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MINUTES OF PROCEEDINGS

OF

THE INSTITUTION

OF

CIVIL ENGINEERS;

WITH

ABSTRACTS OF THE DISCUSSIONS.

Vol. XXXI.

SESSION 1870-71.—PART I.

EDITED BY

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Index, Page 417.

LONDON:
Published by the Institution,
25, GREAT GEORGE STREET, WESTMINSTER, S.W.
1871.

620

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LONDON: PRINTED BY WILLIAM CLOWES AND SONS, STAMFORD STREET AND CHARING CROSS.

CONTENTS.

THE

INSTITUTION

OF

CIVIL ENGINEERS.

SESSION 1870-71.

November 8, 1870.

CHARLES B. VIGNOLES, F.R.S., President, in the Chair.

No. 1,148.—"Description of the Cofferdams used in the execution of No. 2 Contract of the Thames Embankment." By Thomas Dawson Ridley, Assoc. Inst. C.E.

In this Paper it is proposed to describe the Cofferdams which were constructed for the purpose of excluding the water from that portion of the Thames Embankment wall which extends from the landing-pier at Waterloo Bridge to the eastern end of Temple Gardens, a length of 1,970 feet. Also, to give a brief account of the causes and considerations which were influential in determining the kind of dams that should be adopted, and their position, with an epitome of the calculations from which the stability of the dams was deduced; and lastly, to allude to the manner in which the removal of the piles and puddle was effected, after the dams had served their purpose.

The works upon this contract were let, by the Metropolitan Board of Works, to Mr. A. W. Ritson, in January, 1864, and were begun in March of the same year. Mr. J. W. Bazalgette (M. Inst. C.E.) was the Engineer under whom the works were executed, and Mr. Edmund Cooper (M. Inst. C.E.) was the Resident Engineer; the Author having charge of the work for the Contractor.

At the eastern end of Temple Gardens, the new river wall encroaches 200 feet upon the Thames; opposite Arundel Street the breadth of reclaimed land is 270 feet; whilst at the western end of the contract, where the terrace of Somerset House projected beyond the old shore line, the embankment wall is only 110 feet from the existing quay. The depth of water, when the tide is low, varies along the line of the face of the new wall from 1 foot to 6 feet, being deepest in front of the Temple steamboat pier. The borings showed, that the bed of the river consisted of sand and gravel, resting upon clay, at depths varying from 27·58 feet to 33·10 feet below Ordnance datum, which is 6 feet above low-water mark. The depth of the foundation for the river wall and the Temple pier was in all cases designed to be 20 feet below Ordnance datum.

In the specification, the responsibility was laid upon the Contractor of constructing such dams as would exclude the water, power being reserved to the Engineer of the Board to adopt either cofferdams or caissons; but no plan was given of either, and when the works had been let, the Contractor was called upon to submit a plan for approval. In preparing such a plan, the following points were carefully considered:—(1.) The material to be used—wood or iron? The experience of the Author having been chiefly in dams of timber and clay, he may have been somewhat prejudiced in their favour; but careful calculations led him to the conclusion, that in this case a dam of caissons would be much more costly, and could not be so rapidly constructed as a timber dam. (2.) The depth of the water, and the consequent strength of the dam to resist its pressure. The general depth of the water, when the tide is low, in front of the wall, is about 2 feet, and for a very short space only does it exceed 3 feet 6 inches. As the rise of spring tides is 18 feet 6 inches, it was considered that a depth of 22 feet of water would form a safe basis for the calculation. (3.) The position of the dam in reference to the wall. It was considered desirable that the dam should be placed at such a distance from the foundation trench, that the shoring should not extend across the wall. Strutting, when it crosses work in course of construction, offers considerable hindrance to its progress, and interferes with its economical execution. The shores have also to be removed as the work rises, and their removal is frequently followed by a slight yielding of the dam, and a consequent settlement of the puddle. It was therefore decided to place the dam so that its inner face should be at a distance of 25 feet from the foundation trench. (4.) The depth to which the piles should be driven. The upper surface of the clay, which underlies the sand and gravel in the bed of the river, was shown to be at an average depth of more than 30 feet under Ordnance datum, or about 42 feet 6 inches under high-water mark. To have piles, therefore, which would stand 4 feet above high-water mark, and reach 4 feet into the clay, would require timber averaging 50 feet 6 inches in length. To provide a sufficient number of logs of such a length would be a matter of great difficulty; and it was considered that as the dam would be at some distance from the foundation, it would not be necessary to drive the piles more than 12 feet into the ground, nor to carry the puddle more than 4 feet below the bed of the river. The weight of the puddle wall resting on the sandy substratum, added to the effect of the clay banks to be deposited at the front and back of the dam, would, it was thought, render any leakage under the bottom of the piles of small account.

