - m\,d\}}$$

which in the present case becomes

$$t = \frac{\mathfrak{z}}{5\,\{15 - 6 \times 1{\cdot}26\}} = 0{\cdot}806 \text{ inches,}$$

we may, therefore, call the diameter of each rivet $1\frac{1}{4}$ inches, and the thickness of the plate $\frac{13}{16}$ of an inch.

Let us see the ratio of the strength of this joint to that of the solid plate.

The strength of the joint is

$$\mathfrak{z} = 30 \text{ tons,}$$

that of the solid plate is

$$S = 5\,b\,t$$

$$= 5 \times 15\,\frac{13}{16} = 60{\cdot}93 \text{ tons,}$$

or the strength of the joints and plates bear to each other the ratio of

1 to 2.

Mr. Fairbairn has found from experiment that the strength of a simple rivetted joint is 0·56 per cent. of that of the solid plate, which is somewhat more than our calculation gives, but this may be accounted for, as we have not regarded the friction between the rivetted plates. We will now determine the amount of lap requisite for the above joint.

If the joint fails, by tearing pieces out of the edge of the plate, twice as many sections as there are rivets will be cut through.

Let l = the lap in inches.

The strength of the lap will be

$$S = 4 \times 2\,m\,t\,l,$$

but the strength of the joint must eqnal the strength of the lap, therefore

$$8\,m\,t\,l = 3{\cdot}1416\,m\,d^2$$

therefore,

$$l = \frac{3{\cdot}1416\,d}{8\,t} = 0{\cdot}3927\,\frac{d^2}{t}.$$

In the above case the lap will, therefore, be

$$l = 0{\cdot}3927\,\frac{1{\cdot}25^2}{\frac{13}{16}} = 0{\cdot}755 \text{ inches,}$$

this is, of course, the least theoretical area, and it may for safety be measured from the outside of the head of the rivet.

The diameter of the head of the rivet will be about twice the diameter of the centre of the same.

Let us now suppose that the strain was in compression instead of tension ; in this case the effective area of the plate will be that which bears upon the rivets.

Taking the given dimensions as in the above case, we will find the diameter of the rivets, which will allow a sufficient bearing surface, which will be found from the expression

$$S = 4\,m\,d\,t,$$

wherefore,

$$d = \frac{S}{4\,m\,t}$$

but in the above case,

$$S = \mathfrak{z} = 30 \text{ tons } m = 6,\ t = \tfrac{1}{2},$$

therefore,

$$d = \frac{30}{12} = 2{\cdot}5 \text{ inches.}$$

in this case it is evident that we must employ more rivets, or thicker plate, or the sectional area will be much in excess of that which is necessary; in fact, the use of rivets of the above dimensions would give four times the sectional area of rivets required.

Let the plate be 12 inches wide, with 6 rivets, and 1 inch thick, then,

$$d = \frac{30}{24} = 1{\cdot}25 \text{ inches.}$$

A careful observation of these results shows that the single rivetted lap-joint is peculiarly unsuited to resist compressive strains. The ratio between the strength of the joint and that of the solid plate will be as follows:—

The strength of the joint is 30 tons, that of the solid plate is

$$S = 4\,b\,t = 48 \text{ tons},$$

being in the ratio of 2·5 to 4.

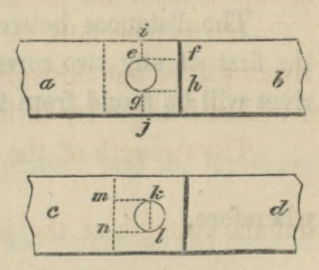
We shall not enter fully into an examination of the various forms of lap-joints, but will merely consider such as are most suitable to the purposes of which we treat.

From the foregoing calculations it will be observed that we cannot, by using a single row of rivets, communicate the entire strength of the plate itself through the joint, but we will now endeavour to show the manner in which this may most conveniently be accomplished.

We shall now assume that the joint is to be made equally strong with the solid plate, but before proceeding with our calculations, it may be desirable to call the reader's attention to the different modes in which a rivetted or bolted joint resists tension and compression.

Let *a b*, Fig. 113, represent a joint in tension, and *c d* the same joint in compression; as the upper and under plates will be similarly affected, we will consider the former only.

Fig. 113.



In the first case *a b*, it is a matter of observation that the strains are as follows:—

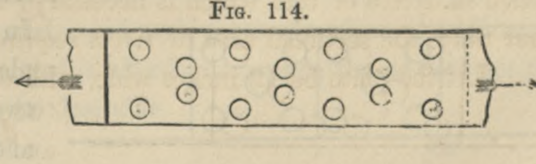
The section of effective area of the plate, is at the lines *i e*, *g j*, equal to the area of the plate, minus the loss by the rivet-hole, and there is a shearing strain along the section *e f*, and also along the section *g h*. In the case of *c d*, the effective sectional area of the plate is at the line *k l*, and is equal to the loss of area by the rivet-hole, which is the diameter of the rivet-hole, multiplied by the thickness of the plate. Shearing strains pass along the sections *k m*, *l n*.

The strain on the rivet, is in both cases, a shearing force. It may be imagined by many, that the friction between the rivetted plate, adds somewhat to the strength of the joints, and there is no doubt that such is the case, and, in fact, it has been experimentally proved that such addition to the strength does exist, when the joint is first made, for it has been found, that at the time when two pieces of iron have been freshly rivetted together, that the greatest strain to which such a joint would be subject, does not amount to that which will cause the plates to slide upon each other.

Calculations have been made with regard to the amount of friction created by the tension of the rivets, but as this is found to be about 20 to 25 tons for every sectional inch of area of rivets, we immediately perceive how devoid of utility this extra resistance is, for it is tolerably certain, that the iron would stretch considerably at from 13 to 15 tons per square inch, wherefore the contraction of the rivet in cooling, would be but partially employed in adding to the strength of the plate, the remainder of the force thus produced being consumed in permanently extending the iron. Although we do not purpose taking into account, the addition of strength to a joint, by friction of the plates which are joined, we will here observe, that we consider it highly probable, that the tension of the rivets would amount to from nine to eleven tons per square inch, but certainly not to more. Our reason for objecting to any reliance being placed on this source of strength, arises from a conviction that it is very liable to become reduced and even destroyed entirely, after the structure has been subject to vibration for some time. The amount of resistance that would be obtained, from any given pressure produced by tension of the rivets, will depend upon the condition of the surfaces of contact, but will in no way be affected by the extent of these surfaces.

Let us suppose that a joint is to be constructed of the form shown at Fig. 114, to resist a tensile strain of 50 tons. There will be 14 rivets in this arrangement, therefore the shearing strain upon each will be

Fig. 114.



$$\frac{50}{14} = 3{\cdot}57 \text{ tons nearly.}$$

We will for the sake of safety, call this 3·58 tons. Then to find the diameter of each rivet, we have,

$$x = 3{\cdot}58 = 3{\cdot}1416\, d^2,$$

therefore,

$$d = \sqrt{\frac{3{\cdot}58}{3{\cdot}1416}} = 1{\cdot}139 \text{ inches.}$$

Let the plate be 16 inches in width, then on any line of rivets, we have a loss of width

$$= 2 \times 1{\cdot}139 = 2{\cdot}278 \text{ inches,}$$

making the least effective width,

$$= 16 - 2{\cdot}278 = 13{\cdot}722 \text{ inches;}$$

the thickness of the plate must therefore be,

$$= \frac{50}{5 \times 13{\cdot}722} = 0{\cdot}729 \text{ inches,}$$

and the strength of the solid plate will be,

$$= 16 \times 5 \times 0{\cdot}729 = 58{\cdot}32 \text{ tons,}$$

the ratio between the strength of the joint and the solid plate will, therefore, be

as 25 to 29·16 nearly,

as 5 to 6.

The distances between the lines of rivets must not be less than the lap required for the rivets in the first of any two rows. Call the thickness of the plate $\frac{3}{4}$ of an inch, then the lap required by any rivet will be found from the expression,

$$S = l\,t$$

but S = 3·58 tons,

wherefore,

$$l = \frac{3{\cdot}58}{6} = 0{\cdot}596 \text{ inches.}$$

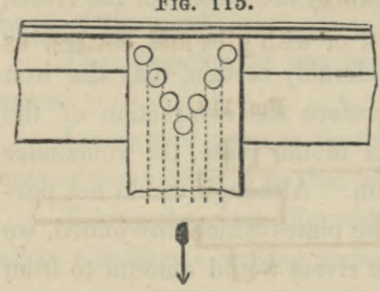
say $\frac{5}{8}$ of an inch.

It is very evident that when a joint is subject to a tensile strain, it should be thinner than when the strain is compressive.

If the strain is compressive, the rivets must be of the same diameter; and it is evident that we may arrange them so as to get the total area of the plate as the effective area.

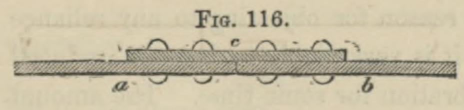
The form of joint which theory indicates as being most economical, is that shown at Fig. 115, which is suitable for the attachment of the diagonals of triangular girders to the flanges of the same. The joint is supposed to be subject to a tensile strain in the direction shown by the arrow. If we suppose the upright tie to be divided into bars, as shown by the dotted lines, we may then consider that each rivet carries the strain transmitted by one of these bars. The same lap will be required in this case as would be in any other.

Fig. 115.



We will now pass on to the consideration of the form and strength of butt-joints with cover-plates.

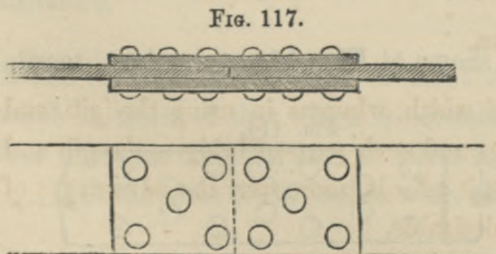
Fig. 116.



If two plates, Fig. 116, *a* and *b* be joined by one cover-plate, as at *c*, then the case is evidently identical with that of two successive lap-joints, and may consequently be treated as such.

It is, however, preferable to use two cover-plates as shown at Fig. 117. In this case each rivet must be sheared at two places before the joint can fail. Let there be a tensile-strain of 30 tons on the joint. We must regard it as a pair of consecutive joints, thus one plate is rivetted to two, which two are rivetted to the following one. In either joint we have five rivets passing through the main plate, and through two cover-plates; but but as each rivet has two sections of resistance, there are ten sections of resistance, and the strain on each will, therefore, be

Fig. 117.



$$= \frac{30}{10} = 3 \text{ tons.}$$

the diameter of each rivet will, therefore, be

$$d = \sqrt{\frac{3}{3{\cdot}1416}} = 0{\cdot}976 \text{ inches nearly.}$$

we will call this one inch.

The two rivets in the first row will take off $\frac{2}{5}$ of the strain, wherefore there will be no inconvenience from the loss of area by subsequent rivet-holes in the main plates: the loss from the first two rivets will be 2 inches in breadth. Let the plate be 12 inches wide, then its least effective width will be 10 inches, and its thickness must be

$$= \frac{30}{5 \times 10} = 0{\cdot}6 \text{ inches;}$$

say $\frac{5}{8}$ of an inch, then the strength of the solid plate will be

$$= 5\ \frac{5}{8} \times 12 = 37{\cdot}5 \text{ tons.}$$

wherefore the ratio of the strength of the joint to that of the solid plate, is

as 6 to 7·5.

The requisite lap for the main plates will be as before.

The thickness of each cover-plate must not be less than half that of the main plates; and the lap must be the same for both.

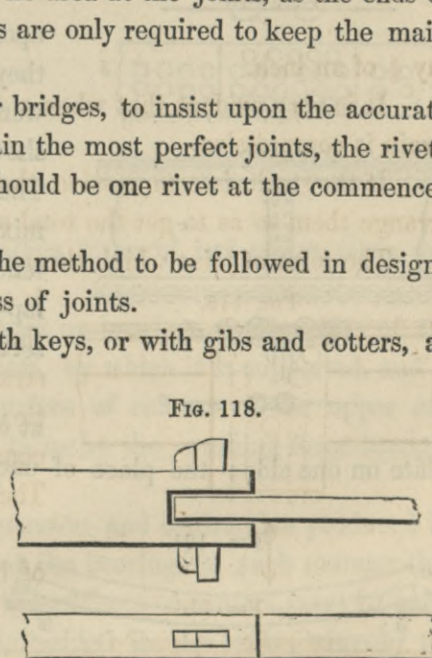
If the strain were compressive instead of tensile, we should lose no area at the joints, as the ends of the plates bear upon each other; therefore, the cover-plates and rivets are only required to keep the main plates in their proper relative positions.

We may here pause to observe the necessity when specifying for bridges, to insist upon the accurate construction of the butt-joints in compression. Theoretically, to obtain the most perfect joints, the rivets should be as small as possible, and proportionately numerous; there should be one rivet at the commencement of the joint, two in the second row, and in all those following.

The foregoing examples will doubtless be sufficient to illustrate the method to be followed in designing a rivetted-joint; let us therefore pass on to examine the next class of joints.

The joints of which we are about to speak, are those formed with keys, or with gibs and cotters, as shown in Fig. 118. In this form of joint we obtain a large amount of sectional area in the wedge or key; but at the same time cause only a very slight loss of the same to the pieces of metal thus jointed. Suppose we wish to connect two rods in this manner, the strain being tensile, and amounting to 6 tons. Let the gib and cotter each be $\frac{1}{2}$ an inch thick, then to obtain the requisite sectional area, the width of the two together must be

Fig. 118.



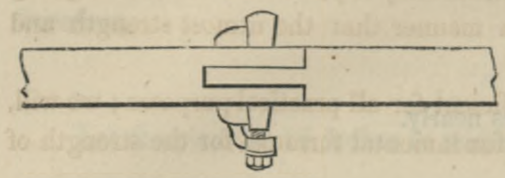
$$\frac{6}{4} \times \frac{1}{2} = 3 \text{ inches.}$$

Let us compare this joint with one of equal strength formed with a bolt or rivet, the diameter of the bolt or rivet would be

$$d = \sqrt{\frac{6}{3{\cdot}1416}} = 1{\cdot}9 \text{ inches.}$$

By using the rivet or bolt, we should lose nearly 2 inches of width, whereas in using the gib and cotter, we only lose $\frac{1}{4}$ this amount, or $\frac{1}{2}$ an inch; hence for bracings and such parts of bridges, the gib and cotter form a joint superior to that produced by the screw-bolt, and it affords, moreover, the advantages of admitting of the tightening-up of the joint whenever this may be desirable.

Fig. 119.



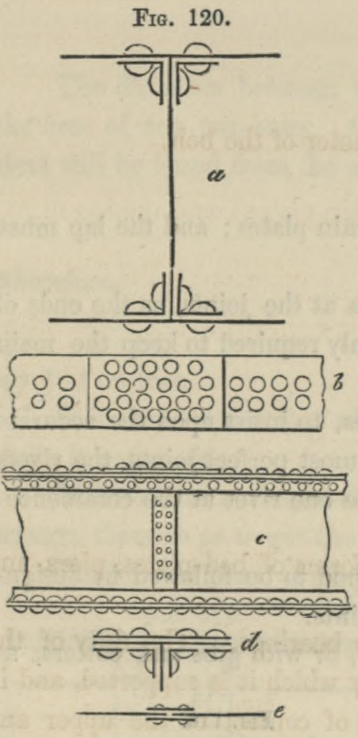
There are various ways of fixing the gib so that it shall not move through vibration or other disturbing cause, the end being sometimes split and a wedge inserted into it; but the method shown in the accompanying figure is by far the most certain way of securing it. The extremity of the cotter is prolonged, and terminates in an eye, which embraces a screw at the end of the gib, which latter having been driven tight into its place, is secured by a screw-nut as shown.

Having thus described separately the different kinds of joints, it remains for us to show the position and application of them to bridge structures; first, however, we will consider the importance of the offices which are filled by these portions of structures.

If a girder were made with welded joints, we might then regard it as without joint, and homogeneous throughout, but we must treat differently the case of a structure rivetted, or bolted together.

Let us examine the case of a plate-girder of very simple section, as shown at Fig. 120; *a* is a section, *b* is a plan, and *c* an elevation of part of the girder, including a joint. It will be observed, that all the joints shown in the figure, may be arranged under the two classes, single rivetted lap-joints, and butt-joints variously rivetted, with two cover-plates.

Fig. 120.

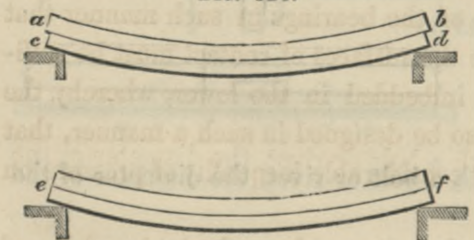


The rivets which attach the angle-irons to the flanges, are subject to a shearing strain, in the direction of the length of the girder, and the mode in which their resistance is brought into action, may be clearly shown by the following illustration:—

Let *a b*, *c d*, Fig. 121, represent two beams of equal length, laid upon each other, and supported at the ends; when deflected by a load, they will assume the position shown in the figure. If, however, the two were firmly bolted together, and loaded, they would assume the form shown at *e f*, the two beams being constrained to act as if they were one. The rivets connecting the web with the angle-irons, will have a somewhat mixed duty to perform, for as well as an action similar to the above, the tendency of the web to buckle must be resisted at this joint. These are lap-joints, and in case of their failure, each rivet will be sheared at one section only.

The joints in the top and bottom flanges, are made as shown in plan at *b*, and in section at *d*. The joint is a butt-joint, with an ordinary cover plate on one side; the place of one on the opposite side being replaced by the two angle-irons, and two narrow cover-plates. These joints are precisely similar in their mode of action, to the butt-joints described above.

Fig. 121.



The joints of the web are also butt-joints, with cover-plates on both sides, but the strain acts at right-angles to the length of the plates. The elevation of this joint is shown at *c*; at *e* is a horizontal section of the same joint.

We conclude, from the foregoing investigation of joints generally, that the cylindrical form, is not the best that can be adopted for a bolt or rivet, on account of the great loss of sectional area caused thereby in the plate, in proportion to the sectional area of the rivet; but that a square section is preferable, and also an elliptical section, provided that the longest diameter is in the direction of the strain.

Bolts, square under the head, are frequently used, and we think that elliptical rivets might also be applied for the junction of plates in which the holes are produced by the action of a punch. The heads of the rivets might be round as usual.

A perfect rivetted joint, that is to say, a rivetted joint, having equal strength with the solid part of a plate cannot possibly be constructed; hence, if we design a joint so that it is sufficiently strong, we may be sure that the plate on each side will have ample strength; hence, we may say, that the art of designing iron structures consists in arranging the various joints in such a manner that the utmost strength and economy may be obtained.

The examples of joints already selected, will doubtless, be sufficient for all practical purposes; we will, therefore, conclude the present Chapter; repeating, however, the fundamental formulæ for the strength of bolts, rivets and gibs :—

Let a = sectional area of bolt, rivet, or gib and cotter,
d = diameter of rivet or bolt (if it be round) in inches,
S = total direct strain to be withstood in tons,
m = number of sections which must be sheared if the joint fails.

Then,

$$a = \frac{S}{4\,m}$$

and,

$$d = \sqrt{\frac{S}{3{\cdot}1416\,m}}$$

The head of the rivet must be at least half the diameter in height.

The nut to fit on any screw-bolt should have a height equal to the diameter of the bolt.

CHAPTER XIV.

ON IRON PIERS AND FOUNDATIONS.

In the present Chapter we propose to examine and describe the various forms of bed-plates, piers, and foundations which are constructed wholly or chiefly of iron.

We will first turn our attention to the duties and form of bed-plates or bearings. The duty of the bed-plates is evidently to transmit the load upon the girders to the piers, by which it is supported, and in order to effect this properly the area of the bearing-surfaces, or surface of contact of the upper and lower plates, must be of sufficient extent to enable the bed-plates to resist the crushing force brought upon them.

The bearings must also be constructed so as to allow for the expansion and contraction produced by changes of temperature, and this is effected by forming one or more of the bearings in such manner that the top bearing-plate may move upon the lower one, and in this case the surfaces of contact must be sufficiently great, that to obviate the danger of the upper one becoming imbedded in the lower, whereby the freedom of motion would be diminished; and the bearings should also be designed in such a manner, that the deflection of the girder under a given load, shall not produce any considerable inequality of pressure on the various parts of the surfaces of the bed-plates.

The most simple bedding for a girder consists in attaching it to the piers without the interposition of bearing-plates, according to the method shown at Fig. 2, Plate LXXI., and the insertion of timber between the girder and the pier, as here shown, is also desirable, as by its elasticity it tends to deaden the effect of percussive force, and to equalise the pressure upon the piers. This method of construction cannot, however, be applied to both extremities of the girder on account of the necessity for making allowance for expansion, and in order to effect this the other end of the rib is carried upon rollers, a bearing-plate being attached to the bottom of the girder and another to the top of the pier, the rollers being placed between them. The extremities of the rollers are furnished with turned journals, capable of revolving in notches in strips of metal, by means of which the rollers are retained at their proper distances from each other. This bearing is illustrated at Figs. 1, 10 and 11, Plate LXXI.

Another form of fixed bearing is shown, Plate LXIX.; it consists of two cast-iron plates, one of which is secured to the top of the pier and the other to the underside of the girder; in these Plates corresponding grooves are made, which may serve to admit the heads of the bolts by which they are retained in their respective positions.

We will now call the reader's attention to some bearings, in which allowance has been made for the alteration of position produced by the deflection of the girders; the structure to which these bearings are employed is the Jumna Bridge, and they are illustrated at Plate XLV.

In these bearings, besides the rolling movement above-mentioned, there is another supplied by an arrangement of the following form.

The plate which rests upon the rollers is furnished on its upper side with a semi-cylindrical projection, or camber, and to the bottom of the girder is fitted a block, in which is a recess forming a counterpart to the cylindrical projection mentioned above, so that the former may revolve through a small space upon the latter, thereby preventing the unequal pressure upon the bearing rollers, which would otherwise be produced by the deflection of the girders.

There are some particular forms of bed-plates, which are required for structures of a peculiar character, but it is unnecessary here to describe them, as those forms to which we have already called the reader's attention may be regarded as types of those which are most generally employed in practice.

We will now proceed to consider the proportions which should be given to the piers, so that the greatest economy may be obtained for any given height and span.

We may here call the attention of our readers to the importance of properly proportioning the spans (when more than one are used), to the height of the bridge from the foundations, as the piers are very frequently of primary importance as regards cost.

The weight of the superstructure will vary as follows:—The depth of the girders always bearing the same proportion to the span, and the total length of all the spans being constant, this length being divided into such number of equal spans as may be most economical, and the breadth constant; let w = load per foot-run, l = length of one span or distance between two piers measured from centre to centre; d = depth of girder; W = total weight of the structure only. The weight of the main girders will vary as the sectional area, or as the strain on the flanges, supposing the web and stiffeners to be be proportioned to these, or the weight of the two girders will

$$= \frac{w\,l^2}{8\,d} \times a$$

where a is a constant, but the value of

$$\frac{l}{8\,d}$$

is constant by the hypothesis, therefore the weight of the man girders will be

$$= w\,l\,a,$$

the weight of the platform will be constant, as it will have to carry a certain length of load upon cross-girders of constant length, hence calling W′ the weight of the whole platform, we have,

$$\mathrm{W} = \mathrm{W}' + w\,l\,a,$$

and supposing the weight of the main girders to bear such proportion to the total load, that the deviations of load produced by altering their dimensions may be safely neglected in practice, we may assume w as constant, a sufficiently high value being assigned to it, and the only part on which a saving may be effected in the superstructure will vary as l. Some idea of the value of this part of the superstructure, compared with the remainder, may be formed of the fact, that in a number of wrought-iron bridges, we have found the weight of the main girders to vary from two-thirds to three-quarters of the total weight of metal, in the superstructure.

From these remarks, we conclude that it is desirable to adopt spans as small as possible; but in diminishing the distance between the piers, we shall be limited by circumstances which we will now consider.

In designing the piers, the first point to be considered is the area required to support the weight of the structure and load. As the weight of the structure acts directly upon the piers without experiencing any deviation from its normal direction, the area to support it will vary as the load supported, hence the amount of metal in the piers for a certain length of bridges will be constant, no matter in what number of piers it may be distributed. From these remarks, it appears that we may form the bridge of any number of spans without requiring any corresponding variation in the amount of metal in, or cost of the piers; but such is not, practically speaking, the case; and we shall find that there is a limit to the number of piers which may be used for a given length of bridges.

The limit may be determined in the following manner : having decided the area of bearing of each pile, the number of piles requisite to impart sufficient stability to one pier must be decided; then, dividing the total bearing area required by the total bearing area of one pier, we find the number of piers for a minimum bearing surface. By using a greater number of piers, we shall shorten the spans, and thereby effect a saving of metal on the superstructure; but at the same time, we shall involve an extra expenditure of metal in the piers; hence it remains for us to determine whether in any case which may be under consideration, the loss or saving is greater, and we shall ultimately find a minimum gross weight for the piers and superstructure together; or, by other calculations, we may find a minimum gross cost which is the desideratum.

We may give the following example of the method of adapting the spans of girders to the height of the piers. We will assume the main girders to constitute $\frac{2}{3}$ of the total weight of the superstructure.

Let us suppose the height of the superstructure from the ground to be about 60 feet, and let the total length of the bridge be 1200 feet, then for spans of 150 feet each we shall require 9 piers, each of which may cost about £1400; the total cost of the piers will, therefore, be about £12,600, and on the same data the cost of the superstructure will be about £5000 per span, or for the total length £40,000, the cost of the structure being £52,600. Let us now determine the cost at the reduced span of 100 feet, supposing that each pier cannot be reduced in dimensions (which is not in the present case correct, but which we will assume). We shall then require 13 piers, the cost of which will be £18,200. We must now determine the cost of the superstructure. One-third of this cost will be constant for the entire length, and the other $\frac{2}{3}$ will vary as the distance between two of the piers. The cost of the constant part of the structure will be £13,333, and that of the remaining $\frac{2}{3}$ of the superstructure,

$$=\frac{100}{150}\times £26,667 = £17,778;$$

adding the three costs together, we find that the total cost of the bridge will be,

$$18,200 + 13,333 + 17,778 = £49,311,$$

showing a saving of £3289, for though we lose £5600 on the piers, we save £8889 on the superstructure, and

$$8889 - 5600 = £3289.$$

It will, however, be evident, that as the weight of the superstructure will be reduced, less bearing area will be required, and also each pier having a less weight to support, the width of base may be reduced.

We will not now dilate further upon this subject, but will refer our readers for more information to our description of the bridges erected on the Bombay, Baroda, and Central India Railway, where tables of relative cost for various spans and heights will be found.

We will now pass on to consider the various forms of iron foundations, commencing with the simplest. Many years since, an engineer proposed to employ cast-iron for piles, to be driven in the same manner as wood-piles, by the ordinary pile-engine; but careful calculation showed that the impact of the monkey used to drive such piles, would, in all probability, fracture them, and this conclusion caused him to abandon the idea. Subsequently, an attempt was made to employ cast-iron piles on the Eastern Counties Railway, and at other places, but on account of breakage they were found to be very expensive; this breakage, however, is now prevented by placing a piece of wood endways of the grain, upon the top of the pile to be driven.

Cast-iron piles of various forms are now frequently used, and at the New Westminster Bridge the foundations are cased in cast-iron plates, which are retained in position by the following means, illustrated in Plate No. 23.

Cast-iron piles of cylindrical form, but with two grooves in a longitudinal direction, were driven, and between the piles were placed plates of cast-iron, the edges of which passed into the grooves, whereby they are supported.

Within the casing thus formed, concrete was put, upon which the masonry of the pier rests.

It is unnecessary here to occupy our readers with any further account of the various forms of cast-iron piles which have, from time to time, been used generally for some special purpose.

In putting together foundations under water, the aid of divers is frequently required; we will, therefore, before proceeding further, explain the means adopted for working conveniently under water. The diving-bell was first used for sub-aqueous operations, but this method of proceeding is attended with many disadvantages; thus, for instance, it is desirable that the divers should move with more freedom than can be obtained in this manner, and, therefore, for most purposes the bell is superseded by the diving-dress, the ordinary form of which is like the diving-bell, too well-known to require any description at our hands; but we will here explain a few improvements of a valuable character, made in diving-dresses, by Mr. J. W. Heinke, whose dresses were used by the divers employed on the foundations of the New Westminster Bridge. Among the most prominent of these improvements is that of the eye-glass frame, to which is affixed a brass-slide, so contrived that in case of accident to the glass, the diver can immediately close it, and thus save himself from drowning. A double valve, fixed in front of the gorget, enables the diver to descend and rise at pleasure with the whole of his apparatus, of which the weight is about 200 lbs.; it also affords him protection in case of anything happening to the air-hose, as by its means a sufficient quantity of air to support respiration for ten minutes can be supplied to the helmet, thus giving time to ascend, even from a very great depth. The joints are manufactured with a double safety-cap, whereby they are enabled to withstand a very great amount of pressure. The new vulcanized india-rubber band completely excludes the water from the dress, and enables it to fit more easily, and with greater comfort to the wearer. A signal dial is also attached to make known the wants of the diver to those above. Mr. Heinke has also devised a lamp for sub-aqueous operations, whereby the manipulations of the diver may be facilitated exceedingly. This apparatus has been tested at various places, the results of the experiments being exceedingly satisfactory. We may here observe the relative costs of this diving-dress and the bell, the latter would cost about £20 per week, whereas the former would cost but £7 per week; and the amount of work executed is much greater with the dress than with the bell, so that by using the former a saving of about 75 per cent. is effected. We will conclude this description with a list of some of the places at which this dress was used, and of the engineers by whom it has been adopted. Ayr and Calder Railway, Bombay and Baroda Railway, Imperial Messagerie Co., Peninsular and Oriental Co., Paris and Orleans Railway, Dover Breakwater, North Eastern Railway, Victoria Docks, East and West India Docks, St. Katherine's Docks, etc. Adopted by Messrs. Bidder, Hawkshaw, Kennard, Smith and Knight, Walker, Peto, Brassey, and Betts, Furnis, Manby, Harris, Royal Engineers, etc.

We will now return to the consideration of the various kinds of iron foundations ordinarily used.

The first foundation of which we shall here speak, is the common iron cylinder foundation, used at Charing Cross, Staines, and other bridges. Cylinders of various diameters are used for this kind of foundation, such as 6 feet, 10 feet, etc.

These cylinders may be sunk as follows; being placed in position, the earth is removed from within the cylinder, which is sunk by loading it with weights, the earth is removed by scoops, or spoons furnished with sharp edges and long handles, by means of which they may be worked from above the surface of the water, through which the cylinders are being sunk. The centre part of the spoon is a leather bag. The cylinder is weighted, so that it may sink as the stratum of earth is excavated, and the weight should be at least equal to the load which the cylinder will have to support, as otherwise, it may sink after the erection

To face Page 163.

IRON FOUNDATIONS.

HUGHES' PNEUMATIC APPARATUS
FOR SINKING
IRON CYLINDER PILES.

FIG. 1.
General Section

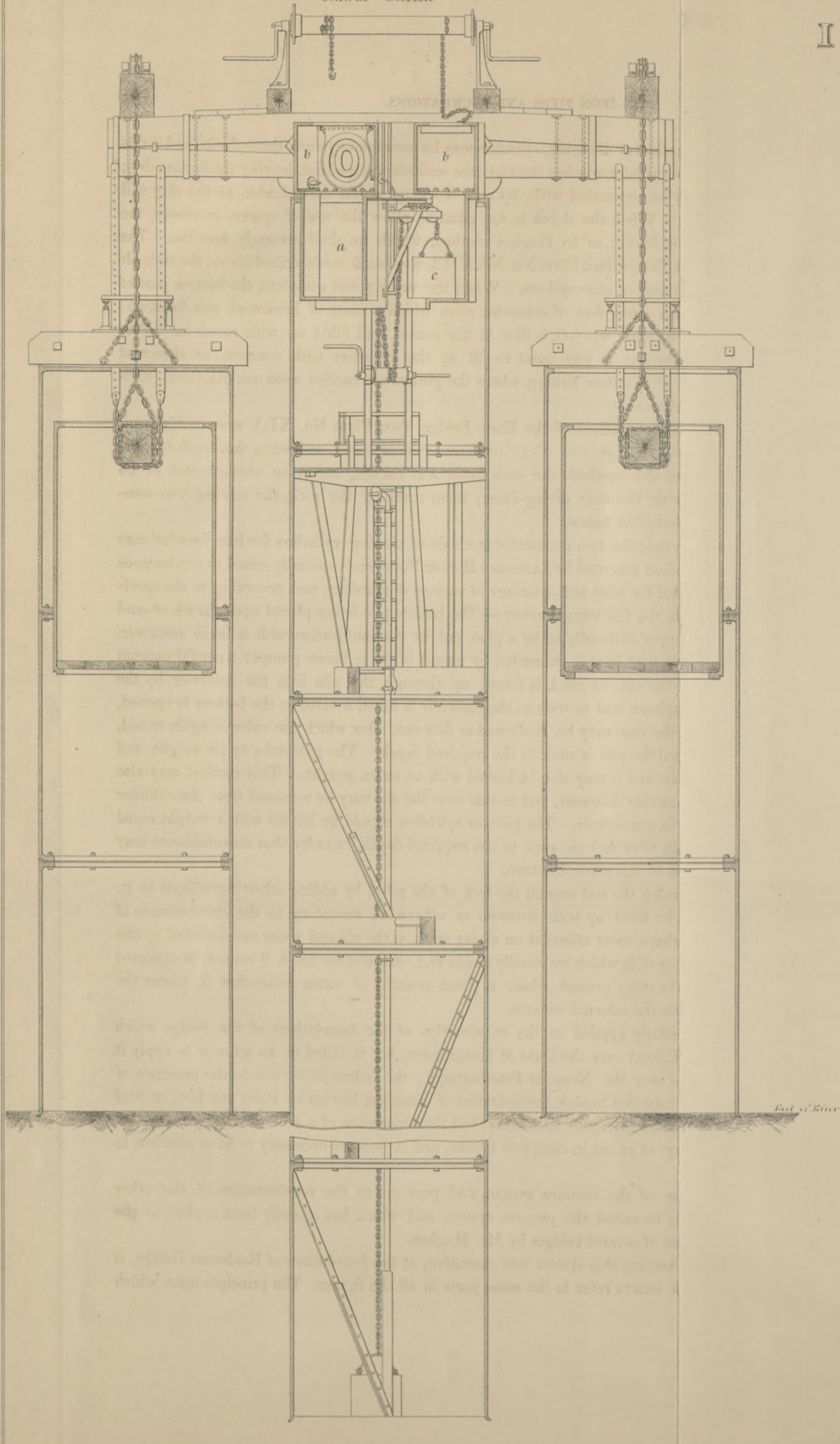


FIG. 2.
General Plan

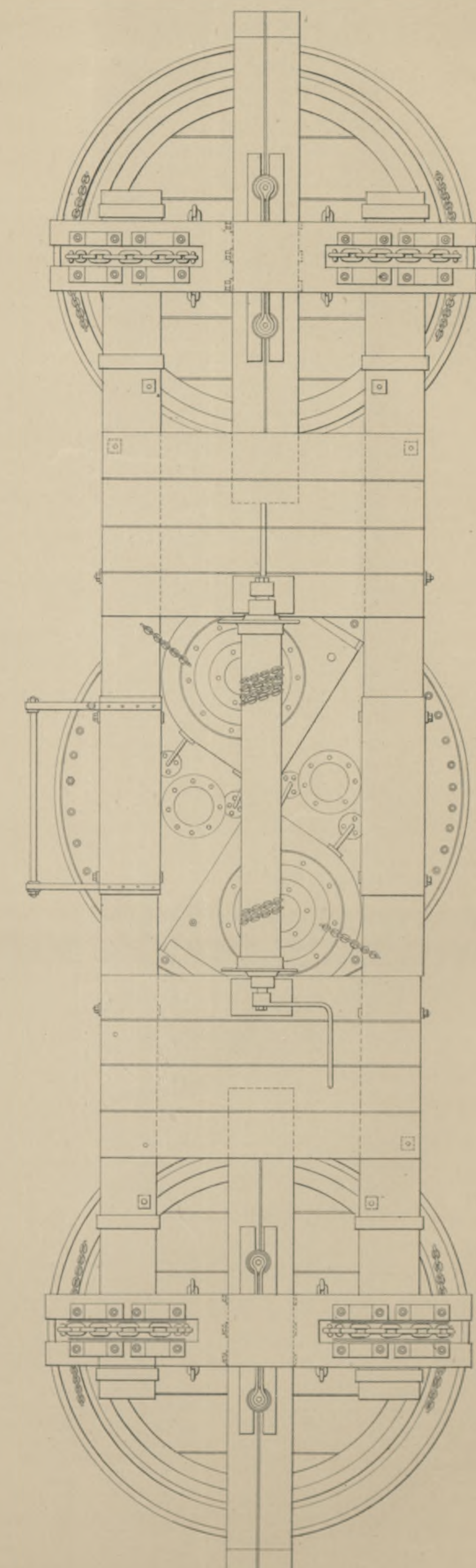


FIG. 3.
Sectional Plan at A. A.

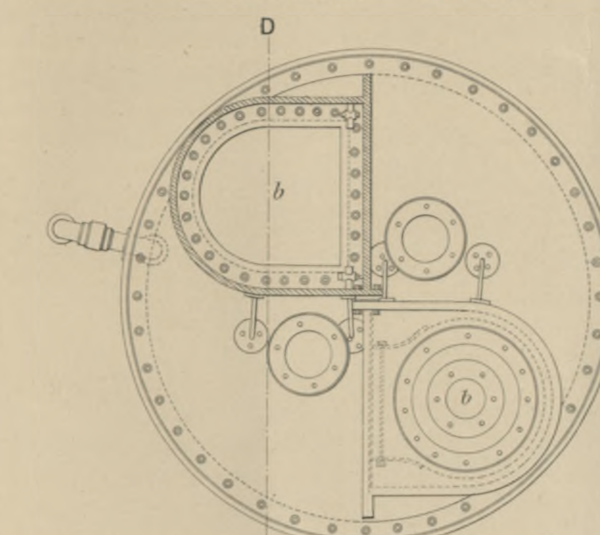


FIG. 4
Sectional Plan at B. B.

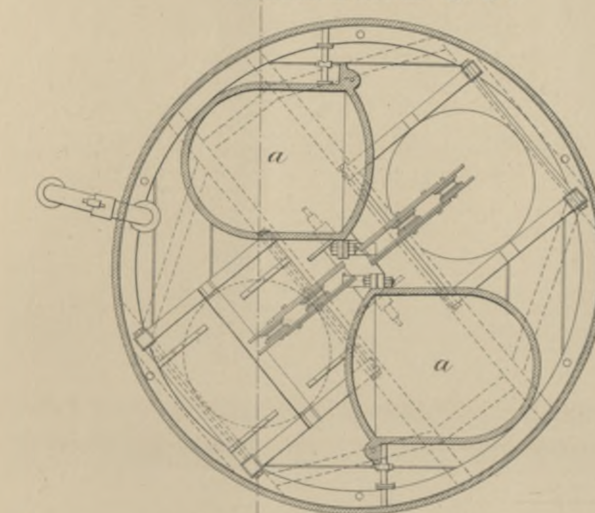


FIG. 5.
Sectional Plan at C.C.

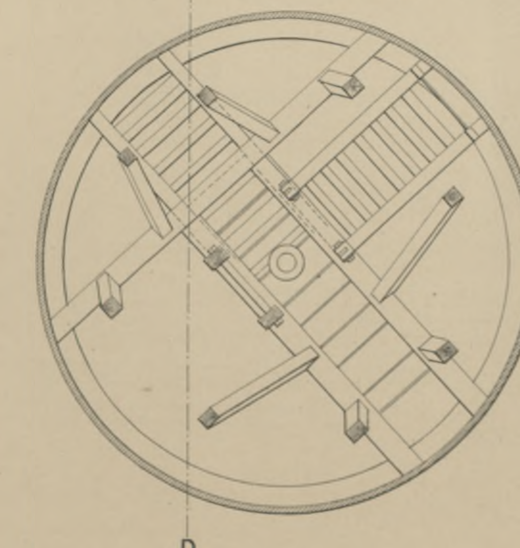
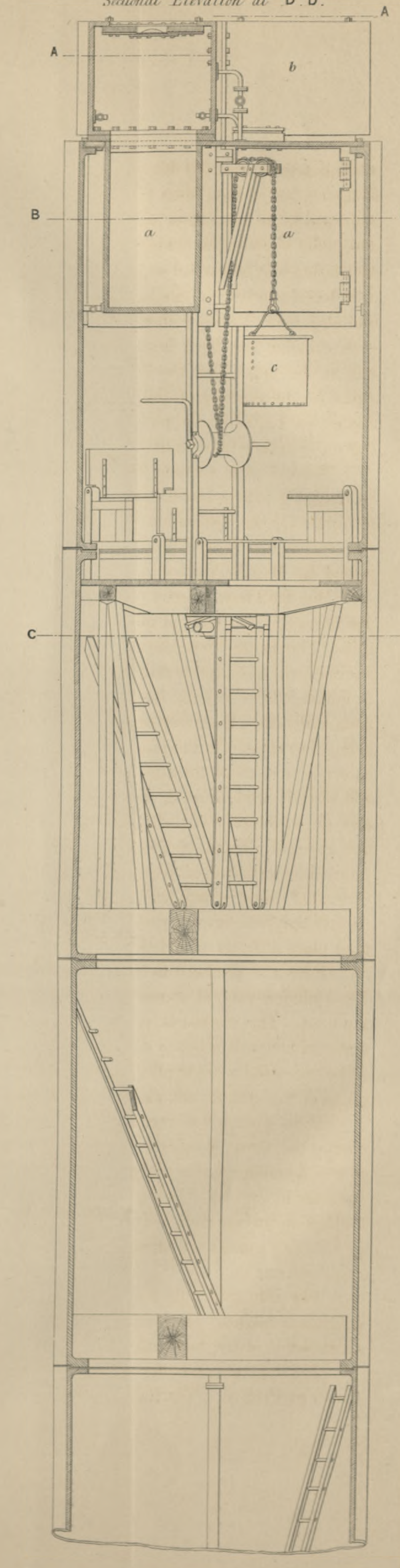


FIG. 6.
Sectional Elevation at D. D.



Scale for Fig 1 – ¾ Inch to 1 Foot

Ins 12 9 6 3 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 Ft

Scale for Figs 2.3.4.5 & 6 – ⅜ Inch to 1 Foot

Ins 12 9 6 3 1 2 3 4 5 6 7 8 9 10 Ft

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Standidge & Co. Litho. Old Jewry

of the superstructure. By this method of dredging, cylinders may be conveniently sunk through silt, and if this be upon a bed of clay, when the cylinder reaches the clay, the water is pumped out, leaving the bottom dry; after which the excavation may be proceeded with, by men working within the cylinder, as the clay will effectually exclude the water. When the depth is too great to allow the use of spoons or scoops, the excavation may be carried on by divers: or by Hughes' method, which we shall presently describe. The cylinders supporting the Staines Bridge, (see Plate No. XXXII.), were sunk, as described above, through silt and clay, being loaded with 80 tons of railway-bars. When the sinking was complete, the bottom part of each cylinder was filled in with a few feet of concrete, upon which a mass of brickwork was built up; but at the upper part the brickwork was left hollow at the centre, and filled up with concrete. Previous to the erection of this bridge, it was usual to fill up the cylinders with concrete, as shown at Plate No. LXXI. in the piers of Windsor Bridge, where the girders are carried upon oak platforms, resting on cylinders filled with concrete.

The cylinders forming the foundations of the Ebro Bridge, (see Plate No. XL.), were partly sunk by dredging with a spoon, at an average cost of £5 per lineal foot of depth, and when the depth became too great for the application of this method, the excavation was carried on, at an average cost of £4 per foot, by divers furnished with Heinke's diving-dress; upon reaching the rock, the sinking was completed by Hughes' method, described below.

We shall now proceed to describe two pneumatic methods of sinking cylinders for iron foundations; the first being the vacuum method patented by Lawrence Holker Potts, and especially suited to argillaceous soils. This method was intended for piles and cylinders of various dimensions, and, according to the specification, was to be conducted in the following manner:—The hollow pile being placed upon the silt or sand in which it is to be sunk, its upper extremity is, by a pipe, put in communication with a large reservoir, from which the air already contained in it is exhausted by means of three throw pumps; a partial vacuum being thus created in the pile, the silt, or sand, is forced up through the pile into the reservoir by the external pressure of the atmosphere, and as soon as the reservoir is filled, a valve at the bottom is opened, and the silt, mud, or sand, as the case may be, is allowed to flow out, after which the valve is again closed, and the operation repeated until the pile is sunk to the required depth. The pile sinks by its weight, and the external pressure of the air, and it may also be loaded with an extra weight. This method may also be used for cylinders of considerable diameter, and in this case the silt may be removed from the cylinder instead of being accumulated in a reservoir. The piles or cylinders should be loaded with a weight equal to that they will have to sustain after they are sunk to the required depth, in order that no subsidence may take place after the completion of the superstructure.

Potts also proposed to harden the soil around the feet of the piles, by adding other ingredients to it. The cylinders may, of course, be filled up with concrete or otherwise, according to the circumstance of the case. This method is, perhaps, most effectual on clayey soils, as the air and water are excluded by this material better than by any other with which we usually meet; and, on the other hand, it cannot be expected to be successful when applied to stony ground, where a great quantity of water would flow in under the edges of the cylinder and vitiate the internal vacuum.

Potts' system was successfully applied to the construction of the foundations of the bridge which carries the Great Northern Railway over the Ouse at Huntingdon, but it failed in an attempt to apply it to the foundations of a bridge over the Nene at Peterborough; the failure being due to the presence of boulders in the clay. It was intended to sink the cylinders of Shannon Bridge by Potts' method, as trial borings showed the river-bed to consist of stiff bluish clay, but the occurrence of a great number of boulders in the clay caused the attempt to result in complete failure, and it became necessary to have recourse to other systems.

We will now take leave of the vacuum system, and pass on to the consideration of the other pneumatic method, which may be called the plenum system, and which has recently been applied to the construction of the foundations of several bridges by Mr. Hughes.

The apparatus used for carrying this system into execution, at the foundations of Rochester Bridge, is shown at Plate E. The italic letters refer to the same parts in all the figures. The principle upon which this method is based, is that of the diving-bell; the water being prevented from entering at the bottom of the cylinder to be sunk, by keeping it full of compressed air. The top of the cylinder is of course closed. At the bottom of the cylinder thus circumstanced, men are employed in excavating the soil; and, as the excavation proceeds, the cylinder sinks, being loaded on the top. Some means are required to allow the workmen to enter or leave the cylinder, without the necessity of refilling it with air after each entrance or exit; and such means are supplied by two air-locks, precisely similar in principle to the ordinary canal locks; through these air-locks the excavators can pass, and the soil removed is also carried through the air-locks. We must now refer to the illustration, Plate E. Fig. 1 shows a general section of a cylinder, 7 feet in diameter, being sunk by Hughes' method. It will be observed that the cylinder is fitted with stagings at every joint, the stagings communicating with each other by ladders, which enable the workmen to pass from the top of the cylinder to the bottom, and *vice versa*. Tackle is also provided, by means of which, the bucket *c* when filled with excavated soil may be raised to the top of the cylinder. *a b* are the air-locks, furnished with two doors each; the upper one opening into the lock, and the lower one into the cylinder, and so arranged, that the pressure of the contained air shall tend to keep them closed. The action of the air-locks is as follows:—An accurate idea of the form of the air-locks having been obtained by inspecting the elevation and section in Fig. 6, the horizontal sections, *a a*, at Fig. 4, and the plan and horizontal section, *b b*, Fig. 3; let us suppose that the bucket, *c*, Fig. 6, is to be passed through the air-lock. In the elevation of the lower part *a*, on the right-hand side of the Figure is shown a door, hinged in the usual manner, communicating with the interior of that part shown in section on the left-hand side of the Figure. Before this door can be opened, the pressure within the air-lock must be made equal to that in the cylinder, and this is effected by opening a stop-cock in a pipe communicating with the lock and cylinder. The pressure being thus equalised, the door is opened, and the bucket, *c*, passed into the lower part of the air-lock; the door is then closed, and the condensed air is allowed to flow out of the lock into the atmosphere, after which, the flap in the upper part, *b*, of the lock is opened, and the bucket is hoisted out.

In the cover of the cylinder, two stout glass lenses are fixed, to give light during the day to the men working the cranes, and air-lock doors. Light is also admitted in a similar way to the chambers of the air-lock. It will be observed, that the air-lock is so constructed that it may be easily fixed upon the top of any cylinder. By means of this apparatus, any kind of soil may be penetrated; but it has its peculiar advantages when applied to rock bottoms, on which divers could not conveniently work, whereas by this method the excavators can operate without inconvenience, as soon as they become accustomed to the increased density of the atmosphere by which they are surrounded.

The piers of the Ebro Bridge were completed by means of Hughes' system, which was also successfully applied to the cylinders, of which the piers of the Shannon Bridge are formed. The cost of sinking in the former case was generally about £15 per foot, but in some of the rock it amounted to £50 per foot.

The next kind of iron-foundation to which we shall refer, is that formed of columns or piles furnished at the lower extremity with belical flanges or screws, by means of which they are caused to penetrate the soil in which the foundation is made. In the first cases, piles thus formed were fitted with a gearing, and hauled round by men, but it has been found more convenient to screw them into the soil by means of bullocks or horses. The screw-piles work their way into the soil without materially disturbing it, and afford a considerable resistance to motion in either direction, vertically; when the force tending to produce it acts vertically, although they may be readily screwed out if necessary; they have been successfully applied to every kind of ground, with the exception of hard rock, and have been found to act very advantageously on chalk; they have also been made to penetrate to a depth of 12 feet in the coral-rocks of the Florida Reefs on the Coast of the United States of America. The screws used are of various forms and sizes, depending upon the circumstances under which they are to be used. Piles of this description, 2 feet 6 inches in diameter, were used for the piers of the Taptee Viaduct, Plate No. XXXVIII., and for various other bridges; there is little doubt that they will be very extensively used after the expiration of the patent, the royalty demanded being now a very serious obstacle to their general adoption. The royalty demanded on these piles, caused Mr. Brunlees to make some experiment with a view to arrive at some other form equally applicable to sand bottoms, and he finally determined to use the disc piles for the Kent and Leven Viaducts, as shown, Plate No. XLIX. Trial borings, on the site of the foundations of these viaducts, showed that the bottom, for a considerable depth, consisted of fine sand; for such a depth, in fact, that the sand below, where the tides and currents have any influence on it, would necessarily be the foundation upon which to build the piers. Experiments were first tried in order to determine the bearing-power of the sand, and to ascertain the form and size of disc best adapted for a permanent foundation. The experiments were tried, upon a bell-mouthed pile closed at the bottom; upon an open bell-mouthed flanged pile rammed full of concrete; upon a plain disc pile, open to the extent of the internal diameter, and filled up with concrete; and upon a disc pile with an orifice of 3 inches in diameter, in the centre of the disc. Of these forms, the last proved most satisfactory, and it was adopted. The piles, filled up with concrete, would probably have been equally efficacious, from the tendency of the concrete to consolidate the sand disturbed in sinking them; but on account of the small interior diameter of the piles, there was a difficulty in clearing out the water, and introducing the concrete. The average result of these experiments proved that the sustaining power of sand is about 5 tons per square foot, and as it was intended that each pile should carry at least 20 tons; the diameter of the disc was fixed at 2 feet 6 inches, giving a bearing area of 4·86 square feet on 24·30 tons sustaining power. The piles were braced together in order to impart sufficient stability and rigidity to the piers.

For sinking the piles, two pontoons were used, and the operations were conducted in the following manner :—

Each pontoon was fitted with a small donkey-engine of about 2-horse power, and with a piling-machine. The pontoons at the ebb of a tide were moored on the site of the intended pier. The pile was then placed in position and lashed at the top to the slide-block in the piling-machine, the bottom being " clipped " by the guiding-apparatus fixed to the side of the pontoon. The aperture in the bottom of the pile received the end of a wrought-iron pipe, about 2 inches in diameter, which, passing up through the pile, was connected by a flexible-hose with a pump worked by the donkey-engine. The donkey-engine being then started, water was thus forced down the pipe and out at the bottom of the pile, blowing out the sand from beneath the disc, and the pile sank rapidly, through the upper layer of sand to a distance of 7 or 9 feet; but below that depth, in the more compact marly-deposit, the sinking was slow, and in order to obviate this obstacle, the piles were cast with radial ribs or feathers upon the discs, by which, when a reciprocating rotatory motion was given to the piles, the material was loosened so as to be more readily blown out by the water. The operation of sinking was much facilitated by keeping a considerable part of the weight of the pile hanging upon the piling-machine, as otherwise the piles sank irregularly, and sometimes so rapidly as to overpower the pumps.

In sinking the piles, the sand beneath the discs was necessarily disturbed and loosened, and to consolidate this when the sinking was finished, each pile was ultimately driven down 2 inches by short blows from a heavy " tup."

We shall now conclude our description of iron foundations; first, however, calling the reader's attention to those of the piers used to support the Beelah and Deepdale Viaducts, shown in Plates, Nos. L. and LIII., where the columns of which the piers are formed, rest upon feet bolted down to masonry.

PART III.

DESCRIPTIVE.

CHAPTER XV.

CAST AND WROUGHT IRON ARCH BRIDGES.

In the present Chapter we purpose to describe as fully as the means at our disposal will allow, the four examples of arch-bridges, illustrated.

Rochester Bridge we have included under this head, notwithstanding the fact, that it partly consists of a wrought-iron straight girder swing bridge, because we deem it desirable, to class our bridges in accordance with their most distinctive features. Standish Bridge is entirely of cast-iron, and constitutes a very excellent example of an ordinary roadway bridge. Westminster Bridge has also been included among the arch bridges, as being in form, though perhaps not in principle, an arch; and it is probably the most elegant specimen of bridge construction now existing. The last structure which we here describe, is the Victoria Railway Bridge at Pimlico, a wrought-iron arch bridge, peculiarly remarkable for the extreme lightness of its construction, and the accuracy of the principle upon which it is designed. It is, however, unnecessary for us here to dilate further upon this subject, as a full description of it will follow, from which our readers will obtain any information they may require.

ROCHESTER BRIDGE.

PLATES I. to XV.

This bridge which crosses the Medway, and connects the important towns of Rochester and Strood, was designed by Sir William Cubitt, and replaced an old stone structure standing in a most inconvenient position, as the traffic from the main street of Rochester had to be diverted at a right angle to reach it, which objection also presented itself on the opposite side, with reference to the High Street of Strood. The site selected for it, is in a line with the principal streets already mentioned, and identical with the position of an ancient wooden bridge, which existed before the erection of the stone structure. It consists of a main bridge, and of a passage to admit masted vessels to the upper parts of the river, over which a swing bridge is placed.

The main bridge is of cast-iron, in length about 485 feet, and in width about 40 feet; it is composed of three arches, viz., a centre arch of 178 feet span, with a rise of 17 feet, and two side arches of 140 feet each, with a rise of 14 feet, each arch consists of 8 main ribs, which in the smaller arches are cast in 5 pieces, and in the larger one in six. The ends of these castings were planed by machinery made expressly for the purpose; they are connected by means of strong wrought-iron bolts, $2\frac{1}{2}$ inches in diameter, accurately fitted to their respective places. The ribs are firmly braced together with cast-iron frames fitted at intervals between them, these frames are fully detailed in Plate VI. The spandrils of the arches are filled in with ornamental cast-iron work, which also serve to carry the covering-plates of the roadway. The whole of the joints of the spandrils and ribs were truly planed, and the holes for the bolts drilled; the spandril girders are connected with the ribs by means of $1\frac{1}{2}$ inch bolts; the arrangements of these will be seen on reference to the plates. The whole is surmounted by a splendid moulded cast-iron cornice and parapet railing, the former intersecting with the moulded stone-work of the piers. The roadway and footpaths are carried by wrought-iron plates, bolted to the tops of the spandril castings, and are made perfectly water-tight with iron cement, these plates are overlaid with 3-inch planking, upon which is placed a granite pitching.

On the 10th June, 1856, the three arches were freed from their supporting centres, when the drop of the crown of the middle arch, did not exceed $\frac{1}{8}$ of an inch, showing the excellence of the joinings of the ribs, and the soundness of the beddings on the skewbacks.

The weights of ironwork in the bridge are as follows:—

	Tons.	Cwt.
Cast-iron	2515	0
Wrought-iron	25	10
Total	2540	10

In the swing bridge the most elaborate system of contrivance is resorted to, to relieve the work from all strain in its ordinary position, fixed for land traffic, and to attain the utmost durability in the working parts. It has a clear span of 46 feet, and is formed of wrought-iron rivetted-plate girders, of which there are 6 placed in direction of its length; the outside girders are 7 ft. $11\frac{1}{2}$ inches deep at one end, and 6 ft. $0\frac{1}{4}$ inches at the other, the 4 intermediate girders being somewhat less. The total length of the outside girders on the centre line is 105 ft. $5\frac{5}{16}$ inches, of the intermediate girders, 107 ft. $7\frac{7}{16}$ inches, and of the inner girders, 108 ft. $9\frac{5}{8}$ inches; the distance between the two inner girders is 6 ft., and between the others, 4 ft. 9 inches. These girders are intersected by boiler-plate cross girders, a row of plate passing on each side the centre pivot for extra strength, and elsewhere by cast-iron pipes; this method of strutting, which has the advantage of extreme lightness, give the requisite amount of solidity to the structure. The footpaths of the bridge project on T-iron cantilevers, 5 ft. apart, which are attached to flat joint covers. The bridge turns in the direction of the stream, the centre, or pivot being on the pier; the bearing points are the pivot, formed of properly prepared iron, and 30 wheels, or rollers, running on a roller path, 30 feet in diameter, which is accurately turned to the proper inclination; the rollers are fixed on the edge of a large horizontal wheel, which keeps them in position; the whole of the machinery employed in turning the bridge is fully detailed in Plates XI., XII., XIII., and XIV. Motion is given to the bridge by a chain, stretched round, and fastened to the roller path, by two men standing on the bridge; the gearing is so admirably constructed, and the whole weight so perfectly in equilibrium, that a very small force is necessary to open and shut the bridge, notwithstanding its total weight exceed 300 tons. At the outer end of the radius there is a stop to prevent the motion of the bridge too far laterally in reclosing, and at the same end there are a number of setting-up screws, to which motion is given by a windlass, for the purpose of taking off the weight which would otherwise exert a great leverage upon the machinery; there is also a horizontal stop to prevent these screws from forcing themselves upon the bridge too far. At the shorter end of the bridge, about 100 tons of stone is filled in the hollow parts under the level of the causeway, so as to secure a proper counterbalance. Taken as an entirety, the whole contrivance may be regarded as a triumph of engineering genius, and reflects the utmost credit alike on the designer, and those who have carried it into effect.

The weight of ironwork in the swing bridge are as under:—

	Tons.	Cwt.
Cast-iron	50	0
Wrought-iron	105	0
Gun-metal	1	5
Total	156	5

The river piers each occupy an area of 1,118 square feet, having a width of 17 ft. 8 inches, and a length between the points of the cutwaters of 70 feet; they stand upon cast-iron cylinder piles 7 feet in diameter, which are carried into the solid ground sufficiently below the bottom of the river to ensure a substantial support for the irresistent weight. The cylinder piles of the river pier are at central distances of 9 feet longitudinally, and of 10 feet transversely, there being 14 in each pier; in the abutments they are 6 feet diameter, the Strood abutment requiring 30, and the Rochester abutment 12; these piles consist of two, three, or more cylinders, 9 feet long, bolted together through stout flanges; the bottom length has its lower edge levelled, so as to facilitate the penetration of the ground; the piles were all pitched with the most studied accuracy, to ensure their being in a true position. In reliance upon a section of the bottom of the river, which indicated beds of soft clay, sand and gravel, interlaying the chalk; it was originally intended to force the cylinder piles into the ground by means of Dr. Potts' pneumatic method, but a few trials on the Strood Pier gave abundant evidence that the bottom there consisted of a mass of hard rock or stone, closely packed together, and as a mere action upon the piles, brought about by a partial vacuum, would be totally misapplied in such a situation; it was resolved upon to abandon this idea and to adopt Mr. Hughes' method, which was attended with complete success; this method consists in closing the top of the cylinders, and injecting air at a greater pressure than that due to the surrounding water, so expelling the water from within the cylinder, and enabling the workmen to enter and remove all material. As a detailed plate and full description of this method have been already given in our pages, we will here abstain from further allusion to it. The cylinders are all filled with concrete and brickwork, and are firmly bound together by cast-iron covering-plates, upon which the masonry of the bridge is placed.

The original contract for the two bridges provided for 2,500 tons of cast-iron, 35 tons of wrought-iron, 2,000 square yards of brickwork in Portland cement, 850 square yards of Portland cement concrete, 2,300 cubic feet of Bramley-Fall stone, set in Portland cement, 9,000 superficial feet of 6-inch granite landings for foot-paving, each stone being 8 feet in length, 5,000 superficial feet of 6-inch York landings, 4 feet in length; and 1,500 yards of Scotch granite-paving for the carriage-way. The masonry of the bridge was completed by Messrs. Fox, Henderson and Co., under the contract of Messrs. Cochrane and Co., of the Woodside Iron Works, near Dudley, and is of a most substantial character. The whole of the erection was under the superintendence of Mr. John Wright, the resident engineer, who discharged the onerous duties devolving upon him in a most able and satsfactory manner.

STANDISH BRIDGE.

PLATES XVI. AND XVII.

The line of the Gloucester and Stonehouse Railway of Midland Company being made parallel with and contiguous to the Great Western for a considerable distance, it became necessary to remove the then existing brick bridges over the line (four in number), and to erect other bridges over both lines of railway without interrupting the traffic on the Great Western.

The brick bridges which had to be removed, were each formed of one large arch, the abutments of which were placed in the slopes of the cutting. There was not sufficient room between the engine-chimneys and the soffit of the arch to admit of any ordinary form of centring, but the arches being of so large a span, it was deemed unsafe to attempt their removal without some effectual support being given during the process. It was therefore decided to use a suspending centring, which was formed of two strong beams, one placed at each face of the arch near the level of the under-side of the crown, and properly supported at their extremities, leaving a clear space of about 28 feet for the passage of the trains.

From these beams, laggings formed of half-baulk were suspended by means of iron rods, which laggings were brought up into contact with the soffit of the arch by screws cut on the suspending rods. The brickwork of each arch was then removed, working equally from both faces, so that when the arch had to be finally ruptured, the weight brought upon the suspended centring was comparatively small.

The bridges employed to replace the brick arches are similar in construction, so that a description of one of them, the Standish, will suffice.

This bridge, which was designed by Mr. W. H. Barlow, consists of ten cast-iron half-arches, resting on skew-backs in the masonry of the abutments; these arches are formed of one piece or casting, so as to avoid the necessity of centring, and are connected at the crown by means of four wrought-iron bolts. The bridge is placed on the skew, and has a clear span of 83 feet 4 inches, and a rise of 11 feet. The cast-iron ribs are firmly braced together by means of a strong cast-iron bracing, which will easily be understood on reference to the plates. Upon these ribs are placed the roadway-plates, which are 232 in number, and formed of cast-iron, having a thickness of metal of ½ inch, over which is laid the ballast 4 feet 8 inches in depth. A parapet-girder 10½ inches in depth, runs the whole length of the bridge, and is fastened to the main outside-rib by means of 1-inch bolts.

The parapet, which is of corrugated iron, is strengthened by the addition of ten cast-iron standards 14 inches in width, and is surmounted by a hollow capping 6 inches wide. The weight of cast-iron in the structure is 94 tons, and of wrought-iron 2·3 tons, making a total of 96·3 tons of metal.

The form of arch was (with a slight modification) a copy of one designed by Mr. Barlow for a bridge over the Trent, and in making that design, he instituted a series of experiments to ascertain the weight which might with safety be placed upon that form of casting.

For this purpose, a large model, $\frac{1}{6}$ full-size, was made, consisting of a pair of arches braced together in the same manner as the intended bridge, and loaded until they gave way.

The results of the experiments showed that, taking the minimum section exposed to compressive action, the strain amounted to 16 tons per inch at the time of rupture; also, that the fracture arose from distortion, the parts which yielded being the ribs and upper horizontal piece; but the arch remained entire.

Previous to erecting the bridge above described, the experiments were again repeated upon a model $\frac{1}{6}$ full-size, and with a precisely similar result as to the fracture of the ribs; but the strain was 17 tons per inch at the smallest section.

In the bridges over the Gloucester and Stonehouse and Great Western Railway, the area of section of the metal is such, that a maximum load produces a strain of 2 tons per inch.

NEW WESTMINSTER BRIDGE.

PLATE XVIII. TO XXIII.

This magnificent structure, now fast approaching completion, is being erected from the designs of Mr. Thomas Page, and more than sustains the reputation which that gentleman has already acquired. From the lightness of its appearance, the elegance and beauty of its design, and the massiveness of its construction, it cannot fail to be considered one of the handsomest structures that has ever crossed the waters of the Thames.

The bridge will consist of seven arches, with three small culvert arches for the passage of the water through the Surrey abutment. The general dimensions of the whole work are as follows:—Extreme length, 1,160 feet 3 inches, width 85 feet, span of largest or centre arch 120 feet, smallest ditto, 94 feet 9 inches, height of centre from Trinity high-water mark, 22 feet 8 inches, height from foundation 52 feet. The rise on the whole bridge is 5 feet 3½ inches. The immense advantages which the new bridge possesses over the old one, which was built by Labelye in 1740, will be easily seen by a comparison of their dimensions; the length of the old bridge was 1,160 feet 8 inches, the extreme width 44 feet, or 41 feet less than that of the new bridge, the width of roadway 26 feet, and of footpaths 8 feet each, the height from foundation to centre arch 57 feet, and depth of foundations below low-water mark 6 feet. The rise of the old bridge was 10 feet 6 inches, but it joined the roadway at so high a level that its centre would be nearly 11 feet, about that of the present structure. The position which the new bridge will occupy, when completed, is almost the whole of the site of the old bridge, and almost 40 feet on the western side of it, next to the Houses of Parliament. One of the greatest advantages is the increased width, which will afford ample accommodation for carriage and passenger traffic.

Each span is composed of 15 arches or ribs, which are placed at a distance of 5 feet apart; except in the two footways for passengers, where the weight, being trifling, they are fixed at intervals of 7 feet apart, the shapes of which are somewhat novel in bridge building, being a curve parallel with an ellipse; and owing to the almost elliptical curve of the arches, and, consequently, their very thin crowns, it was deemed necessary to provide for the security and stiffness of the structure in such a manner as would avoid the use of cast-iron in the central parts, where the traffic subjects them to constant vibration and frequent percussion from moving loads. The arches have, therefore, been designed with crowns of wrought-iron boiler-plate of immense strength and stiffness. These central ribs vary in span with that of the arch. In the centre they are 52 feet long by 28 inches deep; and in the land arches they are only 42 feet 3 inches long by 22 inches deep. The full load strain of the arches is at the rate of 3 tons the square inch, but the wrought-iron portions are tested with a compressive strain of more than 12 tons to the inch, or nearly 2,000 tons to the square foot. The strength of the whole mass is made equal to about thirty times the amount of strain, which the ordinary run of metropolitan traffic can ever place upon it. Provision has been made for expansion in the longitudinal girders by placing vulcanized india-rubber between the joints. A change in the temperature of 50 degrees would increase the length of the centre arch half-an-inch; an amount of expansion which, unless provided against, would unsettle the strongest bridge.

The roadway, as in the case of Mr. Page's suspension-bridge, at Battersea, is formed of wrought-iron buckled plates; the peculiar form of these plates gives them great strength on the convex sides. One was tested with a block of granite, weighing not less than 17 tons, and after remaining under the ponderous mass for nearly a week, there was not the least sign of permanent set. These buckled plates are 7 feet long by 4 broad, and rather less than half-an-inch thick, and are bolted down to the main ribs of the bridge. The channels formed by the slopes of the buckled plates are filled up with blocks of wood and asphalte, till the whole way is a perfect level. Over the asphalte is placed concrete, to the depth of a foot or more, and over this again a stone paving, known as granite pitching. In order to lighten the pressure at the crown, a mixture of cork and bitumen, is substituted for the concrete. The roadway will be divided into two footpaths, each 15 feet wide, and two carriage-ways, each 26 feet wide. The carriage-way will be divided by a centre kerb, separating the going and coming traffic; and each will have a tramway of 7 feet for the heavy traffic. The centre portions of these tramways are composed of granite, and the surfaces on which the wheels of the vehicles will work, are of wrought-iron, bedded on pine baulk timber.

The spandrils of the arches are filled with light Gothic tracery, formed in cast-iron; the parapet, side-rail, and cornice are replete with decorations in the same material. The whole design of these details is bold in the extreme, yet, so easy and graceful, as to be in perfect keeping with the fine line and curve of the bridge itself.

The piers are noble and massive-looking structures, and add in no small degree to the beauty of the bridge. They are surmounted with octagonal pillars, and consist of immense blocks of gray granite with moulded capitals and bases. These blocks are from the well-known Cheesewring and Penrhyn Quarries in Cornwall, and are among the largest and finest ever sent from those works, many of them being 14 feet high and 8 feet thick, and weighing from 20 to 30 tons. The neatness and finish with which these ponderous masses have been put together, more resembles cabinet-work than the masonry for one of the strongest bridges across the Thames.

The abutments of the bridge, together with the dry arches, etc., are constructed of brick, faced with Portland stone, of excellent quality, procured from the old bridge and reworked.

The foundations resemble those of the suspension-bridge at Battersea (full details of which were given in our former work), only that they are stronger in relative proportion to the weight they have to bear. These foundations combine all the advantages of foundations on bearing-piles, made by means of coffer-dams, without the expense and obstruction to the waterway, which the latter involve. A great diversity of opinion has been manifested amongst engineers as to the correctness of this proceeding; and in their evidence before the Parliamentary Committee, Mr. Simpson and the late Mr. Rendel expressed themselves strongly against dispensing with the cofferdams, and adopting the system of foundation as advised by Mr. Page; these gentlemen contended that it would be a safer plan to carry the solid foundations deep into the London clay, as the scour of the river would soon have the effect of undermining and destroying the stability of the bearing-piles proposed by Mr. Page. The permanence, however, of the foundations adopted, may be considered fully established by the durability of the piers of Old London Bridge, which were similar in construction to the New Westminster Bridge, though less bearing-piles were employed. These foundations were exposed to the scour of the ebb-tide, running like a rapid beneath its arches, yet the bridge stood in excellent order up to the time of its removal, and the piles were drawn out in a sound state after an immersion of 600 years. New London Bridge was founded in cofferdams of great size and strength, yet its piers have settled on the down stream-side between 6 and 10 inches.

The elm bearing-piles for the foundations of Westminister Bridge are disposed in rows of 3 and 5 alternately, to the number of 145 in each pier; they are 14 inches in diameter, and 32 feet in length, and are driven a depth of 19 feet into the solid London clay, and 24 feet into the bed of the river. Round these bearing-piles, a casing of cast-iron piles and plates is driven to a great depth, and the whole mass is fastened together by means of a crossing series of wrought-iron tie-bolts. This casing is formed of 44 cast-iron circular or main piles, 25 feet in length, 15 inches in diameter, and 1 inch thick; which are driven, at distances of 5 feet 2 inches apart, 23 feet 9 inches below low-water line; and the same number of cast-iron sheeting piles, fitting into grooves in the cast-iron piles, which are driven to nearly the same depth. The flat sheeting-piles are each 13 feet 6 inches long, and 4 feet wide, and are strengthened by flanges or ribs at their backs. Between the bearing-piles the bed of the river is dredged to the hard gravel, and the whole space is filled with concrete, so as to form a solid mass. Six feet below low-water line the cast-iron plates are stopped, and granite, 18 to 20 inches thick, embedded in concrete; and, resting on the top flange of the flat piles, are substituted; and on the heads of the elm bearing-piles a course of Bramley-Fall stones is placed, covering the heads of two or three piles alternately; upon these the granite blocks are laid, with a projection over the face of the outside piling, so as to form a fender to them. According to a calculation made by Mr. Page, the pressure on the bearing-piles is most unusually light, when compared with the loads generally placed. Supposing that the entire weight of the bridge be carried on these piles, they would be ultimately loaded with a weight of 15 tons each, or 12 tons per square foot in the pile; which contrasts favourably with London Bridge, where the load is 80 tons; and at Hull Docks, 25 tons per pile. It is computed that elm piles, such as are used in these foundations, would bear, without permanent alteration, a compressive force of 200 tons. The pressure per foot superficial on the whole area of the foundations is only 2 tons, while on the old bridge it was 6 tons, and in London Bridge, $5\frac{1}{2}$ tons.

The amount of timber used in the bearing-piles is 45,700 cubic feet; of cast-iron in the foundations, 1600 tons, and in the superstructure, 2600 tons. The wrought-iron in the superstructure is 1300 tons, and in the foundations, 70 tons for tie-rods. There are also 165,000 cubic feet of granite, 21,000 cubic yards of brickwork and Portland cement, and 30,000 cubic yards of concrete.

A vote of £482,000 "for erecting the new bridge at Westminster and approaches thereto," was agreed to in Committee of Supply, as well as an additional sum of £40,000 solely for the approaches; and this amount has been declared sufficient for the completion of the bridge and approaches.

The contract for the bridge was taken by the Messrs. Cochrane and Co. of the Woodside Ironworks, near Dudley, who have entrusted the superintendence of the work to Mr. Phillips.

VICTORIA BRIDGE, PIMLICO.

PLATES XXIV. TO XXIX.

This bridge was erected over the river Thames to carry the Crystal Palace and West End of London Railway, and also the London and Brighton, London, Chatham and Dover, and other Railways, terminating at the Victoria Station, within a short distance of Buckingham Palace. The whole of the work was designed by Mr. John Fowler, who was ably assisted in the construction by Mr. W. Wilson, and others of his staff.

The bridge is composed of four principal openings of 175 feet, each of which is spanned by six wrought-iron segmental arches, with a rise of 17 feet 6 inches; and of two land-openings, one of 70 feet span, and the other of 65 feet, consisting of six horizontal wrought-iron girders. The widths between the parapets of the bridge is 32 feet, allowing spaces for two lines of mixed gauge railway ; the level of the rails is $24\frac{1}{2}$ feet above Trinity high-water. The main-ribs are placed in pairs, and their respective areas are as follows:—

		Inches.
No. 1, Outer rib	Top table,	11·25
	Web,	18·00
	Bottom table,	11·25
		40·50
No. 2, or First inner, . . .	Top table,	18·00
	Web,	18·00
	Bottom table,	18·00
		54·00
No. 3, or centre	Top table,	22·50
	Web,	18·00
	Bottom table,	22·50
		63·00

We will now give an abstract of the quantities of iron employed in the construction of the bridge, and then pass on to a few remarks upon the foundations and piers; a further description of the iron-work being considered unnecessary, as full details will be found in the specification, a copy of which is annexed.

QUANTITIES OF IRON WORK.

WROUGHT-IRON—RIVER ARCHES.

	Tons.	cwt.	qrs.	lbs.
In 8 central and bearing ribs	196	15	2	8
In 8 outside do.	176	17	2	16
In 8 outside do. countersunk face	127	3	2	24
In 8 outside horizontal girders	73	18	1	20
In 16 inner do.	180	0	2	8
In horizontal girder over abutments	5	18	0	0
In 16 inner spandrils	115	16	1	20
In 8 outside do.	35	16	2	24
In 224 cross girders E	53	16	0	0
In 256 do. D	52	11	1	20
In 232 do. F (outside)	8	4	2	18
In 116 do. F (inside)	4	1	3	8
In 64 do. G (long)	6	6	3	12
In 96 do. G (short)	8	9	0	19
In 129 cover-plates, under cross girders	24	6	3	12
In diagonal round iron tie-bars	9	3	3	25
In diagonal and cross stays between spandrils	8	2	2	8
In joint-plates to junctions of segments	5	0	2	24
In cornice	51	16	3	4
In bolts to shoes, bed-plates, junctions, etc.	11	5	3	23
Weight of wrought-iron in river arches	1155	13	3	13
70 feet opening over road.				
In 4 inner bearing girders	72	14	2	0
In 2 outer girders, with countersunk face	12	5	3	22
In 50 cross girders	11	13	2	5
In cornice	5	0	2	0
In bolts and nuts	0	3	0	0
In 25 tie-plates for cross girders	1	7	0	1
	1258	18	1	13

CAST-IRON—RIVER ARCHES.

	Tons.	cwt.	qrs.	lbs.
In outer shoes for arched ribs	51	15	0	16
In inner do. „	26	12	2	24
In bed-plates for horizontal girders (inside)	3	10	0	0
Do. do. (outside)	1	10	1	14
In heels for inside girders	21	19	2	24
Do. outside girders	9	5	1	4
In Parapet Railway, ornamental moulding, etc.	75	18	1	7
	190	11	2	5
70 feet opening.				
In bed-plates to inner girders	2	9	2	16
Do. outer girders	0	16	1	16
In parapet railing	4	15	3	0
Total weight of cast-iron	198	13	1	9

GENERAL SUMMARY.

	Tons.	cwt.	qrs.	lbs.
Wrought-iron	1258	18	1	3
Cast-iron	198	13	1	9
	1457	11	2	12
Additional land arch	90	0	0	0
Wrought-steel for keys and gibs	15	12	3	12
Total weight of the bridge	1563	4	1	24

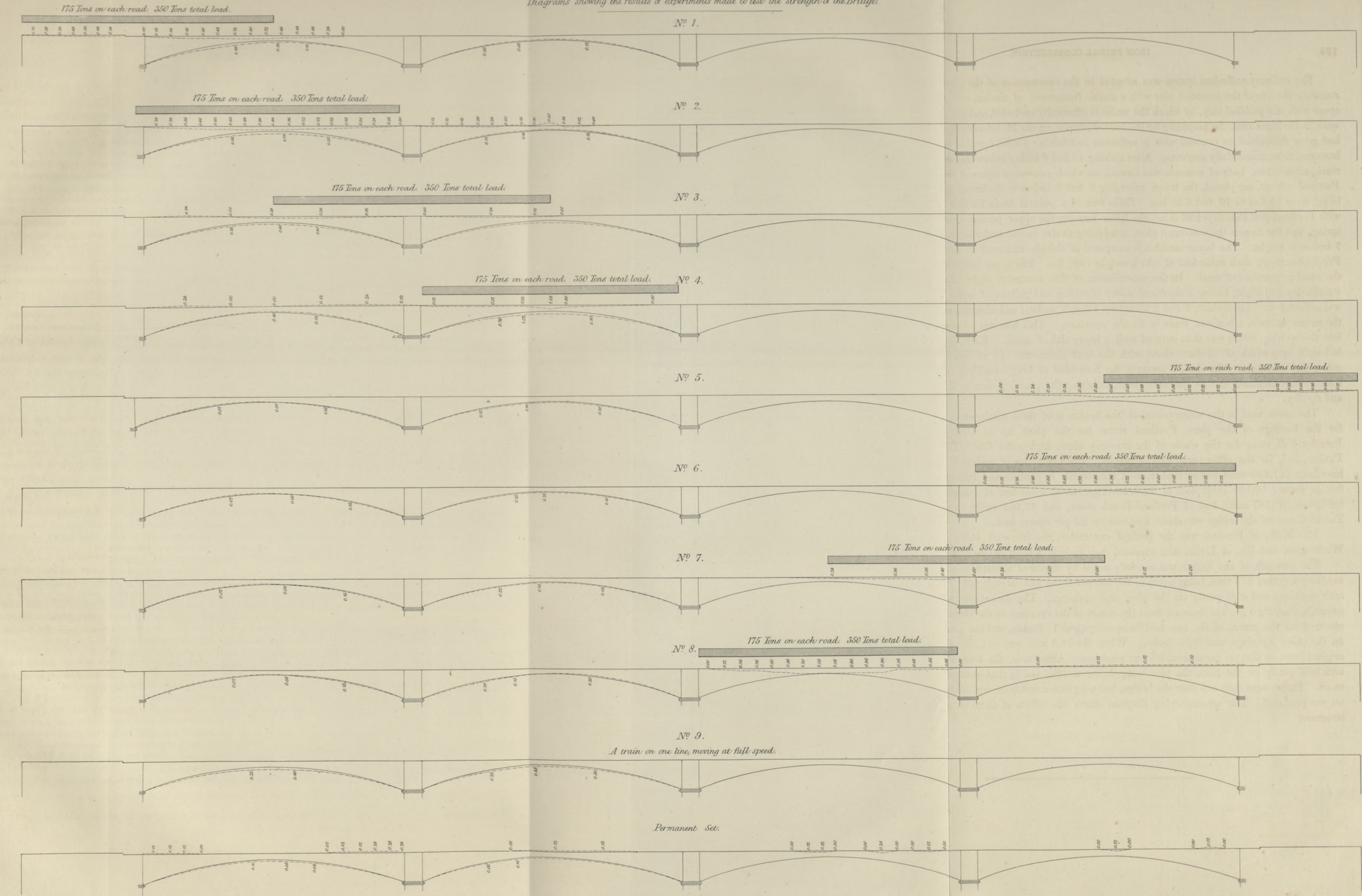
The ordinary cofferdam system was adopted in the construction of the piers, which consisted in surrounding the site of the intended pier with a double framework of timber, and filling in the intervening space with stiff puddled clay, by which the water is effectually excluded, and the operatives are enabled to work in the inner area. In preparing for the land abutment on the Middlesex side of the river, the engineer had great difficulties to contend with in reference to attaining a solid foundation, which difficulties were, however, most successfully overcome. After sinking 46 feet 6 inches below the level of Trinity high-water mark, an immense body of concrete was formed, on which successive layers of brickwork, set in hydraulic Portland cement, are placed, the layers averaging 2 feet 6 inches in thickness, and the masses of concrete being from 10 feet to 10 feet 6 inches. These formed a general basis for the abutment, which is faced with Portland and Bramley-Fall stone, the latter forming the upper portion from which the iron arches spring, and the former the rusticated piers, comprising twelve course underneath, each course being 1 foot 7 inches in height. The lower stratum is composed of shingle and sand in amalgamation with the best Portland cement, each cubic foot of which weighs 143$\frac{3}{4}$ lbs. The upper layers are of blue lias lime, with the usual proportions of sand. In the construction of this abutment eight pumps were employed constantly, day and night, having a combined power of raising 800 gallons of water per minute, which water was collected in a brick-shaft, 6 feet 6 inches by 4 feet 3 inches; and after its overflow was overcome, at the proper altitude, a box was made to fit the aperture. This box was filled with concrete, and dipped into the opening, which was then covered with a heavy slab of stone. Each river pier measures 63 feet in length by a width of 19 feet; these, with the land abutments and arches, form a very elegant and substantial structure. Its design presents the *beau ideal* of bridge-building, charming the eye by its lightness and elegance, and, at the same time, satisfying the practical man by its appearance of strength and rigidity.

The stone used in the construction of the bridge is of three kinds—viz., Yorkshire rag landings for the footings of the piers; Portland stone for the piers up to Trinity high-water mark, and Bramley-Fall stone for the whole of the masonry above high-water line. The bricks in the piers are Paviors, and, for the other parts, Cowley Stocks. The quantities of materials employed in its erection have been 197,800 cubic feet of timber, used temporarily in gantries and cofferdams, and 16,800 cubic feet in floors; 10,700 cubic feet of York landings, 4,050 cubic feet of concrete, 6,500 cubic yards of brickwork, 23,587 cubic feet of Portland-Roach stone, and 57,205 cubic feet of Bramley-Fall stone. The total cost of the bridge was about £90,000, or £3 per square foot.

Mr. Kelk, of Pimlico, was the general contractor, the ironwork being sublet to Messrs. Bray, Waddington and Co., of Leeds, who executed their task in a most satisfactory manner.

The strength of the bridge was carefully tested by means of locomotive-engines and loaded trucks, weighing together 350 tons, being equal to one ton per lineal foot on each rail. The load was placed over each opening, and also partly on the piers and openings. The deflections were observed at regular intervals, showing a gradual decrease from the centre of the openings to the piers. The greatest deflection observed on the centre of the two middle openings was 1$\frac{1}{5}$ inches, and the greatest deflection observed on the other openings was 0·94 inches. When the load was removed, the bridge again resumed its original level, so that no permanent set was produced. Afterwards, the load was placed partly on one arch and partly on the next, the pier being in the middle, and in that position the deflection was very small. Trains were then run over the bridge, but no greater deflection was observed, and no permanent set was produced. The accompanying diagram shows the effects of these tests on various parts of the structure.

VICTORIA BRIDGE, PIMLICO.

Diagrams showing the results of experiments made to test the strength of the Bridge.

Horizontal Scale, 50 feet – an inch.

W. Humber, dir.

75 P. 236.
Standidge & Co. litho Old Jewry London.

London: E. & F. N. Spon, 16, Bucklersbury.

SPECIFICATION.

The contract, to which this specification refers, includes the finding of all material, tools, and labour for the entire construction and erection of all the wrought- and cast-iron work required for the proposed bridge over the river Thames on the line of the above railway as shown on the several drawings numbered 1 to 10 inclusive.

The bridge will consist of four principal openings across the river, each having a clear span of 175 feet, and of one side opening* over the road leading to the suspension-bridge with a clear span of 70 feet.

The principal openings are each spanned by six wrought-iron arched girders, resting on cast-iron bed-plates fixed to the piers and abutments, and will be composed of I shaped ribs, with top and bottom tables, and central web of flat plate-iron, connected together by means of angle-iron and T-iron covers, and the whole rivetted together and constructed in the form and manner shown on the detailed drawings.

Horizontal wrought-iron girders stretch from arch to arch, bearing on the piers in the centre of their length upon cast-iron bed plates fixed on the masonry of the piers. These horizontal girders are also I shaped, and are constructed of flat wrought-iron plates for the top, bottom, and central web, with T- and angle-iron stiffening-bars rivetted together in the same manner as the arched-ribs.

The spandrils, or intermediate spaces between the arches and horizontal girders, are intended to be filled in with wrought-iron framework radiating from the arch. Those for the four inner or bearing-girders will be composed of four angle-irons placed back to back, and rivetted through flat wrought-iron plates of varying thickness; the angle-irons will be rivetted to and form part of the upper table of the rib, and the bottom table of the horizontal girder being joined to the radiating-bars at the points, and in the manner shown on the drawings.