In pursuance of these considerations, a plan (Plate 1) was prepared and submitted to the Engineer, who, however, after having examined it, decided that the outer face of the dam should not be more than 15 feet from the foundation trench, and that the piles and puddle should reach to the clay. The plan was at once modified to suit these conditions, and the work was begun.

The Temple steamboat pier, in length 470 feet, is the most important work in this section of the Embankment, and comprises as much granite, brickwork, and concrete as the remaining 1,500 feet of river wall. It was therefore essential to lay its foundation dry as soon as possible, and it was determined to construct, in the first place, a short dam at each end of the pier. Each of these dams completely enclosed a short length of the river wall, and served as an abutment for the large dam in front of the Temple pier, and for the earth filling, which reached back to the shore, and cut off the water from behind the foundation.

The dam at the western end of the Temple pier, called No. 1 Dam (Plate 1), was first begun. With the view of saving time, the ground was not dredged before the piles were driven, and the driving was in consequence a slow and difficult operation. In many cases it was all but impossible to force the piles down, and about one-sixth of the whole number pitched, having, in the process of driving, appeared to have failed, were drawn, and other piles were substituted. Whenever a pile was observed to show symptoms of failure in driving, it was drawn; and in this dam ninety-five piles were so removed and replaced. Generally, the piles when drawn were found to have cast their shoes, and their points were bruised into a mass of tangled shreds. The failure usually occurred whilst the point of the pile was passing through a bed of close compact sand containing fragments of shells, which rested on coarse open gravel. Beneath the gravel, and resting on the clay, was a layer of septaria, which presented a serious obstruction to the passage of the piles. Once through this stratum and into the clay, the driving became comparatively easy. Notwithstanding the precautions which were taken to draw and replace injured piles, it was afterwards ascertained, when the foundations were excavated inside this dam, that about one-fourth of the piles which remained were bruised and broken, and had not penetrated the clay.

The internal dimensions of this dam were 111 feet 6 inches by 25 feet, with a clear space of 6 feet between the piles for puddle. The piles were of whole timbers, in lengths of from 40 feet to 48 feet, and from 12 inches to 14 inches square. They were shod with cast-iron shoes, weighing 70 lbs. each, and were driven, or were intended to be driven, 4 feet into the clay. Cast-iron shoes were used in preference to those of wrought iron, as giving, at an equal cost, a much larger base for the timber. Where the driving was very difficult, shoes having cast-iron bases and wrought-iron straps were employed. The piles were secured by three rows of walings, of whole timbers, 13 inches to 14 inches square, through which, and passing through the puddle space, were bolts, $2\frac{1}{2}$ inches in diameter in the lower waling, and 2 inches in diameter in the middle and upper walings. These bolts were placed at an average distance of 6 feet 6 inches apart in each waling. Their heads and nuts pressed against cast-iron washers, 8 inches square and $2\frac{1}{8}$ inches thick, with splayed edges. The washers were afterwards made circular, 9 inches in diameter and $2\frac{3}{4}$ inches thick, and it was frequently found that the pressure of the puddle had forced them into the waling to the full extent of their thickness. To avoid the difficulty of procuring long timbers, the heads of the piles were driven below the level of the top of the dam, which was finished to 4 feet above high-water mark. Lengthening pieces were half-lapped to the heads of the piles at short intervals, and were bolted to each other across the puddle space by bolts $1\frac{1}{4}$ inch in diameter. Between these lengthening pieces deals were filled in, longitudinally, up to the level of the top of the puddle. Temporary walings, of half timbers, to guide the piles in driving, were fixed to the inner faces of the gauge piles by bolts $1\frac{1}{4}$ inch in diameter, and were removed before the puddle was deposited.