The radiating-bars will be intersected longitudinally by a flat wrought-iron plate passing through the centre of each bar, and rivetted to them at the points of intersection.

The two outer spandrils will also be fitted in with a radiating framework composed of T-iron bars placed back to back, and rivetted through flat wrought-iron plates of varying thickness; the T-irons are rivetted to and form part of the upper table of the arch rib and the lower table of the horizontal girder, and will be connected at the points and in the manner shown on the drawings.

The whole of the external face of these outer spandrils will be countersunk, rivetted, and the greatest care must be used to ensure all the cranked angles being set perfectly true, so that every part may be strictly in accordance with the detail drawings.

The four inner or bearing-ribs, with the spandrils or horizontal girders, are all exactly similar, both as to sectional area and mode of construction; the two external or face-girders will be diminished in section, and constructed so as to form a plain fascia, the central web forming the face, all the external rivetting being countersunk.

The arched ribs and horizontal girders will all be constructed in pairs, connected by means of the transverse covering-plates, which will be rivetted to the upper and lower flanges of each, at about every 6 feet apart, and each arched rib will be divided into five segments, which will be connected by bolts passing through transverse flat iron-plates, extending through the entire width of each pair of girders.

The whole surfaces of these joints are to be planed perfectly true, so as to ensure the most perfect contact throughout the entire bearing surface.

The *m m m* roadway will be carried by means of bars, wrought-iron transverse bearers placed about 3 feet apart, and bolted through the horizontal-girders and arched ribs; each alternate bearer being continued through the centre and external spaces. They will consist of I shaped bars rolled to the several dimensions shown on the drawings, and those resting on the horizontal girders having the bottom flanges rivetted to flat wrought-iron plates extending over the entire width of the bridge, and connected with the bottom tables of each horizontal girder. The cross-bearers which occur between the arched ribs will not have the separate plate, but will be rolled with an increased section of bottom flange.

The arched ribs will spring from cast-iron bed-plates, resting on, and bolted through the masonry of the piers and abutments; a segmental cast-iron shoe will be bolted to each end of the arch, and will rest in a corresponding concave bearer, which will work loose in the bed-plate, and will be provided with proper keys for adjusting it in position; cast-iron bed-plates are also provided for the bearings of the horizontal girders, which will be bolted to the masonry of the piers.

The 70 feet of opening over the road will be spanned by six horizontal wrought-iron girders, which will be composed of I shaped beams, with top and bottom table, and central web of flat wrought-iron plates, with T and angle-iron stiffening bars, rivetted together and constructed in the manner shown in the drawings; of these, the four inner, or bearing girders will be precisely similar in sectional area and construction; but the two outer girders will be of diminished section, and will be rivetted countersunk, so as to form a plain fascia on the external elevation.

The roadway over this opening will be carried by means of wrought-iron cross bearers, placed three feet apart, bolted through the horizontal girders, and each alternate bearer being continued through the centre external spaces. These bearers will be composed of wrought-iron I shaped rolled bars, of the dimensions shown on the drawings; and those carrying the roadway, will be cranked over the bottom tables of the girders, so as to bring the bottom flange down to the flat wrought-iron plates, which, as in the main openings, will extend across the entire width of the bridge, and to which they will be rivetted in the same manner as before described.

The whole of the girders for this opening will bear on cast-iron bed-plates, resting on, and bolted through the masonry of the abutments, and one end of each of the four girders will be connected with the horizontal girder of the adjoining main opening.

On the top of the external horizontal girders, a cornice will run along the entire length of the Viaduct on each face, and will be composed of wrought-iron plates, rolled or bent to the several forms and dimensions, shown on the drawings.

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* Provision is only made for one land opening in this specification, as the other was executed by the London and Brighton Railway Company.

All the cast-iron work, must be of the best quality, having an admixture of one part Scotch, one part Yorkshire or Derbyshire, and one of old metal.

All the castings must be true, straight and exact to the shape and dimensions; sound, clean, and free from air-holes of every description, and the greatest attention must be paid to the execution of the ornamental castings, in order to ensure the arris being sharp and true, and the neat and accurate fitting of the several parts.

All the bed-plates for the arch springing, and bearings of the horizontal girders will be of cast-iron, and the upper surface of the lower plates, and the lower surface of the upper plates, as well as the convex and concave surfaces of the segmental springing shoes, are all to be planed perfectly true, and in such a manner as will ensure a perfect contact throughout the entire bearing surface.

The whole of the wrought-iron used in the work, is to be obtained from the Monk Bridge Iron Company, and to be of the quality known as "Monk Bridge Crown plates guaranteed," or of such other iron of equal quality as the engineer may approve. And the engineer is to have the power of testing and rejecting from time to time, any portion of the wrought-iron used in the construction of the girders. The tests will be applied in the following manner, viz:—

A piece of iron shall be cut, at the discretion of the engineer, from any or every plate about to be used in the work, two inches wide and half-an-inch thick throughout, and of such length as to have 7 inches under actual tension. The following tensile strains shall then be applied, and if the extension of the iron experimented upon exceeds the lengths given opposite each strain, the plate from which the piece of iron has been cut shall be absolutely rejected.

18 tons strain must be borne without a greater

extension of length than,	- - - - -	$\frac{1}{8}$	of an inch extension.
21 tons strain,	- - - - -	$\frac{1}{4}$	"
23 tons strain,	- - - - -	$\frac{1}{2}$	"
24 tons strain,	- - - - -	$\frac{3}{4}$	"

And all bars must bear a tensile strain of 26 tons before fracture.

The engineer shall have the power of rejecting any part of the work which in his opinion does not satisfactorily bear the tests applied, or which may be otherwise unsuited for the purpose, whether arising from imperfect workmanship or defective materials.

All costs and expenses attending the testing to be borne by the contractor.

All plates are to be perfectly true, of uniform thickness throughout; and all joints must be made, by planing the surfaces true, so as to fit close and accurate throughout their entire length.

All the angle-irons are to be rolled accurately, so as to apply exactly to the surface of the plates to be connected by them, and must have all their joints planed true and covered by angle-iron covering-plates. The joints in the angle-irons used in the external face of the outer girders will be neatly and securely welded, so as to form one continuous bar throughout each segment of the arch.

The outer edges of all the plates in the external girders are to be made perfectly smooth by planing.

All covering-plates must be sheared perfectly square, and must also be made perfectly smooth and regular at the edges by planing.

The joints in all plates, angle-irons, &c., are to be made only in such positions as are shown in the drawings, and all covering-plates or T-iron stiffening-bars, are to be truly cut at the ends, so as to ensure perfect fitting, and must in all cases be cranked, curved, or pounded to the exact form shown on the drawings.

The rivetting throughout the entire work must be executed in the neatest and most workmanlike manner. All the holes must be accurately punched, so that the rivetting may bring each plate into its proper position, and into the most perfect and accurate contact with the adjoining pieces. All rivets are to fit the holes perfectly, and will be furnished with cup-heads, or such other shape as may be approved or ordered, and where rivets are shown to be countersunk, the sinking must be effected by drilling, and the heads finished in the neatest and most workmanlike manner, so as perfectly to conceal the rivet-head. All bolt-heads and nuts, where exposed to view, must be hexagonal, and bolt-heads countersunk where necessary.

As soon as any part of the ironwork is constructed, it is to receive one coat of boiled linseed-oil, to be laid on hot; and all the iron-work employed is, in addition, to receive one coat of redlead before erection.

All paint must be of the very best quality, and free from adulteration, and the greatest attention must be paid to protect all the ironwork before any oxidation commences.

All planed, bored, or twined surfaces, whether in cast-iron or wrought-iron, are to be coated with a mixture of whitelead and tallow, to prevent and preserve them from oxidation.

The principal features of the work to be executed are stated in this specification, or shown on the drawings; but the works, although parts of the same may be shown on the drawings, or referred to in this specification, is considered as included in the contract as much as if it had been particularly set forth and described; also any work that may be mentioned in the specification only, without being shown on the drawings, or as may be shown on the drawings without being alluded to in the specification, are to be included in the contract, the same as if they had been particularly set forth and shown on both the drawings and the specification.

Whenever dimensions are written or figured upon the drawings, they are to he considered as correct (excepting in cases of obvious clerical errors), although they may not correspond with the dimensions taken by the scale, which must only be used when no written dimension is given.

And all details which are drawn to enlarged scales are to be taken as more correct than those of former scales. And it is further to be understood that the contractor is, without any claim to extra charge, to execute all work in compliance with any further working or detailed drawing or instructions which the engineer may, from time to time supply; which, in his opinion, may be required for the proper completion of the work intended to be included in this specification, and described by the drawings.

The works are to be conducted under the direction and superintendence of the engineers of the company, and must be executed and completed to their entire satisfaction, and their judgment, both as regards the quality of materials or the workmanship, shall be absolute and without appeal, and in case of any difference between the company and the contractor relative to this particular work, either during its progress or after completion; or as to the meaning of any part of this specification, or as to the drawings, or the duty of the contractor, the decision of the engineer-in-chief shall be final and conclusive.

The engineer shall have the power of making any alteration in the dimensions, by either increasing or diminishing the thickness of any of the plates or other parts of the works, and shall be at liberty to alter or vary the design in any manner he may think fit; and any increase or diminution of cost thereby occasioned shall be paid or allowed by the one party to the other; to be settled, if the parties differ, by arbitration; but no alteration affecting the cost of the bridge shall be made without the consent of Mr. Kelk.

FOUNDATIONS AND MASONRY.

The piers and abutments are all to be carried down to the depths of at least 8 feet, into solid clay, as the foundations for the concrete, and to any greater depths which the engineer may consider necessary.

THE STONE to be used in the construction of the Bridge shall be of three (3) kinds, viz., for the footing of the piers, Yorkshire Rag Landings.

For the piers up to the level of Trinity high-water, Portland Roach stone; and for the whole of the masonry above high-water line, Bramley-Fall Stone, or some other stone of equal quality, which must be first approved by the engineer.

The whole of the stone used in the bridge shall be the best of its respective kind—hard, sound, and free from all beds and flaws.

THE LANDINGS used in the foundations are to be squared to their proper dimensions, and to be dressed truly in the joints; and, if necessary, on the beds also.

THE PORTLAND STONE, used in the facework of the piers, shall be the best hard Roach, and shall be truly dressed on the beds and joints, and left rock-faced on the external face with horizontal rustic joints, neatly tool-dressed.

ALL the Bramley-Fall stone used in the structure shall be selected with great care, so as to be fine-grained and uniform in quality and colour, and neatly tool-dressed.

The joints, mouldings, and other ornamental members shall be worked in such a manner as may be shown on the working drawings, and the whole to be well and truly bedded and squared throughout.

It must be distinctly understood that if the contractor is unable to obtain Bramley-Fall stone, of such good quality with regard to fineness of grain and uniformity of colour and freedom from defects, as to be, in the opinion of the engineer, suitable for the design, Bramley-Fall stone must not be used, but some other kind obtained, to be approved by the engineer.

THE BRICKS used in the river piers are to be of the kind known as "paviors," and all the other brickwork in the abutments, whether in foundations or superstructure, are to be "Cowley's Stocks;" the whole of the bricks used in the work shall be well-shaped, sound, and hard-burnt, and laid (in such bond as the engineer may direct) in well-tempered mortar or cement; no joint to exceed a quarter of an inch (¼″) in thickness, and shall be less if the nature of the bricks will permit; and every course shall be well grouted. No broken bricks shall be used in any of the work, except what may be requisite for closing a course, and the work shall be equally good for the interior as the exterior.

THE MORTAR to be used in all masonry, or brickwork below the level of high-water, shall be composed of Greave's blue lias lime, obtained in barrels, and used as fresh as possible, in the proportion of one (1) of lime to two (2) of clean, sharp river sand; and for all other work above high-water level, and where not otherwise specified, it shall be composed of well-burnt Greystone Medway lime in the same proportions. In all cases the mortar shall be well ground and mixed together in a mill erected for the purpose immediately adjoining the work. All mortar shall be used perfectly fresh-ground, and the contractor shall not have more mortar on the work than is absolutely requisite, day by day; and for each day's consumption, any mortar mixed the previous day shall not be used in any part of the work.

CEMENT.—In all cases where brickwork, or other masonry is shown in the drawings for specified or ordered by the engineer, to be set in cement. It shall be composed of Portland cement of the best quality, mixed with an equal quantity of clean, sharp sand, and be mixed only in small quantities and used quite fresh.

CONCRETE.—All concrete used in the work, whether in foundations or otherwise, shall be made with Portland cement, and mixed in the proportion of one yard of cement to seven of good strong clean gravel, the gravel to be carefully selected, so that it contains, as nearly as possible, pebbles of all sizes found in strong Thames gravel, and then to have all the sand screened out of it, which is of a size suitable for building purposes; these ingredients are to be well and thoroughly mixed in a dry state, and then all are to be beaten and worked with a small quantity of water in such a manner as to ensure their being thoroughly amalgamated together, and in this state are to be tipped into the work from a height of at least 10 feet.

TIMBER.—The timber used for the sheet pilings and wallings shall be prepared so as to enable it to be driven straight and true, and creosoted under pressure, with 8lbs. creosote to the cubic foot. The greatest care must be nsed that the sheet pilings be driven so as to be perfectly tight, and as true as possible; and all space between the sheet pilings and the cofferdams up to the bed of the river filled in solid with concrete made with Portland cement; the inner piles of the cofferdam to be cut off and not drawn.

Any stone, bricks, mortar, cement, or other materials which the engineer may think unfit for the purpose for which it it proposed to be applied, shall be removed from the works by the contractor within twenty-four hours of the receipt of instructions to that effect from the engineer, and replaced by proper and sufficient materials.

CHAPTER XVI.

WROUGHT-IRON PLATE GIRDER BRIDGES.

In this Chapter we shall include descriptions of such bridges as are carried by girders having plate-webs; they are Staines Bridge, Trent Lane Bridge, and the Victoria Bridge, Australia. The girders of these three bridges show different methods of obtaining the required rigidity, but they are all characterised by great neatness, indicative of careful designs.

We have, in a previous part of this work, mentioned the necessity of devoting a considerable amount of attention to the determination of such details as cannot be calculated, and it is by such attention that the pleasing results which we have just mentioned are attained.

Staines Bridge is carried by continuous girders of three spans, and is a very convenient example for calculation, wherefore, we selected it for our example of the continuous girder of three spans; the moments of strain on which are shown, Diagram No. 3. The Trent Lane Bridge is also carried by continuous girders of three spans, and is built on the skew, and we may here observe that we seldom see a cross section better designed than that of this structure. The Victoria Bridge, Australia, is carried by tubular plate girders, of light design. The strains produced by a full load, and also by a moving load, together with the weight of the structure, may be found from Diagram No. 1. Having made these brief observations upon the general character of the structures included in the present Chapter, we will proceed to describe them.

BRIDGE OVER THE RIVER THAMES AT STAINES.

PLATES XXX., XXXI., AND XXXII.

This bridge has been constructed from the design of Mr. John Gardner, for the purpose of carrying the Staines, Wokingham, and Woking Railway across the river Thames at Staines. It consists of three openings, viz., a centre opening of 88 feet 6 inches span, and two side openings of 85 feet 3 inches each. The bridge is composed of three wrought-iron plate continuous girders, having a total length of 279 feet 8 inches, and a depth of 7 feet 6 inches; these girders are supported on 6 cast-iron cylinders, sunk deep into the bed of the river, the centre girder having a double duty to perform, is made much stronger than the others, and consists of the following parts: A top flange of a triangular shape, which is composed of plates of an average length of 10 feet 3 inches, and of varying thicknesses from $\frac{1}{4}$ to $\frac{5}{8}$ of an inch; at the points of greatest strain, additional plates are introduced, which will be observed on reference to Figures 4 and 5, plate LI.; the joints are covered by wrought-iron strips 6 inches by $\frac{3}{8}$ inch. A bottom flange, which for nearly the whole of its length, is composed of a double row of plates 10 feet 3 inches in length, 2 feet 6 inches in width, and varying in thickness from $\frac{3}{8}$ to $\frac{11}{16}$ of an inch, and of a web which is connected to the top and bottom flanges by means of angle-irons of varying dimensions, and composed of wrought-iron plates 7 feet 6 inches in length, 3 feet 5 inches wide, and of thickness from $\frac{1}{4}$ to $\frac{11}{16}$ of an inch; these plates are all connected by double covering strips $4\frac{1}{2}$ inches by $\frac{1}{4}$ inch throughout; considerable rigidity is imparted to the web by means of stiffening pieces, which are placed at distances of 10 feet 3 inches apart, along the whole length of the girder. The outer girders require no description, as they are similar in construction to the centre girder, though much diminished in strength. The roadway is supported on 56 transverse beams, which rest upon the bottom flanges of the main girders; these beams are of a uniform depth of 2 feet, and consist of a $\frac{1}{4}$ inch plate, and 4 angle-irons, $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches $\frac{7}{16}$ by inch.

Longitudinal bearing-girders are placed under each rail; they are composed of a plate 1·5$\frac{5}{8}$ inches in depth, and four angle-irons, and are rivetted to the web of the cross girders.

Cast-iron sliding-plates are attached to the underside of the bottom flanges of the main girders, where they rest upon the piers and abutments, and are supported on the cast-iron bed-plates, which on the piers are bolted down to the upper flanges of the cast-iron girders, placed at a distance of 3 feet apart, from centre to centre; these girders are fastened to the tops of the cylinders, and are 2 feet 8 inches in depth, with top and bottom flanges of 1 foot, and a thickness of metal of 1$\frac{1}{4}$ inches.

The piers are composed of three cast-iron cylinders, 6 feet in diameter, which are constructed by means of a cast-iron framing, placed between and bolted to them; the cylinders vary in thickness from 1 inch to the top lengths to 1$\frac{1}{2}$ inches for the bottom; they were sunk by weighting each with 80 tons of rails, and then dredging out the gravel with a Thames spoon until the clay was reached, after which the water was pumped out and the clay excavated; they were partly filled with brickwork in cement, and partly with concrete. See Fig. 1, Plate XXXII.

The weight of the bridge is as follows:—

	Tons	cwt.	lbs.
2 single girders	81	13	94
1 double girder	57	10	61
56 cross girders	25	0	0
4 longitudinal do.	28	8	0
	192	12	43

which with the rails, timber, and ballast, gives a total of 360 tons, or about 1·33 tons per foot-run.

The cost of the bridge was £10,000, or somewhat less than £40 a foot.

The contractors were the Messrs. Kennard of the Crumlin Iron Works, who performed their work in a most satisfactory and efficient manner.

TRENT-LANE BRIDGE.

PLATES XLIII. AND XLIV.

This bridge was erected to carry the extension into Nottingham, of the Ambergate, Nottingham, and Boston and Eastern Junction Railway, over the old Trent-Lane public road, and the Railway from Nottingham to Lincoln, belonging to the Midland Railway Company. It is constructed from the designs of John Underwood, Esq.; Messrs. Lloyds, Fosters and Co., of Wednesbury, being the contractors.

It is a skew-bridge of a very acute angle, 23$\frac{1}{2}$ degrees, and consists of one large span of 96 feet across the railway and public road, and two smaller spans for affording communication with adjoining lands.

The total length of the bridge, as measured on the skew face, is 286 feet; the length of each of the two main girders is 264 feet; the centre span in the clear of the bearings is 96 feet, and the side openings have each a clear span of 60 feet. On the piers and abutments, cast-iron bed-plates are fixed with moveable rollers, upon which the main girders rest. The platform is supported on transverse girders, formed of plate- and angle-iron, having a depth at the centre of 1 foot 4 inches, and at the ends, of 1 foot 1 inch; which are placed 2 feet 6 inches apart, except where the bearing is reduced by reason of the skew, when they are placed 5 feet apart. These girders are covered with with 4 inch Memel planking, having half-baulks of timber, 14 inches by 7, placed longitudinally, upon which the bridge rails, 80 lbs. per yard, are fixed.

The depth of the main girders at the centre is 8 feet, and at the extreme ends, 5 feet; the top is cellular, being 2 feet 9 inches in width, the form being considered advisable, as giving more lateral strength; the bottom flange is composed of four plates 7 feet 6 inches long, 3 feet wide, and $\frac{7}{16}$ of an inch thick, middle section, breaking joint at equal intervals, and thus dispensing with the necessity of joint covering plates, and giving a more uniform appearance to the underside of the girder.

The following is an abstract of the weight of this bridge:—

	Tons	cwt.	lbs.
Weight of two longitudinal girders . . .	152	6	98
Weight of transverse girders, 67 in number . .	98	5	99
Total weight of wrought-iron . .	250	12	85
Weight of longitudinal sleeper	10	0	48
Weight of 1425 cubic feet Memel planking . .	22	2	14
Weight of rails	10	5	80
Total weight of bridge . . .	293	1	3

or 1·1 tons per foot-run.

The bridge was tested by the Government Inspector, who had both lines of rails over the whole platform at the centre span, covered with some of the Great Northern Engines, (about 120 tons) as dead weight; these were afterwards brought on simultaneously from opposite ends along the two roads at a considerable velocity. The utmost deflection obtained, $3\frac{1}{2}$ tenths of an inch, this being the amount at each trial, without the smallest discernible permanent set.

The deflectometer used, was constructed so as to register an upward deflection as well as a downward one; the former being plainly visible when the weight was on the outer spans, showing satisfactorily the action due to the continuity of the girder.

BRIDGE OVER THE SALT-WATER RIVER, VICTORIA.

PLATE XXXV.

This bridge was executed under the direction of Messrs. W. Fairbairn and Co., of Manchester, for the Melbourne and Williamstown Railway over the Salt-Water River near Melbourne. The main girders are each 216 feet in length, and of a uniform depth of 14 feet 3 inches; they are constructed on the tubular principle, with a cellular top of plates, and bars of wrought-iron; the sectional area of each of the outside girders at the centre is 83·81 square inches, and of the centre girder it is 14·606 square inches. The total weight of the bridge is about 490 tons, or nearly 2·27 tons a foot-run. As the accompanying specification will be found fully descriptive of the various parts of the bridge, we do not consider it necessary to offer any further remarks.

SPECIFICATION.

DESCRIPTION.

The bridge required, is to cross the Salt-Water River in one span of 200 feet in the clear between the piers, and to consist of three wrought-iron tubular girders, each of 200 feet clear span, supported upon two stone piers 21 feet 6 inches above high-water level; each girder is to have 8 feet bearing at each end upon the piers, making the total length of each girder 216 feet. The girders are to be placed parallel to each other, and 15 feet 9 inches apart. The roadway is to be carried upon longitudinal sleepers, resting upon and bolted to the angle-irons of the cross-bearers. The cross-bearers are to be of wrought-iron in one length, reaching from outside girder to outside girder, and rivetted to the bottoms of the outside girders, and rivetted and bolted also to the centre girder.

DIMENSIONS.

OUTSIDE GIRDER.

Extreme dimensions :—Depth 14 feet 3 inches; width of bottom and cell, 3 feet 6 inches; depth of cell, 1 foot 4 inches; thicknes of bottom, 1 inch and $\frac{11}{16}$ in the centre, and $1\frac{1}{4}$ inches at the end; length, 216 feet.

CONSTRUCTION.

The outside girders are to be of one uniform depth and width, but curved to 3 inches camber to allow for deflection.

The dimensions, thickness, and arrangement of the plates and angle-iron, and the general mode of construction, are to be carried out in all respects according to the drawing approved of by inspecting engineer.

SIDES.

The plates are to be as long as can be conveniently rolled, so as to make as few joints as possible.

The plates are to be reduced from the centre, towards the ends, into the ratio shown in the drawings.

Each joint is to be broken by the plate above, and all the joints are to be equidistant, and the joints of the top and bottom plates are to have a jointing strip of the same width and thickness as the plates.

The holes are to be rimered and made fair, and the rivets must thoroughly fill the holes; the plates are to be brought together with butt-joints. The bottom is to be fastened to the side-plates by angle-irons, as shown on the drawings.

TOP, OR CELL.

The cell is to be 1 foot 4 inches deep, and 3 feet 6 inches wide, and constructed with boiler-plates, with a rise of $\frac{3}{4}$ of an inch in the centre, so as to throw off the rain.

DIMENSIONS.

CENTRE GIRDER.

Extreme dimension:—Depth, 14 feet three inches; width at centre, 4 feet; width of bottom and cell, 4 feet 7 inches; depth of cell, 1 foot 4 inches; thickness of bottom $2\frac{5}{8}$ inches in centre, and $1\frac{7}{8}$ inches at the ends; length 216 feet.

CONSTRUCTION.

The girder is to be of uniform depth and width, but curved to 3 inches camber, to allow for deflection. The plates are to be as long as can be conveniently rolled, so as to make as few joints as possible.

Each joint is to be broken by the plate above, and the whole construction to be made in strict accordance with the drawings approved and signed by the inspecting engineer.

CROSS BEARERS.

The cross-bearers for carrying the roadway, are to extend the whole width of the bridge in one length of 42 feet; if found possible to ship them, and to be rivetted to the bottom of the outside girders; the rivets in the angle-irons of the girders passing through the angle-irons of the cross-bearers, as shown in the drawing. The cross-bearers to be rivetted and bolted, also to the bottom of the centre girder by four bolts, $1\frac{1}{8}$ inch in diameter, as shown in the drawing.

The bolts to be $15\frac{1}{2}$ inches apart, so as to clear the flanges of the cross-bearers. The bolts and nuts must be of the best iron, and to have the threads clean and perfectly cut up with " Whitworth's Patent Taps and Dies." The nuts are to be hexagonal, and to have a washer under them of $\frac{1}{4}$ of an inch thick; the nuts to be tightened up, and the end of the bolt rivetted over, to prevent the nut jarring loose.

The cross-bearers are to be 14 inches deep, as shown in the drawing.

LONGITUDINAL SLEEPERS.

The longitudinal sleepers are to be of red pine, 8 inches by 12, sound, clear, and free from knots and shakes, and are to be bolted down to the cross-bearers by bolts $\frac{5}{8}$ of an inch in diameter, passing through the flanges of the angle-irons of the cross-bearers, where the intermediate rivets are left out. The heads of the bolts are to be underneath the angle-irons of the cross-bearers, and the nuts to be let into the timber with washers underneath. The holes in the timber which receive the nuts are to be round, and of sufficient size to allow them to be screwed up by a box-spanner; the holes to be afterwards filled up with iron cement, so as to prevent them from turning and becoming loose. The flooring to be of Baltic deals, 9 inches wide by 3 inches thick, laid longitudinally, with 1 inch space between them. The planks are to be bolted to each of the top flanges of the cross-bearers, with bolts $\frac{5}{8}$ of an inch in diameter, passing through the holes where the rivets are left out. The nuts are to be sunk into the planks with round holes which are afterwards to be filled up with iron cement, to prevent them from turning and becoming loose.

CAST-IRON BED-PLATES.

The main girders—viz., the middle and two outside girders, are to rest upon cast-iron bed-plates, each 8 feet 2 inches long; such bed-plates, and all the details of the rolling arrangements, to be made according to the drawing approved by the inspecting engineer.

CAST-IRON PIER FOR RECEIVING CENTRE GIRDER.

The centre girder is to be received at each end by a cast-iron pier, the ends of the two outside girders being built into masonry. The cast-iron pier is to be $1\frac{1}{4}$ inches thick at the sides and back, with strengthening ribs at the sides, projecting $7\frac{1}{2}$ inches, as shown in the drawing. The top of the pier may be cast separately and bolted together afterwards. This pier may also be cast in two parts and bolted together in the middle, as shown in the drawing; all the bolts to be $1\frac{1}{4}$ inches in diameter, and 3 inches apart, the holes to be rimered, and the bolts turned and driven in tight. The pier is to have flanges running round, as shown in the drawing, agreeing with the flanges of the bottom bed-plates. The side flanges, as well as the back flange, are to be 7½ inches wide, and 1¾ of an inch thick, as shown in the drawing; and to be bolted to the flanges of the bottom bed-plate by bolts 1¼ inches diameter; the holes to be rimered, the bolts turned and driven in tight. The inside width of the pier is to be 5 feet 7½ inches. The pier is to be 5 feet 8¾ inches deep in the clear inside, leaving 6 inches clear behind the girder. The snugs for bolting the pier at the back are to be arranged so as to fall inside of the girder, so as to allow for the girder expanding. The centre side flanges, for bolting the pier in the middle, are to be inside and outside the pier: the dimensions will be found upon drawing No. 3. The upper part of the pier is to have a distance or guide-piece cast on, as shown in drawing No. 3., giving 1 inch clearance on each side between the distance-piece and the cell of the girder.

GENERAL STIPULATIONS.

All the plates and jointing-pieces are to be sheared, and the rivet-holes in the plates, jointing-pieces, and angle-irons, to be punched by a punching and shearing-machine, so as to ensure squareness and accuracy of distance. Should the slightest inaccuracy occur, so that any of the holes either in the jointing-pieces, angle-irons, &c., should not come fair with the plates, the holes are to be carefully rimered, and care taken that the rivets fill the holes. All the rivets must be of the very best iron, and thoroughly fill the holes, and they must be rivetted up with such weight of hammer as may be directed by the inspecting engineer.

All the plates, angle-irons, and T-irons are to be of best iron. All the rivets and bolts for carrying the cross-bearers are to be of the very best iron. The bolts for fastening the sleepers and flooring are to be of mitre-iron; the whole of the angle-irons are to be jointed by pieces of the same sectional area and rivetted with four rivets, or else carefully drawn down and a tap-joint made, taking care not to diminish the sectional area of the angle-irons at the joints. The joints of the upper angle-irons must have jointing-pieces, and be connected with butt-joints.

The whole of the plates in the upper portion or cell of the girder are to be carefully butted, squared, and close-jointed, so that the whole of the ends of the plates may have a bearing one against another, and not depend upon the rivets for the thrust.

The whole of the plates, angle-irons, &c., of such girders to be completed, and the girders erected complete with the cross-bearers and tie-girders upon a platform in or near the works where manufactured; and the whole is to be cottered and fastened together, and all the holes rimered, and made fair and complete before leaving the works. The cells and bottom to be rivetted up and completed in as long lengths as can be shipped, and the whole of the cross-bearers rivetted up complete before shipping, so as to leave as little rivetting to be done in the erection as possible. The whole of the workmanship and materials must be approved of and certified by the inspecting engineer appointed by the government of Victoria, and resident in England, before leaving the works for final erection.

The whole of the materials required in the construction of the bridge are to be submitted to the inspecting engineer for his approval before being used, and to be approved of and certified by him before the work is commenced; but even after such approval should any of the materials prove unsound, or be objected to by him, such materials so objected to shall be immediately removed and other materials supplied, subject to his approval.

All the workmanship to be subject to the approval of the inspecting engineer, as also the mode of construction; and should any portion of the workmanship, workmen, or plan of construction be objected to by the inspecting engineer, such workmanship, workmen, or plan of construction, to be altered, discharged, or renewed as he may direct.

Should the manufacturer or contractor refuse to conform with any of these stipulations, no money or further sums of money (should any have been advanced) will be paid until such stipulations are duly complied with and carried out.

The whole of the bridge is to be completed and temporarily erected, and approved of by the inspecting engineer by the 15th day of March, 1857. The bridge is to be painted throughout, both inside and outside of the girders, with three coats of redlead and boiled oil; and all the parts are to be numbered, and a complete schedule, fully describing every part, to be made out in duplicate, one copy accompanying the bridge, and the other forwarded by the first ship. The whole is to be carefully taken to pieces and all the minor parts packed carefully in cases and shipped by the 31st day of March, 1857. And for each and every week that the manufacturer or contractor is over or beyond that time in the completion of such bridge, he shall be subject to a forfeiture of the sum of two hundred pounds sterling.

The contractor or manufacturer is to include in his tender all contingencies. Any slight deviation from the dimensions or construction, or omission that may arise which the inspecting engineer may deem necessary to alter, is to be included in the tender, as no extras will be allowed without the written authority of the engineer.

No deviation from the drawing or specification will be allowed, without the authority in writing of the inspecting engineer.

The contractor is to include in his tender, the amount of all patent rights and royalties, which may be claimed by any patentee or patentees, and must undertake to liquidate the same, when required to do so by such patentee or patentees.

The superintending engineer, in England, will have full power to increase or diminish the quantities, and to make any alteration in the work that he may from time to time deem necessary, such alteration or deviation from the quantities (whether the quantities be increased or diminished), to be allowed or deducted from the contract price.

CHAPTER XVII.

TRIANGULAR-GIRDER AND TRELLIS-GIRDER BRIDGES.

Of the various bridges illustrated, those that will be included under this head, are the Taptee Viaduct, the Ebro, Jumna, and Morecambe Bay Bridges; the Beelah and Deepdale Viaducts, and the Londonderry and Charing-Cross Bridges.

Taptee Viaduct is one of a numerous series of bridges erected on the Bombay, Baroda and Central India Railway, and in looking at this one we may judge of all. In designing these structures, care has evidently been taken to ensure the highest degree of economy; that is to say, to construct the bridges at the lowest cost, consistent with a proper amount of strength and rigidity; and, in carrying out this very important point, the engineer has by no means sacrificed the beauty of the structure, for its general appearance is exceedingly light and elegant, although it has withstood tests of a most serious character. We may here observe that on one of the bridges of this series, a passing train was thrown off the line by an obstacle maliciously placed on the rails; the engine was running tender foremost, and the tender tearing through the roadway, hung suspended from the broken cross-girders; and it must be evident to all, that unless the joints of the structure had been excellently executed, the tender must have fallen completely through the platform.

Great attention has also been paid to the construction of the piers, which are probably as light as they can be made, to withstand the storm-floods to which the localities in which they are erected are subject; and the distances between the piers have been nicely adjusted to their heights, so as to obtain a minimum gross cost for the piers and superstructure taken together.

Ebro Bridge, of which we shall next speak, is most remarkable for its lightness, the weight of the superstructure amounting to only 6 cwt. per lineal foot for one line of railway, which, at first sight, lead us to expect that a very great strain upon the flanges would be produced by a full load; we have, therefore, calculated the direct strains upon the flanges at the centre of the span for a total load, and find, for the top or compression flange, a strain of nearly 4 tons per square inch, and for the bottom or tensile member, allowing for rivet-holes, a strain of about 4·6 tons per square inch, both these strains being sufficiently small. From this it appears, that the great saving of metal is not in the flanges, but in the bracing, etc., which, as we have previously observed, cannot be calculated; hence time and wear only can test the durability of this part of the structure. It will be observed that in this bridge the lattice-bars do not cross each other at the joint, but are separately rivetted to the flanges, thereby gaining certain advantages as regards convenience, but accompanied with some loss as to appearance. The bed-plates have evidently been very carefully designed, allowance being made for the variations of position produced by the deflection of the girders, so that the load may be equally distributed upon the rollers in every case, and this is a point which is usually neglected, although it is evidently desirable to provide for it, especially as the extra cost involved is very trifling.

Jumna Bridge, a structure of considerable magnitude, at once attracts the observer's attention by reason of the disposition of the ties and struts. The struts are placed vertically, whereby greater rigidity is obtained, for the tendency of the struts to bend increases in greater ratio than the length of the same; whereas the sectional areas of the ties need in no way depend upon their lengths. The top member is well adapted to resist compressive strain, being of a very rigid section. The greatest strain that can be brought upon the tension bars is about 4·5 tons per sectional square inch, and on the top flange about 4 tons. The weight of the structure is moderate when we consider the extent of the span. In concluding these remarks, we must call the reader's attention to the diagram of the deflections of Jumna Bridge, in which the regularity of the curve is very remarkable, indicating most excellent workmanship on the joints, and this great regularity is fully accounted for by the fact that the sections of which the top flange is made, were faced on the ends in a lathe, dead true, that is to say, that the lengths specified, were actually obtained.

Our next description refers to the Kent and Leven Viaducts, of which the foundations form the most characteristic feature. The disc piles, of which these consist, appear to be peculiarly adapted to the purpose, to which we here find them applied; and we think that there can be but little doubt that so ingenious and useful an invention will be very generally used, and the more especially as the inventor has not patented it, thereby sacrificing, in some measure, his own interests to those of the public generally. The drawbridge, which forms part of the structure now under consideration, is, perhaps, the most suitable form that can be adopted, and the entire viaduct has the appearance of lightness; considerable saving being effected by placing the lines of railway upon the main girders, instead of carrying them upon cross-girders.

The Beelah and Deepdale Viaducts are of the ordinary trellis form, but the spans are very short, which, however, is a great saving, if it does not necessitate a greater quantity of metal in the piers than would otherwise be required. The foundations are very simple, the columns resting upon feet, bolted down to masonry. The method adopted for the erection of this structure was exceedingly novel and inexpensive, no scaffolding being used, the pieces of the structure being placed in position by means of a large jib-crane.

The Londonderry and Charing-cross Bridges exhibit the same principle of construction, but the latter is most worthy of attention. Its great width necessitated the use of very strong cross-girders, which could not be done away with, as the level of the railway would not allow the platform of the bridge to be placed upon the top of the main girders. The width of the centre part of the platform is 49 feet, so that the cross girders are of greater span than the main girders of many bridges.

Having now concluded our general observations, we will proceed with the detailed descriptions of those bridges which are included in the present Chapter.

TAPTEE VIADUCT.

PLATES XXXVI., XXXVII., AND XXXVIII.

Taptee Viaduct, of which a general view is shown in the Frontispiece of the Volume of Plates, is an average specimen of a long series of bridges designed by Lieut.-Col. J. P. Kennedy, to carry the Bombay, Baroda, and Central India Railway across the various rivers by which it is intersected. These rivers must be crossed within tidal influence, and most of them are swept by fierce monsoon currents, and their beds offer the worst class of foundations for masonry piers.

The country traversed by the Bombay, Baroda, and Central India Railway in its course of 313 miles, from Bombay to Ahmedabad, is crossed by rivers opposing, on the average, 103 feet per mile of line, to be crossed by first-class bridges, exclusive of minor bridges and culverts. So great was the estimate of these difficulties, due to the character of the country, that the absolute practicability of constructing the line was seriously disputed by professional men of very considerable experience, and it appears probable that it could not have been executed, except by a greater expenditure of capital than the undertaking would warrant, had the old methods of construction been adhered to, and, even with the adoption of the most recent improvements, it was necessary to scrutinize every detail, in order to obtain the greatest possible degree of economy.

After due consideration, it was decided that the most economical method that could be adopted for bridging the rivers, would consist in employing cast-iron for the piers, and wrought-iron for the superstructure.

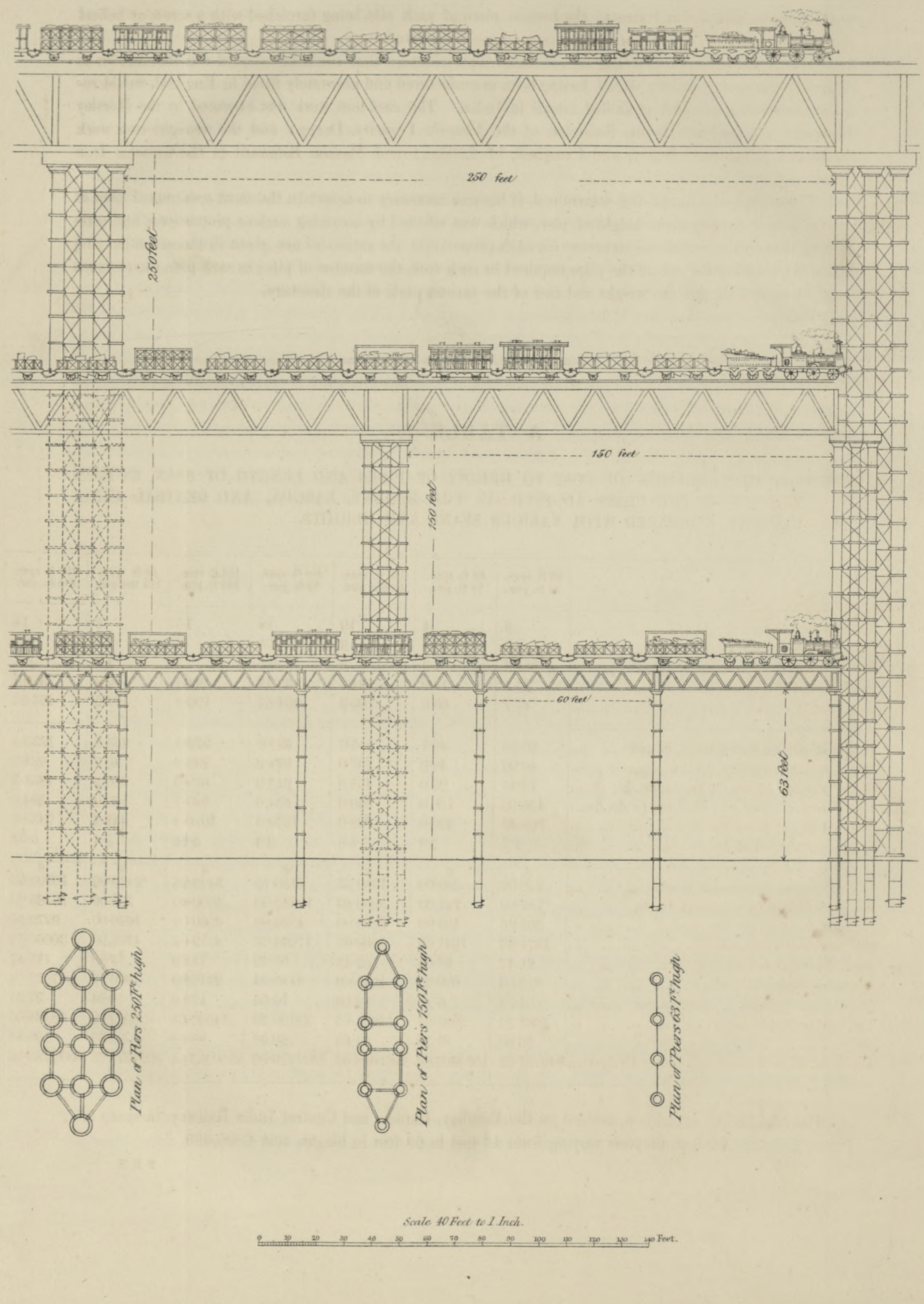
It was concluded also that the piers should consist of cast-iron hollow columns, 2 feet 6 inches in diameter, cast in lengths of 9 feet each, and bolted together at strong flanged joints, and thoroughly braced

To face page 185.

Diagram No 7.

A DIAGRAM

Showing the proportions of Piers and superstructures of various heights as determined from the estimates in the foregoing table.



W. Humber, dir.

Standidge & Co. Litho. 36, Old Jewry, London, E.C.

London: E. & F. N. Spon, 16, Bucklersbury.

with wrought-iron ties and struts; the bottom piece of each pile being furnished with a screw or helical flange, except when the piers are to be erected upon rock foundations.

It was also decided that the best-class of superstructure that could be adopted was that known as Warren's triangular girder; which, having been manufactured and accurately fitted in England, would require the smallest amount of skilled labour in India. The cast-iron work was executed by the Horsley Company, Tipton, and Messrs. Swingler, of the Victoria Foundry, Derby; and the wrought-iron work by Messrs. Westwood, Bailey, and Campbell, of London; and Messrs. Kennard of the Crumlin Iron Works.

These general points being determined, it became necessary to ascertain the most economical span of superstructure for any given height of pier, which was effected by assuming various proportions, and estimating the entire cost of the structure for each proportion; the estimates are given in the accompanying Table (*a*), showing the size of the piles required in each case, the number of piles in each pier, the pressure on the foundations, and the weight and cost of the various parts of the structure.

A TABLE (*a*),

SHOWING THE RELATION OF COST TO HEIGHT OF PIERS AND LENGTH OF SPAN, IN IRON BRIDGES OF THE CLASS ADOPTED ON THE BOMBAY, BARODA, AND CENTRAL INDIA RAILWAY, COMPARED WITH VARIOUS SPANS AND HEIGHTS.

		60 ft. span. 45 ft. pier.	60 ft. span. 63 ft. pier.	150 ft. span. 63 feet pier.	250 ft. span. 63 ft. pier.	150 ft span. 153 ft. pier.	60 ft. span. 153 feet pier.	250 ft. span. 252 ft. pier.
Description.	No. of piles in pier	3	4	10	14	10	8	14
	Exterior diameter of piles, in feet...	2·5	2.5	3	4·5	3	2·5	4·5
	Do. do of pile screw, in feet	4·5	4·5	5	6·5	5	4·5	6·5
	Bearing of pile, in square feet	15·9	15·9	19·63	33·18	19·63	15·9	33·18
	Total bearing of pier, in feet	47·7	63·6	196·3	464·52	196·3	127·2	464·52
Weight.	Weight of pier for double-track, in tons	26·55	46·1	155·0	321·0	372·0	258·3	1230·5
	Do. of superstructure do. do. do.	46·90	46·9	298 0	624·0	298·0	46·9	624·0
	Do. of pier and superstructure do. do. do. ...	72·45	93·0	453·0	945·0	670·0	305·2	1854·3
	Do. of moveable load, and platform do. do. do....	136·00	136·0	340·0	584·0	340·0	136 0	584 0
	Do. total on pier foundation do.	208·45	229·0	793·0	1529·0	1010·0	441·2	2438·5
	Do. per square foot of do. do.	4·3	3·6	4·8	3·3	5·14	3·4	5·24
Cost.		£	£	£	£	£	£	£
	Cost of one pier delivered in London	222·00	389·00	1389·25	2756·75	3423·25	2479·90	10900·55
	Do. superstructure double-track do. do.	787·00	787·00	5006·40	10483·20	5006·40	787·00	10483·20
	Do. erection at £5 per ton	362·25	465·00	2265·00	4725·00	3350·00	1526·00	9272·50
	Do. total erected in London	1371·25	1641·00	8660·65	17694·95	4779·65	4792.90	30656·25
	Do. do. per foot-run do. do.	21·42	25·64	52·48	65·32	71·39	74·88	111·47
	Do. extra charges for India	315·95	390·95	1991·00	4140·34	2810·90	1206·10	7527·75
	Do. do. do. per foot-run	4·93	6·10	12·06	15·05	17·03	18·84	27·34
	Do. total erected in India	1687·20	2031·95	10651·65	22105·29	14590·55	5999·00	38184·00
	Do. do. do. per foot-run	26·36	31·75	64·55	80·38	88·42	93·73	138·84
	Do. of 32,237 feet, about six miles of Bridges ...	849767·32	1023202·38	2080898·35	2591210·06	2850395·54	3021574·01	4475785·08

The 32,237 feet of bridges, erected on the Bombay, Baroda, and Central India Railway, in spans of 60 feet, on piers varying from 45 feet to 63 feet in height, cost £900,000.

The proportions of the various parts of the structures required for different spans and heights, are shown on the diagram facing the Table (*a*).

We must now explain the general arrangement of the cast-iron columns, of which the piers are composed. It was originally proposed to construct all the piers on the plan illustrated, at Fig. 3, Plate F., for the bridges over the Nerbudda, and all similar rivers, where there are strong tidal-currents in both directions; and where the piles are screwed to a depth of about 20 feet into the bed of the river. The inclined piles are not intended expressly as fenders, but to act as struts, in opposition to the force of the current acting upon the opposite side of the pier; their best points of junction with the upright piles are, at the highest flood level; but the first pile, whether upright or inclined, which receives the blow of a floating body, should be protected by timber, as shown; which, by its elasticity, will deaden the force of impact. A pier was being erected in the deepest and fiercest current of the Nerbudda river, at 1062 feet from the north bank, and had advanced to the condition shown in Fig. 2, when the monsoon of 1860 set in, and which the pier, in this unfinished state, most satisfactorily resisted; the proportions of strength of this part to that of the finished structure, were as follows :—Bearing-area, $\frac{2}{5}$; bracings, $\frac{1}{4}$; and length of base, $\frac{1}{7}$.

The apology of the engineers, charged with the construction, for having left the work in this unprepared and hazardous condition, was the sudden dispersion of their workmen on account of cholera, shortly before the occurrence of the monsoon.

At Fig. 1, is shown a diagram of the average class of pier used on the Bombay, Baroda, and Central India Railway; on each element the cost of that element, with its accompanying bracing, is written, so that from this diagram we can estimate the cost of any similar pier.

We have also inserted a Table (*b*) showing the weight, cost, and composition of piers of the class used on the Bombay, Baroda, and Central India Railway.

The heights of the piers on this railway, varying from 45 feet to 63 feet, the proper spans for the superstructure would be about 60 feet; wherefore this span was adopted.

Before recommending the adoption of the Warren-girder system of superstructure, Colonel Kennedy tested a girder of 60 feet span, constructed upon that system, to the breaking point, and found that the results fully justified the adoption of that system. The regular tests, to which the girders intended to carry the superstructure were submitted before leaving England, consisted in applying a load of two tons per lineal foot, which is twice as great as any practical load to which they would be subject.

This test-load was rolled on in trucks from a siding, and it produced a maximum deflection of $\frac{5}{8}$ of an inch only in each 60 feet span; and upon the removal of the load the girders immediately recovered their original camber.

The greatest direct strain to which any part of the structure is subject, under the greatest load which occurs in practice, is 3·75 tons per square inch of sectional area.

In Table (*c*) will be found the weights of the various parts of one 60 feet span; both when the roadway is fixed on the top of the main girders, and when it is fixed on the bottom of the same.

The piers and superstructures for ninety-five bridges, comprising 477 spans, and making about 6 miles of viaduct, have been already shipped for the construction of the line, and 14 bridges, containing 57 spans of 60 feet each, are now giving passage to the trains between Surat and Baroda.

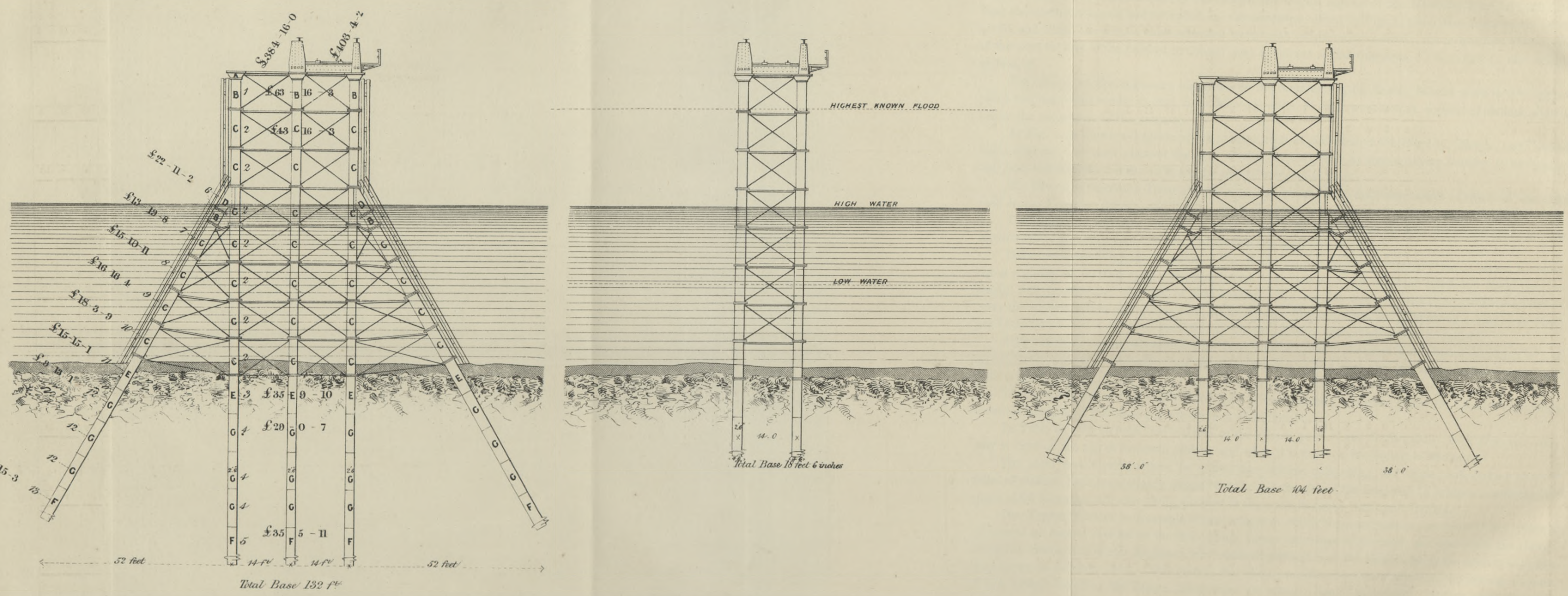
The Taptee Viaduct is a sample of the bridges of which the construction has already been effected on second or central division of the line, which covers an extent of 80 miles, and was under the charge of Mr. J. Burns, as District Engineer, who obtained possession of the land as far as the 48th mile, south from Surat, in October, 1858. This portion was the most difficult of the entire line, the average amount of iron bridges including the Taptee, being twice the average rate of the whole. In fact, about 40 miles in this locality, or $\frac{1}{8}$ of the entire line, included $\frac{1}{4}$ of the gross amount of iron bridges.

The Taptee Viaduct, of which the total length is 1891 feet, spanning a tidal river, and built on an alluvial bed, was opened for the passage of the trains in November 1860, within one year from the sink-

BOMBAY BARODA AND CENTRAL INDIA RAILWAY.

BRIDGE PIERS FOR ALLUVIAL DISTRICTS.

FIG. 1. FIG. 2. FIG. 3.



Scale 20 Feet = 1 inch.

A TABLE (*b*),

SHOWING THE COMPOSITION AND COST OF BRIDGE PIERS, OF THE CLASS USED ON THE BOMBAY, BARODA, AND CENTRAL INDIA RAILWAY.

The superstructures being constant quantities for spans of 60 feet—viz., 1st track, 24 tons of wrought-iron, at £16. 16s. a ton, £406. 11s. 2d. 2nd track, 22 tons 14 cwt., at same rate, £380. 17s. 2d.

Class.	No.	Description.	Cast-Iron, at £7 10s. a ton.							Wrought-Iron, at £17 15s. a ton.							Total Cast- and Wrought-Iron.						
			Weight.				Cost, delivered in London.			Weight.				Cost, delivered in London.			Weight.				Cost, delivered in London.		
			Ton.	c.	qr.	lbs.	£	s.	d.	t.	c.	qr.	lbs.	£	s.	d.	t.	c.	qr.	lbs.	£	s	d.
1	3	Pile caps, A	2	12	3	14																	
	3	Piles, ...	3	19	3	14																	
	2	Horizontal T-iron struts....									3	3	24										
	4	Diagonal L-iron bracings ..									6	2	20										
	3	L-iron rings									3	2	22										
	54	Bolts, nuts, & washrs. 1″ × 4½″									1	0	6										
	4	„ „ 1½″ × 7½″										1	3										
	4	Gibs and cotters											17										
		Total	6	12	3	0	49	15	7		15	3	8	14	0	8	7	8	2	8	63	16	3
2	3	Piles C	4	8	2	9																	
	2	Horizontal T-iron bracings .									4	0	6										
	4	Diagonal L-iron bracings ..									6	3	0										
	36	Bolts, nuts, & washrs. 1″×4½″										2	22										
	4	„ „ 1½″×7½″										1	3										
	4	Gibs and cotters											17										
		Total	4	8	2	9	33	4	4		11	3	20	10	11	11	5	0	2	1	43	16	3
3	3	Piles E	4	3	0	7																	
	2	Horizontal T-iron bracing..									4	0	6										
	30	Bolts, nuts, & washrs, 1″ × 4½″										2	9										
	4	„ „ 1½″ × 6½″										1	2										
		Total	4	3	0	7	31	2	11		4	3	17	4	6	11	4	7	3	24	35	9	10
4	3	Piles G	3	16	0	5																	
	30	Bolts, nuts, & washrs. 1″ × 4½″										2	9										
		Total	3	16	0	5	28	18	4			2	9		10	3	3	16	2	14	29	0	7
5	3	Piles F (screw)	4	14	0	14	35	5	11								4	14	0	14	35	5	11
6	1	Pile D	1	9	1	0																	
	1	Pile B	1	6	2	14																	
	30	Bolts, nuts, & washers, 1″ × 4½										2	9										
	1	L-iron ring									1	0	26										
		Total	2	15	3	14	20	19	0		1	3	7	1	12	2	2	17	2	21	22	11	2
7	1	Pile C	1	9	2	3																	
	1	Horizontal T-iron bracing ..										2	16										
	1	Diagonal L-iron bracing....									1	1	20										
	1	„ „										3	26										
	12	Bolts, nuts, & washers, 1″ × 4½″											26										
		Total	1	9	2	3	11	1	5		3	1	4	2	18	3	1	12	3	7	13	19	8
8	1	Pile C	1	9	2	3																	
	1	Horizontal T-iron bracing..									1	1	26										
	1	Diagonal L-iron bracing									1	3	12										
	1	„ „									1	1	25										
	12	Bolts, nuts, & washers, 1″ × 4½″											26										
		Total.	1	9	2	3	11	1	5		5	0	5	4	9	6	1	14	2	8	15	10	11
9	1	Pile C	1	9	2	3																	
	1	Horizontal T-iron bracing ..									2	1											
	1	Diagonal L-iron bracing....									1	3	26										
	1	„ „									2		14										
	12	Bolts, nuts, & washers, 1″ × 4½″											26										
		Total	1	9	2	3	11	1	5		6	2	10	5	16	11	1	16	0	13	16	18	4
10	1	Pile C	1	9	2	3																	
	1	Horizontal T-iron bracing..									4	0	18										
	1	Diagonal L-iron bracing....									2	1	24										
	1	„ „									1	0	18										
	12	Bolts, nuts, & washers, 1″ × 4½″											26										
		Total	1	9	2	3	11	1	5		8	0	2	7	2	4	1	17	2	5	18	3	9
11	1	Pile E	1	7	2	21																	
	1	Horizontal T-iron bracing..									5	2	17										
	10	Bolts, nuts, & washers, 1″ × 4½″											22										
	2	„ „ 1½″ × 4½″											15										
		Total	1	7	2	21	10	7	8		5	3	26	5	7	5	1	13	2	19	15	15	1
12	1	Pile G	1	5	1	11																	
	10	Bolts, nuts, & washers, 1″ × 4½″											22										
		Total	1	5	1	11	9	9	9				22		3	4	1	5	2	5	9	13	1
13	1	Pile F	1	11	1	14	11	15	3								1	11	1	14	11	15	3
14	1½	Pile C, without bracing	2	4	1	4	16	11	3								2	4	1	4	16	11	3

ing of the first pile; and 41 miles of this portion of the line, including 18 iron bridges, comprising more than a mile and a-half of viaduct, are so far advanced, that one-half of this section has already been opened, and it was anticipated that the remainder would be completed by the 1st of May, thus occupying only 2½ years in its construction, a feat unparalleled in railway operations, elsewhere effected.

Mr. Burns and his assistants have reached the point of being able to erect as many piers at a time as it might be found advisable to carry on simultaneously; each pier being completed in a fortnight, and they could cover those piers with their superstructures at the rate of 2 days per span.

The latest advices from India show that the 81 miles of line already opened, would, before the 1st of May, be increased to 132 miles; that by the 15th of May, 156 miles would be opened; that, by the 1st of June, 173 miles would be completed, including the most difficult viaduct on the line, viz., the Nerbudda bridge, 3700 feet long, crossing a rapid tidal river, and on an alluvial bed, where many of the pier foundations are 43 feet deep, the extreme height of those piers being 90 feet*; that in December, the length opened for traffic would be 191 miles, and finally, that in the working-season of 1862-3, the entire line of 313 miles is expected to be completed.

A TABLE *(c)*,

SHOWING THE DIMENSIONS AND WEIGHTS OF THE VARIOUS PARTS OF THE SUPERSTRUCTURE OF 60 FEET SPAN, USED ON THE BOMBAY, BARODA, AND CENTRAL INDIA RAILWAY.

	ROADWAY ON TOP OF GIRDERS.										ROADWAY ON BOTTOM OF GIRDERS.									
	Length.		Breadth.		Depth.		Weight.				Length.		Breadth.		Depth.		Weight.			
	Ft.	In.	Ft.	In.	Ft.	In.	Tons.	cwt.	qrs.	lbs.	Ft.	In.	Ft.	In.	Ft.	In.	Tons.	cwt.	qrs.	lbs.
2 Compression Beams in 3 lengths each	62	5	1	5¼	0	8	7	3	3	0	62	5	1	5¼	0	8¼	7	3	3	0
1 Cross Girder in Two Pieces	12	2½	0	4½	1	9½		15	0	10	12	2½	0	4½	1	9½		15	0	10
9 Roadway Girders	15	2	0	6⅜	1	0	3	13	1	4										
7 Roadway Girders											15	2	0	6⅜	1	0	2	17	0	0
1 Double Standard	7	0⅝	3	8	1	8	1	9	3	4	6	11⅜	3	8	1	8	1	9	3	4
1 Single	7	0⅝	3	4½	1	8		14	1	4	6	11⅜	3	4½	1	8		14	1	4
14 Small Standards											5	6	1	8	0	6⅜	1	7	1	0
12 Bottom Tension Bars..	15	11¼	0	6½	0	0½		18	2	0	15	11¼	0	6½	0	0½		18	2	0
24 Bottom Tension Bars...	15	11¼	0	6½	0	0$\frac{7}{16}$	1	12	1	6	15	11¼	0	6½	0	0$\frac{7}{16}$	1	12	1	6
16 Bottom Tension Bars...	8	2¼	0	6½	0	0⅜		10	2	14	8	2¼	0	6½	0	0		10	2	14
8 Vertical Tension Bars...	7	7½	0	6½	0	0$\frac{11}{16}$		8	1	14	7	7½	0	6½	0	0$\frac{11}{16}$		8	1	14
8 Diagonal Tension Bars	8	6¼	0	6½	0	0$\frac{11}{16}$		9	1	14	8	6¼	0	6½	0	0$\frac{11}{16}$		9	1	14
8 Diagonal Tension Bars	8	6¼	0	6½	0	0$\frac{9}{16}$		7	2	14	8	6¼	0	6½	0	0$\frac{9}{16}$		7	2	14
4 Struts	8	7	0	6	0	0		11	3	16	8	7	0	6	0	0⅞		11	3	16
20 Struts	8	7	0	6	0	0½	2	6	2	14	8	7	0	6	0	0½	2	6	2	14
16 T Braces for Roadways	13	10	0	4	2½×¾		1	4	1	2	13	10	0	4	2	½	1	4	1	2
16 L Braces for Struts	14	6	0	2½	2½×⅜			11	2	8										
1 Case of Main Pins	3	7	1	6	1	2		7	0	0	3	7	1	6	1					
1 Case of Bolts and Nuts								6	2	2										
½ Case of Rollers and Washers								3	1	14							1	7	1	5
1 Case of Bolts, Nuts and Washers ...								4	3	0										
Total weight......................................							23	19	0	0	Total weight...						24	4	0	5

There was but one bridge constructed with the roadway on the top of the main girders, as, in all other cases, circumstances were unfavourable to the adoption of this method.

In Table (*d*), will be found the general dimensions, cost, time occupied in erection, etc., of the various iron bridges, between Taptee and Bulsar.

* Since the above was written, advices have been received, stating that this part is complete, and opened for traffic.

In concluding this account of iron bridges erected on the Bombay, Baroda, and Central India Railway, we will call the reader's attention to Table (*e*), exhibiting the weight and cost of the piers and superstructure, as also the cost of freightage, insurance, and erection of the same, and also to the accompanying specification, where full particulars of the structures may be found.

A TABLE, (*d*),

SHOWING THE GENERAL DIMENSIONS, COST, ETC., OF IRON BRIDGES FROM TAPTEE TO BULSAR, A DISTANCE OF 43 MILES.

Name of Bridge.	No. of Spans	Length of Bridge including abutments.	Total Cost of Bridge.	Cost per lineal foot.	Time occupied in erection.	Cost of abutments.	Cost of erection in India, including abutments.		Remarks.
							Pr.spn. 60 ft.	Pr.foot-run.	
		Ft.	£	£ s. d.		£	£	£	
1 Taptee ...	30	1891	42,000	22 4 2½	12 months	None.	350	5·33	This bridge was erected in 12 months, including 3 months of the monsoon.
2 Kaki Karee...	2	135	2,090	15 9 7½	2 months	480	375	6·	Erected during the monsoon.
3 Kaki Karee...	2	135	2,000	14 16 3	2 months	480	330	5·28	Do.
4 Ooukair ...	2	135	1,630	12 1 6	2 months	200	175	2·80	Clay, 5 feet. Rock, 4 feet.
5 Humane Karee	1	72	1,100	15 5 6½	1 month	500	580	9·28	On strong clay.
6 Uesor Karee	1	72	1,500	20 16 8	2 months	900	980	15·68	Foundation 25 feet below surface of ground.
7 Mindole River	11	693	12,380	17 17 4	7 months	500	330	5·28	Coarse gravel foundation, one abutment on pile, with dwarf retaining wall.
8 Sagra	2	135	2,000	14 16 3	1 month	400	360	5·79	Strong Clay and Conkar.
9 Poona (N.) ...	13	847	13,550	15 19 11	6 months	420	266	4·25	Piles screwed into rock 3 to 4 ft.
10 Poona (S.) ...	6	385	6,540	16 19 9	6 months	480	290	4·64	Masonry abutments difficult to lay,—coarse gravel foundations.
11 Uumbuca ...	14	885	14,380	16 4 11½	3 months	360	263	4·28	Piles let into clay, 12 feet. Rock, 3 feet.
12 Cavery (N.) ...	11	697	11,950	17 2 10	3 months	480	272	4·35	Bed of piles, a coarse conglomerate.
13 Cavery (S.)...	10	635	10,960	17 5 2½	3 months	480	274	4·38	The third and 4th pile piers 10 feet clay,—from the south end let into the solid rock, 4 feet, without the screw.
14 Nulla	1	72	1,000	13 17 9	1 month	380	460	7·36	Rock foundation.
15 Nulla	2	135	1,850	13 14 0	2 months	700	405	6·48	Do.
16 Nulla	2	135	1,750	12 19 3	2 months	610	355	5·68	Do.
17 Nulla	2	135	1,750	12 19 3	2 months	610	355	5·68	Do.
18 Busbar	15	948	15,350	16 13 10	9 months	360	263	4·28	Do., and piles let into solid rock, for a depth of from 3 to 5 feet, with screw attached.

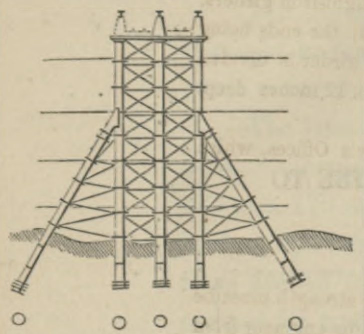
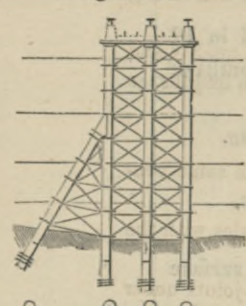
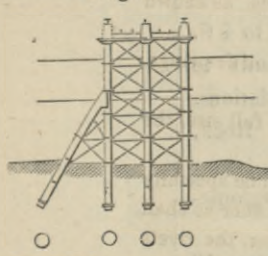
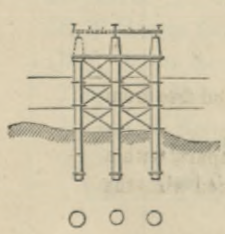
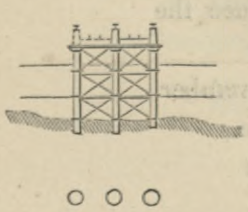
The above return includes the staging and tackle, etc., purchased in India; a large portion of which is available for other bridges (that of the Taptee having been removed to the Nerbudda). The probable value of the whole would be about £7,000, which, when deducted from the total cost would reduce the rate per lineal foot to £16 16*s*.

The whole of the above bridges were executed within a period of 15 months, from the 1st November, 1859, to 1st February, 1861, this included 3 months of the monsoon, when little work can be done.

A TABLE, (*e*),

SHOWING THE WEIGHT AND COST OF THE IRON BRIDGES USED IN THE CONSTRUCTION OF THE BOMBAY, BARODA, AND CENTRAL INDIA RAILWAY, ADAPTED TO PIERS OF VARIOUS HEIGHTS AND STRENGTHS, SUITABLE TO FOUNDATIONS ON ALLUVIAL OR ROCK FORMATIONS.

The span of 60 feet is uniform. The cost is shown for piers for a double-track and for superstructures for single or double-tracks, per span and also per foot-run.

ELEVATIONS.	DESCRIPTION.	Piers 60 ft. apart, Double-track.			Superstructure 60 ft. span. Single-track.			Total for Pier and Superstructure.			Superstructure 60 ft. span. Double-track.			Gross total. Pier and Double-track.		
		Weight.	Cost.		Weight.	Cost.		Weight.	Cost.		Weight.	Cost.		Weight.	Cost.	
			Total.	per foot.		Total.	per foot.		Total.	per foot.		Total.	per foot.		Total.	per foot.
Scale 70 feet 1 in.		Tons.	£	£	Total.	£	£	Tons.	£	£	Tons.	£	£	Tons.	£	£
Diagram A. 	Diagram A 5 pile pier															
	Height from foundation 90 ft.....															
	Delivered in London............	80·6	690	10·78	24·2	406	6·34	104·8	1096	17·12	22·7	381	5·95	127·5	1477	23·07
	Erection at £5 per ton..........		403	6·28		121	1·89		524	8·18		113	1·76		637	9·95
	Total, erected in England........	80·6	1093	17·06	24·2	527	8·23	104·8	1620	25·3	22·7	494	7·71	127·5	2114	33·02
	Freight to India £2 per ton......		161	2·51		48	0·75		209	3·26		45	0·70		254	3·96
	Insurance 3 per cent............		25	0·39		13	0·20		39	0·60		12	0·18		51	0·76
	Government duty at 10 per cent..		69	1·07		40	0·62		109	1·71		38	0·59		147	2·29
	Coast freight in India 10s. per ton		40	0·62		12	0·18		52	0·31		11	0·17		63	0·98
Piers suited to rivers with strong tidal currents on alluvial beds, heights 70 to 80 feet.	Total		298	4·59		113	1·75		439	6·31		106	1·64		515	7·99
	Gross total..............	80·6	1388	21·65	24·2	640	9·98	104·8	2059	31·57	22·7	600	9·35	127·5	2629	40·61
Diagram B. 	Diagram B 4 pile pier															
	Height from foundation 90 ft.....															
	Delivered in London............	64·9	556	8·68	24·2	406	6·34	8·91	962	15·03	22·7	381	5·95	111·8	1343	20·98
	Erection at £5 per ton..........		324	5·06		121	1·89		445	6·95		113	1·76		558	8·71
	Total, erected in England	64·9	880	13·74	24·2	527	8·23	8·91	1407	21·98	22·7	494	7·71	111·8	1901	29·69
	Freight to India £2 per ton		129	2·01		48	4·75		178	2·78		45	0·70		223	3·48
	Insurance 3 per cent.		20	0·31		13	0·20		34	0·53		12	0·18		47	0·73
	Government duty at 10 per cent.		55	0·85		40	0·62		96	1·50		38	0·59		134	2·09
	Coast freight in India 10s. per ton		32	0·50		12	0·18		44	0·68		11	0·17		55	0·85
Piers suited to inland rivers, with strong currents on alluvial beds, heights 70 to 80 feet.	Total		236	3·67		113	1·75		352	5·49		106	1·64		459	7·15
	Gross total.....................	64·9	1116	17·41	24·2	640	9·98	8·91	1759	27·47	22·7	600	9·35	111·8	1360	36·84
Diagram C. 	Diagram C 4 pile pier															
	Height from foundation 63 ft.....															
	Delivered in London	46·1	389	6.07	24·2	406	6·34	70·3	795	12·42	22·7	381	5·95	93·0	1176	18·37
	Erection at £5 per ton..........		230	3·59		121	1·89		351	5·48		113	1·76		464	7·25
	Total, erected in England	46·1	619	9·66	24·2	527	8·23	70·3	1146	17·90	22·7	494	7·71	93·0	1640	25·62
	Freight to India £2 per ton		92	1·43		48	0·75		140	2·18		45	0·70		185	2·89
	Insurance 3 per cent......... ...		14	0·21		13	0·20		27	0·42		12	0·18		39	0·60
	Government duty 10 per cent....		38	0·59		40	0·62		78	1·21		38	0·59		116	1·81
	Coast freight in India 10s. per ton		23	0·35		12	0·18		35	0·54		11	0.17		46	0·71
Average class of pier used on the Bombay, Baroda, and Central India Railway.	Total		167	2·58		113	1·75		280	4·35		106	1·64		386	6·01
	Gross total.....................	46·1	786	12·24	24·2	640	9·98	70·3	1426	22·25	22·7	600	9·35	93·0	2026	31·63
Diagram D. 	Diagram D 3 pile pier															
	Height from foundation 45 ft.....															
	Delivered in London............	26·55	222	3·46	23·95	398	6·22	50·04	620	9·68	22·45	377	5·89	72·18	997	15·57
	Erection at £5 per ton..........		132	2·06		119	1·85		251	3·91		112	1·75		363	5·67
	Total, erected in England........	26·55	354	5·52	23·95	517	8·07	50·04	871	13·59	22·45	489	7·64	72·18	1360	21·24
	Freight to India £2 per ton		53	0·82		48	0·75		101	1·57		44	0·68		145	5·26
	Insurance 3 per cent		8	0·12		13	0·20		21	0·32		12	0·18		33	0·51
	Government duty 10 per cent ..		22	0·34		39	0·60		61	0·95		37	0·57		98	1·53
	Coast freight in India 10s. per ton		13	0·20		12	0·18		25	0·39		11	0·17		36	0·56
Pier adapted to reduced level where the floods do not exceed 20 ft.	Total		96	1·48		112	1·75		208	3·23		104	1·60		312	4.83
	Gross total.....................	26·55	450	7·00	23·95	629	9·80	50·04	1079	16·82	22·45	593	9·24	72·18	1672	26·07
Diagram E. 	Diagram E 3 pile pier															
	Height from foundation 33 ft.....															
	Delivered in London	19.9	169	2.64	24·2	406	6·34	44·1	575	8·98	22·7	381	5·95	66·8	956	14·93
	Erection at £5 per ton		99	1·54		121	1·89		220	3·43		113	1·76		333	5·19
	Total, erected in England........	19·9	268	4·18	24·2	527	8·23	44·1	796	12·41	22·7	494	7·71	66·8	1289	20·12
	Freight to India £2 per ton......		39	0·60		48	0·75		87	1·36		45	0·70		132	2·06
	Insurance 3 per cent.		6	0·09		13	0·20		19	0·29		12	0·18		31	0·48
	Government duty 10 per cent.....		16	0·25		40	0·62		56	0·87		38	0·59		94	1·46
	Coast freight in India 10s. per ton		10	0·15		12	0·18		22	0·34		11	0·17		33	0·51
Piers of piles without screws suited to rock foundations, heights 70 to 80 ft.	Total		71	1·09		113	1·75		184	2·86		106	1·64		290	4·51
	Gross total.....................	19·9	339	5·27	24·2	640	9·98	44·1	979	15·27	22·7	600	9·35	66·8	1579	24·63

SPECIFICATION FOR 60 FEET SPAN WROUGHT-IRON GIRDER BRIDGES.

GENERAL DESCRIPTION. This Specification is for the construction of wrought-iron girders, for 60 feet spans, each span is formed of 2 wrought-iron girders, (Warren's patent principle), 62 feet 5 inches long in the top flange, 7'·9" deep, and 13 feet apart from centre to centre; the ends being arranged to slide on rollers, fitted on the cast-iron* standards, the whole arranged as shown on the drawings. Each girder is divided into 8 — 7'·6" equilateral triangles. Each pair of girders is connected together by wrought-iron transverse girders, 12 inches deep, cross braced with T-irons, as shown on the drawings.

The construction and dimensions of the various parts of the work, are shown in the drawings at the Company's Offices, which must be carefully worked to in the manufacture.

WROUGHT-IRON WORK.

BARS, HORIZONTAL AND DIAGONAL. The iron for these bars must be of such quality and make as shall be specially approved by the engineer, and the strength must be such as to withstand without breaking, a tensile force of 24 tons per square inch; sample bars will be selected by the engineer from each class of iron employed, to be torn asunder in the presence of himself or his assistant, and should any of them break with a less strain than 24 tons, none of the bars supplied by the same iron master will be allowed to be used in the bridges. The bars are to be shaped at the ends, and the holes for the main pins bored with extreme accuracy to gauge.

No bar having an error in the length between the holes amounting to 1-64th of an inch, or in the diameter of the holes amounting to 1-100th of an inch, will be permitted to be used.

THE TOP OF GIRDER STRUTS. The top member of the girder, and the diagonal struts, are to be formed of bars and plates of the best iron, of such quality and make as the engineer shall approve. The top of the girder is to be formed of plates and angle-irons, rivetted together in lengths, as as shown on the drawings; but jointed with joint-plates attached by bolts.

The struts are to be formed of bars, with T-irons on the sides, rivetted together, and shaped at the ends.

HOLES TO BE TRUE WITH EACH OTHER. The holes for the main pins, in both top and bottom of struts, are to be bored with the same accuracy, and under the same conditions as described for the tie-bars.

The holes in the 2 sides of the top, are to be in true line with each other, so that the pins may be exactly at right angles with the plane of the girder.

CAST-IRON DISTANT PIECES. The cast-iron distance pieces, between side plates, are to be formed and connected with bolts as shown. All butt-joints under compression, are to be accurately and truly fitted, and care taken to allow them to bear well against each other over the whole surface.

MAIN PINS. The main pins are to be forged out of Low Moor, Hood and Cooper's, Bowling, or Farnely best iron, as must be attested by the certificate, or invoices of the firm supplying the same.

They are to be turned perfectly parallel, and exact to gauges; no error of more than $\frac{1}{100}$ of an inch in diameter will be allowed; the ends to be screwed $2\frac{1}{8}$ inches in diameter, with hexagonal nuts and split cotters outside the nuts and through the pins, as shown in the drawing.

CROSS BEAMS. The cross-bearers supporting the roadway are to be made of the best Staffordshire plates, rivetted together as shown on drawings. The bracket pieces supporting the cross-bearers are to be soundly forged out of good iron and bored to fit the pins.

DIAGONAL BRACING. The horizontal diagonal bracing is to be formed from best description of T-iron of the section shown on the drawing, full size, the ends being carefully formed, punched, and provided with bolts and washers.

BOLTS, PLATES, RIVETS, &c. All bolts to be made of Low Moor, Bowling, Farnley, or Cooper and Hood's best iron, or such other iron as shall be specially sanctioned by the engineer. They are to be carefully forged and screwed. All rivets shall be of S C crown iron, or such other as shall be specially approved by the engineer. The rivetting generally is to be executed in a careful and workmanlike manner, the rivets thoroughly fitting the holes; great care must be taken that the rivet-holes exactly correspond; should any of them not do so, they shall be rimered out, as no drifting will be allowed.

The engineer may require any holes to be drilled, and the bolts turned for the same, or he may order bolts to be used in the place of rivets, or other changes of this kind, in any such place as he may consider necessary without any extra charge.

CAST-IRON WORK.

GENERAL. The casting packings to be used in the work are to be made from suitable metal, and to be sound, sharp, clean, true, and free from flaws or defects of any kind.

Testing-bars to be cast from every cupola 3' 6" + 2" + 1", one of which is to be tested on edge or bearings 3 feet apart, with a weight of $1\frac{1}{4}$ ton upon the centre, from which the girders, standards, and packings are cast; the other being carefully marked with the date of the cast and put away for the subsequent inspection of the engineer.

STANDARDS TO BE FITTED TO TRANSVERSE GIRDERS, AND ARRANGED FOR THE ROLLERS. Standards to be accurately and securley bolted to transverse girders, with inch bolts, the upper surfaces to be planed perfectly true and faced for the rollers to run on, and receive and connect the wrought-iron girders. The rollers must be of the best Low Moor iron turned perfectly parallel, and fitted to the links with the greatest nicety.

The ends of the top member of the girders to be filled in up to the second pin-hole, with four $\frac{13}{16}$ ihch plates inside the angle-irons' and of the same depth; the plates nearest the angle-irons, on each side to be cut away for the suspension bars. On the outside of these angle-irons, a smalier angle-iron to be put at the bottom and on each side.

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* It was subsequently found desirable to replace the cast-iron standards by others of wrought-iron.

DISTANCE PIECES.

The distance pieces for the top member of the girders are to be cast and fitted as per drawing, and if required by the engineer, a proportion of the holes in them shall be cast small, and afterwards bored to the required size.

WOOD PLATFORM.

BOLTS AND HOOP IRON.

The wood platform and guard rails are not to be included in the contract, as they will be provided in India; but all the bolts necessary for fixing the same must be provided.

MISCELLANEOUS WORK.

THREADS.

All screw threads used throughout the work shall be Whitworth's patterns.

GAUGES.

Proper gauges for the sizes and distances of the main pin holes in the various bars for the main pins themselves, and for any other parts which require accuracy and uniformity, shall be prepared by the contractor at his own expense, and submitted to the engineer for his approval, after which they must be carefully worked to.

TESTING APPARATUS.

The apparatus for proving the power of the resistance of the various parts to tension and compression, and for pulling asunder the sample tension bars, and of any other description required by the engineer, shall also be provided by the contractor at his own cost and charge, and he shall find all assistance necessary for the process testing, or for any other experiments the engineer may desire to try upon the works.

ERECTION AND PROOFS.

The engineer to have the power of calling on the contractor to put every span together complete on his works, if he thinks it desirable to do so, but, at the same time, if from his own opinion of the accuracy and uniformity of the work, he may choose to dispense with the full carrying out of this condition, he may limit the number erected. It is to be so arranged that each girder, when erected, and without other than its own weight shall have a camber of $1\frac{1}{4}$ inch.

Each of the spans so erected must be proved in the presence of the engineer, or his assistant, in the following manner:—A temporary platform to be laid on the cross girders, upon which bridge rails are to be fastened, upon the rails, six trucks (not exceeding 10 feet in length from the extreme ends of buffers), and loaded to 22 tons each, to be pulled backwards and forwards over each span, as the engineer may direct, and the effect on the bridge to be carefully observed.

When the above number of trucks loaded are equally distributed over the bridge and at rest, the entire deflection in the centre on both girders shall not be more than $\frac{3}{4}$ of an inch. And after the trucks have been removed from the bridge there shall remain no visible permanent set, but each girder shall go back to its original camber. Ten per cent. of girders, going down $\frac{7}{8}$ of an inch with the load, and returning to the $\frac{1}{16}$ of its original camber, will be passed; but should any go beyond these dimensions, and return with more than $\frac{1}{16}$ permanent set they will be rejected. The engineer may, should he think proper to do so, call upon the contractor to alter or amend any portions of the rejected work, and should any dispute arise between the Company and the contractor in reference to any of the before or hereinafter mentioned conditions and stipulation in whatsoever manner or thing whatever, the engineer shall have full power to decide, and his award shall be final and binding on all parties.

The contractor is to provide, at his own cost, proper foundations for erecting the spans upon, and all necessary arrangements and labour for the above operation as the engineer may direct.

MARKING.

The several parts of each span shall be distinctly and legibly marked with punch-marks, or in any other form the engineer or his assistant shall define, so as to facilitate the putting together of the work in India; complete references to the marks relating to all shall be prepared and forwarded to the engineer.

PAINTING, PROTECTING, AND PACKING.

The whole of the work must be carefully protected and packed in a manner satisfactory to the engineer, in order to preserve it from injury during a sea voyage to India.

The main pins, and all other bright work, must be carefully coated with one coat of boiled linseed oil, two coats of white paint and one coat of whitelead and tallow mixed, and these, as well as all other small or minute parts, must be packed in strong boxes, well secured.

The whole of the remaining portions of the work must immediately, on being formed (being previously well cleaned), be painted with one coat of best redlead and boiled linseed oil, mixed; after which, to receive three distinct coats of strong white paint and red paint, as instructed by the company's inspector.

GENERAL CONDITIONS OF CONTRACT.

COMPLETENESS.

The contract is intended to include the entire completion of the bridges ready for fixing, and the provision of the whole of the ironwork necessary for the same. It is, therefore, expressly to be understood that all bolts, rivets, washers, and other minor parts, which may not be shown in the drawings or mentioned in the specification, but which may reasonably be considered as being requisite for the proper completion of the work, are to be provided by the contractor and included in the contract sum.

QUALITY OF MATERIALS AND WORKMANSHIP.

The whole of the materials shall be of the best quality of their respective kinds, and shall be subject to the special approval of the engineer, who shall have power to prohibit any materials being used which he may consider unfit for the purposes for which they are intended.

The quality of the workmanship throughout, and more particularly in regard to the fitting of the main pins, is to be of the most perfect description, and is to be more of the nature of that known as engine-work than such as is ordinarily employed on girder-bridges.

CONTRACT.

TO BE EXECUTED TO THE SATISFACTION OF THE ENGINEER.

The whole contract is to be executed in every respect in perfect conformity with the drawings and this specification, and to the entire satisfaction of the company's engineer, who shall have the power to inspect, or of appointing a person to inspect, the entire manufacture, and of testing the work at any time during its progress in any manner he may think fit, and of rejecting all such portions as, in his opinion, are incorrectly made or inferior in strength or quality, and in case any dispute shall arise as to the meaning of any part of the drawings or specification, or as to the quality of work or materials, or any other matter or thing connected with this contract, the engineer shall have the power of deciding the same, and his decision shall be final and binding on all parties.

The contractor shall attend to and forthwith execute all directions in reference to the construction of the work, which may, from time to time, be given to him by the engineer; and in case any slight deviations from the drawings or the specification shall be considered advisable, the engineer shall have the power of ordering them to be done, and no extra charge shall be allowed for the same, unless the engineer shall agree to such extra charge, in writing, before the work it refers to is executed.

TIME OF COMPLETION. The entire contract is to be completed by the One third to be delivered on or before the and

PLACES OF DELIVERY. The ironwork is to be delivered free alongside at any dock or any wharf, or in any part of the stream in the port of London, as the engineer may direct.

The contractor shall not assign or sublet this contract, or any part thereof, nor allow any other portion of the work to be done other than in his own establishment, without the express written consent of the engineer being first obtained.

PATENT RIGHT. The patent rights of the bridge shall be borne by the Company, and need not, therefore, be included in the tender.

The contractor is to include in his tender the expense of preparing the contract, and all painting, packing, and packing-cases, together with all drawings and templates that may be required by the engineer or his inspector.