The 6-feet space between the piles was dredged to the level of the clay, by means of a bag and spoon, and the puddle, which had been previously prepared, was filled in. It was composed of

London clay, from the excavations of the Metropolitan Railway near Smithfield, mixed with one-sixth part of its bulk of gravelly loam, forming, when well wrought and tempered, a consistent and tenacious mass. Part of the clay which was used for puddle, and all that which was deposited against the outer face of the dam, were supplied by the Conservators of the Thames, who dredged it from the bottom of the river. Before the puddle was raised above the level of low-water mark, a sluice, formed of elm planks, $4\frac{1}{2}$ inches thick, and having an internal section of 8 inches by 8 inches, was fixed through the river side of the dam, and rested on the lower waling. It was closed at the outer end by a hanging flap door, to which a ring and chain were attached, to lift it when required.

The transverse struts, of which there was a tier to each waling, were of whole timbers, and were 8 feet apart in the length of the dam. Those in the upper tier were secured by angle-plates and bolts reaching through the piles, the puddle, and the walings. The struts abutting against the middle and lower walings were kept in their places by wooden cleats, bolted to the timbers. To preserve the vertical position of the whole dam, that was, to hinder it from swerving, either towards the river or towards the land, back-stays were fixed, which had angle plates attached to them, bolted through the thickness of the dam to the corresponding plates of the upper row of struts. The tide was excluded from this dam on September 19, 1864, and the foundation stone was laid two months later.

As soon as the construction of No. 1 Dam was fairly in operation, No. 2 Dam was begun. It was precisely similar to No. 1 Dam, but was a few feet longer. It was completed, and the tide excluded, on December 9, 1864.

Whilst these two dams were in progress, the filling-in of the foreshore was going on, so as to embank the space from one to the other behind the Temple pier. The dredging of a trench for the Temple Pier Dam, called also No. 3 Dam (Plate 1), was in operation, the work being executed by one of the dredgers belonging to the Conservators of the Thames. No difficulty was in this case experienced in driving the piles, and the beneficial effect of the dredging was at once apparent. The Temple pier is irregular in outline, portions of it projecting into the river upwards of 30 feet beyond the line of the ordinary wall, and it was necessary to push out the dam so as to enclose the greatest projection. The wall here is also of considerable thickness, the breadth from front to back of the foundation trench along the central part being

57 feet. To avoid the necessity of having to use a large number of struts of this length, it was determined to strengthen the dam by means of buttresses, similar in principle to those which had been used in the cofferdam constructed for the Grimsby Docks.[1] These buttresses were placed at intervals of 20 feet, and were 11 feet in breadth from the walings of the dam. They were tied together by three walings, corresponding in level with the walings which were fixed to the piles of the dam. The walings of the buttresses were secured by long bolts, which passed through to the outer face of the dam. The scantlings of the timber used in this dam were similar to those described for No. 1 Dam, except the walings, which were from 14 inches to 15 inches square.

Two sluices were fixed in this dam, and rested on the lower waling. They were formed of American elm planks, $4\frac{1}{2}$ inches thick, with bolts and stays of wrought iron. Their length was about a foot greater than the thickness of the dam, and each had an internal section, 3 feet high and 1 foot across. The volume of water to be discharged by these sluices during the ebbing of the tide was 986,031 cubic feet, or 164,388 cubic feet per hour; and this discharge was effected in such a manner, that the difference in level between the water inside and the water outside was always less than 2 feet.

As soon as the puddle was raised to the level of low-water mark, the spaces which had been dredged at the front and back of the dam were filled with a mixture of gravel and clay, to prevent the pressure of the puddle from breaking the piles between the lower walings and the solid ground into which they were driven. This backing was carried up to the level of the lower walings, and was of great value in increasing the stability of the dam. Before the tide was excluded, one horizontal strut and one raking strut, both stretching across the line of wall, were fixed against each buttress. They abutted against pairs of piles, driven into the solid ground, and backed with large rubble stones. This dam, which was 481 feet 6 inches in length, was completed, and the tide excluded, on March 24, 1865. Gauges were fixed at four points along the upper waling, and at high water the dam was observed to have yielded 3 inches, $3\frac{1}{2}$ inches, $1\frac{3}{4}$ inch, and $3\frac{1}{2}$ inches, at the several points of observation.

It was found that the trench, which had been excavated for this dam by the Thames Conservators' dredger, was much too wide, and was, therefore, objectionable, both on the score of economy, and as

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. ix., p. 4.

regarded the stability of the dam. For No. 4 Dam, therefore, and for all the dams subsequently constructed, the trench was dredged by means of a bag and spoon, wrought by a small steam-engine. The engine was fixed upon a timber framework having flanged wheels, which travelled upon rails laid upon the stage from which the piles were to be driven. This dredging machine, which was found to be very efficient and economical, was fitted up, under the direction of Mr. B. Cooke, at the workshops of the Contractor. As soon as a sufficient length of trench had been dredged, the piling of No. 4 Dam was begun. In it, and in every dam except that for the Temple pier, the inner row of piles was placed so as to coincide with the river face of the concrete in the foundation of the wall. The mode of strutting adopted in No. 4 Dam was as follows:—Across the breadth of the wall the struts were horizontal, and abutted against walings of whole timbers, bolted to pairs of piles driven into the solid ground behind the foundation of the wall. These coupled piles were placed at intervals of 18 feet from centre to centre, and were strengthened by three back struts to each pair, two of which were horizontal, and one was raking. These back struts rested against a row of piles driven into the slope of the embankment filling, and to the piles deals were fixed, so as to admit of their being backed up with earth and clay to the level of the upper struts. Immediately behind each pile, a mass of rubble stones was roughly built, to give further stability, and to divide the pressure over a larger surface of the earth filling. A sluice, whose internal dimensions were 3 feet by 1 foot, and two smaller sluices, each 8 inches by 4 inches, were fixed in this dam. The sluices were shut, and the tide excluded, on June 26, 1865. The dam yielded at high water, along the line of the upper waling, $2\frac{1}{2}$, $2\frac{3}{4}$, $2\frac{1}{2}$, $4\frac{1}{2}$, $4\frac{1}{2}$, and $4\frac{1}{2}$ inches, at the several points where the gauges were fixed. The internal length of this dam was 382 feet 3 inches.

On September 26, 1865, the tide was shut out from No. 5 Dam, which was 347 feet 6 inches long, and was precisely similar in all its details to the dam last described. The amount which it yielded at high water, along the line of the upper waling, varied from 1 inch to 4 inches. The yielding of the dams was, in every instance, due to the fact that the ground behind the back piles, against which the struts abutted, was not sufficiently compact to resist the enormous pressure of the tide. The rubble stones behind the piles distributed this pressure over a considerable surface, but the new filling material, having been thoroughly saturated with tidal water could, not resist the force of the thrust without a slight yielding.

In executing these works, it was found that the level of the upper surface of the clay, underlying the sand and gravel which form the bed of the Thames, was nearly uniform throughout the extent of this contract, and was not so low as appeared from the borings, being generally about 24 feet under Ordnance datum.