CONSEQUENCES OF DELIVERIES NOT BEING PUNCTUAL. In the event of the deliveries not being made according to the contract—the punctual deliveries being important, with a view to the Company's general arrangements—it must be distinctly understood to be a condition of the contract that in the event of delay in delivery, deductions will be made by the Company upon the agreed price according to the following scale:—1 month, 4 per cent., 2 months, 10 per cent, 3 months, 20 per cent., and in the event of any delay, exceeding 3 months, they shall be at liberty, at any time thereafter, notwithstanding any subsequent acceptance by them of any portion of the work, to declare the contract at an end, and in which case they shall be liable to pay only for such portions of the work as may have been delivered up to the time of such declaration, deducting any loss or damage sustained by the breach of the contract.

PAYMENT. No payment will be considered as legally becoming due to the contractor until the completion of the contract, but advances of money on account will be made to him at such times and to such amounts as the engineer shall recommend. The balance shall be paid within one month after the engineer shall have given his certificate of the satisfactory completion of the contract.

PAYMENT NOT TO BE CONSIDERED AS EVIDENCE OF WORK DONE. It is, however, to be expressly understood, that such advances are in no case to be considered as evidence of any particular quantity of work having been executed, or of the manner of its execution being satisfactory, or in any other way to relieve the contractor from the liabilities he may sustain under the terms of the contract.

SPECIFICATION FOR SCREW-PILE PIERS FOR BRIDGES.

DESCRIPTION.

The bridge piers are to be constructed of cast-iron screw piles braced together with wrought-iron struts and braces, as shown on the drawing; and hereinafter set forth and described.

CLASS OF PILE, AND NUMBER OF DISTINCT PIECES IN EACH. The piles are to be of the description known as Mitchell's patent screw piles, and are to be cast in lengths of 9 feet, from six distinct patterns, as shown on the drawing and marked **A**, **B**, **C**, **D**, **E**, **F**.

CLASS OF IRON TO BE USED. These piles are to be cast from the very best toughened cast-iron, and to be perfectly free from all flaws, cracks, holes, or defects of any kind; the class of iron used, to be to the entire satisfaction of the Company's engineer, or his inspector, who will have the power of testing the quality of the iron in any way, or at any time they may think proper, the contractor providing every assistance for this purpose.

Testing bars to be cast from every cupola from which the piles are cast, 3′ 6″ × 2″ × 1″, one of which is to be tested on edge, on bearings 3 feet apart, with a weight of one ton and a-quarter upon the centre; the other being carefully marked with the date of the cast, and put away for the subsequent inspection of the engineer to the railway.

The piece of pile marked **A**, to be made to the exact form and size shown on drawing; the top, or bearing part for girder, to be planed perfectly level, and inside bearing part for top of pile, to be perfectly parallel with the top when planed.

DESCRIPTION OF PART OF PILE MARKED **B**. The piece of pile marked **B**, to be 2 feet 6 inches outside diameter, 1 inch thick, and 2 feet long outside. To be left plain at top, and without flange inside or out; to have an outside flange at bottom end 3½ inches, and 1½ inches thick; to have 4 lugs cast on flange end, with oblong slots 3 inches long, by 1 inch wide, for gibs and cotters; these holes are to be perfectly true with each other, and to be clean throughout, and square with both lugs, to be in the positions shown on the drawing.

DESCRIPTION OF PART OF PILE MARKED **C**. The part of pile marked **C**, to be 2 feet 6 inches outside diameter, 1 inch thick, and 9 feet long outside; to have one outside flange at each end, 3½ inches wide, and 1½ inches thick, and to have eight lugs, cast on four at bottom, and four at top end, with holes 1½ inches diameter, cast in the top lugs for cotter bolts and slots; 3 inches long, by 1 inch wide, in bottom lugs for gibs and cotters, in the positions as shown on the drawing; and same as described in pile marked **A**.

DESCRIPTION OF PART OF PILE MARKED **D**. The part of pile marked **D**, to be made to the sizes shown on the drawing; to join the top of the oblique pile, with the perpendicular piles, in the form of a clam, or jaw; to have holes cast in the clam, and at the bottom, in the positions as shown; to be clean throughout, and 1 inch diameter.

DESCRIPTION OF PART OF PILE MARKED **E**. The part of pile marked **E**, to be 2 feet 6 inches outside diameter, 1 inch thick, 9 feet long; to have an inside flange at one end, 3½ inches wide, and 1½ inches thick; and outside flange at the other end, 3½ inches wide, and 1½ inch thick; to have 4 lugs cast on the end having the outside flange, and in the positions as shown on the drawing; the lugs to have holes for cotter bolts, 1½″ diameter; and to be, in every respect, similar to those on pile marked **A**.

DESCRIPTION OF PART OF PILE MARKED **F**. The part of pile marked **F**, to be 2 feet 6 inches diameter, 1 inch thick, and 9 feet long; the upper part to have an inside flange, 3½ inches wide, and 1½ inch thick; and at bottom end, to be furnished with a helical flange, or screw, projecting 1 foot from the outside of the pile; and of the form and dimensions as shown on the drawing. This screw must be cast on the pile, and must be perfectly true and regular in its pitch. The lower, or cutting edge, must be carefully sharpened, or chamfered off on its upper side.

JOINTS OF PILES.

The several lengths of such piles are to be connected by means of the flanged joints, before mentioned, which are to be perfectly true and square, with the entire line of pile; the outside flanges bolted with 12 bolts, 1″ in diameter; and the inside flanges 10 bolts, 1″ in diameter. The flanges, in all cases, are to be strengthened by feathers, as shown.

The bolt-holes, in the outside flanges, are all to be bored from one template throughout the work, so that all the holes may be uniform; the holes in the inside flanges to be bored from one templet also, for the same purpose.

STRUTS AND BRACES.

The braces are to be made from the best Staffordshire angle and T-irons, and to be of the forms and sections shown in the drawings; to be secured to lugs, cast on the piles by means of the cotter bolts, and gibs, and keys.

HORIZONTAL STRUTS.

The horizontal struts are to be of T-iron, of the section shown on the drawing, and connected to the lugs of the piles by 1½ inch bolts.

DIAGONAL BRACES.

The diagonal braces are to be of angle-iron, of the section shown on the drawing, placed back to back, and bolted, or rivetted, firmly together at their crossing.

The upper part, or ends, of these braces, to be secured within the same lugs, and by the same bolts, as the horizontal struts; and the lower ends secured to distinct lugs, and fastened by gibs and cotters.

BOLTS AND NUTS.

The bolts and nuts are to be made from the best Low Moor, Farnley, Hood, and Cooper, or Bowling Iron; to be the exact form and dimensions shown on the drawing; to be screwed full throughout, and nut to fit well on bolt; to be Whitworth's thread.

GIBS & COTTERS.

To be made to the form shown on the drawing; to be of the best Staffordshire iron.

PAINTING, PACKING.

The whole of the bolts, &c., and gibs, and cotters, are to be heated, and dipped into boiled linseed oil, and to be packed in strong cases, hooped with iron.

The whole of cast-iron work is to be painted with 2 coats of good strong paint—stone, or red colour; and the wrought-iron to receive 1 coat of linseed oil, and 2 coats of good strong paint, as above.

CONTRACT TO BE EXECUTED TO THE SATISFACTION OF THE ENGINEER.

The contract to be executed, in every respect, to the satisfaction of Company's engineer, who shall have the power of inspecting, or appointing a person to inspect, the entire manufacture of the work, at any time during its progress; and of rejecting all such, as in his opinion, are incorrectly made, or inferior in strength or quality; all such defective parts shall be immediately replaced at the contractor's expense.

THE CONTRACTOR PROVIDING ASSISTANCE.

The contractor is to provide, without charge, all labour or assistance the engineer, or his assistant, may require for testing the accuracy of the work.

DELIVERY.

The whole of the work, complete, to be delivered free alongside at any dock, or on any wharf, or in any part of the stream in the ports of London, Liverpool, or Glasgow, as the Company may direct.

TEMPLATES AND CARRIAGE OF SAMPLES, &c.

The contractor is to include in his tender the expense of preparing the contract, and the cost of all painting, packing, packing-cases; also of all drawings, templates, carriage of samples, &c., that may be required by the engineer, or his inspector.

THE CONTRACTOR TO REPLACE THE BREAKAGES.

The contractor is to replace, at his own cost and charge, any part or parts of the before-mentioned work, that may be broken, or damaged in carriage or delivery, or otherwise, previously to its coming into the possession of the Company.

DELIVERY.

In the event of the delivery not being made according to the contract—"the punctual delivery being important, with a view to the Company's general arrangements"—it must be understood to be a condition of the contract that, in the event of delay in delivery, deductions will be made by the Company, upon the agreed price, according to the following scale, viz.:—

For 1 month, 4 per cent.
" 2 months, 10 per cent.
" 3 months, 20 per cent.

And, in the event of any delay exceeding 3 months, the Company shall be at liberty, at any time thereafter, notwithstanding any subsequent acceptance, by them, of any portion of the work, to declare the contract at an end; and, in which case, they shall be liable to pay only for such portions of the work as may have been delivered up to the time of such declaration, deducting any loss or damage sustained by the breach of the contract. The engineer, in all cases, shall have the power to decide on any point of dispute that may arise in reference to this contract; and his award thereon shall be final and binding on all parties.

The total quantity of cast-iron pieces, forming piles, required by the Company, as per mark, are as follows, viz.:—

Pieces of pile marked **A**, required for bridge.
" " **B**, Do.
" " **C**, Do.
" " **D**, Do.
" " **E**, Do.
" " **F**, Do.

PAYMENT.

No payment will be considered as legally becoming due to the contractor until the completion of the contract; but advances of money will be made to him at such times as the engineer shall recommend, to an extent not exceeding 90 per cent. of the value of goods delivered; the balance being retained, without interest, until the engineer shall have given his certificate of the satisfactory completion of the contract. It is, however, to be expressly understood that such advances are, in no case, to be considered as evidence of any particular quantity of work having been satisfactory; or, in any way, to relieve the contractor from the liabilities he may sustain, under the terms of the contract.

EBRO BRIDGE.

PLATES XXXIX. AND XL.

This bridge was constructed for the purpose of carrying the Saragossa and Alsasua Railway, over the River Ebro at Cadrieta, near Tudela, in Spain, under the superintendence of Signor Don Angel Retortillo, of Madrid. It consists of 21 spans, of 101 feet 10 inches each, from centre to centre of the piers, the total length, exclusive of the abutments, being 2138 feet 6 inches, and is supported upon 20 piers, each formed of two cast-iron cylinders, sunk deep into the bed of the river.

The superstructure is for a single line, and is chiefly composed of wrought-iron, and consists of a top compression member formed of wrought-iron plates, and a bottom tension member formed of wrought-iron bars, which are connected by means of two series of equilateral triangles, the length of the side being 10 feet, $11\frac{1}{4}$ inches. The top member at the centre of the span, is composed of one plate 12 inches by $\frac{11}{16}$, 2 bars 6 inches by $\frac{1}{2}$ inch, and two angle bars 7 inches by 3 inches by $\frac{5}{8}$ inch, the sectional area being 24·75 square inches, and the greatest strain 4 tons per sectional inch; this area is gradually diminished from the centre to the ends, where it is but 9 square inches. The bottom member, at the centre of the span, consists of 4 bars 9 inches by $\frac{5}{8}$ inch, and at the ends, of 2 bars 9 inches by $\frac{1}{2}$ inch, the tension beams are braced together by a wrought-iron triangular bracing, as shown in Fig. 2, Plate XXXIX. Those of the diagonals acting as struts, are made of bar- and angle-iron, and those acting as ties, of flat bars; the end strut consists of one flat bar 6 inches by $\frac{3}{8}$ inch, and two angle bars 4 inches by 3 inches by $\frac{5}{8}$ inch, and the end tie, of 2 bars 6 inches by $\frac{3}{8}$ inch, the centre struts are formed of one bar 6 inches by $\frac{1}{4}$ inch, and two angle bars 4 inches by $3\frac{3}{8}$ inches by $\frac{3}{8}$ inch, and the centre ties, which also act as struts, are formed of two T-iron bars 4 inches, 3 inches by $\frac{3}{8}$ inch.

The roadway is supported on 19 transverse girders in each span, placed 5 feet $5\frac{5}{8}$ inches apart; these girders are composed of $\frac{1}{4}$-inch plate, 9 inches deep, and four angle-irons 3 inches by 3 inches; they rest on the longitudinal main girder, and are braced together by means of a strong wrought-iron bracing.

The ends of the girders rest upon cast-iron rollers, moving in bed-plates, fixed on the tops of the piers.

The bridge is surmounted by a handsome cast-iron parapet, which adds much to the general effect.

The cylinders were made in lengths of 6 feet, each length being formed of four castings, with vertical flanged joints bolted together; the general diameter is 6 feet, but as it is in contemplation to make a double line of railway, and the inner cylinder would then have to carry an additional girder, its diameter at the base is increased to 8 feet; the weight is 1 ton per foot-run for each pier. The cylinders were sunk by means of scroops or dredging, until the depth was too great, when divers were employed, using Heinke's diving-dress; the two river piers in deep water were entirely sunk by means of Hughes' pneumatic process; all the others were completed by this process; the depth from the surface to the conglomerate was from 20 to 30 feet, and the difficulty of sinking was much increased by the rapidity of the current, which at flood-time reached 15 miles an hour; the cylinders were all sunk through the alluvial and conglomerate, and some feet in the solid rock, and were filled in with concrete. The sinking advanced by the dredging process from 16 inches to 4 feet in 12 hours.

The following are the weights of a single span:—

	Tons.	cwt.	qrs.	lbs.
Two compression beams	7	8	0	6
Two tension beams	6	2	0	22
Standards at ends	1	10	3	6
Struts and ties	6	17	1	4
Nineteen transverse girders	3	4	3	25
Cross bracing	0	17	0	9
End bracing	1	2	0	18
Handrail tubes	0	17	2	20
Pins and nuts, &c.	0	1	3	23
Rivets, &c.	0	5	0	16
Bolts and nuts for fixing cross girders, &c.	0	1	2	0
End bracing (cast-iron)	1	0	0	0
Handrail standards	1	16	1	4
Standard brackets	0	8	1	7
Saddles at ends of girders	0	5	2	24
Total weight of superstructure for a single span, or 6·2 cwt. per foot-run	31	19	0	10

These bridges were designed by Mr. T. W. Kennard, of 36, Great George Street, Westminster, and erected for Exmo Sr. Don José de Salamanca, and the whole of the ironwork was made by Messrs. Kennard Brothers, at their works at Crumlin, near Newport, Monmouthshire.

A similar bridge, but of 16 spans, was erected over the River Aragon, at Marcilla, and several others have been erected on this principle in Spain, Portugal, and Italy.

JUMNA BRIDGE.

PLATES XLI. TO XLV.

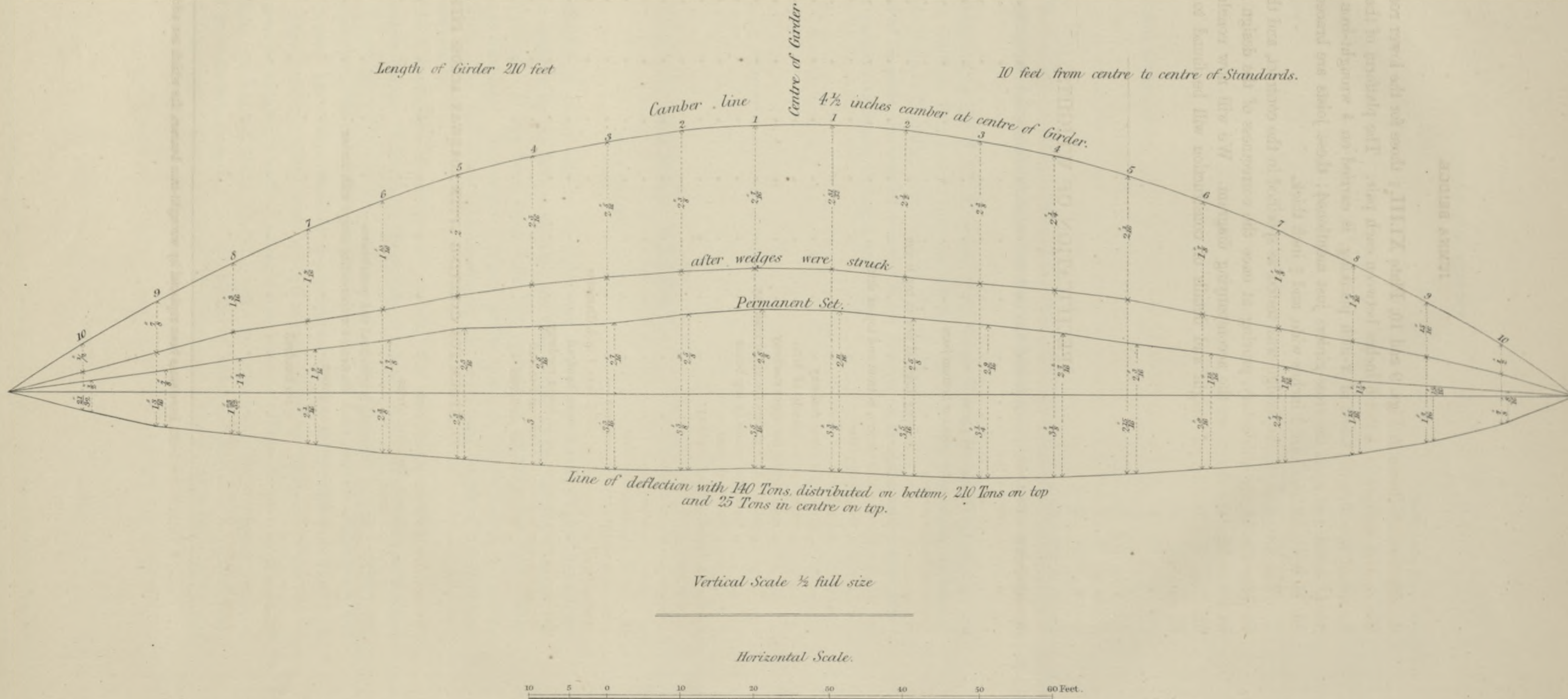
We feel much satisfaction in being able to lay before our readers full details of this magnificent structure, which has been designed by the Messrs. Rendel, and constructed by the Canada Company, at the Canada Works, Birkenhead. It has been erected for the purpose of carrying the East Indian Railway over the Jumna, at Allahabad. It consists of fifteen similar spans, of 205 feet each, of fourteen piers, each 14 feet 6 inches in width, and of two land abutments, and has a total length of 3,300 feet, or considerably more than half-a-mile.

The design provided for a double line of railway and roadway, the rails being carried on the tops of the main girders at a height of about 17 feet from the surface of the roadway. Each span was intended to consist of two distinct bridges, placed side by side, and each bridge was designed to carry a railway and roadway. The structure, however, has only been completed for a single line, though the masonry and iron superstructure of the piers have been executed in accordance with the design shown in Figs. 1, 2, and 3, Plate XLI. The main girders, of which there are two to each bridge, are composed of the following parts, viz., a top compressive member formed of wrought-iron boxes or cells, a bottom tension flange of wrought-iron bars or links (see Fig. 13, Plate XLIII.), and a web which is of the triangular principle, and consists of a series of wrought-iron struts and ties, the former of which are placed in a vertical position. The cross girders for the upper roadway are placed at distances of 1 foot 6 inches apart, and are of

To face page 197.

DIAGRAM SHOWING DEFLECTIONS DURING PROOF OF ONE SPAN OF JUMNA BRIDGE.

MESSRS. A. M & G. RENDEL, ENGINEERS.



W. Humber, dir.

87 P. 236.

Standidge & Co. Litho. 36, Old Jewry, E.C.

London: E. & F. N. Spon, 16, Bucklersbury.

the construction shown in Figs. 9 and 10, Plate XLIII.; those for the lower roadway are placed in pairs at a central distance of 4 feet 6 inches between each pair. The platform of the lower roadway, which is formed of a double thickness of 2-inch planking, is carried on 4 wrought-iron continuous longitudinal joists,* which rest upon the cross-girders just mentioned; these joists are braced together on the underside by wrought-iron bars 3 inches wide, and $\frac{3}{4}$ inch thick.

The first span of the bridge was tested as specified in the contract, and the results, which must be considered as highly satisfactory, proving at once the correctness of the design and the accuracy of the workmanship, are shown on the accompanying diagram. We will now conclude our remarks, as the following specification of weight and details of construction will be found to supply most ample information.

SPECIFICATION OF WEIGHTS.

	Tons.	cwt.	qrs.	lbs.
Wrought-iron in bottom bars	60	1	1	21
„ „ diagonal ties	38	12	0	21
„ „ struts	16	14	3	7
„ „ shaped washer plates	0	8	2	14
„ „ 320 turned bolts for bottom bars	2	13	0	7
Cast-iron distance pieces	0	14	2	14
Wrought-iron in 296 bolts, connecting diagonal ties to top boxes	1	19	2	14
„ „ end framings	6	4	1	21
„ „ connecting girders, bottom and top of ditto	2	12	1	21
„ „ chequered plates	0	5	2	0
„ „ top boxes	68	17	2	21
„ „ girders for lower roadway	6	19	0	14
„ „ washer plates at ends of ditto	2	8	2	7
„ „ main girders for upper roadway	5	12	2	0
„ „ intermediate girders for upper roadway	10	14	0	14
„ „ horizontal braces to top boxes	2	18	2	14
„ „ 84 lugs to ditto	1	0	2	14
Cast-iron in curbs to upper roadway	11	11	2	7
Lead in washers for ditto	0	3	1	0
Wrought-iron in screw bolts	0	2	0	21
„ „ drain-pipes	0	0	1	14
„ „ clamps for ditto	0	0	1	0
Cast-iron in railing for upper roadway	2	12	0	21
Wrought-iron in handrail bars, rolled iron, 1 inch diameter	0	9	3	14
„ „ hollow pipes, forming handrail	0	12	2	7
Cast-iron in brackets for railing of lower roadway	1	3	2	0
Wrought-iron in hollow pipes forming handrail	1	12	1	7
„ „ bolts for timber platforms	0	13	3	7
	251	6	1	14

PLATFORM FOR CARRYING UPPER ROADWAY ACROSS PIERS.

Wrought-iron in plates and angle-irons	1	2	3	14
„ „ $\frac{5}{16}$ inch chequered plates	0	0	3	21
„ „ 2 straps	0	0	0	10
„ „ 28 bolts connecting longitudinal and cross beams	0	0	2	21
„ „ 8 screw bolts, $\frac{5}{8}$ inch diameter for connecting straps with timber	0	0	0	14
„ „ 6 ditto for securing gutter	0	0	0	7
„ „ handrail bars	0	0	1	0
„ „ hollow pipes for upper handrail	0	0	3	0
Cast-iron in one length of curb	0	6	2	14
„ „ 2 standards	0	1	1	0
Lead in washers for bedding ditto	0	0	0	4
	1	14	0	0

* These timber joists have since been replaced by wrought-iron bearers, for which see additional specification.

PIER SUPERSTRUCTURE.

	Tons	cwt.	qrs.	lbs.
Wrought-iron in plates and angle-irons	2	3	0	0
„ „ 12 1-inch bolts with lewis ends	0	0	1	21
„ „ 70 screw bolts to connect cornices	0	0	2	21
„ „ ditto to connect cast moulding	0	0	0	21
Cast-iron in moulded base	0	7	1	0
„ „ mouldings	0	3	3	14
„ „ moulded and enriched cornices	2	2	1	14
„ „ lower ornamental railing	0	3	3	7
„ „ upper ditto	0	12	0	0
	5	13	2	14

ONE SET OF BED-PLATES FOR PIERS, WITH ROLLER ARRANGEMENTS.

Cast-iron in 4 bottom plates, 4 intermediate ditto, and 4 upper ditto	14	4	1	21
„ „ 20 rollers	1	18	2	0
Wrought-iron, in 4 frames, for ditto	0	13	1	14
„ „ 64 screw bolts and 24 with lewis ends	0	9	2	7
„ „ screw bolts	0	6	0	0
„ „ 8 cover plates at ends of upper bed-plates	0	3	0	7
	17	14	3	21

ONE SET OF BED-PLATES FOR PIERS, WITH FIXED BEARINGS.

Cast-iron in 4 bottom and 4 upper plates	13	11	1	14
Wrought-iron in 8 covers at ends of upper plates	0	3	0	7
„ „ in screw bolts	0	9	1	7
	14	3	3	0

ONE SET OF BED-PLATES FOR LAND PIERS, WITH ROLLER ARRANGEMENTS.

Cast-iron in 2 bottom, 2 intermediate, and 2 upper plates	7	2	1	0
„ „ 2 abutment plates	0	12	0	7
„ „ 10 rollers	0	19	1	0
Wrought-iron in 2 frames for rollers	0	6	2	21
„ „ 32 screw bolts	0	3	0	14
„ „ 24 ditto, with lewis ends	0	3	1	7
„ „ 4 tie bolts	0	4	1	0
„ „ 56 screw bolts	0	2	2	0
„ „ 4 cover plates at ends of upper bed-plates	0	1	2	7
	9	15	0	0

ONE SET OF BED-PLATES FOR LAND PIERS, WITH FIXED BEARINGS.

Cast-iron in 2 bottom and 2 upper plates	6	15	3	0
„ „ 2 abutment plates	0	12	0	7
Wrought-iron in 4 covers at ends of upper plates	0	1	2	7
„ „ 24 screw bolts	0	3	1	7
„ „ 4 tie bolts	0	4	1	0
„ „ 56 screw bolts	0	2	2	0
	7	19	1	21
Total weight of duplicate parts supplied	75	15	0	0

In order to arrive at the aggregate weight of the bridge it will be necessary to multiply the above quantities as follows :—

Tons.	cwt.	qrs.	lbs.						
251	6	1	14	× 15 the No. of spans =	3769	15	2	14
1	14	0	0	× 14 platforms =	23	16	0	0
17	14	3	21	× 7 sets of bed-plates, with roller arrangements =	124	4	2	7
14	3	3	0	× 7 sets of bed-plates for fixed bearings =	99	6	1	0
9	15	0	0	× 1 for land pier =	9	15	0	0
7	19	1	21	× 1 for land pier =	7	19	1	21
				Weight of superstructure for one line of railway and roadway	4034	16	3	14
				Weight of ironwork in piers	159	4	0	0
				Total weight of ironwork in bridge as executed	4194	0	3	14

which, for a length of 3,330 feet, gives a total weight of iron per foot-run of 1·235 tons.

SPECIFICATION.

The work to which this specification has reference, is the construction; temporary erection on the contractor's premises; painting; packing; and delivery at some port in England; all, as hereinafter set forth, of 15 similar spans of a wrought-iron girder bridge, intended to carry the East Indian Railway across the river Jumna, at Allahabad.

THE WORK CONSISTS.

FIRST, of 15 wrought-iron girders, of 205 feet clear span, arranged to carry a rail and roadway.

SECONDLY, of the bed-plates on which those girders rest, viz., seven sets for intermediate piers, and one set for a land pier, arranged with rollers to meet expansion; seven sets for intermediate piers, and one set for a land pier, without expansion rollers.

And THIRDLY, of the ornamental superstructure for the intermediate piers (which is to be supplied at once), as for a double line of girders, making, in all, 28 single frames.

It is intended to comprise, under this contract, the whole of the ironwork, cast and wrought, ornamental, or otherwise, shown on these 10 drawings, or mentioned in this specification, excepting only the rails and chairs. It is also intended to comprise the supply of all bolts, lewis bolts, wood screws, washers, pins, &c.; of all the rivets necessary for the erection of the work in India; and of the extra parts intended to cover loss and waste, enumerated in the list annexed to this specification.

MATERIALS.

OUGHT-IRON. The whole of the wrought-iron, used for the work to which this specification refers, is to be free from scales, blisters, laminations, and all other defect; and is to be of a quality to be specially approved by the engineers to the Company; and to be accurately of the sections and weights shown in the drawings. The angle-irons and bars, in particular, are to be sound, and regular on the edges; and any that are not so will be rejected. No plates, bars, or angle-irons, will be approved which are found to exceed, or fall short, of the specified weights and dimensions, more than 5 per cent.

The bars, and all parts of the girders, whatsoever, which may be subjected to tension, are to be capable of bearing a tensile strain of, at least, 20 tons on the square inch of section, under the concussion of a blow struck with a heavy hammer.

RIVETS. The rivets are all to be of the best quality of Staffordshire rivet-iron; they are to be cup-headed at each end (except where shown or required to be countersunk), and are to have well-shaped heads, containing not less than would form a length of $1\frac{3}{16}$ times the diameter of the rivet. The engineers will be at liberty to have rivets cut out from the work in their presence, from time to time, or to take any other means, they may think fit, to judge of the toughness and quality of the iron used.

.TS AND NUTS. The bolts and nuts, all wood screws, lewis bolts, and other bolts, and the horizontal braces, are to be made of the Monk Bridge "best," or of Bradley's S. C. crown iron, or of such other iron of equal quality, as shall be specially approved by the engineers.

.LER FRAMES. The frames of the expansion roller-bearings, are to be forged from Low Moor, or Farnley, or the best Monk Bridge iron.

'AST-IRON. All the castings are to be clean, sharp, true to form, and free from air-holes or other defects. The bed-plates and rollers are to be cast from a mixture specially selected for hardness.

MATERIAL ENERALLY. All the materials, whatever, used in the work to which this specification applies, are to be thoroughly good, and the most suitable of their respective kinds; and, in all cases, are to be approved by the engineers, in writing, before being used. They will also have to bear the before-mentioned, or any other tests, to which the engineers may think fit to subject them; and the contractors are to provide any machinery that may be required to make such tests, at their own expense, as well as all labour and other charges incident thereon.

The contractors will be required to give the engineers all the particulars, they may desire to have, of the mode and place of manufacture of any of the materials being used, and will be expected to facilitate, in every way, the inspection of the same.

RKMANSHIP GAUGES. Before commencing to punch, or otherwise to prepare for construction of any of the ironwork, the contractors are to make, at their own expense, a complete set of gauges, and of any special tools necessary for securing the accuracy of the fitting of the various portions of the girders.

All similar parts of the girders are to be marked off for punching, and to accurately correspond with the gauge made specially for those parts. Every possible care is to be taken to make these gauges true; and they must bear every test of accuracy to which the engineers may think it requisite to subject them.

The engineers will also be empowered to order any special tools to be made at the contractor's expense, which they may consider necessary to ensure the due performance of their contract.

The holes for the rivets are to be punched truly parallel, and at right angles to the plane of the plates. All holes which do not coincide in such a way as to allow a rivet, $\frac{1}{16}$ of an inch diameter, less than the holes, to pass freely through them, are to be rimed out till they coincide, and to be then rivetted with rivets of the full diameter of the rimed holes.

The rivets are, in all cases, to completely fill the holes; and the engineers, or their inspector, will be at liberty to cause any rivets to be cut out and replaced, which may rattle when struck by a hammer, or otherwise show themselves to be loose.

Wherever necessary for the division of the work for shipment, the rivets are to be left out; but the holes are, in all cases, to be punched, and particular care is to be taken that they correspond accurately with the dimensions given on the drawings.

The bolt-holes in the bottom bars, and in the diagonal ties, are to be made as follows:—Holes not more than $1\frac{1}{4}$ inches in diameter, are first to be punched through the bars, as nearly as possible over the centres of the finished holes, their position having been accurately marked off with a template. The bars, in sets of from 6 to 12, are then to be stacked on, and bolted firmly, to the bed-plate of a boring tool, made, or adapted specially for the purpose. In this position all the holes are to be bored, without the bars being shifted from the bed-plate, the boring tool being passed through the holes punched in the bars, and guided in steel bushes, both above and below the tool.

The holes are to be finished off perfectly smooth, so that no marks of the tool may be perceptible in them; and all burr on the edges of the holes is to be removed with a rimer, so that nothing may interfere with the stacking of the bars flat together.

The accuracy of the position of the finished holes is to be such, that when any of the bars, taken indiscriminately, are laid upon any others, turned pins $\frac{1}{100}$ of an inch less than the diameter of the holes, may be passed freely through all the holes, at both ends of the bars at the same time.

BOLT-HOLES THROUGH STRUTS, WASHER-PLATES, &c. The bolt-holes through the struts, through the washer-plates, which carry the lower roadway girders, and through the top boxes, must be drilled as far as practicable in the same manner, and, in all respects, with the same accuracy as is specified for the bars.

BOLTS FOR BOTTOM BARS. All the bolts used for linking the bottom bars are to be turned accurately parallel and true to the required diameter of $1\frac{1}{2}$ inch, for their whole length. The marks of the tool are to be entirely removed, and the bolts finished off perfectly smooth.

BOLTS FOR DIAGONAL TIES. The bolts which secure the diagonal ties to the boxes, are to be finished in the same style, but need not be turned for their whole length. They may be swaged down instead, between the bearing surfaces for 14 inches of their length, to a diameter of $1\frac{3}{8}$ inches, should the contractors prefer that mode of manufacture.

Every bolt through top boxes, or bottom bars, is to be screwed at each end in a lathe, and the threads, in no case, to pass within the washers, so as to interfere with the bearing surface of the bars. Two round turned wrought-iron plate washers are to be provided with every bolt.

NUTS. The nuts are to be forged from the solid, to be finished off neatly to an hexagonal form, and to be screwed so as to fit properly on the bolts.

ENDS OF BARS TO BE CUT. The ends of all the bars, horizontal and diagonal, are to be cut neatly, as shown. and care is to be taken to remove any burr, or prominence, which may interfere with their being packed flatly together.

It is assumed that 16 bars, 1 inch thick, will, when painted with two coats, as specified, packed into a space of $16\frac{3}{4}$ inches, or one bar, become 1,047 inches thick.

Should they occupy less space, the difference is to be made up by inserting thin turned wrought-iron washers, at the centre joints, and increasing to the necessary extent, the length of the cast-iron distance pieces at all the other joints.

No washers, or distance pieces, less than $\frac{1}{2}$ inch in thichness, are to be of cast-iron.

TIE-BARS. The end diagonal ties, marked A, are to be of bar, or plate, 12 inches wide; and are to be welded at their lower ends to slabs of similar thickness, of the form shown; to take all the bolt-holes.

Particular care is to be taken to make sound welds, and the engineers will cause every bar to be tested, should they see reason to doubt their efficiency.

ANGLE-IRONS IN ONE LENGTH. The angle-irons of the boxes, struts, roadway girders, and all angle-irons whatsoever, not expressly excepted, are to be in single lengths, without welds, or joints. In the struts and intermediate girders, they are to be carefully bent, at a sufficient heat, to the required form, without being cut; and are to be neatly finished under the smith's hammer.

ANGLE-IRONS OF THE PLATFORM. The angle irons of the platform, which connect the upper roadway across the piers, are to be cut on one flange, and forged at a welding heat, square and true to the form shown.

SIZES OF PLATES. The plates of the horizontal boxes are to be in one length, without joints; and throughout all the girder work no joints, or joint-plates, will be permitted, beyond those shown on the drawings.

Wherever plates are cast to an irregular form, as in the intermediate roadway girders, the bearers of the pier platform, &c., &c., the edges are to be finished neatly off on completion, by chipping, and beating with the hammer.

PIER-FRAMES. The plates of the pier-frames are to be bent between rollers, or on moulds, to the form required. The arch shown is to be punched out from the plate before it is turned, and is to be afterwards carefully trimmed and filled out to a true curve.

Particular care is to be taken to finish this part of the work in the highest style of wrought-ironwork.

ROLLER FRAMES, HOW MADE. The frames for the expansion rollers are to be forged under the hammer, true and square; they must have all their meeting faces planed, and are to be put together with turned bolts fitted in drilled holes. The holes for the expansion rollers are likewise to be accurately drilled parallel, at distances apart, centre to centre, of $7\frac{1}{2}$ inches.

LUGS FOR BRACES. The lugs by which the horizontal braces on the girders are affixed to the top boxes are to be of forged scrap-iron; after being drilled to take the rods, they are to be rivetted on the boxes accurately in the line of the braces.

BRACES. The braces themselves are to be rods of $1\frac{1}{4}$ inches in diameter for their general length, but having pieces of rod of larger diameter welded on their ends to take the screwed portion without reducing the section below that due to a diameter of $1\frac{1}{4}$ inches. They are all to be screwed at each end for a length of not less than 6 inches, and provided with double hexagonal nuts, as shown on Sheet No. 5.

HANDRAIL BARS. The upper bar of the top roadway handrail is to be formed of $1\frac{1}{2}$ inch wrought-iron pipe, $1\frac{7}{8}$ inches in outside diameter; the lower bar of rod-iron 1 inch in diameter. They are both to be in lengths of 15 feet, and are to be butted and fitted in every third standard; the upper bars are to be fixed at every joint by pins passed through drilled holes: the lower bars by keys in a slot.

The two bars of the lower roadway handrail are to be formed of wrought-iron pipe 2 inches in internal, and $2\frac{3}{8}$ inches in external diameter; they are to be in lengths of 10 feet, to be fitted and butted in, the cast-iron brackets rivetted to the struts for the purpose, and to be secured in them by pins driven through drilled holes.

DRAIN-PIPES. Wrought-iron pipes of the same diameter and thickness, as specified above for the lower roadway handrail, are to be supplied for draining the upper roadway; the number of these required will be eight to each span (besides those mentioned in the lists of extra parts). They are all to be bent to follow the line of the struts, and to be in two parts divided where directed. The top ends of the pipes are to be bulged by driving a taper plug into them, so as to make dovetail joints with the curb castings.

A cast-iron double socket piece is to be provided and sent with every drain-pipe to make the joint.

Three clamps or fastenings of the kind shown by Fig. 14, on Sheet 5, are also to be provided and sent with every drain-pipe for attaching it to the strut. The holes in the struts for taking these clamps will be drilled in India.

CAST-IRON WORK.

CURBS. The curbs are to be cast in lengths of 10 feet, and are to be chipped off at the ends, so as to form a neat joint.

The shoulders of the sockets are to be turned or planed, and key-slots to be cut in the boss; the slots are to be filed out square, and split keys of wrought-iron to be fitted in them to secure the handrail standards.

Eight curbs in every span are to have holes made in them to receive the drain-pipes; the holes to be formed conical, so that the pipe may be secured by a dovetail rust-joint.

CURBS TO BE SEATED ON LEAD WASHERS. The curbs will be fixed to the roadway girders on lead-washers, and a sufficient quantity of sheet lead, weighing 7 lbs. per superficial foot, is to be provided and sent for the purpose. On the intermediate girders these washers are to be 6″ + 7″, but at the joints on the main girders they are to be 6″ + 14″ to admit of their being turned up under the curb, so as to form a water-tight joint.

HANDRAIL STANDARDS. The handrail standards are to be cast solid, and the holes for the rail bars drilled out; they are to be turned on the shoulders of the bottom socket, so as to stand upright on their curbs.

MOULDINGS OF PIER-FRAMES. The mouldings of the pier-frame are to be cast in such lengths, not exceeding those shown, as will give the greatest accuracy in their form; the pieces are to be bolted together through flange joints, as shown by Fig. 6, Sheet 8, and all meeting edges are to be brought to true line, and accurately fitted by chipping and filing.

The pierced ornament of the entablature and the ornamental handrails are to be filed up and finished in the best style.

The bed-plates are to be planed on the bearing surfaces, where tinted red on the drawings, and must be brought to a specially fine surface where the rollers bear.

The upper and intermediate bed-plates are to be planed to a radius of 6½ inches, and they are to be further filed together by grinding, till the surfaces accurately correspond in any position in which they can be placed.

It is to be borne in mind that similar bed-plates are to be all finished to one pattern, so that they may be used indiscriminately in India.

ROLLFRS. The rollers are to be finely turned and brought accurately to one diameter, and when fitted complete in their forged iron frames, are to be tested by being shown to run easily over a planed surface of any length that can be obtained.

ACCURACY REQUIRED.

It is to be expressly understood, that as a rule, the greatest possible accuracy is to be attained in every part of the work, the object being to facilitate the erection of the bridge in India by perfection of workmanship in this country. It is therefore intended that all similar parts shall fit indiscriminately in all similar places and in any span. To ensure this every similar piece is to be tested on completion, as to the accuracy of its form and dimensions, in a gauge-test, and all those that do not correspond with such gauges will be rejected.

The contractors are to construct these gauges in a manner and of the form the engineers may require, and are to include the expense of the same in their tenders.

TEMPORARY ERECTION.

The first span of the bridge is to be erected rivetted together, and finished complete in every respect, except as regards timber-work, on the contractors' premises, so that it may be tested under load. For this purpose it is to be put together upon a timber platform, as specified in the following paragraphs for the other spans; but bearings are, in addition, to be provided at two ends to carry the girders when the wedges beneath the platforms are withdrawn.

These bearings are to be of solid masonry, built upon concrete, and (if necessary) piled foundation, to prevent the possibility of settlement during the testing of the span.

Should any settlement take place whilst the whole or any portion of the load is on the bridge, the contractors will be required to remove all such load, to reinstate the bridge in its proper position by means of the platform wedges, and to recommence anew the testing. The load to be placed is 450 tons of pig-iron, ¾ of which are to be put upon the upper roadway, and the remaining ¼ upon the lower roadway. It is to be laid upon temporary planks of sufficient strength, and distributed uniformly over the roadway to the satisfaction of the Company's engineers.

On the receipt by the contractors of the engineer's certificate of the satisfactory completion of the testing; the temporary rivets are to be cut out and the span taken to pieces, packed and delivered for shipment precisely as specified for the rest of the work.

Every other span and pier superstructure is to be temporarily erected in the contractors' yards, and to undergo thus the examination of the engineers or of their inspector before removal. For this purpose the contractors are to provide, at their own expense, the requisite number of bolts to fix securely in its place every part which cannot be rivetted in this country, so that, if required by the engineers, the girder may be lowered on to its end bearings and carry its own weight.

The girders are to be erected on timber platforms, and must rest immediately on cross cills, with carefully-made folding wedges of hard wood beneath each cill, the cills and wedges being sufficiently close to admit of the girders being readily raised or lowered by driving or withdrawing the wedges.

These cills are to be carried on piles, if the nature of the ground renders piling advisable, or if not, they are to be secured by other means from the slightest settlement.

CAMBER. The required camber must be carefully given to the platform in the arc of a circle, the chord or camber in the centre being 4½ inches.

DIVISION OF WORK FOR SHIPMENT.

The divisions in which the work is to be prepared for shipment may be easily recognised by reference to the drawings, but must, in all cases, be subject to the decision of the Company's engineers.

The top boxes, upper and lower roadway girders, and the horizontal and diagonal bars will go separately without packing.

The struts, end boxes, and such other parts as may appear to the engineers to need the protection, are to have checks inserted between the end cheek plates, and secured by temporary bolts.

All bolts and nuts, screws, washers, pins, &c., are to be packed in strong iron-bound boxes; the nuts to be stowed in compartments separate from the bolts.

The ornamental handrails and casting of the pier frames are to be packed also in cases.

All cases to have the words "Jumna Bridge," and a statement of the contents branded on them, externally, in large letters.

PAINTING. The whole of the materials delivered under this contract are to be painted with two coats of good zinc paint, or of Woolston's Torbay paint, of any colour the engineers may desire, one to be laid on as soon after the formation of the parts, and the other subsequent to the taking to pieces of the parts after their temporary erection.

All machine-cut surfaces, including bored holes, bolts and nuts, are also to be painted with two coats of similar paint before leaving the contractor's yards; and they are to be further protected, when required, by the engineers with planks or roping fixed over them.

The work, during and for at least two days subsequent to painting, is to be protected from the weather, and with this view it will be preferred that the painting be done entirely under cover.