The cross dams, which bounded the western ends of Nos. 4 and 5 Dams, were formed of piles and puddle, reaching down to the clay for a length of 26 feet, being 2 feet more than the breadth of the wall foundation. The piles for the remaining portion of these dams became much shorter as they rose up the slope of the earth filling, and were only used for the purpose of holding up a puddle wall, whose base rested upon the slope of the bank, and whose top corresponded in level with the puddle of the dam, and united it with the earthwork behind. Notwithstanding that the puddle was carried down to the clay in these cross dams for a distance of 26 feet only from the front dam, the leakage through the gravel, around the end of each puddle wall, did not appear to be greater than at any other part of the foundation.

There was some delay in completing No. 6 Dam at the eastern end of the contract, on account of difficulties with the adjoining wharfingers, which rendered it necessary to make some change in the plan, and not until June 6, 1866, was the water excluded from this dam, which was similar to Dams Nos. 4 and 5. The return dam shown upon the plan at the western end of the contract was not constructed, and the works there remained in abeyance, until the front line of Dam No. 5A had been joined by the dam constructed for the works of the adjacent contract.

Two pump-wells, formed of cast-iron cylinders, 8 feet in diameter, were sunk in the Temple Pier Dam, to a depth of 4 feet below the foundation level; and one such well was sunk to a similar level in each of the Dams Nos. 4, 5, 5A, and 6. The sinking of these cylinders was carried on so as to have the pumps ready for work as soon as the sluices were closed and the tide shut out. The quantity of water to be dealt with in the Temple Pier Dam varied from 620 gallons per minute, to upwards of 1,200 gallons per minute, which was the volume passing through the pumps when a large area of the lowest level of the foundation was exposed. In the other dams, a much smaller volume of water was lifted out of the foundations; and as soon as the wall in any of the dams had been raised 6 feet above low-water mark, no further pumping was found necessary. In such cases, all the water which gathered whilst the tide was above the level of the sluices, was passed through them into the river when the tide was low.

Murray's chain-pumps were used for draining the foundations and dams throughout these works, and were found to answer admirably. They are seldom out of order, are easily repaired, and lift mud as readily as water.

From the lower waling to the solid ground at the bottom of the trench which was dredged for the piles, the depth was considerable, and all this depth being under low-water mark, no bolts could be applied to hold together the two sides of the dam. In two instances, where the filling of the dredged space, or the inner face of the dam, had not been carefully attended to, a bulging out of the piles occurred, from the pressure of the puddle. Solid material was immediately deposited against the piles where the symptoms of weakness were seen, and further injury was prevented. But on excavating the foundations at both these places, it was found that several piles were fractured, and were forced about a foot out of the vertical line. Careful timbering, in carrying down the excavation, was sufficient to prevent any further movement.

In depositing the puddle, a tie-bolt failed in three or four instances. In these cases, either a new bolt was substituted, or two new bolts were passed though the dam, one on each side of that which had failed. It was always a work of some difficulty to fix a bolt through the dam after the puddle had been deposited, but a little practice soon increased the proficiency of the workmen in this matter. In tidal streams, on account of the ever varying pressure of the water, cofferdams are seldom in a state of rest, and one consequence of their continual motion is a settlement of the puddle, producing thereby channels underneath the bolts, along which leakages are of frequent occurrence. To remedy such defects, and to ensure the safety of the dams generally, a man was appointed, whose sole duty was to examine the dams carefully once a day throughout their entire length. He attended to the repair of any defects, stopped leaks, and was charged to report immediately any movement, or appearance of weakness. When any leakage shewed itself in the dam, a hole was bored by a 3-inch auger through the inner pile, immediately below the bolt where the leakage appeared, and through this hole cylindrical pellets, formed of clay and sawdust, were forced by means of a wooden staff driven by a mallet. Pellet after pellet was driven into the puddle, until any vacant spaces around the bolt were thoroughly filled up, and the leakage was subdued.