No pieces are to be sent from the works until the paint has had at least three entire days to dry.

The whole of the interior surface of the boxes is to be payed over with some preparation of coal-tar, laid on as hot as possible.

MARKING. Every portion of every span is to be marked for guide in erection, as directed by the engineers, both with steel punches and paintings.

Such shipping marks as may be required by the Company's agent are also to be painted on the various pieces.

As a general rule, the principal erection marks will be the letters affixed to each part of the drawings.

LIST OF EXTRA PARTS TO BE SUPPLIED IN ADDITION TO THE NETT QUANTITIES.	No.
Bottom bar, ½ inch thick	60
Ditto, 1 inch thick	150
Diagonal ties, A	2
Ditto B, C, D E, F, G, H, I (four of each)	32
Ditto K and L (two of each	4
Struts (one of each letter	10
Washer-plates	32
Turned bolts for bottom bars	90
Cast hollow distance pieces and octagonal washers	60
Bolts at top of ties	90
Girder for lower roadway	10
Washer plates for end of ditto	20
Main girder for upper roadway	5
Intermediate ditto	10
Horizontal braces	15
Curbs to upper roadway	15
Screw bolts to curbs	90
Length of drain-pipe	3
Cast double sockets for ditto	3
Clamps to secure pipes	9
Standards for upper railings	60
Wrought-iron keys to ditto	60
Handrail bars, 1 inch diameter	(feet) 300
Hollow pipes for handrail	(feet) 300
Cast brackets for lower handrail	30
Hollow pipes for ditto	(feet) 300
Screw bolts for timbers of lower roadway	75
Ditto do. for ditto upper roadway	450

PIER SUPERSTRUCTURE.

Bolts, with lewis ends to ditto	24
Bolts to connect cast cornices	140
Bolts to secure cast moulding round openings	192
Wood screws	32
Cast circular moulded bars	2
Cast moulding round openings	2
Cast and enriched cornices	2
Cast ornamental lower railing	2
Cast ornamental upper railing	2
Cast ornamental spiked heads	8

BED-PLATES FOR PIERS WITH ROLLER ARRANGEMENTS.

Bottom plates	2

	No.
Intermediate plates	2
Upper plates	2
Rollers	10
Wrought frames for rollers	2
Turned bolts to ditto	32
Bolts, with lewis ends, for bottom plate	12
Bolts to connect bottom plates	9
Bolts to secure cover plates	8
Bolts to attach upper plates to end framing of girder	48
Cover plates	4

BED-PLATE FOR PIERS WITH FIXED BEARING.

Bottom plates	2
Upper plates	2
Cover plates	4
Bolts and lewis ends	12
Bolts to connect bed-plate	9
Bolts to secure cover plates	8
Bolts to attach upper plates	48

GENERAL CONDITIONS OF CONTRACT.

The contract is intended to include the entire completion of the various parts ready for fixing, and the provision of the whole of the ironwork necessary for the same, except where otherwise specified. It is, therefore, expressly to be understood, that all bolts, screws, rivets washers, keys, pins, spikes, and other minor parts which may not be shown in the drawings, or mentioned in the specification; but, which may reasonably be considered as being requisite for the proper completion of the work, are to be provided by the contractors, and included in their contract amount.

The contract is to be executed, in every respect, in conformity with the drawings and specifications, and to the entire satisfaction of the Company's engineers, who shall, either personally, or by deputy, have the power of inspecting and testing the work, during its progress, in any manner they may think fit; and by rejecting any such parts as they may disapprove, on any ground whatsoever.

The contractors shall attend to, and forthwith execute all directions, in reference to the construction of their work, which may, from time to time, be given to them by the engineers; and, in case any deviations from the drawings, or specification, shall be considered desirable; the engineers shall have the power of ordering them to be done, and they shall be paid for in the shape of additions to, and deductions from the contract amount, at the rates named in the schedule of prices attached to the contractors' tender.

The whole of the work is to be delivered free alongside, at any dock, or in any wharf, in the port of London, or Liverpool, as the Company may direct.

Parties tendering, are, however, at liberty to name in their tender a separate price for delivery at any other port, should they desire to do so.

The first span is to be erected as specified, ready for testing within five calendar months of the date of the letter by which the Company intimate their acceptance of the contractors' tenders; and, after being tested, it is to be delivered, as specified, at London, or Liverpool, complete, with one pier superstructure, and one set of bed-plates, within six weeks of the date above specified for its completion. The remaining spans are to be delivered, each with one pier superstructure, and one set of bed-plates, at equal intervals of about three weeks, commencing from the date above specified, for the completion of the first span, so that the whole may be delivered within fifteen calendar months of the date of acceptance of the contractors' tender.

Failing the proper deliveries of the work, as above specified, the contractors shall be bound to pay to the Company, as ascertained any liquidated damages, and not as penalty, a sum of £30 sterling, for each and every week that any span remains undelivered after the expiry of the contract time for its delivery.

Should, however, the contractors not progress with the work, in the complete and careful manner intended by the contract, or should they refuse or neglect to comply with the directions given them by the Company's engineers, or, in any other respect, act contrary to the terms of the contract, the Company shall have power to declare the contract at an end; and shall be liable only to pay for such portions of the ironwork as shall have been delivered at the time of such declaration, deducting the value of any loss or damage they may have sustained by the contractors' breach of the contract.

No payment will be considered as legally becoming due to the contractors, until the completion of their contract; but advances of money on account will be made to them, at such times, and to such amount, as the engineers shall recommend; the balance to be paid within one month after the engineers shall have given their certificate of the satisfactory completion of the contract.

It is, however, to be expressly understood, that such advances are in no wise to be considered as evidence of any particular quantity of work having been executed, or of the manner of its execution being satisfactory, or in any other way, to relieve the contractors from the liabilities they may sustain under the terms of their contract.

The contractors shall not assign, or sublet, their contract, or any part thereof; nor allow any portion of the work to be done other than in their own establishments, without the express written consent of the Company being first obtained.

SPECIFICATION FOR ADDITIONAL WORK.

1st.—Four rows of continuous wrought-iron joists are to be laid from end to end of the bridge, to carry the lower roadway. The outside rows consist of bars $7 \times \frac{5}{16}$, with two angle-irons at each edge, each $2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$. The lower flange is strengthened by a plate $10'' \times \frac{1}{2}''$, also running the whole length of the bridge. These joists, with the 10″ plate rivetted to them, are to be sent out complete in 20 feet lengths, the connections between the plates being made as shown.

The intermediate joists consist of bars 8″ × $\frac{5}{16}$, with two angle-irons, $2\frac{1}{2}$″ × 2″ × $\frac{1}{4}$″ at each edge, to be made up in 20 feet lengths as before.

All these lengths are to be so arranged as to break joint with each other.

The cross-girders are to be punched out where these joists cross them, so that they may be rivetted together in India.

The rivets are all to be $\frac{3}{4}$″ diameter. The rivets which are to be executed in India, are all tinted red.

These joists are to be braced together on the underside, as shown, by wrought-iron bars, 3″ wide, by $\frac{3}{4}$″ thick.

2ND.—The vertical struts are to be connected together at the lower ends, with plates $\frac{3}{8}$″ thick, 15″ deep, and of such width as will fill up the distance between the vertical angle-irons of the strut rivetted to the struts with four $\frac{3}{4}$″ rivets on each side.

3RD.—The end bars of the lower member of the girder are to be stiffened by the insertion between each pair of $\frac{1}{2}$″ bars of a bar 3″ × 1″. All these bars are to be rivetted together.

The pairs of bars on each side of the girder are to be further connected together by 3″ × $\frac{3}{4}$″ bars, forming a bracing as shown. None of the rivetting is to be executed in this country, except which is necessary to form the intermediate bars into a bracing. All the punching of every kind is to be done to template in the most accurate manner.

KENT AND LEVEN VIADUCTS.

PLATES XLVI. TO XLIX.

These viaducts have been erected by Mr. Brunlees, for the purpose of carrying the Ulverstone and Lancaster Railway across the tidal estuaries of the River Kent and Leven in Morecambe Bay. As they are similar in design, situation, and mode of construction, we do not consider it necessary to treat of them separately, but purpose giving a general description.

The estuaries formed by these rivers where the railway crosses them, are each about $1\frac{1}{4}$ mile in width, and the whole bed is a mass of shifting sand, as found by borings carried to a depth of 70 feet. To carry a viaduct upon such a foundation was a task of no ordinary difficulty, the large volumes of water at low tides which scour the beds of the estuaries, caused the navigable channels to fluctuate from one side to the other, in such a manner as to set all calculation at defiance; and as the Admiralty required provision to be made for keeping open the navigation, it became necessary to confine the channels to a fixed course, to admit of drawbridges being placed over them; this was accomplished by means of weirs, composed of quarry rid, which was tipped from waggons into the sand, at the ebb and flow of the tide; by this means the sand was scoured out to a depth of 9 feet, and the channels were gradually deepened, and fixed to one place.

As it was evident that no firmer foundation than the sands could be obtained for the bearing piles of the viaducts, a series of experiments were undertaken to test the bearing power of the sand, and to ascertain the size and form of piles best adapted for a permanent foundation.

The experiments were firstly tried upon a bell-mouthed pile (Fig. 15, Plate XLIX.); secondly, upon a bell-mouthed pile with a flat outer rim of metal, the interior being filled and rammed with concrete (Fig. 16); thirdly, upon a plain disc pile open to the extent of the internal diameter, and filled with concrete (Fig. 14); and, fourthly, upon a plain disc pile, with an orifice of 3 inches diameter in the centre (Fig. 11). Of these forms the last proved the most satisfactory, and was consequently adopted. The experimental piles were sunk from 16 feet to 23 feet in the sand, and were then weighted, the amount of depression being observed for every 2 tons up to 20 tons.

The average results of these experiments proved that the sustaining power of the sand was equal to about 5 tons per square foot, and as it was intended that each bearing pile should carry at least 20 tons; the size of the disc was fixed at 2 feet 6 inches in diameter, giving an area of 4·86 square feet, or 24·30 tons sustaining power.

The method adopted by Mr. Brunlees for sinking these piles was novel and most ingenious, and requires, we consider, a somewhat detailed account. A pontoon was moored in the channel, where the piles were intended to be sunk, on board of which was a small steam-engine and donkey-pump of about

6-horse power. The piles were then pitched in the position assigned to them, and secured by means of a guiding apparatus to ensure accuracy. A wrought-iron pipe of 2 inches in diameter was then placed within the pile, and carried through the hole in the disc (Fig. 11) to a distance of about 2 feet; at the top of this pipe was attached a flexible hose, which was connected with the donkey-pump. By these appliances the water was forced down the pipe into the sand, which, being loosened, permitted the easy descent of the pile. It was found that the piles sank rapidly through the upper or more recent layer of sand for a depth of from 7 to 9 feet, but below this depth, in the more compact marly deposit, the progress of sinking was slow. To obviate this, ribs or cutters were cast on the lower surface of the disc (Fig. 13), by which, when an alternating rotatory motion was given to the pile, the material was loosened and was washed away by the water. This plan was found so effective, that two piles were generally put down from each pontoon during the ebb of the tide. To consolidate the sand beneath the discs, which had been disturbed in sinking the piles, each pile was ultimately driven down two inches, by short blows from a heavy 'tup.' After the first tide the sand was at its full height, and then the piles were quite stationary.

The guide-piles, 36 in number, forming the approach to the drawbridges, are of baulks of timber 14 inches square, and are fitted into cast-iron sockets having a disc of 2 feet 6 inches (Fig. 9). These piles were sunk by the same process as the hollow piles, the water being carried down the outside of the baulk, and passed through a hole in the side of the disc. The piles after being sunk to their proper depth by the process already described, were braced together by a strong timber walling, to prevent them from being shifted from their positions.

Each ordinary pier consists alternately of 4 and 5 piles and columns. The temporary taking pile in every other pier, now used as a stay to the pier, will, on the widenings of the viaducts for a double line of rails, be put into a vertical position and become one of the bearing-piles.

The piles are 10 inches external, and $8\frac{1}{2}$ inches internal diameter; the longest column above the wallings in either viaduct is 22 feet. On testing one of the columns, laid horizontally between two supports 20 feet apart, it broke with a weight of 10 tons in the centre, or 20 tons equally distributed; taking the same length of column in a vertical position; its breaking-weight, by Hodgkinson's formula, would be 480 tons, or 24 times the horizontal breaking-weight equally distributed.

The roadways of the viaducts are formed of longitudinal timbers laid on open planking 3 inches in thickness, supported by three continuous trellis girders, and one light outside girder. On referring to Figs. 2 and 3, Plate XLIX., it will be perceived that one line of bearing girders, and one of outside girders, will be sufficient to complete the design for a double line of way. The top flange of the bearing girders consists of a malleable iron plate 14 inches in width by $\frac{1}{2}$ inch in thickness, and two angle-irons 3 inches $\times$ $3\frac{1}{2}$ inches $\times$ $\frac{1}{2}$ inch, and the bottom flange of a plate 9 inches in width and $\frac{1}{2}$ inch in thickness, and two angle-irons 3 inches $\times$ $3\frac{1}{2}$ inches $\times$ $\frac{1}{2}$ inch; the web is composed of diagonal bars of merchant iron 4 inches $\times$ $\frac{1}{2}$ inch, and is strengthened at every 3 feet by vertical T-irons $4\frac{1}{2}$ inches $\times$ 3 inches $\times$ $\frac{1}{2}$ inch. The girders are well braced together at distances of 7 feet apart, by means of wrought-iron diagonal cross-bracing composed of T- and angle-irons. The planking which is 12 inches in width, also serves to increase the rigidity of the structures.

Several of these girders were tested in lengths of 30 feet, up to a load of 30 tons in the centre, the deflection being $\frac{1}{2}$ inch. They were further tested after erection, with two engines, loaded up to $1\frac{1}{2}$ tons per lineal foot; and where the girders are continuous, no deflection was observable, but the end openings yielded nearly one-eighth of an inch, rising again on the removal of the load.

We will now pass on to a description of the drawbridges, which will be found most worthy of attention. The waterway for the vessels in the Leven estuary is 36 feet wide, and is spanned by a moveable platform, formed of two girders of similar construction to those already described. The method of accomplishing the opening of the bridge, consists in making the moveable platform 78 feet long, which is double the length of the open part, and 6 feet over for surplus counterbalance, and then causing it to glide under the fixed roadway on one side the opening, thus forming a kind of telescope bridge; the provision for this consists of a lower line of rails fixed on wrought-iron lattice beams, which have a slight inclination at the counterbalance end. There are three pairs of wheels attached to the girders, and these, resting on the lower line just mentioned, facilitate the movement of the platform, a rack and pinion worked by one man being sufficient to overcome the friction. The fixed roadway is formed of cross I-irons (Fig. 2, Plate XLVII.) for a length equal to the open span, thus affording clear space for the admission of the platform beneath it. When the bridge is closed, by passing the platform over the span, it will be perceived that owing to the inclination of the lower or platform rails, the counterbalance end is on a somewhat lower level than the fixed line; to raise it to the same plane, an eccentric is placed under each girder, the eccentrics being connected by a shaft, which is worked by a rack and screw motion.

In the construction of the drawbridge over the Kent estuary, considerable improvements were introduced, which will be at once manifest on a comparison of the two structures. The lower rails and girders were dispensed with, thus considerably diminishing the cost of erection. By referring to the longitudinal section of this viaduct (Fig. 1, Plate XLVIII.), it will be perceived that one end of the moveable platform rests upon two eccentrics; when these are lowered, and the platform is moved along by means of a rack and pinion, its weight causes it to fall upon the wheels, shown in the figure, which are supported on columns 27 feet apart. When the bridge is opened to its full extent, the end of the moveable platform rests upon two cast-iron supports, attached to two of the bearing-piles of the viaducts. Guide rails are fixed on the under sides of the girders, which rest upon the wheels already mentioned, and facilitate the movement of the platform. The fixed roadway is similar to that of the Leven Viaduct, being supported on I-iron for a length of 40 feet, thus allowing ample space for the admission of the moveable platform beneath it. The advantages of this design must be evident to all who make themselves acquainted with its details. Small expense at first cost, with great facility for opening and shutting, under all circumstances; and no extra provision for foundations, the weight of the whole moveable platform being dispersed in its bearings.

The drawbridges have been now a considerable time in operation, so that their merits have been most thoroughly tested, and they are so steady under the trains that their position is not apparent in passing over the viaducts.

The Leven Viaduct consists of 48 bays, or openings, 30 feet each from centre to centre of the columns, and one bay 36 feet in the clear for the opening-bridge. The height to the rails is 26 feet above low-water. The length over all is 1,563 feet. 395 tons of wrought-iron, and 438 tons of cast-iron have been employed in its construction. The cost, including the stone abutments, the drawbridge, with guide fenders, and the rubblestone weirs, has been £18,604, for a single line; when completed, the cost for a double line, at the same prices, will be £24,361, or £46 15s. per lineal yard. The piling was commenced on the 1st April, 1856, and was finished on the 25th October, of the same year: the whole erection was completed on the 14th June, 1857. Messrs. W. and J. Galloway, of Manchester, were the contractors.

The Kent Viaduct consists of fifty bays, each 30 feet span from centre to centre of the columns, and one bay 36 feet in the clear, for the opening-bridge. The height of the rails is 23 feet above low-water. The length over all is 1,566 feet. In its construction, 406 tons of wrought-iron, and 368 tons of cast-iron have been used. The total cost has been £15,056; and when completed for a double line, it will be £20,813, or £39 17s. per lineal yard. The piling was commenced on the 21st October, 1856; and was finished on the 31st January, 1857: the whole structure was completed on the 24th July, 1857. Mr. James Featherstone, of Manchester, was the contractor.

The railway was opened in August, 1857; and, since then, in addition to the ordinary passenger trains, heavy trains laden with minerals and goods, have passed over daily, without causing the slightest depression in the piles, and but little vibration.

BEELAH AND DEEPDALE VIADUCTS.

PLATES L. TO LV.

The Beelah Viaduct, which is situated about 4 miles from the town of Brough, in Westmoreland, and which carries the South Durham and Lancashire Union Railway over one of the wild mountain gorges of Westmoreland, may perhaps be considered one of the lightest, and cheapest of the kind, that has ever been erected. The Crumlin Viaduct, of which a description has been given in our previous work, was doubtless the model after which the Beelah was designed, but the Engineer, Mr. Thomas Bouch, of Edinburgh, did not follow Mr. Kennard's example, except in the general scheme of skeleton trussed piers, composed of cast- and wrought-iron, and a superstructure composed of lattice girders crossed with timber. In these general features the two structures are identical, but in almost all others they are dissimilar. Mr. Bouch has reduced the spans to 60 feet, and has consequently been able to make the piers much lighter, as well as to reduce the depth and scantlings of the platform or main girders.

The Beelah Viaduct consists of 15 piers of varying heights, according to the section of the valley; each pier being composed of six hollow columns, placed in the form of a tapering trapezium, and firmly braced together with cross-girders, at distances of 15 feet perpendicular, and by horizontal and diagonal wrought-iron tie-bars. The illustration at Plate L. will show this construction more clearly, and will more fully indicate the arrangement of the cast- and wrought-iron in combination.

The columns are 12 inches external diameter, and of varying thickness, from $1\frac{1}{4}$ inches to $\frac{7}{8}$ inches, according to the height of the pier and their position in it. The columns are placed at distances of 50 feet centre to centre, in the breadth of the pier at the base, and taper towards each other, until at the top, immediately under the platform girders, they are 22 feet apart, from centre to centre.

The taper is given in the foundation piece, at the base of each column, which foundation piece is firmly bolted to a stone base, the upper surface of which is bevelled at such an angle, as will produce the taper required for the columns. Thus the columns have all their flanges square to the centre line, which simplified the fitting very materially. The depth of the stone foundations, varied according to the nature of the ground, and the height of the piers, but in almost all cases they went down to the solid rock.

It is a distinguishing feature in this viaduct, that the cross, or distance girders of the piers, encircle the columns which are turned up at that point, the girders being bored out to fit the turned part with great accuracy. No cement of any kind was used in the whole structure, and the piers when completed, and the vertical and horizontal wrought-iron bracings keyed up, are nearly as rigid as though they were one solid piece.

The platform girders, or wrought-iron superstructure, were of the lattice construction, and were tested with 1 ton on every lineal foot; under which weight, they manifested scarcely any visible deflection. There are three girders to each span, two outside girders which are 5 feet $10\frac{1}{2}$ inches deep, and one middle girder, of equal depth to the latter; being, of course, much stronger in the scantlings than the outside girders. These three girders were firmly stayed to each other, by metal cross-bracings, and wrought-iron ties, and are crossed by baulks of timber, dressed and squared to 12×9 inches, which overhang the outside girders about 2 feet 6 inches. These baulks are again crossed by 3 inch planks, and these planks covered by a good thickness of finely-broken stones. The rails are carried by longitudinal baulks, the whole length of the viaduct; and guard rails are fixed on both sides, so as to render it nearly impossible for the wheels of an engine or carriage to get off the way. The whole is surmounted by an elegant hand-railing of cast-iron in which the design of the viaduct is well maintained.

The weight of the whole viaduct was,

Cast-iron	776 tons.
Wrought-iron	398 tons.

and the superstructure contains 12,343 cubic feet of Memel timber.

One important element which has to be provided for in these erections is that of expansion, which in such a length as the Beelah—1000 feet—would become a serious difficulty if it were not arranged for. This Mr. Bouch has done by the application of sliding-plates lined with brass, on which every third girder rests, with liberty to move in its length within certain limits.

The expansion of these girders which are secured together, is thus provided for at one joint; and experience has proved that this is sufficient, as the girders move easily without producing the slightest effect on the piers. The total length of this viaduct is 1000 feet, and the greatest depth from the rail to the ground 195 feet.

The two prominent and distinctive features of this class of railway viaducts, are their cheapness and the rapidity with which they can be erected; the latter point being often of vital importance, as it not unfrequently happens that a wide gorge becomes the key to a whole line of railway, and the power of bridging it rapidly may convert the capital of the railway, which would otherwise lie dormant, into active and remunerative stock.

No example could better illustrate this than the Beelah viaduct, which was erected in the incredibly short space of four months by Messrs. Gilkes, Wilson, and Co., of Middlesbro'-on-Tees. A stone viaduct over the same spot would have occupied three years in its erection. Much of this quickness was, however, attained by the admirable managements made by this firm for the fitting and erection of the work. The fitting was all done by machines, which were specially designed for the purpose, and finished the work with mathematical accuracy.

The flanges of the columns were all faced up and their edges turned, and every column was stepped into the one below it with a lip about ⅝ of an inch in depth, the lip and the socket for it being accurately turned and bored. That portion of the column against which the cross-girders rested was also turned. The whole of these operations were performed at one time, the column being centred in a hollow mandril-lathe. After being turned, the columns passed on to a drilling-machine, in which all the holes in each flange were drilled out of the solid simultaneously. And as this was done with them all in the same machine, the holes of course perfectly coincided when the columns were placed one on the other in the progress of the erection. Similar care was taken with the cross girders, which were bored out at the ends by machines designed for that purpose. Thus, when the pieces of the viaduct had to be put together at the place of erection, there was literally not a tool required, and neither chipping or filing to retard the progress of the work.

The mode of erection adopted by Messrs. Gilkes, Wilson and Co., was novel, but remarkably successful. They used no scaffold, but having prepared a crane, with a gib of sufficient length to reach from the abutment over the first pier; and, of course, from the first pier over the second, and so on; they commenced the erection by setting the first pier from the abutment, on the Brough side of the valley. The crane was balanced by a shifting weight-box, so that each piece of the pier, whether light or heavy, was counterbalanced as it swung; and they were thus enabled to lower it slowly and steadily to its place, whilst it was swinging round from the point of its first suspension. As soon as the first pier was erected, they placed two baulks across from the abutment to the pier, and ran the girder (which was placed on two low bogie wagons) over, dropping it into its place with the assistance of the crane, by which each end of each girder was lifted and lowered.

On the completion of the first span the crane was moved forward, and the erection of the second pier was commenced; and this process was pursued with singular success, not a single accident occurring during the whole erection, and not a man receiving any injury; a fact almost without parallel in the annals of railway engineering.

Some planks were placed upon the cross-girders, which, with some hand-ladders, enabled the workmen to go up and down readily. In the erection of this important work there were about 130 men engaged. The time occupied has been before stated, and, we think, we are justified in saying that the whole history of railway erections, either in stone, wood, or iron, can furnish no parallel to this almost magical creation.

The Deepdale Viaduct is, in its details, identical with the Beelah; all the piers and other parts being similar; but the Deepdale Viaduct is built upon a curve. The wrought-iron work in these structures cost £22 per ton, and the cast-iron work £11 per ton.

BEELAH VIADUCT.

ESTIMATE OF WORK, EXCLUSIVE OF MASONRY.

	Tons.	cwt.	qrs.	lbs.
PIERS.—WROUGHT-IRON WORK.				
Bar-iron in tie-bars and clip-plates	84	18	0	0
Bolts, nuts, gibs and cotters	21	3	0	0
CAST-IRON WORK.				
510 columns and 78 foundation pieces	353	4	0	0
738 girders	308	16	2	0
SUPERSTRUCTURE.—WROUGHT-IRON WORK.				
42 main girders	224	3	2	0
29 horizontal and transverse bracing	34	18	2	0
Bolts and spikes to platform	2	5	0	0
CAST-IRON WORK.				
Bearings and brackets	8	2	0	0
Railing (552 yards lineal) at 1 cwt.	27	12	0	0
BRASS.				
In sliding bearings	0	2	2	0
TIMBER.				
Fir, bearers and planking	9815 cubic feet			
Oak, bearings	15 cubic feet.			

DEEPDALE VIADUCT.

ESTIMATE OF WORK, EXCLUSIVE OF MASONRY.

PIERS.—WROUGHT-IRON WORK.				
Bar-iron in tie-bars and clip-plates	64	6	1	14
Bolts nuts, gibs and cotters	13	13	1	14
CAST-IRON WORK.				
408 columns and 60 foundation pieces	277	12	0	0
Girders	248	2	0	0
SUPERSTRUCTURE.—WROUGHT-IRON WORK.				
Main girders	176	2	3	0
Horizontal and transverse bracing	27	9	1	0
Bolts and spikes to platform	1	15	0	0
CAST-IRON WORK.				
Bearings and brackets	6	8	0	0
Railing (432 yards lineal) at 1 cwt.	21	12	0	0
BRASS.				
In sliding bearings	0	2	0	0
TIMBER.				
Fir, bearers and planking	7712 cubic feet.			
Oak, bearings	12 cubic feet.			

LONDONDERRY BRIDGE.

PLATES LVI. TO LIX.

This bridge, designed by Mr. Hawkshaw, and contracted for by Messrs. Butler and Co., of the Kirkstall Forge Works, near Leeds, is now in course of erection, and is intended to carry a roadway and a single line of railway; the former being placed on the top of the girders, and the latter attached to the bottom of the same. It crosses the River Foyle, at Londonderry, and consists of a centre swing-bridge of two spans; on either side of which are three main spans, each of 119 feet, supported by continuous girders; and two land openings, each of 62 feet span, also supported by continuous girders.

The main girders are constructed on the trellis system; the struts and ties being placed at an angle of 45° to the horizon; the girders being stiffened by uprights at every junction of the diagouals, with the flanges, which occurs at distances of 15 feet.

The swing-bridge is supported upon a pier composed of seven cylinders, each 8 feet in diameter; the whole being surrounded at the top by a cylindrical casing, 30 feet in diameter.

The remaining river-piers, each consist of two 10-feet cylinders, braced together by a cast-iron distance piece. The piers supporting the land openings, each consist of two columns, 2 feet in diameter, bolted down to masonry.

The girders supporting the land openings are constructed on the lattice principle; the bars being placed at an angle of 60° to the horizon; the web is stiffened by T-iron uprights, placed 7 feet 4 inches apart. For further particulars of this structure, we refer our readers to the following list of quantities, and specification.

QUANTITIES.

The Weights and Quantities are taken Nett, without regard to any local custom.
Estimate of Works to be tendered for if the cylinders should be 60 feet in length.

CAST-IRON.

	Tons.	cwt.
In No. 1 cylinder, 30 feet diameter	114	0
„ 16 „ 11 „	667	0
„ 2 „ 5 „	31	0
„ 1 upper roller frame turntable	33	0
„ 1 lower frame ditto	12	10
„ 18 roller wheels for ditto	9	0
In parapet openwork and capping	130	0
In stanchions for ditto	15	0
In curved ornamental brackets to stiffen ditto outside	15	15
In moulded upper facia string	29	0
In girders to support parapet and footpath	85	0
In moulded lower facia string	19	0
In plain half-inch casting	14	10
In channel gutters	66	0
In 16 pilasters from top of cylinders to lamps	144	0
In parapet cap along top of lattice girders to land openings	3	15
In string course to ditto	15	0
In No. 16 expansion roller frames for girders to rest upon	9	15
„ 5 cast-iron columns, 2 feet diameter	21	5
„ 4 upright girders to support ends of land opening girders	22	0
„ 8 stretches between pairs of 11-feet cylinders	3	15
„ 6 shoes for piles to timber gearing, 8 springers to timber, bowstring girders	2	15
„ 4 girders to carry expansion rollers clear over locking-gear of swivel bridge	1	15
„ 4 saddles to carry expansion rollers over locking gear of swivel bridge	1	5
Carried forward	1465	10

	Tons.	cwt.
Brought forward	1465	10
In No. 4 slide troughs for sliding	1	5
Key to lock bridge		
In No. 4 sliding keys and 4 boxes under ends of paving girders	2	0
In iron pipes between diagonal bars, &c.	3	5
Total	1472	10

MACHINERY.

No. 4, large screws attached to keys to lock swing girders, 2 feet 8 inches long by 3 inches diameter, each.

No. 8, bevil cogs, each 12 inches diameter.

TURNING GEAR FOR SWIVEL BRIDGE.

No. 1 Moveable capstan-head, with No. 4 sockets, 18 inches diameter to 4 inches thick, with No. 4 lever bars to fix into same, 5 feet long each and 2½ inches diameter, with 1 vertical capstan pillar 3 feet 9 inches high, with vertical axle to fit capstan pillar upon, with cog-wheel to turn large wheel 5 feet diameter, moving swivel bridge round by another verticle axle, 4 feet 6 inches long, on which is fixed a small pinion working in rack, 7 inches deep, and one-fourth the circumference of large, 30 feet.

Cylinder, length up rackwork, 24 feet, including gun-metal work for anti-friction. The above is attached to a cast box frame for working.

WROUGHT OR MALLEABLE IRON.

	Tons.	cwt.
In No. 1 roller frame, holding cast wheels or rollers	13	0
T-irons to L- do., for stiffening buckle-plates	26	0
In cross girders to carry railways, including rivet-heads	124	10
In cross girders to carry roadway over land openings, including rivet-heads	61	5
In cross girders to carry roadway above the railway, including rivet-heads	163	5
In principal *lattice* girders over land openings, also do. resting upon five feet cylinders, including rivet-heads	90	0
In No. 11 principal girders, exclusive of diagonal to vertical bars, to do.	393	5
Patent diagonal to vertical bars, to do. with swelled ends	178	15
In No. 2 principal girders for swivel bridge, exclusive of diagonal and vertical bars, to do.	50	0
Patent diagonal to vertical bars, to do. with swelled ends	24	10
In No. 16 expansion roller frames under ends of girders	0	10
„ 2 extra cross girders at ends of swivel bridge, including rivet-heads	1	5
In Mallet's buckle-plates for roadway	117	10
Rails, 390 lineal yards, single line	26	0
Total	1269	15

Gun-metal washers, &c., in wrought-iron rolling frame . . . . 1080 lbs.

BOLTS AND NUTS.

	Tons.	cwt.
1¼″ bolts and nuts in securing plates composing 30-feet cylinder	3	0
1¼″ „ „ in securing plates composing No. 16, 11-feet cylinders	16	0
1¼″ „ „ in securing lengths of 5-feet cylinders	0	10
1½″ „ „ in securing cast-iron upper roller frame, or turntable	0	10
1½″ „ „ in securing cast-iron lower roller frame, or turntable		
¾″ „ „ in securing brackets to outside of parapet	1	5
¾″ „ „ in securing stanchions to parapet cast girders		
¾″ „ „ in securing parapet cast girders to wrought *w w* cross girders	0	10
½″ „ „ to secure buckle-plates at intersections of cross girders	0	5
¾″ „ „ to suspend railway cross girders to principal girders, 8 to each cross girder	1	5
¾″ „ „ to secure roadway cross girders above railway, 4 to each cross girder	0	15
½″ „ „ to secure channel gutter to cross girders at intersections		
¾″ „ „ to secure pilasters to cylinders and flanges of pilasters	1	0
¾″ „ „ to secure longitudinal sleepers to cross beams	0	10
¾″ „ „ to secure longitudinal sleepers to cross girders		
1½″ lewis bolts to secure fixed ends of principal girders to stonework on cylinder, &c.		
1¼″ bolts and nuts to secure cast expansion roller frames to girders and saddles		
1¼″ countersunk bolts to ditto	0	10
1½″ bolts and nuts to secure cast columns to girders at top		
1½″ „ „ lewis, to secure base of ditto to stonework		
1¼″ „ „ to secure *w* vertical cast girder to ends of principal girders, to receive land opening girders upon the same		
1½″ „ „ to secure swivel girders to cast-iron roller frame or turntable	0	10
1½″ lewis bolts to secure cast lower roller frame or turntable to stonework		
1¼″ bolts and nuts to secure cast stretchers between pairs of 11-feet cylinders		
Carried forward	26	10

	Tons.	cwt.
Brought forward	26	10
¾″ bolts and nuts to secure timber cross-heads, &c., &c., to piling		
1½ lewis bolts to secure cast girders and saddles to stonework	0	10
1¼ bolts and nuts to secure cast-iron troughs for slide keys		
⅞ „ „ passing through iron distance pipes between diagonal bars of principal girders, &c.	1	10
½″ wrought-iron spikes, 6″ long, to secure planking to cross girders, and clenched	1	0
½″ „ „ to secure planking to timber		
Straps of wrought-iron round timber bowstring girders cross beams		
Straps and bolts at ends of cross beams to suspend them to bowstring girders		
	29	10

Laying 390 yards of single line of railway, including fastening.
Sinking No. 1 cylinder, 30 feet diameter, 27 feet below bed of river.
Sinking No. 4 cylinders, 11 feet diameter, 45 feet each below bed of river.
Sinking No. 6 ditto 25 feet each below bed of river.
Sinking No. 6 ditto 30 feet each ditto ditto.

STEEL.

No. 1 steel swivel pins or axis for turntable, 5 feet long and 8″ diameter.
No. of steel bolts and nuts to hold diagonal and vertical bars to top and bottom of principal girders.
No. 52, 18″ long, 5″ diameter.
No. 52, 18″ long, 4½″ diameter.
No. 52, 18″ long, 4″ diameter.
No. 78, 18″ long, 3½″ diameter.
No. 20, ornamental lamps and glasses.

	Cube feet.
Timber-work in bowstring girders, cross-beams, piling (20 feet long), and planking over ditto, including extra for longitudinal sleepers	2717
	Square yards.
Wood-paving for paving roadway, 4 inch cubes	2600
	Cube feet.
4-inch planking for railway, 8″ wide, each in	6992
Fenders to protect openings, including fastenings	4320
	Cube yards.
Concrete inside of 30-feet cylinder	1600
„ in No. 16, 11-feet cylinders	3424
„ in No. 2, 5-feet ditto	90
	Square yards.
Asphalte for roadway footpaths, 1 inch thick	3900
	Cube feet.
Granite on concrete in 30-feet cylinder	678
„ „ in No. 16, 11-feet cylinder	640
	Square yards.
9-inch brickwork on concrete between granite.	200
	Lineal feet.
Stuffing joint cavities 3″ × ½″ with iron cement inside 30-feet cylinder	1536
„ „ 3″ × ½″ with ditto ditto inside No. 16, 11-feet cylinders	8239

PAINTING, &c.

All ironwork above low-water line to be one coat in boiled oil, and three coats in oil colour.

	Square yards.
One coat in boiled oil, and three coats in oil colour upon all outside work	30,758

SPECIFICATION.

CAST-IRON. All cast-iron must be of the best cold-blast grey metal, and the castings must be straight, and exact to shape and dimensions, clean and sound, and entirely free from air-holes and flaws of every description. Great attention must be paid to the execution of all ornamental castings, the arrisses must be left sharp, and the several parts neatly and accurately jointed and fitted.

The mixture of the metal will be left to the contractor's own judgment; but it must be of such quality as the engineer shall approve; and, in respect of strength, must stand the following test:—

Two bars are to be cast each day from any of the castings, being run 3 feet 6 inches long, 1 inch broad, and 2 inches deep. The person appointed to inspect the work will then test one of these by placing it edgewise on bearings 3 feet apart, and loading the middle of the bar, and should it break with any less weight than 27 cwt., all work cast with that metal will be rejected. The other test bar is to be marked with the date of the cast, and put away for the subsequent inspection of the engineer.

WROUGHT IRON. The whole of the wrought-iron is to be equal in quality to the very best Staffordshire, and the parties tendering are to mention the iron they propose to use.

BRASSES. The brasses are to be made of hard gun-metal of the very best quality, composed as follows:—

Copper	. . .	84 per cent.
Tin	. . .	13 „ „
Zinc	. . .	3 „ „
		100

The engineer shall have power to alter this mixture if he thinks proper. The whole of the brasses are to be bored, and the fitting to be done in the best and most careful manner.

IRON CEMENT. The iron cement is to be composed of clean iron borings, sal-ammoniac, and flour of sulphur, in proper and approved proportions.

No iron cement will be allowed to be used except in the situations specially referred to in the specifications.

SINKING CYLINDERS. The cylinders are to be sunk to such depths as the engineer shall direct; but the contractor must tender in one sum on the assumption that all the cylinders will be sunk to a depth of 60 feet, from the underside of the lower roadway cross-girders; and in another sum on the assumption that they will all be sunk to a depth of 80 feet, from the underside of the said cross-girders.

Any variation in the depths to which the cylinders may be sunk will be calculated, and be added to, or deducted from, the contract sum as the case may be, in accordance with the schedule of prices.

The cylinders are to be sunk by dredging the material from the inside, and then weighting them. When a water-tight stratum is met with, the cylinders must be pumped dry, and the material dug from the inside in the usual way; should no sufficiently water-tight stratum be met with, then, after the cylinders have been sunk by dredging and weighting, to such depth as the engineer may approve, the contractor must provide suitable apparatus, and force the water from the interior of the cylinders by compressed air, so as to enable the foundations to be examined; and the sinking must be continued by the pneumatic process.

Any other means that the engineer may think it desirable to adopt for sinking the cylinders, are to be resorted to.

All the apparatus and machinery necessary for sinking the cylinders shall be such as the engineer shall approve.

All the material excavated from the cylinders is to be deposited in such places as may be ordered.

All the 11-feet cylinders, and the 30-feet cylinder, are to be made in segments, as shown on the drawings.

The spaces left between the flanges, both vertical and horizontal, are to be caulked with iron cement, so as to render the cylinders perfectly water-tight.

The 5-feet cylinders are to be of 1 inch metal, and are to be composed of lengths of 9 feet, cast entire, and with top and bottom flanges. These flanges are to be similar, in every respect, to the horizontal flanges of the larger cylinders, and the joints are to be planed, and the lengths united together in precisely the same manner.

Each pair of 11-feet cylinders, and the pair of 5-feet cylinders, after having been sunk in place, are to be united together by a cast-iron transverse distance tie, bolted to each cylinder near the top by 4 bolts 1½ inch diameter.

CONCRETE. After each cylinder has been carried down to the proper depth, and after all the material has been excavated from the inside, they are to be filled in with concrete.

The concrete is to be brought up to the underside of, and filled in all round the granite blocks, and the work is to be finished level with the top of the blocks with 9 inches of brickwork, or stone-paving set with Portland cement.

Great attention is to be paid to ensure the cylinders being thoroughly filled in all round the edges, and underneath the ribs and flanges, with the concrete.

The concrete must be composed of clean gravel and sand, and best London Portland cement, in the proportion of 3 parts of gravel, 2 parts of sand, and 1 part of Portland cement.

The materials are to be accurately measured, and mixed in the dry state, the mass is then to be moistened and well beaten, until a thorough admixture of all the parts is effected, and it is brought to a proper consistency.

It is to be deposited in place as soon as possible after mixing. It is to be brought up in layers not exceeding 12 inches in thickness; each layer to be spread to a level surface before another layer is commenced.

The engineer reserves to himself the right of substituting Lias lime for the Portland cement, if he thinks proper.

GRANITE BLOCKS. The whole of the granite blocks, used throughout the bridge, whether in the cylinders, or in the foundations for the columns, in the landward openings, are to be of approved quality. The blocks are to be truly squared, and set perfectly level; the top surfaces are to be finely axed, and left perfectly smooth; and the other surfaces are to be pitched true.

The blocks are to be set on a bed of Portland cement.

WEIGHTING CYLINDERS. The main piers of the two 11-feet cylinders, and the piers for the swing-bridge, after being filled with concrete, to the underside of the granite blocks, and before the latter are in place, are to be loaded each with a dead weight of 400 tons, spread equally all over the surface; this dead weight is to remain on for, at least, one month.

The attached 5-feet cylinders are similarly to be loaded separately with a weight of 100 tons, or together with a weight of 200 tons.

PILLARS FOR CARRYING LATTICE GIRDERS. The foundations of the pillars supporting the lattice girders, and dividing the landward space into two openings, consist of footings of masonry or brickwork 2-feet deep, set in Portland cement. These footings are to be laid on a bed of concrete (composed of the same materials, and mixed in a similar manner, as the concrete in the cylinders), 3 feet deep, and projecting 1 foot all round, from the bottom of the masonry or brickwork; or the concrete is to be carried to such depths as may be directed.

Granite blocks are to be set on a bed of Portland cement on the brickwork or masonry.

These blocks are 4 feet square, and 2 feet 6 inches deep.

The bases of these pillars are to be bolted down to granite blocks by 4 strong dovetailed bolts, let into the blocks, and run with lead.

The pillars are to be of 1¼ inch metal, and are to be finished by an ornamental cast-iron cap, as shown, to which the lattice girders are to be bolted.

The pillar continued up on the 5-feet cylinder, dividing the south westermost side of the north westermost principal opening into two equal parts, is to be generally similar to the pillars just described, and the base is to be similarly bolted to a block of granite.

PLATEWORK RIVETTING. All plates are to be perfectly true and uniform in thickness; they are to butt closely and accurately together; and all vertical and horizontal joints of all plates in the top and bottom of the main girders, and all joints on the upper and lower flanges of the lattice, and upper and lower roadway cross-girders, and in all other similar situations, are to be planed.

Angle-irons are to be accurately rolled, so as to apply exactly to the surfaces of the plates connected by them, and are to be planed at the joints.

All vertical and horizontal joints in all plates in the top and bottom of the main girders, and all joints in the upper and lower flanges of the lattice girders, and upper and lower roadway cross-girders, and in all other similar situations, are to be secured by two covered plates, one on either side of the joint.

Each joint in the angle-irons is to be secured by a covering angle-iron, and by a $\frac{1}{2}$ inch plate at the top of the upper flange of the girders, or the bottom of the lower flange, as the case may be.

The outside edges of all plates are to be made perfectly smooth and regular by planing, or otherwise.

All cover-plates are to be sheared perfectly square, and are also to be made perfectly smooth and regular at the edges by planing, or otherwise.