The quantities of materials used in the construction of these dams were as follows:—In Dams Nos. 1 and 2, timber 117 cubic feet, iron 202 lbs., and puddle 9 cubic yards, per lineal foot of dam, measured along the centre of the puddle. In the Temple Pier Dam, the timber amounted to 152 cubic feet, the iron to 285 lbs., and the puddle to 9 cubic yards, per lineal foot of dam. In Dams Nos. 4 and 5, the quantities of timber, iron, and puddle, were almost identical with those in the Temple Pier Dam. The staging from which the piles were driven consumed 19·6 cubic feet of timber, and 13 lbs. of iron per lineal foot. The total length of dam constructed was a little over 2,500 feet.

The pile-driving was executed partly by manual labour and partly by steam power. It was found most advantageous to drive the gauge piles by hand machines, and likewise to pitch, by the same machines, the piles which were to be driven by the steam engines. The steam pile-drivers used in the execution of the works were supplied by Sissons and White, of Hull, and by Appleby Brothers, of London. Those manufactured by Sissons and White cost £290 each, and were very efficacious, driving from eight to ten piles daily, where the ground had been dredged. Messrs. Appleby's machines were less costly, but not so rapid in their action as those of Sissons and White. The cost of driving the piles, for labour and steam power only, was rather less than fourpence a cubic foot where the ground was dredged, but varied from sixpence to eighteenpence a cubic foot where the ground was not dredged. These prices are for the quantity of timber in the full length of the pile, and not for the length driven. They are exclusive of hire of plant and of superintendence. The preparation of the piles cost seven-eighths of a penny per cubic foot, and the fixing of the walings and shores cost fourpence halfpenny per cubic foot for labour, exclusive of the fixing of the tie-bolts.

The timber used was from Memel, Dantzic, and Riga. For the long struts in the Temple Pier Dam, American red pine was employed, and where the driving was very difficult, in Dams Nos. 1 and 2, a considerable number of piles of American elm were used.

In carrying down the excavation of the foundations, careful shoring was required to support the dam; but as this was similar to the timbering used for supporting the sides of excavations in the London clay, it has not been thought necessary to describe it.

In estimating the pressure of the water to be resisted, and the requisite strength of the dam, the following calculations were made:—

P = the pressure of the water in lbs. to be resisted.
d = the depth of water = 22 feet.

Taking 62·5 lbs. as the weight of a cubic foot of water—

$P = \frac{62 \cdot 5\, d^2}{2} = \frac{125\, d^2}{4} = 15,125$ lbs. $= 6 \cdot 752$ tons per lineal foot of dam . . . . (1)

$M =$ the moment of water tending to overthrow dam $= P \times \frac{d}{3}$

$$\therefore M = \frac{125\, d^2}{4} \times \frac{d}{3} = \frac{125\, d^3}{12} = 110,916 \quad . \quad . \quad . \quad . \quad (2)$$

Let w = the specific gravity, or the weight, of a cubic foot of the dam, which, where there are 2 feet of timber and 6 feet of puddle, may be taken at 100 lbs.

h = the height of the dam above the ground = 26 feet.

t = the thickness of the dam = 8 feet.

D = the moment of the dam, considered as a wall, or as a mass of clay in a box or coffer, to resist overthrow

$$= \frac{h\, t^2\, w}{2} = 83,200 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

To produce equilibrium, D must equal M, or equation (2) must equal equation (3); that is, $\frac{h\, t^2\, w}{2} = \frac{125\, d^3}{12}$.

In this case, $t = 5\, d \sqrt{\frac{5\, d}{6\, h\, w}} = 9 \cdot 23$ feet . . . . . (4)

That is, with a thickness of 9·23 feet, the dam would exactly balance the pressure of the water, and any increase in depth would produce overthrow.

p = the thickness of the outer row of piles = 12 inches.

p_1 = the thickness of the inner row of piles = 12 inches.

r = the breaking load of a piece of timber of similar quality to the piles, 1 inch square and 1 foot between the supports = 400 lbs.