All joints in all plates and angle-irons, are to occur only where shown, or where specially allowed.

All T-irons are to be truly cut at the ends, so as to ensure accurate fitting.

The whole of the rivetting throughout the bridge is to be performed in the most workmanlike, neat, and effective manner.

The rivet-holes are to be accurately punched, so that the rivetting may bring each piece into its proper position, and into perfect and accurate contact with the adjoining pieces. All rivets are perfectly to fill the holes, and are to be cup-headed, or shaped, as may be approved or ordered. The rivets are to be countersunk, where shown or ordered; and no two rivets shall, in any place, be further apart than $3\frac{1}{2}$ inches, centre and centre; all bolt-heads and nuts, where exposed to view, must be six-sided; and bolt-heads countersunk where ordered.

MAIN GIRDERS. The main girders are to be of the form and dimensions shown on the drawings.

At the places where the pin-holes occur, a plate 5 feet long, and covering either side of both the vertical plates of the top and bottom, and bent on the top to the shape and size of the horizontal flange of the angle-iron, is to be firmly rivetted to the top and bottom vertical plates. The angle-irons are to be united to this plate, where they butt against it, by a covering angle-iron and cover.

The angle-irons between the 5-feet plates are to be in one length.

The struts and ties are to be composed of "Howard's patent rolled suspension links," rivetted together in pairs, as shown, by rivets $\frac{3}{4}$ inch diameter. They are to be of the several widths shown on the drawings; the two bars composing each pair when together, must be of exactly the same shape and dimensions.

The struts and ties are to be stayed by 6 bolts, $\frac{7}{8}$ inch diameter, passing through cast-iron distance pipes, placed between them; and where they cross at the centre they are to be similarly stayed, and an ornamental casting is to be screwed on either side on to this centre bolt.

The bolt-holes and rivet-holes in these bars are to be drilled.

The vertical stiffening bars are also composed of "Howard's patent rolled suspension links," one being placed outside, and on either side of the vertical plates of the top and bottom. At the several points where they occur, they are to be united together by 2 wrought-iron bars, bent zigzag, as shown, which are to be rivetted together, and to the vertical stiffening bars at the several points of contact. Each of these zigzag bars is to be in one piece.

The holes in the swelled ends of the struts and ties, through which the pins are inserted, are to be bored with the greatest accuracy, and the pins must be truly turned; and the utmost attention must be paid, to ensure all these parts being fitted with the utmost precision, and in the most perfect and workmanlike manner. These pins are to be made of steel.

Each pin passes through a cast-iron distance pipe; and, on its outside, an ornamental casting is to be fixed, as shown.

The spaces between the vertical sides of the top and bottom are to be stayed by $\frac{7}{8}$ inch bolts, passing through pipes. There are to be three such stays in each horizontal space between the steel pins.

The sides of the girders at the places where they rest on the cylinders, are to be composed of $\frac{3}{8}$ inch wrought-iron boiler plate, strengthened by vertical T-irons, as shown, and by stays; each of the two opposite vertical T-irons being stayed by three stays of bolts and pipes.

At the top and bottom of all the inside vertical stiffening bars, wrought-iron gusset pieces cranked to give a space at the pins, are to be inserted, and firmly rivetted to the bars and to the adjoining cross-girders.

The rivets throughout the whole of these girders are to be $\frac{7}{8}$ inch diameter, unless otherwise shown.

To the outside of the ends of the main girders, adjoining the upper roadway lattice girders, upright castings, resting on bed-plates, on granite blocks, are to be securely bolted.

These upright castings are for the purpose of carrying one end of the upper roadway lattice girders, as shown.

LATTICE GIRDERS. The top and bottom of the upper roadway lattice girders, and of the lower roadway lattice girders, are to be composed of plate-iron. The sides are to be of lattice bars, rivetted between angle-irons at the top and bottom; the latter being rivetted to the upper and lower plates. Filling pieces are to be inserted between the angle-irons, where ordered. The lattice sides are to be stiffened by T-iron uprights, placed at intervals on each side of the girder, as shown. These T-irons are to be rivetted together, and also to the lattice bars, and the top and bottom angle-irons.

The ends of the upper roadway continuous lattice girders, and of the lower roadway lattice girders, are to consist of plate-iron, stiffened by T-iron uprights, as shown.

The rivets throughout the whole of these lattice girders are to be $\frac{3}{4}$ inch diameter, excepting at the crossing of the lattice bars, where they are to be $\frac{5}{8}$ inch diameter.

The lattice bars are to be placed at an angle of 45° with the top and bottom flanges.

CROSS GIRDERS. The upper and lower roadway cross-girders are to consist of a vertical web of $\frac{3}{8}$ inch boiler-plate, to which angle-irons are to be rivetted on either side at the top, and bottom, and ends.

The joints in the middle webs are to be secured by flat plates on either side, which are to be cranked over the vertical web of the angle-irons.

The angle-irons on either side are each to be of single pieces, the whole length of the girder, and turned square at the ends, so that the joint will occur at the middle of the ends.

The rivets are to be $\frac{5}{8}$ inch in diameter at the parts where the cross-girders rest on or are united to the main girders; the spaces between the rivets are to be filled in with iron cement, so as to make the bearings and points of contact perfect over the whole surface.

BED-PLATES, BOLTING DOWN GIRDERS, &c.

BED-PLATES, BOLTING DOWN GIRDERS, &c.

The lower fixed plate and the upper sliding plate of the bed-plates are to be of cast-iron, the upper surface of the lower plate, upon which the rollers run, and the lower surface of the upper plate, which runs out the rollers, are to be planed.

The rollers are to be of cast-iron, with wrought-iron axes or pins cast in at each end; the rollers and pins to be accurately turned and fitted in a wrought-iron frame work; the whole work is to be accurately put together. The holes for receiving the axes or pins of the rollers are to be drilled, great care being taken to preserve the axes of the rollers exactly parallel to each other.

The lower fixed plates are bolted down, and the girders are fastened to the upper sliding plates, as shown.

The girders that rest on the granite blocks are securely bolted to them by strong dove-tailed bolts, let into the blocks and run with lead.

SWING-BRIDGE.

SWING-BRIDGE.

The remarks referring to the main girders refer also to the large girders of the swing-bridge wherever applicable. The bars comprising the three centre vertical stiffeners are to be united by boiler-plates rivetted between angle-irons, the latter being rivetted to the bars. The ends of the girders are to be constructed as shown.

The joints of the several parts composing the upper roller frame are to be planed, and the whole bolted together. The top surfaces on which the girders rest, and to which they are bolted, are to be planed, and the roller path moving on the rollers is also to be planed. The centre pivot-hole is to be bored.

The joints of the several parts composing the lower roller frame are to be planed and the whole fitted and bolted together. The roller path, over which the rollers run, is also to be planed. The lower roller frame is to be bolted down to granite blocks by dove-tailed bolts let into the blocks and run with lead. The centre pivot-hole is to be bored.

The live ring is composed of wrought-iron; the part composing the centre radial frame is to be of solid malleable iron, neatly welded in the most perfect and workmanlike manner.

The ends of the radial bars are to be turned. The outer turned ends of the radial bars work in brasses fitted and fixed in the inside and outside vertical parts of the outer frame of the live ring. Between these vertical parts the rollers move, and are firmly keyed to the radial bars.

The inner turned ends of the radial bars work in brasses fitted and fixed in the outer circle of the centre radial frame.

There is a split cotter at either end of the radial bars, as shewn, through which a small keep-ring is inserted.

The rollers are to be of cast-iron, and are to be turned, and the centre holes bored.

Tangent bores, with a provision for tighfening-up, are fixed as shown.

The centre pivot is to be of wrought-iron, and the portion above the lower frame is to be case-hardened. Before case-hardening, the pivot is to be turned.

The pivot is to be fitted and firmly keyed into the centre hole of the lower roller frame.

The live ring and the upper roller frame work round the pivot; a brass is fitted into the former.

On the top of the pivot a cast-iron centre inverted cup, connected with four adjusting screws to the centre of the upper roller frame is fixed, as shown. The underside of this cup is to be case-hardened. All holes into which brasses are fitted are to be bored. Holes for oiling are to be made wherever necessary.

To the underside of the large girders a cast-iron wedge piece is to be bolted. The underside of this wedge-piece is to be planed, and the top surface of the adjusting wedge, which slides underneath it, as also its lower surface and the top surface of the bottom plate are also to be planed. The bottom plate, where it rests on the granite block, is bolted thereto by dovetailed bolts let into the block, and run with lead. The main rackwork connected with the machinery for opening the bridge is to be bolted to the outside of the lower roller frame.

This machinery, and also the machinery for working the adjusting apparatus is to be fitted in the most approved and workmanlike manner.

All centre holes are to be bored and filled with brasses, and all journals are to be turned.

A loose capstan working on a centre, and four capstan bars are to be provided for working the machinery for opening and shutting the bridge, also the necessary loose handles for apparatus for working the adjusting machinery.

All bolt-holes in the parts of the swing bridge, and in all situations generally throughout the bridge are to be drilled.

TIMBER, &c.

All the timber is to be of the first quality, Baltic or Quebec Yellow Pine, and is to be creosoted.

All timber is to be sound and straight, and free from sap, shakes, loose or dead knots, and from defects of every kind.

The timber, when cut out to the proper scantlings for the work, is to be creosoted with 50 gallons of creosote per load of 50 cubic feet. The absorption by the timber of this quantity of creosote is to be tested by weighing the timber before it is tanked and weighing it after it is taken out of the tank and dried, or in such manner as shall be satisfactory to the engineer, and one gallon of creosote is to be considered as weighing 10lbs. In all cases where any cutting of timber takes place after creosoting, the new surface so cut is to have 4 coats of hot creosote, the timber being first dried.

TIMBER TRUSSES AND PILEWORK FOR SAME.

The timber trusses are to be framed in the most workmanlike manner, and bolted and put together, as shown on the drawings, or as may be directed. All the timbers, whether in trusses or pilework, are to be sawn all round. The laminated planks, or timbers of the trusses, are to be planed, and are to be trenailed together. The whole of the timbers are to be of uniform dimensions, and are to be perfectly straight and square. All the joints are to be made with whitelead, oakum, or chopped hair and Stockholm tar; and allowance must be made for shrinkage.

LOWER ROAD-WAY PLATFORM.

The piles are to be driven to such depths, and in such manner, as is hereinafter specified for the piles at the swing-bridge.

All bearers (longitudinal or cross) must be sawn all round, as described for the timbers of the timber-trusses. The planking is to be grooved, and tongued with iron, and caulked with tar and oakum; the whole surface is to be left smooth and regular, no one plank projecting above the adjoining.

The longitudinals and planking are to be fastened to the wrought-iron cross-girders, by ¾ inch hook-bolts, to avoid weakening the latter by bolt-holes.

The whole of the timber-work is to be bolted, and put together in the most workmanlike manner, and as may be directed; and all timber is to be square and straight, and of uniform dimensions throughout.

UPPER ROAD-WAY.

The cross-girders of the upper roadway are to be rigidly connected by "Mallet's patent buckled plates." These plates are to be laid butt to butt. The ends of each plate are to be rivetted to the top angle-irons of the tranverse girders, and they are to be connected together longitudinally by a continuous T-iron bar rivetted to the plates, so as to give proper stiffness to the whole surface.

At the edges of the outside buckled plates, and on each side of the roadway, a cast-iron gutter is to be bolted down to each of the cross-girders. This gutter is to be cast in lengths of not less than 17 feet. At the junction of the length, flanges are to be cast underneath, and they are to be securely bolted together. The water is to be discharged from these gutters by pipes leading from them near to each pier, and so conducted down that the surface water can fall free of the structure, and of any craft passing beneath. At the junction of the swing-bridge the gutters are to have stopped ends, and at the swing-bridge. The discharge-pipe is carried down near the centre of its length, and must be made to clear the large cylinder. At these discharge pipes, provision must be made for expansion as may be directed. The gutters are to be cast with a regular fall.

The buckled-plates are to be covered with find sand, to a depth of, at least, 1 inch over their centre. The sand is then to be covered with 1 inch of asphalte, formed of hot sand and gravel, passed through a ⅜ inch sieve, and boiled asphalte of the best quality, or the asphalte is to be made as may be directed. The surface of the sand and asphalte is to be smoothed and rounded to the curve of the roadway. Before the sand and asphalte are put on, the upper surface of the plates are to be coated with hot coal-tar.

Upon the surface of the asphalte "Hosking's patent wood paving," or an approved paving of oak, set in marine glue, is to be laid. The wood-paving is to be neatly fitted and finished at the junction with the cast-iron gutter.

FOOTWAYS.

The footways are also composed of "Mallet's buckled plates," laid as shown. They are to be covered with fine sand to a depth of, at least, 1 inch over the shallowest part of the plates. This sand is to be uniformly laid over, and covered with 1 inch of asphalte, made as before described, but with finer sand and gravel, or made in any way that may be directed. The surface is to be finished perfectly smooth and parallel with the plates, which are to be laid so as to preserve a fall of 2 inches towards the gutter.

CYLINDERS, LAPS, PILASTERS, LAMPS, &c.

The 11-feet cylinders are to be covered with cast-iron caps, and cast-iron ornamental pilasters and cornices are to be carried up thereupon.

These pilasters are to be surmounted by lamp-posts and lamps.

The cornice is to be continued all along from pilaster to pilaster, and along the bottom of the upper lattice girder, covering the ends of the cross-girders, and bolted, as shown.

The lamps are to be of a design approved of by the engineer, and are to be glazed with stout glass, and fitted with approved burners.

The outer ends of the lower roadway cross-girders are to be carried by a longitudinal moulded casting, bolted, as shown.

PALING.

Between the pilasters carried above the footpath, an ornamental paling is to be fixed.

The paling consists of cast-iron standards, fixed at the intervals shown, by being bolted to the outside longitudinal footway-girder, and further strengthened by cast-iron ornamental brackets.

The ornamental lengths of the paling slide into grooves in these uprights, and are fastened thereto by screws tapped in and countersunk.

The joints are to be made good with iron-cement. The whole is to be surmounted by a hand-railing, cast in long lengths, united together, and to the paling. A casting is also bolted on the top flange of the roadway lattice girder.

PILEWORK AT SWING-BRIDGE.

The whole of the timber in the pilework are to be sawn all round. The piles are to be well hooped for driving; shod with approved shoes, and driven perfectly square and upright. They are to be driven until they will not drive more than one inch with 8 blows of a monkey, weighing not less than 16 cwt., falling through 20 feet, or to such depth as the engineer may direct. Any piles that are split, or in any way injured in driving, are to be drawn and new ones substituted.

The whole of the timberwork is to be well bolted, and put together in the most workmanlike and approved manner; and all timbers are to be square, straight, and of uniform dimensions throughout.

RAILS.

The rails to be laid throughout the lower roadway are to weigh 75 lbs. per yard. They are to be bridge rails, flat-bottomed, or of any section that the engineer may approve; and are to be fastened and laid as may be directed.

PAINTING AND PROTECTING.

Before erecting, the whole of the ironwork is to receive one coat of boiled linseed oil, laid on hot; and the whole of the ironwork above low-water mark is, in addition, to receive before erection, one coat of redlead. After the erection the whole of the ironwork above low-water mark, is to receive another coat of redlead, and to be finished with two coats of the best oil paint, of a colour to be determined by the engineer; and portions of the ornamental work, if required, are to be picked out with additional coats of paint.

Great attention is to be paid to protect all the ironwork before oxidization commences.

All paint is to be the very best, and perfectly free from adulteration.

All planed, bored, or turned surfaces, are to be carefully coated with whitelead and tallow, to preserve them from oxidization.

PATENT RIGHT.

The contract sum is to include all charges for patent rights whatever, and the contractor is to bear the commissioners harmless against all claims of all kinds connected therewith.

TESTING.

The testing is to be done in the following manner:—

FOR THE MAIN GIRDERS

Two girders are to be placed transversely, as they will be when in place, and on immoveable supports, leaving a clear span of 119 feet. When in this position, and before being loaded, each girder is to have a camber in the centre, of one inch and a-half, or they are to be constructed with such other camber as may be deemed desirable by the engineer.

They are then to be loaded with 400 tons, spread equally all over the two girders. With this weight the deflection is not to exceed 1½ inch in the middle.

FOR THE UPPER ROADWAY LATTICE GIRDERS.

Two girders, continuous over two openings, are to be placed transversely, as they will be when in place, and on immoveable supports leaving two clear spans of 62 feet. When in this position, and before being loaded, the girders are to have a camber of one inch in the centre of each of the openings, or they are to be constructed with such other camber as may be deemed desirable by the engineer. The weights are then to be applied as follows :—

One of the openings is to be loaded with a weight of 90 tons, spread equally all over the two transverse girders of the opening, with this the deflection is not to exceed 1 inch in the middle.

Before the removal of the load, the other opening is to be similarly loaded with 90 tons, spread equally all over the two transverse girders of the opening, with this load the deflection of the two openings is not to exceed ⅝ inch in the middle of each.

The former load is then to be removed, when the deflection of the girders of the latter opening is not to be more than 1 inch in the middle.

FOR THE LOWER ROADWAY LATTICE GIRDERS.

The two girders are to be placed transversely, as they will be when in place, and on immoveable supports, leaving a clear span of 62 feet. When in this position, and before being loaded, each girder is to have a camber in the centre of 1 inch, or they are to be constructed with such other camber as may be deemed desirable by the engineer.

They are then to be loaded with 65 tons, spread equally all over the two girders; with this weight the deflection is not to exceed 1 inch in the middle.

FOR THE ROADWAY CROSS GIRDERS.

These girders are to be placed on immoveable supports, leaving a clear span, such as they will have when in place, and when in this position, they are to be perfectly straight, and without any camber.

The lower roadway cross girders are then each to be loaded with 6 tons, spread all over. With this weight the deflection is not to exceed ¼ inch in the middle.

The upper roadway cross girders are also to be loaded with 6 tons spread all over. With this the deflection is likewise not to exceed ¼ inch in the middle.

After the removal of the loads, the whole of the girders, main lattice, and roadway, must resume their original conditions, in every respect.

Should any of the supports sink during the testing, the load will have to be removed, the supports made good, and the testing done over again.

The wrought-iron used in the bridge will be tested from time to time, and it must stand such tests as the engineer shall consider satisfactory.

The engineer is to have the power to alter any of the previously-described modes of testing; and to test the girders, wrought and cast ironwork, and iron timber trusses, and any other part of the work, in any way he may think desirable.

CHARING CROSS BRIDGE.

PLATES LX. TO LXIII.

This structure, which is now being erected across the Thames at Hungerford, was designed by Mr. John Hawkshaw, to carry the London Bridge and Charing Cross Railway; and the work will be executed by Messrs. Cochrane and Co., of the Woodside Iron Works, near Dudley.

It is intended to carry four lines of railway, and two footways, one on either side; the central part of the platform being 49 feet in width, and each footway 9 feet.

The platform will be supported by two main girders, of great strength and solidity, very carefully constructed. From the bottom flanges of these main girders, cross girders are suspended, the extremities of which are prolonged in the form of cantilevers to carry the footways; the total length of each cross girder, including the cantilevers, being 67 feet. The bottom flange of each cross girder is curved, so as to give a greater depth at the centre than elsewhere. The general principle of the main and cross girders, is that of the trellis girder, but the struts and ties of the main girders are exceedingly massive.

The superstructure, which is in discontinuous spans, is to be carried by seven piers and two abutments; the two piers which formerly carried the Hungerford Suspension Bridge, will be enlarged to form two piers for the support of the future bridge; the remaining five will consist of two cylinders each, the diameter of each cylinder being 14 feet at the bottom, and 10 feet at the top.

The cylinders are sunk by dredging out the soil until a water-tight stratum is arrived at, when the water is pumped out, and the excavation continued in the usual manner.

The rivet-holes in the plates forming the main girders, are drilled by a machine especially constructed for the purpose; it drills all the holes in one plate, 80 in number, at once; a ⅝ inch plate being drilled in 15 minutes. A full description of this machine having been given at page 141, we shall not now dilate further upon this subject.

The holes in the plates for the top flange, are 4 inches apart, from centre to centre, and those in the bottom flange, are 3·995 inches apart, the difference being made in order to obtain the requisite amount of camber.

All the work in this structure is being executed in a most careful manner, the joints being planed to fit with extreme accuracy.

For further information as regards the method of constructing the various details, our readers are referred to the subjoined specification.

SPECIFICATION.

CAST-IRON. All cast-iron must be of the best cold-blast grey metal; and the castings must be straight, and exact to shape and dimensions; clear and sound, and entirely free from air-holes and flaws of every description.

Great attention must be paid to the execution of all ornamental castings; the arrisses must be left sharp, and the several parts neatly and accurately joined and fitted.

The mixture of the metal will be left to the contractors' own judgment, but it must be of such quality as the engineer shall approve; and, in respect of strength, must stand the following test:—Two bars are to be cast each day that any of the castings are being run, three feet six inches long, one inch broad, and two inches deep.

The person appointed to inspect the work, will then test one of these by placing it edgewise on bearings, three feet apart, and loading the middle of the bar, and should it break with any weight less than twenty-seven hundred weight, all work cast with that metal will be rejected.

The other test bar is to be marked with the date of the cast, and put away for the subsequent inspection of the engineer.

WROUGHT IRON. The whole of the wrought-iron is to be equal in quality to the very best Staffordshire.

IRON CEMENT. The iron cement is to be composed of clean iron borings, sal ammoniac, and flour of sulphur, in proper and approved proportions.

No iron cement will be allowed to be used, except in the situations specially referred to in the specification.

SINKING CYLINDERS. The cylinders for the bridge across the Thames, are to be sunk to such depth as the engineer shall direct.

The cylinders are to be sunk by dredging the material from the inside, and then weighting them; when a water-tight stratum is met with, the cylinders must be pumped dry, and the material dug from the inside in the usual way; should no sufficiently water-tight stratum be met with; then, after the cylinders have been sunk by dredging and weighting, to such depth as the engineer may approve, the contractor must provide suitable apparatus, and force the water from the interior of the cylinders by compressed air, so as to enable the foundations to be examined; and the sinking must be continued by the pneumatic process. Any other means that the engineer may think it desirable to adopt for sinking the cylinders, are to be resorted to.

All the apparatus and machinery necessary for sinking the cylinders, shall be such as the engineer shall approve.

The spaces left between the flanges in the cylinders, both vertical and horizontal, are to be caulked with iron cement, so as to render the cylinders perfectly water-tight.

CONCRETE. After each cylinder has been carried down to the proper depth, and after all the material has been excavated from the inside, they are to be filled in with concrete, or brickwork, or partly with one and partly with the other, as the engineer may direct.

The concrete or brickwork is to be brought up to the under side of, and filled in all round the stone blocks, and the work is to be finished level with the top of the blocks. Great attention is to be paid to ensure the cylinders being thoroughly filled in all round the edges, and underneath the ribs and flanges, with the concrete and brickwork.

The concrete must be composed of clean gravel, and clean sharp river sand, and best London Portland cement, in the proportion of three parts of gravel, two parts of sand, and one part of Portland cement.

The materials are to be accurately measured, and mixed in the dry state; the mass is then to be moistened, and well beaten until a thorough admixture of all the parts is effected, and it is brought to a proper consistency.

It is to be deposited in place as soon as possible after mixing. It is to be brought up in layers not exceeding twelve inches in thickness, each layer to be spread to a level surface before another layer is commenced.

The engineer reserves to himself the right of substituting Lias lime for the Portland cement, if he thinks proper.

STONE BLOCKS FOR CARRYING ENDS OF GIRDERS. The whole of the stone blocks used throughout the bridge, whether in the cylinders or on the piers, are to be of approved quality; the blocks are to be truly squared, and set properly level; the top surfaces are to be finely axed, wrought to a form suitable for receiving the ends of the girders and bed-plates, and left perfectly smooth; and the other surfaces are to be picked true. The blocks are to be set on a bed of Portland cement.

WEIGHTING CYLINDERS. The piers formed of cylinders, after being filled with concrete to the underside of the stone blocks, and before the latter are in place, are to be loaded each with a dead weight of seven hundred tons, spread equally all over the surface; this dead weight is to remain on for, at least, one month.

PLATEWORK RIVETTING.

All plates in the girders of the Thames and other bridges, are to be perfectly true, and uniform in thickness, they are to butt closely and accurately together; and all vertical and horizontal joints of all plates in the top and bottom of every girder, wherever used, are to be planed.

Angle-irons are to be accurately rolled, so as to apply exactly to the surface of the plates connected by them, and are to be planed at the joints.

All vertical and horizontal joints, in all plates in the top and bottom of the girders, and all joints in the angle-irons, are to be amply secured by cover plates.

The outside edges of all plates are to be made perfectly smooth and regular by planing, or otherwise.

All cover plates are to be sheared perfectly square, and are also to be made perfectly smooth and regular at the edges by planing, or otherwise.

All joints in all plates and angle-irons, are to occur only where shown, or where specially ordered by the engineer; all T-irons are to be truly cut at the ends, so as to ensure accurate fitting.

The whole of the rivetting in the Thames and other bridges, is to be performed in the most workmanlike, neat, and effective manner.

The rivet-holes are to be accurately punched, so that the rivetting may bring each piece into its proper position, and into perfect and accurate contact with the adjoining pieces. All rivets are perfectly to fill the holes, and are to be cupheaded, or shaped as may be approved or ordered; the rivets are to be countersunk where shown or ordered; and no two rivets shall in any place, unless permitted by the engineer, be further apart than three inches and a-half, centre to centre. All bolts, heads, and nuts, where exposed to view, must be six-sided, and bolt heads countersunk where ordered.

The struts and ties of the Thames Bridge, unless otherwise directed, are to be composed of "Howard's patent rolled suspension links," rivetted together in pairs, as shown, by rivets three-quarters of an inch diameter; they are to be of the several widths shown on the drawings; the two bars composing each pair when together, must be of exactly the same shape and dimensions.

These struts and ties are to be of the best malleable iron, neatly formed, and stayed by stays and cross bracing, in such manner as the engineer shall direct; and when they cross at the centre, they are to be similarly stayed, and an ornamental casting is to be screwed on either side on to this centre bolt.

The bolt-holes and rivet-holes in these bars are to be drilled. The vertical stiffening bars are also, unless otherwise directed; to be composed of "Howard's patent rolled suspension links;" one being placed outside, and on either side of the vertical plates of the top and bottom. At the several points where they occur they are to be united together by stays and cross bracing, in such manner as the engineer shall direct.

The engineer will, if he thinks fit, obtain a portion of the stiffness required for the struts and vertical stiffening bars, by rivetting to them angle or T-iron, and connecting these by cross bracing, or by solid plates, as he may determine.

The holes in the swelled ends of the struts and ties, and in the girders through which the pins are inserted, are to be bored with the greatest accuracy, and the pins must be truly turned, and the utmost attention must be paid to ensure all these parts being fitted with the utmost precision, and in the most perfect and workmanlike manner; these pins are to be made of steel.

Each pin, should the engineer require it, may have to pass through a cast or malleable iron distance pipe, or pipes, and on its outside an ornamental casting is to be fixed, as shown.

The spaces between the vertical sides of the top and bottom of the main girders of the Thames Bridge are to be stayed, unless otherwise directed, by seven-eighths of an inch bolts, passing through cast or malleable iron plates. There are to be three such stays in each horizontal space between the steel pins.

The expansion rollers for the Thames and other bridges are to be of cast-iron with wrought-iron axes, or pins cast in at each end; the rollers and pins to be accurately turned and fitted in a wrought-iron framework; the whole work is to be accurately put together. The holes for receiving the axes or pins of the roller are to be drilled, great care being taken to preserve the axes of the rollers exactly parallel to each other, or the whole of the expansion gear shall be done in such other manner as the engineer shall direct.

The lower fixed plates are bolted down, and the girders are fastened to the upper sliding plates, as shown.

All girders that rest on stone blocks are, where directed, to be securely bolted to them by strong dovetailed bolts, let into the blocks, and run with lead.

TIMBER.

All the timber used in the bridges is to be of the first quality of Baltic, is to be sawn all round, and is to be creosoted.

All timber is to be sound, and straight, and free from sap, shakes, loose or dead knots, and from defects of every kind.

The timber, when cut to the proper scantlings for the bridges, is to be creosoted with fifty gallons of creosote per load of fifty cube feet; the absorption by the timber of this quantity of creosote is to be tested by weighing the timber before it is tanked, and again after it is taken out of the tank and dried, or in such manner as shall be satisfactory to the engineer, and one gallon of creosote is to be considered as weighing ten pounds. In all cases where any cutting of timber takes place after creosoting, the new surface so cut is to have four coats of hot creosote, the timber being first dried.

All bearers (longitudinal or cross), and all planking must be sawn all round and planed on those sides exposed to view. The planking of all the bridges is to be grooved, and tongued with iron, and caulked with tar and oakum; the whole of the upper surface of the planking, when not intended to be covered with ballast, is to be planed and left smooth and regular, no one plank projecting beyond the adjoining.

The longitudinal and planking are to be fastened to the wrought-iron cross girder by hook-bolts, to avoid weakening the latter by bolt-holes.

The whole of the timber-work is to be bolted together in the most workmanlike manner, and as may be directed; and all timber is to be square and straight, and of uniform dimensions throughout.

PAINTING AND PROTECTING.

Before erection, the whole of the ironwork for all the bridges, colonnades, &c., is to receive one coat of boiled linseed oil, laid on hot; and the whole of the ironwork is, in addition, to receive before erection one coat of red paint. After the erection the whole of the ironwork of the Thames Bridge above low-water mark, and all the ironwork of the other bridges, is to receive another coat of redlead, and to be finished with two coats of the best oil paint, of a colour to be determined by the engineer; and portions of the ornamental work, if required, are to be picked out with additional coats of paint.

Great attention is to be paid to protect all the ironwork before oxidization commences.

All paint is to be of the very best, and perfectly free from adulteration.

All joints of all ironwork, cast and wrought, are where the engineer directs, to be caulked with iron cement.

All planed, bored, or turned surfaces are to be carefully coated with whitelead and tallow, to preserve them from oxidization.

TESTING. The testing is to be done in the following manner :—

MAIN GIRDERS.

Two girders are to be placed on immoveable supports, leaving clear spans or spaces of one hundred and fifty-four feet. When in this position, and before being loaded, each girder is to have a camber in the centre, of one inch and a-half, or they are to be constructed with such other camber as may be deemed desirable by the engineer.

They are then to be loaded with eight hundred tons, spread equally all over the two girders. With this weight the deflection is not to exceed one and a-half inches in the middle.

The other girders are to be tested in like manner, but with such weights as the engineer may determine; they are to be constructed with such camber as the engineer shall determine or approve.

The wrought-iron used in all the bridges will be tested from time to time, and it must stand such tests as the engineer shall consider satisfactory.

The engineer is to have the power to alter any of the previously-described modes of testing, and to test the girders, wrought and cast ironwork, and iron timber trusses, and any other part of the work in any way he may think desirable.

CHAPTER XVIII.

WROUGHT-IRON LATTICE GIRDER BRIDGES.

The bridges to be described in the present Chapter, comprise the Lerida, Alcanadre, Murillo, and Carlos Gomes. The first of these, is a very good example of a five-span continuous girder bridge, on the lattice principle. It presents a light, but strong appearance, and is, on the whole, well suited to the purpose for which it has been designed.

The Alcanadre Bridge, like the Lerida, was intended to be erected in five spans, but the great difficulties encountered in sinking the foundations for the two outer piers, induced the engineer to dispense with the two centre piers, which has detracted somewhat from the general appearance of the structure, which, however, shows a careful design.

The Murillo Bridge is, in its general aspect, very similar to the foregoing structures, with the difference that it is not supported by continuous girders.

The last bridge of which we shall here speak : that of Carlos Gomes, is of very peculiar construction, for the web although of the lattice form, is composed of bars placed at an unusual angle ; they are, moreover, of uniform size, as are also the plates composing the top and bottom flanges; which, however, has probably been decided upon in order that the various parts of the structure might be counterparts to each other, so that in the case of the loss of any plate, its place might be supplied by some other, whereby delay would be avoided. It must also be remembered, that this structure had to be erected in a locality where skilled labour is exceedingly scarce; which may account for the peculiarity of the design, as the cost of erecting a more accurately planned bridge, would probably have exceeded that of the excess of material used in the structure as designed.

Had circumstances allowed of a more economical distribution of the material, the sectional area of each flange, would have been greatest over the centre pier, from whence it would have diminished on both sides to the points of contrary flexure; then, after increasing up to points about five-eighths of each span, from the centre pier, it would again diminish to a minimum at the abutments.

LERIDA BRIDGE.

PLATES LXIV. AND LXV.

This bridge carries the Barcelona and Saragossa Railway over the River Segre, at Lerida; it has been designed, constructed, and erected by Messrs. Charles de Bergue and Co., of Strangeways Iron Works, Manchester. It crosses the river in five clear spans of 131 feet $2\frac{1}{8}$ inches each; but the actual distance, being the points of bearing on the rollers, measured from the centre of end roller on one pier, to centre of end roller on the next, is 135 feet $10\frac{3}{4}$ inches; its total length over all is 712 feet. It carries a single line of rails, and is composed of two wrought-iron lattice girders, on the continuous principle; placed 16 feet 3 inches apart from centre to centre, giving a clear width between the top flanges of the girders of 13 feet $8\frac{1}{2}$ inches; the total depth of the girder is 10 feet 11 inches.

The cross beams are made to rest on the angle-irons, connecting the lower flange to the web, and are placed 3 feet $2\frac{15}{16}$ inches apart, from centre to centre; on these are laid planking and rails.

On the piers, and abutment piers, cast-iron bed-plates 3 inches thick are fixed, by means of holding-down bolts, the upper surfaces of these plates are planed to receive the friction rollers, which are 4 inches in diameter, and 3 feet long, they are arranged in sets, in wrought-iron frames formed of flat bars 4 inches by 1 inch, connected by distance pieces, or stretchers; the flat bars are drilled with holes 2 inches in diameter, and 6 inches from centre to centre, into which the ends of the rollers fit; the roller frames for the piers contain ten rollers, giving a bearing of 4 feet 7 inches, measured from centre to centre of end rollers; those for the abutments contain seven rollers, with a bearing of 3 feet 4 inches. On the under side of the bottom flange of main girders, corresponding to the respective position of the rollers in place on the piers, are fixed cast-iron roller plates $3\frac{1}{2}$ inches thick; on the side towards the girder, these plates are fluted, or hollowed to receive the heads of the rivets, the other side being truly planed to run upon the rollers; rollers are provided for the two abutment piers, and one of the intermediate piers, the others being fixed, from which the girders expand and contract.

The main girders, as previously stated, are of lattice framing, on the continuous principle; each girder is constructed as follows:—the top flange, or boom, is of a T-shape in section, (see Fig. 6, Plate LXIV.), and is composed of 2 horizontal plates, *b b*, which are connected to the vertical plate, *a*, by the two angle-irons, *c c;* on the outside edges of the horizontal plates are angle-irons, *d d*, and the joints of these plates are covered by the strip, *e;* the several members composing this flange, are increased in thickness according to the sectional area required, except the plate, *a*, which is of uniform thickness throughout, and where the section required is great, the plates, are doubled; the plates, *a* and *b b*, through the whole of the girder, are in lengths of 12 feet 9 inches, and the angle-irons, *d d* and *c c*, are in lengths of 25 feet 6 inches. The horizontal plates, *b b*, are further connected to the vertical plate, *a*, by wrought-iron stays, *f*, 7 inches by $\frac{3}{8}$ inch, these stays are placed at distances along the boom of 6 feet $5\frac{7}{8}$ inches, from centre to centre.

The bottom flange is also T-shape in section (see Fig. 7, Plate LXIV.), and is composed of two horizontal plates, *b b*, a vertical plate, *a*, and two angle-irons, *c c*; the vertical and horizontal members are connected by the angle-irons, *c c*, and the cover strip, *e*, and, as in the case of the top flange, they are further connected by means of the stays, *f*, with this exception, that on the inner side of each girder on the bottom boom there are no stays, the cross-beams coming in their stead. The several members composing the bottom flange are increased in thickness in proportion to the area required, except *a*, which is of vertical thickness throughout; and where the section required is great, the plates, *b b*, are doubled, the plates and angle-irons are of similar length to those in the top boom; they are made to break joint, and are covered as shown in drawings.

The web of the main girders throughout is of lattice framing, formed of angle and flat bars, placed at an angle of 45 degrees; the angle bars are used in compression, and vary in scantling from 5 inches by 3 inches by $\frac{5}{8}$ inches to 5 inches by 3 inches by $\frac{3}{8}$ inch; the flat bars are used in tension, except at the centre of each span, where a few are introduced as struts, the compressive strain there being comparatively light; these bars vary from 5 inches by $\frac{9}{16}$ inch to 5 inches by $\frac{1}{2}$ inch. The lattice framing is 3 bays in height, 3 feet $2\frac{15}{11}$ each, measured from centre to centre of crossing; the extreme height between the crossings of the centre of the lattice bars, is 9 feet $8\frac{13}{16}$ inches. The lattice bars are connected to the vertical plate, *a a*, of the top and bottom flanges by a system of rivetting, and to each other at their crossings, by two rivets through each pair. For a distance of six bays from each pier a plate 8 inches by 8 inches by $\frac{1}{2}$ inch, is introduced between the crossings of the lattice bars, which affords additional means of rivetting, to meet the severe strains induced in these bars when the bridge is loaded. For a similar reason the vertical plates, *a a*, are also increased to 15 inches deep, for a distance of four bays from the piers; they then taper down in a distance of four bays to 12 inches deep, which depth they maintain until they approach the same distance from the opposite pier, when they again increase to 15 inches, and so on to the end of the bridge.

The lattice web over the piers is further stiffened by the introduction of a pier-plate, 8 feet 2 inches high, 6 feet wide, and $\frac{1}{2}$ inch thick, resting on the upper edge of the bottom vertical plate, *a*, to which it is connected by cover strips on both sides; it reaches to the under edge of the top vertical plate, *a*, to which it is similarly connected. A plate $\frac{5}{8}$ inch thick runs up the centre of this pier-plate on the inside of the girder, and extends from the under side of the cross beam to the web of which it is rivetted, and at which place it is 1 foot $2\frac{1}{2}$ inches wide, to the under edge of the angle-iron, *c*, in the top boom, where it is only 7 inches wide, and is rounded off (see cross section, Fig. 3, Plate LXIV.). This plate is denominated a wing-plate, and is fastened to the pier-plate by lengths of angle-iron. In order that the wing-plate may go home to the pier-plate, notches are cut in its edge to receive the lattice bars where they cross.

The lattice framing is still further stiffened by having strips or plates of wrought-iron, 7 inches wide by $\frac{1}{2}$ inch thick and 8 feet 2 inches high, extending from the upper edge of the vertical plate, *a*, of the bottom boom to the under edge of the vertical plate, *a*, in the top boom, to which plates they are connected by covers of wrought-iron. These plates are so placed along the framing of the web as to be vertically over the position of the cross bearers, and to these several strips are attached wing-plates, which are fastened to the ends of the cross beams, and are in all respects similar to those above described.

The cross girders are formed of a plate-web, $\frac{1}{4}$ inch thick and 1 foot deep, with two angle-irons for the top and two for the bottom flange. These angle-irons are $2\frac{1}{2}$ inches by $2\frac{1}{2}$ inches by $\frac{5}{16}$ inch; on each end are two angle-irons, 3 inches by 3 inches by $\frac{3}{8}$ inch, which are rivetted to the vertical plate, *a*, of the bottom boom; the cross beams, which come in where the several wing-plates are disposed, are of a stronger class, having the web-plate $\frac{7}{16}$ inch thick, and the angle-irons $2\frac{1}{2}$ inches by $2\frac{1}{2}$ inches by $\frac{7}{16}$ inch; where the wing-plates are attached to the cross beams, the angle-irons forming the top and bottom flanges are cut away for a distance of 1 foot 2 inches on one side only, to permit the wing-plate to pass down by the side of the cross girder to which it is rivetted, as above stated (see Fig. 3, Plate LXIV.).

The pitch of the rivetting throughout was 4 inches, except in the cross girder, where it was 3 inches. The arrangement of the rivets in the booms, lattice bars, and other portions of the work, was such as to give the required number of rivets with the minimum waste of cross sections.

The weight per foot-run of this bridge, not including the planking and rails, is $9\frac{1}{2}$ cwt., the gross weight, including bed-plates, rollers, holding-down bolts, &c., is 358 tons 6 cwt. 3 qrs. 11 lbs. It is considered a cheap structure, and for its lightness is remarkably rigid; it exhibits the best form of lattice bracing that has come under our notice. The bridge is erected at a height of about 27 feet from the surface of the water, which in some places is from 20 to 23 feet deep; it has been put in position without any scaffolding, and this operation has been performed in an ingenious and very successful manner. The structure was in the first instance framed and rivetted on the embankment; it was then moved forward on rollers of a very peculiar description, and passed over the abutment pier, until it reached the first of the intermediate piers, when rollers similar to those used on the embankment were placed on the pier to receive the forward end of the bridge. As soon as these rollers commenced to bite, they were worked by four men, by means of handspikes, and the bridge was again moved forward; and this process continued until the whole length of the superstructure was in position; it was then raised about 1 foot 6 inches, by means of small presses, to free the capstan rollers, and to insert the frames carrying its permanent rollers; after which it was again lowered, and was ready to receive the planking and rails.

The following deflections were observed on loading the bridge as described:—

Load.				Deflection.	Rise of next span.
One locomotive engine at the centre of the span				0·59	—
	do.	do.	running at 30 miles per hour	0·66	0·04
Two	do.	do.	at rest	0·78	0·04
	do.	do.	running at 30 miles per hour	0·86	0·08
Three	do.	do.	at rest	0·96	0·16
	do.	do.	running at 30 miles per hour	0·96	0·40
Two	do.	do.	and four loaded waggons	—	—
	do.	do.	running at 30 miles per hour	0·86	0·12

One locomotive engine was furnished with a tender, but the others were not. They were of Stuart's heaviest class.

The most perfect accuracy was obtained in punching the holes by means of a machine patented by Mr. Charles de Bergue, of the firm of De Bergue and Co., at whose works, in Manchester, the above bridge was made. A small pointer is attached to the machine, and is so adjusted that the distance from its point to the centre of the punch is exactly equal to the pitch of the holes to be punched in the plates, which are stamped by hand with a centre-punch, instead of being marked with paint or whiting in the ordinary way. The pointer, in the process of punching, drops into the centre punch-holes, or indentations, and holds the plate slightly in position, while each successive hole is being punched, the punch having a small projecting hole or nipple on the under side, which corresponds to or fits the mark or hole made by the hand centre-punch. In addition to greater accuracy, a larger amount of work may be got through by the use of this machine than by the old process.

We were much pleased with the action and working of these punching-machines, and were convinced of the great advantage to the manufacturer, of having tools well adapted to the work in hand; to the engineer who plans and designs such works as bridges, roofs, or any other description of work requiring punching, and rivetting, it is a matter of much importance that great accuracy should be obtained, so that in cases where several layers of plates, or angle-irons and plates come together, the rivet-holes should coincide, and positively require little or no drifting, for by accurate work in this respect, the holes are well fitted by the rivets, and their edges all bear; this ensures the calculations of the engineer, and the ultimate character of his design as an enduring work.

We venture these remarks, in consequence of having closely observed the accuracy with which the several layers of plates, punched to the same pitch, coincided; we were, however, much inclined to doubt whether they had not been all rimed, but a closer examination, fully convinced us they had not.

A drawing of the above machine will be found on Plate D.

ALCANADRE BRIDGE.

PLATE LXVI.

The width of the River Alcanadre at the point where the railway crosses, is about 364 feet, and the height from the ordinary water line to the level of the rails, is about 89 feet, it was proposed by the engineer to make the bridge in 5 spans, of 20 metres each (65 feet 7½ inches), the piers being each 25 metres.

at the top, 8 feet $2\frac{1}{2}$ inches. The land or abutment piers were first completed, and then the piers next thereto on each side, 20 metres each from the abutments; but in executing, these the material composing the river bottom was found highly unfavourable for foundations, and many serious difficulties were encountered from the dangerous freshes in the river; it was, therefore, deemed prudent to abandon the idea of erecting any more piers in the waterway, and to design a structure to span the three openings, viz., one centre span of 213 feet 3 inches, and two side openings of 65 feet $7\frac{1}{2}$ inches each. It was accordingly designed as illustrated in Plate LXVI., and as explained in the following description:—

It carries a single line of rails, and is composed of two wrought-iron lattice girders, placed 11 feet 6 inches apart, from centre to centre; each girder is continuous, and is 374 feet long, and 17 feet $\frac{13}{16}$ inch deep, they are connected by cross girders, and diagonal bracing, as shown in Figs. 2 and 6, Plate LXVI., the cross beams are attached to the top flange of the main girders, and are similar in construction to those already described for the Lerida Bridge. A light handrail of iron wire is fastened to the sides of the top flanges of the main girders, and serve as a protection to workmen, and others crossing the bridge.

The top flange is T-shaped in section, and is composed of two horizontal plates, *b b*, Fig. 8, each 1 foot 6 inches wide, a vertical plate, *a*, two angle-irons, *d d*, $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{1}{2}$ inch, and two angle-irons, *c c*, $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{1}{2}$ inch; the width of the top flange when rivetted up, is 3 feet $\frac{1}{2}$ inch; this member is further stiffened by stays, *f*, which are 7 inches by $\frac{1}{2}$ inch, and placed along the outside elevation of the flange at intervals of 9 feet $11\frac{1}{4}$ inches, from centre to centre. The cross beams are connected to the vertical plate, *a*, and the horizontal plate, *b*, by means of rivetting; packing pieces the thickness of the web of the angle-iron, *c*, being introduced respectively between the cross beams, and the plates, *a* and *b*. The several members composing the section of top flange, are 16 feet $6\frac{13}{16}$ inches long, they are made to break joint, and are covered as shown in Figs. 4 and 5, they vary in thickness in proportion to the sectional area required in each length of the flange, and where this area is greatest, the plates, *b b*, are quadrupled.

The bottom flange is in every respect similar in section and composition to the top flange, with the exception that the stays, *f f*, are introduced on both sides of the vertical plate, *a*.

The lattice framing, or web, is composed of angle and flat bars, placed at an angle of 45 degrees, which are connected to the top and bottom vertical plates by a system of rivetting. The angle bars vary in thickness, from 5 inches by 3 inches by $\frac{3}{4}$ inch, to 5 inches by 3 inches by $\frac{3}{8}$ inch, and the flat bars, from 5 inches by $\frac{5}{8}$ inch, to 5 inches by $\frac{1}{2}$ inch, the angle bars are used as struts, and the flat bars as ties, excepting towards the centre, where the compressive strains being light, a few are introduced as struts. The web is stiffened over the piers, and abutment piers, by the introduction of wrought-iron plates $\frac{5}{8}$ inch thick between the lattice bars, and which are rivetted to them, and to vertical plates, *a*; in the top and bottom flanges over the piers, these plates are 6 feet wide, and over the abutment piers they are 4 feet 11 inches. Packing pieces 8 inches by 8 inches by $\frac{5}{8}$ inch, are introduced between the crossings of the lattice bars over the piers and abutments, for the purpose of affording additional rivets at those points, to meet the increased strain.

At intervals of 5 bays, or about 16 feet $6\frac{13}{16}$ inches, diagonal cross-bracings, are placed between the main girders; each brace is formed of two angle-irons 3 inches by 5 inches by $\frac{1}{2}$ inch, rivetted together, thereby making a T-section, the diagonal brace thus formed, cross each other with the head of the T of one brace, towards the head of the T of the other brace; their extremities are attached to the cross beam, which carry the railway, and to cross girders connected with the lower flanges; which latter, are only introduced where the diagonal bracing comes in. At the intersection of each pair of diagonal braces, a flat bar of wrought-iron 6 inches by $\frac{1}{2}$ inch, is placed on edge transversely across the bridge, and is passed between the diagonal, to which it is rivetted, the ends of this bar are twisted from the vertical at an angle of 45 degrees, in order to take the back of the web of the angle-iron lattice bars, to which they are rivetted.

The whole superstructure is mounted on cast-iron rollers, running in bed-plates, fixed on the tops of the piers, similar in detail to those already illustrated and described for the Lerida Bridge.

To erect this bridge on scaffolding at so great a height, and in a water-way, subject to such heavy floods was considered too hazardous an experiment; M. de Bergue, therefore, resolved to proceed in a somewhat similar manner to that adopted for the Lerida Bridge, on the same line of railway; but as the centre span here was 213 feet 3 inches, and the end span only 65 feet $7\frac{1}{2}$ inches each, it was obvious that to run the superstructure on rollers over the piers, as done in that case, it would have toppled over into the river, from the fact that the forward end would have overbalanced the portion resting on the pier, and abutment pier, when it arrived at about three-quarters of the middle span. To meet this difficulty, the superstructure was framed, and rivetted up in two equal portions, one on each side of the river, in the line of the bridge: and then each half was moved forwards on capstan rollers, until the first pier in the water-way was reached, it was then necessary to load the back end of each half, or the portion resting on the embankments, with a load equal to a little more than the difference between the weights of half the middle and the end span; the load being made up of rails run through the lattice bars, and resting on the lower flanges; they were now again moved forwards, until the leading ends joined at the centre of the middle span, when the cover plates were put on, and the rivetting made good; the whole superstructure was then eased off the capstain rollers, which were replaced by the permanent ones. The bridge was then ready for the planking and rails.

Practical men will readily understand the difficulties to be encountered and provided for in moving heavy masses, more particularly at such a height, without the advantages of floor space at the same level, yet these operations were performed in a comparatively short space of time, and to the entire satisfaction of all concerned.

MURILLO BRIDGE.

PLATE LXVII.

This structure was designed and erected by Mr. John H. Porter, of Birmingham, under restrictions imposed by the Spanish Government, to carry an ordinary road, it is supported by three girders of the ordinary lattice form. The clear span of the bridge is 118 feet $1\frac{1}{2}$ inches, and the total length of each main girder 131 feet 3 inches. The main girders are divided into twelve bays by standards or uprights placed at a distance of 9 feet $10\frac{1}{8}$ inches from each other. The lattice bars are varied in size in the different bays, according to the strain to which they are liable, the ties being made of flat bars, and the struts of angle-iron, both being rivetted together at the intersections. The sectional areas of the top and bottom flanges are greatest at the centre, being reduced towards the abutments.

The platform is carried by cross girders 13 inches in depth, made of plate and angle iron, upon which longitudinal timber bearers are laid, and to these bearers the planking which constitutes the roadway is fixed. The longitudinal timber bearers are eleven in number, the width of the platform being 23 feet.

As stated above, three main girders are used to carry the roadway, being placed at a distance of 9 feet $10\frac{1}{8}$ inches from each other. The top flanges are in the form of triangular tubes, in order to enable them to resist more effectually the compressive strain to which they are subject. The depth of the main girders, from centre to centre of rivet-holes, is 9 feet $10\frac{1}{8}$ inches, or one-twelfth of the clear span of the bridge. The bracing employed to impart to the structure the requisite amount of rigidity is exceedingly strong, vertical bracing being used at the standards which divide the main girders into bays, and horizontal bracing under the platform; the whole being disposed as shown in Plate LXVII. The main girders are also further stiffened in the top flanges by cast-iron stiffeners placed within the tubes, at the junctions of the several lengths. The extremities of the main girders are each placed upon eight cast-iron rollers, each 3 inches in diameter (1 foot 4 inches in length), and placed $7\frac{1}{2}$ inches apart, from centre to centre; they are surrounded and retained in position by wrought-iron frames firmly bolted together.

The rivets are placed at distances of 4 inches apart, from centre to centre, in the cross girders, and 4½ inches apart in the main girders.

The test-load prescribed by the Spanish Government, was 71·7 lbs. per square foot of platform, which is, according to the usual practice in England, exceedingly moderate as a maximum test-load, for with the roadway fully loaded we should have a pressure of about 120 lbs. per square foot for the maximum moving load.

The weight of metal in the whole structure is as follows:—

	Tons.	cwt.	qrs.	lbs.
WROUGHT-IRON.				
Three main lattice girders	60	12	0	0
Transverse girders, bracing and longitudinal ties	28	0	0	0
Total weight of wrought-iron	88	12	0	0
CAST-IRON.				
Cast-iron stiffeners, bed-plates, etc.	4	14	1	14

The gross sectional area of the top flange of the centre of the girder, is 31 square inches, or for the three flanges 73·125 square inches; the nett sectional area of each bottom flange at the centre, is 28 square inches, and for the three flanges 67·5 square inches. Taking the depth at one-eleventh of the span and calculating the strain for 120 lbs. per square foot of platform, the strains will be as follows:—compressive strain upon the top flange 3·00 tons per square inch of sectional area, and tensile strain upon the bottom flange 3·00 tons per square inch of sectional area, both being taken at the centre of the span.

CARLOS GOMES BRIDGE.

PLATES LXVIII. AND LXIX.

This bridge was designed by Captain A. M. de Oliviera Bulhoes, to carry the Unias e Industria Road, over the river at Carlos Gomes, near Rio de Janeiro; and constructed by Messrs. E. T. Bellhouse, & Co., Manchester. This structure is supported by tubular girders, with lattice webs; it is in two spans, each of 113 feet 4½ inches in the clear, advantage being taken of a natural rock in the middle of the stream, for the erection of a central pier of worked masonry. The width of the road within the main girders, is about 22 feet 6 inches, and the roadway is supported upon planking of native timber, resting upon sixty-two wrought-iron lattice web cross girders, each 2 feet 7½ inches deep; these cross girders are placed 3 feet 11¼ inches apart, from centre to centre; each of the main girders for the two spans forms a continuous double webbed lattice girder, in length 241 feet 5½ inches, from end to end; and 9 feet 2¼ inches deep. The whole of this length for each girder, has been prepared in pieces in this country; and rivetted together on the site on which it is erected.

The section of the main girders is the same throughout, the top and bottom members being exactly alike; the latter consists of two plates 2 feet 3½ inches wide by ⅝ inch thick, each; at right angles to these plates are fixed two others 1 foot 7⅝ inches apart, 10 inches deep, and $\frac{5}{16}$ inch thick, to which the lattice bars are rivetted.

The upright angle-irons on each side of these, connecting them with the bottom member, are 4 inches by 2¾ inches by $\frac{5}{16}$ inch thick. The whole of the rivets for these angles are ⅝ inch diameter.

The top and bottom members are connected together by upright pieces of T-iron, outside the upright plates, 3 feet 11¼ inches apart, between centres. The back of this T-iron is 5 inches by ⅝ inch, and the depth over the web is 2½ inches, and thickness ⅜ inch.

Opposite to each of the cross girders, there are internal stiffening plates $\frac{5}{16}$ inch thick, having the side edges flanged at right angles, and rivetted through the lattice bars, and T-iron uprights, both on the outside and inside of the main girders. These plates are 4 feet $3\frac{1}{2}$ inches in depth, and 1 foot 7 inches across, when flanged.

The lattice bars are placed at an angle of 34 degrees with the horizon. The bars are each 4 inches by $\frac{1}{2}$ inch, and are rivetted together at the places where they cross each other by rivets of 1 inch diameter.

The cross girders for supporting roadway are on the lattice principle. Each cross girder is 22 feet 6 inches long, 2 feet $7\frac{1}{2}$ inches deep. In this case, as in that of the main girders, the section is the same throughout the length, and the top and bottom tables are alike. The plates for top and bottom tables are each $9\frac{1}{2}$ inches by $\frac{3}{8}$ inch, and the central plates at right angles to these are each $6\frac{1}{2}$ inches by $\frac{3}{8}$ inch thick, and the angle-irons are $2\frac{3}{8}$ inches by $\frac{3}{8}$ inch thick, connected by $\frac{5}{8}$ inch rivets. The lattice bars are at an angle of 45 degrees, fastened to the central plates, and at their intersections by $\frac{3}{4}$ inch rivets, the bars being 3 inches by $\frac{5}{16}$ inch thick. Each end of the cross girder is fastened to the T-iron upright of main girder by two plates $\frac{5}{16}$ inch thick, on each side of the central plate, the dimensions being 2 feet $2\frac{1}{4}$ inches deep by 1 foot $3\frac{1}{2}$ inches wide.

Upon the central pier of masonry are bedded two cast-iron foundation plates, each 4 feet long by 3 feet wide. The upper surface of each is planed, and upon this rests the planed surface of a cast-iron plate 2 feet $6\frac{1}{2}$ inches wide, rivetted to the underside of the main girder. There are six $1\frac{1}{4}$ inch holding-down bolts for each bed-plate.

Upon the masonry, at the ends of the main girders, are four sets of cast-iron bed-plates on stonework, and upper plates on the girders planed on the surfaces, and having ten turned wrought-iron rolllers in a wrought-iron frame to each set. These rollers are 2 feet $7\frac{1}{2}$ inches long on the face, and $3\frac{1}{4}$ inches diameter; the end bearings on each are 2 inches diameter and $1\frac{1}{2}$ inch long.

By proof with 10 tons of actual dead weight, placed over 10 feet of the length in the centre of one of the cross girders, the bearings being 20 feet 6 inches apart, the deflection was found to be $\frac{3}{8}$ inch bare.

The weight of the bridge is as follows:—

	Tons.	cwt.	qrs.	lbs.
No. 1. Main girder complete, 241 feet $5\frac{1}{2}$ inches extreme length, and 9 feet $2\frac{1}{4}$ in. deep	49	6	2	0
No. 2. Ditto ditto ditto	49	6	2	0
62 cross girders for roadway, each 22 feet 6 inches long, 2 feet $7\frac{1}{2}$ inches deep . .	47	13	0	0
Extra rivets for the above	1	10	0	0
Wrought-iron in rollers and frames, holding-down bolts, &c., for bed-plates	1	18	1	0
Total wrought-iron.	149	14	1	0
Cast-iron in planed bed-plates, and ornamental name-plates.	4	18	0	0
Lead	0	2	2	20
Tons.	154	14	3	20

CHAPTER XIX.

WROUGHT-IRON BOWSTRING BRIDGES.

In the present Chapter we shall treat of the Windsor, Shannon, and Saltash Bridges. The first, Windsor Bridge, is very remarkable for accuracy and neatness of design, in which points it is excelled by none which have hitherto been brought under our notice. The circumstances of traffic, etc., required a very strong bridge, and we have here a structure possessing that quality; but presenting the appearance of lightness. It is very evident that the great beauty of this bridge, is due to the talent and judgment with which the details have been determined, for it is very certain, that without a perfect knowledge of the subject in hand, it would be quite impossible to plan a work so perfect in every particular.

Shannon Bridge has evidently been designed with a view to obtain the greatest economy, and other points appear in a great measure to have been sacrificed to this one, and certainly with success, as the structure has been completed at a very low cost.

The Saltash Bridge, our last but most magnificent example, is undoubtedly one of the most stupendous undertakings which has ever been attempted, and it has, moreover, been executed with perfect success. This structure, like the Windsor Bridge, bears on every feature, the characteristic appearance of having been designed with great care and talent, each detail being peculiarly suitable to the part it is intended to play, and, at the same time, presenting an appearance in perfect keeping with the remaining parts of the work, whether taken singly, or as a whole.

WINDSOR BRIDGE.

PLATES LXX., LXXI., AND LXXII.

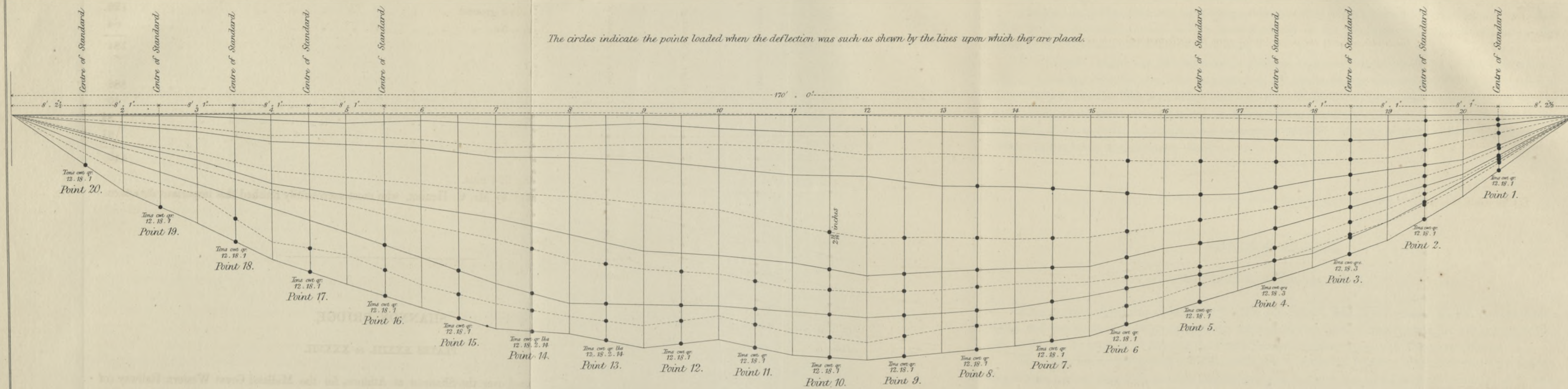
The structure, which was designed by the late Mr. Brunel, belongs to a class of bridges known as bowstring, and may be considered a very excellent example of that form of construction. It was erected in the year 1849, for the purpose of carrying the Great Western Railway across the Thames at Windsor.

The bridge has a clear span of 187 feet, and a rise or versine of 25 feet; it is constructed for a double line of rails, and consists of three main longitudinal girders, which are placed at central distances of 17 feet 6 inches apart; these girders are supported at each extremity on two cast-iron cylinders 6 feet in diameter, which were sunk to the required depth rapidly and easily without pumping, by excavating inside with the bag and spoon, in the manner used by the Thames bargemen in getting gravel from the bed of the river. The centre girder is of a stronger construction than the others, and consists of the following parts; an arched girder 3 feet 6 inches, which is of a triangular shape in section, and composed of a double thickness of $\frac{1}{2}$-inch plates; the joints are covered by wrought-iron covering plates 6 inches in width (See Plates LXXI. and LXXII.), and a tie girder 5 feet $10\frac{1}{2}$ inches in height, which consists of a web and top and bottom flanges; the top flange is 2 feet 6 inches in width, and is formed of two $\frac{3}{4}$-inch plates, the bottom flange is of the same width and similarly constructed; the web which is united to the flange by 3-inch angle-irons is made of four $\frac{1}{2}$-inch plates, which are placed two and two, a distance of 3 inches being kept between them, and is stiffened by angle-iron uprights placed at distances of 5 feet apart. The arched and tie girders are connected by means of wrought-iron standards or uprights, which, for the inner girder are made of plate and angle-iron, and for the outer girders of rolled H-iron: these standards are firmly tied together, by means of wrought-iron bars, varying in width from 6, at the end of the girders, to 4 inches at the centre. The main girders are braced together at the top by a strong bracing composed of T-irons, by which means, considerable rigidity is imparted to the structure. The cross girders which rest upon the lower flanges of the main girders, are placed on the angle of skew, and occur at distance of 5 feet apart; they are composed of a $\frac{1}{4}$-inch plate, 12 inches in depth, and of four $3\frac{1}{2}$ inch angle-irons two for the top and two for the bottom flange, which are rivetted to the web by $\frac{3}{4}$ inch rivets, 4 inches apart. The platform is formed of 4-inch planking which is bolted down to the cross girders. The ends of the longitudinal girders are bedded on American oak, and, at one extremity of the bridge, provision is made for the expansion and contraction of the metal, by allowing this bedding to move freely on rollers placed on the tops of the cylinders. These rollers which are $4\frac{5}{8}$ inches in diameter are fitted in a wrought-iron roller frame, which is of the

DIAGRAM SHOWING DEFLECTIONS DURING PROOF OF FIRST RIB OF SHANNON BRIDGE.

G. W. HEMANS, ESQ^RE ENGINEER.

The circles indicate the points loaded when the deflection was such as shewn by the lines upon which they are placed.



		Tons . cwt . qr.
Load on	1st Standard	12 . 18 . 1
" "	2nd "	12 . 18 . 1
" "	3rd "	12 . 18 . 1
" "	4th "	12 . 18 . 1
" "	5th "	12 . 18 . 1
" "	6th "	12 . 18 . 1
" "	7th "	12 . 18 . 2 . 14
" "	8th "	12 . 18 . 2 . 14
" "	9th "	12 . 18 . 1
" "	10th "	12 . 18 . 1
" "	11th "	12 . 18 . 1
" "	12th "	12 . 18 . 1
" "	13th "	12 . 18 . 1
" "	14th "	12 . 18 . 1
" "	15th "	12 . 18 . 1
" "	16th "	12 . 18 . 1
" "	17th "	12 . 18 . 3
" "	18th "	12 . 18 . 3
" "	19th "	12 . 18 . 1
" "	20th "	12 . 18 . 1
Total load on all the points		258 . 6 . 3

The method pursued in proving this rib was as follows: ——— Points 1 & 2 were first loaded with a weight of 12 Tons, 18 cwt, 1 qr. each; and the amount of deflection caused by these loads is shewn by the upper dotted line passing through the two circles. —— Points 3 & 4 were then loaded in addition to 1 & 2, and the deflection produced by these four loads is shown by the line passing through the four circles. —— Points 5 & 6 were then loaded (making 6 loads) and the deflection again taken as shown by the second dotted line —— two more loads were placed on at each time, in addition to those already on, until the whole 20 Points were loaded.

Total deflection when loaded with 258 tons . 6 cwt . 3 qrs (distributed) = $2\frac{15}{16}$ inches.

set load removed = $\frac{7}{16}$ inch.

The section of iron in top of rib is $64\frac{1}{2}$ square inches, and in the tie bar 50 square inches. In the latter case, the available section is $39\frac{3}{4}$ square inches, after deducting full allowance for the rivet holes. — With 258 tons as the proof load the crushing strain upon the rib will be less than 8 tons per square inch; and the tensile strain on the lower part, or tie, will be less than 10 tons per square inch.

Horizontal Scale of Feet.

10 5 0 10 20 30 40 50 60 70 Feet.

Vertical Scale. Full Size.

construction shown in Figs. 1, 10 and 11, Plate LXXI. A cast-iron parapet of bold appearance is placed at each end of the bridge, which adds much to the effect of the design.

The weight of cast-iron in this work was,

	Tons.
For cylinders above and below ground	120
Parapet mouldings, castings, &c.	64
	184

And of wrought-iron:—

In main trusses	386
Roadway girders, &c.	40
Expansion rollers and plates	$8\frac{1}{2}$
Bolts, nuts, washers, &c.	$5\frac{1}{2}$
	440

The total weight being 624 tons.

The contract was taken by Mr. G. Hennet, who most successfully finished this important structure.

SHANNON BRIDGE.

PLATES XXXIII. TO XXXVII.

This bridge was erected over the Shannon at Athlone, for the Midland Great Western Railway (of Ireland) Company, in the year 1850, from the designs of Mr. G. W. Hemans, engineer; Messrs. Fox, Henderson & Co., being the contractors.

The bridge consists of two handsome stone abutments, built of the grey-blue limestone of the district, and of six openings spanned by girders; one of 50 feet span, for the high road on the western shore, one of 40 feet span, for the high road on the eastern shore, two fixed spans of 165 feet each, and two opening spans of 43 feet each, turning on a centre swivel for navigation purposes.

These girders are all of wrought-iron, and were built up, and rivetted together "*in situ,*" on a temporary scaffolding provided for the purpose; the width between the girders is 28 feet, and the bridge carries a double line of rails. The span for the 165 feet openings are each composed of two wrought-iron bow-string girders, with an extreme length of 178 feet 4 inches, and a rise at the centre of 20 feet 2 inches; these girders consist of a horizontal or straight girder 2 feet $7\frac{1}{2}$ inches high, and of an I shaped arched girder 2 feet wide, and 2 feet 2 inches deep, which are connected together by means of struts and ties; each main girder, exclusive of the flooring, may be considered as weighing 90 tons. The rails rest on longitudinal timbers, 12 inches by 6 inches, supported on wrought-iron cross girders, 1 foot $7\frac{1}{2}$ inches deep, and placed 4 feet apart; the roadway being composed of galvanised wrought-iron corrugated floor plates. Previous to removing the first rib from Messrs. Fox, Henderson's works at Smethwick, it was put together, and subjected to many severe tests, with a distributed load of 258 tons 6 cwt. 3 qrs., the greatest deflection amounted to $2\frac{15}{16}$ inches, the permanent set being $\frac{7}{16}$ of an inch; for a description of the method adopted in proving this rib, we beg to refer our readers to the accompanying diagram; which also shows the actual deflection of the girders when subjected to various distributed loads.

The piers in the river are all composed of cast-iron cylinders, constructed in segments $1\frac{1}{2}$ inches thick, which are bolted together by means of strong wrought-iron bolts. The lower edge of the cylinders are knife-shaped, in order to facilitate their descent; they are all 10 feet in diameter, their lengths varying from 25 to 35 feet. It was originally intended to sink them by means of Dr. Potts' pneumatic, or exhausting process, as the bed of the river had been examined with great care, and found to consist, in several trial holes of a very homogeneous bluish clay, which was considered very suitable for Dr. Potts' process. It is, however, only one of the many instances of a similar nature, in which the trial holes have proved very deceptive; as large boulders were found, where the cylinder had to be sunk, which proved very troublesome, although none appeared in the trial pits. Dr. Potts' principle was applied by inserting a second short cylinder of boiler plate, in the upper part of the main cylinder; as soon as the latter had been erected, and slightly sunk in the bottom by its own weight. This machine was technically called the "Doctor," its upper flanges were firmly and hermetically cemented (temporarily), to the top of the main cylinder. The flexible hose of a powerful air-pump was connected to its cover, and in the bottom was a valve communicating with the cylinder. As soon as the air-pumps had created a vacuum in the "Doctor," the bottom valve was opened, and allowed the air beneath the cylinder to enter; by this means a partial vacuum was created in the whole cylinder, and the surplus pressure of the atmosphere above, caused the latter to descend. A barometer was attached to indicate the amount of vacuum, which did not on any occasion exceed a half vacuum.

Sometimes a very sudden descent of several feet would occur to the cylinder on the opening of the valve, but generally the motion was extremely small, and the air simply leaked in. Moreover, it was soon ascertained that large boulders lay under the edges of the cylinders in some places, and it was found that the atmospheric pressure could not displace them. The expense of working the air-pumps was very considerable, as many as 25 men being required, with long handles to the pumps, to produce the vacuum. In consequence of these difficulties, it was resolved to abandon the pneumatic principle as being too expensive.

Recourse was then had to the ordinary system of pumping the water from the inside of the cylinders, excavating the clay and stones, and sinking the cylinders by weights. This plan, though tedious, was perfectly successful, and not very costly, the water not exceeding a depth of from 8 to 10 feet on an average; the leakage under the edges was not considerable. The weight laid on the tops of the cylinders, to cause them to sink, consisted of railway rails, and the amount of pressure was easily regulated. When no movement arose from a weight double of that which could ever be brought upon the piers, the operation of sinking ceased, due regard having been had to the probability of scour in the river bed. The interiors were then filled with good rubble masonry set in hydraulic lime. Some of the cylinders were turned a little off the perpendicular by the action of boulders, but though visible to the eye, where not sufficiently so to justify the expense of drawing and resinking. The top sections were cast specially to meet any irregularities in level or verticality, and when bolted on, the whole of the piers were perfectly flush and ready for the cornice caps and girders. Had the water been deeper, or the foundation less sound, it would probably have been cheaper to use the diving-bell or compressed air-apparatus as at Rochester Bridge, instead of either of the modes adopted for sinking the cylinders.

The general effect of the bridge is very pleasing; and the cost is considered very moderate.

The following is the abstract of weight and cost of the above bridge:—

	Tons	cwt.	qrs.	lbs.	£	s.	d.
Cast-iron in cylinders, girders, and bearing plates	506	17	3	4			
Ditto ditto in shoes and packing for ribs	6	17	0	27			
Ditto ditto in pilaster caps and girders, to carry load	48	15	2	84			
Ditto ditto in caps and mouldings to piers, including the casing for centre of swing bridge	87	13	0	6			
Ditto in other ornaments	3	5	0	20			
Ditto in girders of swing bridge	46	1	1	2			
Ditto in roller, and rollers for ditto	27	8	2	0			
Dito in gearing, and machinery for ditto	12	1	1	11			
Ditto in covering plates for ditto	15	10	0	5			
	754	10	0	25			
At £10 per ton					7545	2	0

	Tons cwt. qrs. lbs.	£ s. d.
Gun-metal in bearings, &c., for swing bridge, 175 lbs. at 1s. 8d.		14 11 8
Wrought-iron in bolts and nuts, for cylinders	20 11 1 22	
Ditto ditto ditto ditto for swing bridge	7 5 3 16	
Ditto in fittings for swing bridge	10 7 2 16	
	38 4 3 26	
@ £26 per ton		994 9 8
Ditto in bolts and nuts for wrought-iron girders and joints	0 8 3 1	
Ditto in plates for girders	34 17 3 4	
Ditto ditto joists	42 19 3 15	
Ditto ditto bowstring ribs	189 9 3 14	
Ditto in iron for girders	15 14 2 2	
Ditto ditto joists	95 6 0 25	
Ditto ditto bowstring ribs	88 11 3 3	
Ditto ditto in rivets for girder joists and bowstring ribs	19 15 0 11	
	486 13 3 9	
@ £23 per ton		11,193 18 0
Galvanised wrought-iron corrugated in floor plates...	34 5 2 8	
@ 38s. per ton		1302 14 0
Ditto for rivets, washers, &c.	2 0 1 4	
@ 42s. per ton		84 12 0
Additional expense of patterns, labour, &c.	90 18 0 26	
@ £5 10s. per ton		500 0 3
Ditto ditto for castings, &c., for swing bridge	154 9 0 7	
@ £8 10s. per ton		1312 17 0
Expense of two lamp posts, with lamps complete, fixed on swing bridge		37 10 0
Additional expense incurred in sinking cylinders on account of the soil or ground being different from what was anticipated	3367¼ days at 2s. 9d.	463 0 0
Expense of testing cylinders and 50 feet girders to order. Men's time ...	3148 at 2s. 3d.	354 3 8
Men's time, labour, &c., pumping water out of cylinders...	1942¾ days at 2s. 6d.	242 16 11
Use of pumps, 5 of 9 inch diameter, and 1 of 7 inch-diameter	39 days at 3s. 3d.	289 5 0
Expense of temporary footway		2 17 10
Sundry expenses incurred in erecting and testing the first rib at Birmingham		139 2 9
Expense of laying down timber for rails and corrugated iron—carpenters' time—492¼ days ...		135 7 5
		£24,632 8 2

Some extra claims made the whole amouut to £25,000.

ROYAL ALBERT BRIDGE.

PLATES LXXVIII. TO LXXX.

This stupendous structure, remarkable for the boldness of its conception, and for the ingenuity of its construction, was designed to carry the Devon and Cornwall Railway across the Tamar, at Saltash, near Plymouth, by the late lamented Mr. Brunel, and it has been successfully executed, under the able superintendence of Mr. R. P. Brereton. This work, unprecedented in magnitude, will stand forth to future generations, a lasting and worthy monument to the skill of the justly celebrated engineer by whom it was planned.

The whole structure consists of nineteen openings, two of which have each a clear span of 445 feet; the remaining seventeen are each 69 feet 6 inches in length. The two larger spans constitute the remarkable feature of the bridge, and these we illustrate in the Frontispiece, and in the Plates mentioned above. The River Tamar is crossed by the two large spans, the smaller ones bringing the railway from the hills, on either side of the valley, down to the banks of the river; the total length of the entire structure being 2,240 feet. The smaller spans are carried by wrought-iron plate girders, resting on massive double columns of solid masonry, 11 feet square. The plate girders are 8 feet deep, with flanges curved into the form of half a circular tube of plate $\frac{1}{2}$ an inch thick, joined by $\frac{3}{4}$ inch rivets; the diameter of these semi-tubes, or the horizontal width of the top flange is 2 feet; the web to which this flange is united by $3\frac{1}{2}$ inch angle-irons, $\frac{3}{8}$ inch thick, with $\frac{7}{8}$ inch rivets, consists of $\frac{3}{4}$ inch plate; at the bottom of which is fixed also by $3\frac{1}{2}$ inch angle-irons a bottom flange, slightly curved, of $\frac{1}{2}$ inch plate, 3 feet in width. The platform is carried by cross girders, made of plate and angle-iron, 16 feet 10 inches in length, and 13 inches in depth; it consists of 4 inch timber, firmly bolted down to the cross girders.

The main stone piers are at the water's edge, and support the ends of the great spans crossing the river. These are of course of the most solid construction, and more resemble the massive columns of Egypt than the works of modern engineers. Each is of granite 29 feet wide by 17 feet thick, and 190 feet in height from the foundation to the summit. The strength required in each of these piers was far surpassed by the resistance which that in the centre of the river must offer, and for this a column was required of such proportions, that nothing short of the solid rock would suffice for its foundation. But to reach the rock was a matter of no ordinary difficulty, inasmuch as it lay beneath 20 feet of mud and concrete gravel, over which flowed 70 feet of salt-water. To erect a stone pier in the ordinary manner would be here entirely out of the question, but by an ingenious contrivance the granite column requisite to sustain the enormou load to which it is subject was reared. An immense wrought-iron cylinder, 100 feet high and 37 feet in diameter, was sunk upon the site of the intended pier, and proper means being taken to exclude the water from the interior of this cylinder, the above-mentioned column was raised within it. Upon this column four octagonal cast-iron pillars, each 10 feet wide and 88 feet 9 inches high, were erected 10 feet apart, and strongly braced together, forming a square of about 30 feet. The weight of each column is 150 tons, each being in pieces 6 feet long, 2 inches thick, and strengthened inside by stout ribs and brackets. As fast as these pieces were cast, planed down, and accurately fitted, they were sent piecemeal to the centre pier ready for erection. Upon these columns the top framing of the pier is fixed. We abstain from describing in detail the means by which this pier was constructed, as we believe a paper on this subject will shortly be submitted to the Institution of Civil Engineers.

Upon this centre pier, and the two side piers, rest the massive ribs by which the great spans are sustained. They consist each of an arched tube and a suspension chain strongly braced together, to which the small side girders are attached. The arched tubes are in section of an elliptical form, the major axis of the ellipse being placed in a horizontal position; they are made of stout wrought-iron plates strongly rivetted together, and rendered more rigid by stiffeners and diaphragms. The width of each tube is 16 feet 9 inches, and its depth is 12 feet 3 inches, the diaphragms being placed about 20 feet apart.

The ends of the tubes rest upon, and are rivetted to, strong frameworks of wrought-iron plate, and at the centre pier the adjacent ends of the two tubes abut against, and are rivetted to, each other, being surrounded by a hoop, rivetted on after the tubes were in position. The supporting frames are each about 8 feet in length and 23 feet wide, and are surrounded by the wrought-iron shells forming the exteriors of the upper parts of the main piers. We now come to the main chains which form the tension members of the large ribs. These chains each consist alternately of 14 bars 7 inches by 1 inch thick, and 15 bars 7 inches wide by $\frac{15}{16}$ inch thick, except at the extremities, which consist of 12 bars $10\frac{1}{4}$ inches deep by $\frac{13}{16}$ inch thick. The bars are similar to those of an ordinary suspension chain; they are 20 feet long, and were each rolled in one length, by Messrs. Howard, Ravenhill, & Co., of the King and Queen Iron Works, Rotherhithe. There are two chains on each side of the roadway, and their extremities are firmly fixed to the main tubes by 7-inch bolts, in the manner shown at Fig. 1, Plate LXXX. Besides the main standards, or uprights, which we shall shortly describe, there are also intermediate suspenders of flat bars, 7 inches wide by $\frac{7}{8}$ inch thick, from the main chains to the roadway girders.

We now describe the standards, or uprights, of which various views are shown in Plates LXXIX. and LXXX. These are in section of the form of a cross, being formed of two plates and eight angle-irons, the dimensions of which in the centre standard are as follows:—the narrower plate which lies in the direction of the main girders is in two parts; together, 1 foot 6 inches wide by $\frac{3}{8}$ inch thick, and the other plate, which is at right angles to this, is 2 feet 6 inches wide by $\frac{1}{4}$ inch thick; these plates are united at their intersection by means of four $2\frac{1}{2}$ inch angle-irons $\frac{3}{8}$ inch thick; there are also two $2\frac{1}{4}$ inch angle-irons $\frac{1}{4}$ inch thick at each edge of the $\frac{1}{4}$ inch plate. The standard is thus fastened to the main tube: the $\frac{3}{8}$ inch plate is carried straight up to the main tube in the direction of a tangent to the same, to which it is firmly bolted; one of the plates of the tube being formed to lap over it. The $\frac{1}{4}$ inch plate on the outer edge tapers off to meet the tube at the point of junction, and to the inner edge, an additional plate is attached by means of cover plates, 5 inches wide by $\frac{3}{16}$ inch thick, the plate being $\frac{1}{4}$ inch thick, and of the form shown in Fig. 1, Plate LXXX. Through the standards, at varying distances from their junctions with the roadway girder, pass the main chains which are retained in position vertically by wedges, of which the ends are cut off flush after they are driven up tight. On each side of the main chains, there is an extra plate 13 inches wide by $\frac{3}{8}$ inch thick, which is rivetted on after the roadway girder is attached to the main chains. The roadway girders are attached to the uprights by bolts and rivets as shown in Plates LXXIX. and LXXX. The roadway girders are themselves in general form similar to those already described for carrying the railway across the shorter spans which communicate with the large spans and with the sides of the valley.

The main tubes are braced to the standards at the points where the latter are intersected by the main chains, by strong ties 7 inches wide, and of varying thickness, $1\frac{1}{4}$ inch at the centre, and 1 inch at each end of the span. Each brace consists of four ties, which meet between two $\frac{3}{4}$ inch plates of the form shown at Fig. 8, Plate LXXVIII; and to these plates the ties are firmly keyed, as shown; the plates being kept close up to the ties by a $1\frac{1}{2}$ inch bolt. The lower extremity of each lower tie is bolted to lugs on the main standards; and the upper extremity of each upper tie is bolted to the side of the main tube, being covered by the same plate which overlaps the top of the standard, as shown at Fig. 7, Plate LXXIX. The standards are also braced together by transverse bracing, as shown at Fig. 5, Plate LXXVIII; the bars composing it are formed of T-iron.

The rib complete, presents the appearance of a double bow, and it may be regarded as such, the tensile action of the chain upon the bed-plates being counteracted by the thrust exerted upon the same by the arched tube. The depth of the rib from the centre of the main tube to that of the main chain is 56 feet 3 inches, or about $\frac{1}{8}$ of the clear span. The alteration of length of the rib, by contraction and expansion under variations of temperature, is provided for to the amount of six inches, although the greatest difference yet observed amounts only to three inches in the entire length of both spans, by placing the frames which carry those ends of the main tubes which are supported by the side piers, upon 48 wrought-iron rollers, each 3 feet 3 inches long, and $3\frac{1}{2}$ inches diameter, in a double cast-iron frame or bed-plate.

The total quantity of wrought-iron used in this structure is 2700 tons; of cast, 1300 tons; masonry and brickwork 17000 cubic yards, and about 14000 cubic feet of timber.

Each of the main ribs was constructed entire, adjacent to the site of the intended structure, and after being tested was floated out on pontoons, and raised by hydraulic presses of immense power. The foundations intended to support the bridge were used to sustain these presses. As the spans were raised by the means described, the iron columns were built up under them. The pressure upon the foundation of the centre pier will amount to more than 8 tons per square foot of bearing area, or double the pressure upon the foundations of the Victoria Tower.

In the Frontispiece is shown a view of this bridge, with one span in position, and the other ready to be raised; it will be observed that the extremities of the roadway girders, are raised above the central part of the same; and thus it was raised, these parts being subsequently lowered to the general level.

The following extract from Colonel Yolland's Report, dated 25th April, 1859, shows the results of the tests to which the ribs were subjected :—

"When the tube had to carry itself, and its portion of the bridge being, with the floor, about 1100 tons, the deflection observed at the centre was under $2\frac{1}{4}$ inches on the east side, and $2\frac{1}{2}$ inches on the west side, with a load of $1\frac{1}{4}$ tons per foot lineal uniformly laid on the roadway; the deflections were respectively 5·1 inch east, and 5·25 inch west, with the load increased to $2\frac{3}{4}$ tons per foot lineal, 1200 tons; the deflections were $7\frac{1}{2}$ inches east side, and $7\frac{3}{4}$ inches west side; but it was noted, that during the last two experiments, the supports on which the tube rested had sunk about $\frac{1}{2}$ an inch, which quantity must therefore be deducted from the preceding deflections observed with the load on. After the load of $2\frac{3}{4}$ tons per foot had been taken off, a permanent set of 1·2 inch on east side, and 1·25 inch on the west side was observed, beyond that resulting from its own weight, and caused by the heavy weights that had been applied on the roadway.

"On 20th instant, the day being exceedingly favourable, with a spirit-level placed under the roadway and resting on the iron stays at the central pier, I observed the deflections at these large openings produced by a load made up of two heavy engines near the centre, and a number of trucks having their springs packed up, loaded with ballast, iron, &c., making up an aggregate weight of 384 tons, or as large a weight as can possibly be brought on the viaduct, unless the train is made up of a large number of locomotive engines. The staffs observed were placed at the centres of each opening, and midway between the rails, and thus the deflections observed were the sum of all the deflections, including that of the cross girders carrying the roadway.

"With the above load, the east tube, &c., (not previously tested) deflected 1·14 inch. With the above load the west tube (test already referred to) 1·20 inch; and when the weight was taken off, there was not the least indication of a permanent set. I then tried to register the deflection caused by passing the whole of this load over the bridge at a speed estimated at 30 miles an hour; but the spirit-level did not remain sufficiently steady to allow of the staff being read, but vibrated, as might be expected, as the train passed. I regard these results as highly satisfactory, and so far as my knowledge goes, I believe them to be greatly superior to anything of the kind that has been attained elsewhere, and accomplished at less expenditure of money and materials.

"With a load of 1 ton per foot run, and not deducting for any portion of the ends of the tubes immediately resting on the piers, I estimate the amount of the strain on the trussed tubes at about 4·2 tons per square inch of section, and allowing 170 tons as the amount of this superincumbent weight not in any way trying the tube, the strain would be reduced to about 3·8 tons per square inch.

"The strain per square inch of section for the girders over the land openings is also in all cases less than 4 tons."

This structure was opened for general traffic, by His Royal Highness the Prince Consort, in the beginning of May, 1859.

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THE END.

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London: Printed by George Watson, 5, Kirby Street, Hatton Garden.