R = the resistance of the piles to fracture at the ground-line per lineal foot of dam, in respect to a force acting at the centre of pressure of the water = $r\ (p^2 \times 12 + p_1^2 \times 12)$ $\div \frac{4\, d}{3} = \frac{9\, r\, (p^2 + p_1^2)}{d}$ . . . . (5)

In this case $p = p_1$, and the equation becomes $18 \frac{p^2\, r}{d} = 47,127$ lbs.

If $\frac{r}{3}$ be taken as the factor of safety, R = 15,709, whilst the pressure of water is 15,125 lbs.

s = the height above the ground at which any strut abuts against the dam.

θ = the angle which such strut makes with the horizon.

l = the distance which the struts are apart = 18 feet.

L = the load upon any strut in pounds

$$= \frac{l \mathrm{M} \sec \theta}{s} = \frac{125\, d^3\, l \sec \theta}{12\, s} \quad . \quad . \quad . \quad . \quad (6)$$

For the lower strut, which is horizontal, and therefore $\sec \theta = 1$, let $s = s_1 = 4$ feet.

$$\mathrm{L} = \frac{125\, d^3\, l}{12\, s_1} = 499{,}125 \text{ lbs.} = 222 \cdot 82 \text{ tons.}$$

For the middle strut, also horizontal, let $s = s_2 = 11{\cdot}5$ feet.

$$\mathrm{L} = \frac{125\, d^3\, l}{12\, s_2} = 173{,}608 \text{ lbs.} = 77 \cdot 5 \text{ tons.}$$

For the upper strut, $\theta = 12°$, and $\sec \theta = 1{\cdot}022$, let $s = s_3 = 19$ feet.

$$\mathrm{L} = \frac{125\, d^3\, l \sec \theta}{12\, s_3} = 107{,}388 \text{ lbs.} = 47 \cdot 94 \text{ tons.}$$

The whole pressure has been calculated as depending upon each strut; but as the struts are of equal scantling, the load should be alike on each.

Let x = the load upon each strut when all are exerting the same thrust.

$$s_1\, x + s_2\, x + \frac{s_3\, x}{\sec \theta} = l\, \mathrm{M}$$

$$\therefore \; 4\, x + 11 \cdot 5\, x + \frac{19\, x}{1 \cdot 022} = 18 \times 110{,}916$$

$$x = 58{,}563 \text{ lbs.} = 26 \cdot 14 \text{ tons.}$$

Therefore, the whole weight of the water, if distributed equally over these three struts, will exert a pressure of 26·14 tons on each. According to Professor Moseley, the load which a strut of Dantzic timber, 26 feet long and 13 inches square, will sustain, is—

$$7 \cdot 81 \times \frac{13^4}{26^2} \text{ tons} = 329 \cdot 97 \text{ tons.}$$

According to Professor Rankine, the breaking load would be—

$$\frac{3{,}000{,}000 \times 13^4}{2{,}240 \times 312^2} = 392 \cdot 9 \text{ tons.}$$

If one-tenth of each result be taken as a factor of safety, then 32·99 tons or 39·29 tons will represent a safe load, according as one other of these authorities is followed. The experience of the Author leads him to think that both results are too high.

There are, therefore, three forces tending to give stability to the dam, the amount of each of which has been calculated. (1) The *vis inertia* of the dam considered as a wall standing on its base; (2) The strength of the piles to resist fracture at the ground-line; and (3) The resistance of the struts. The first force cannot be depended upon to its full extent as calculated, because cofferdams are scarcely ever quite perpendicular; and the second force cannot be properly exerted unless the ground be perfectly firm and solid. Therefore, it was considered prudent to have the struts sufficiently strong to resist the whole pressure of the water.

No satisfactory data could be ascertained by the Author, from which the pressure of the puddle upon the tie-bolts could be calculated with precision. It was, however, assumed, that the puddle would not press more heavily than an equal volume of water. On this supposition, with bolts at 22 feet, 14·5 feet, and 7 feet below the top of the puddle, and with 8 feet horizontal intervals, the approximate pressures and requisite sizes of the bolts would be—

On the lower bolts the pressure $= 22 \times 7{\cdot}75 \times 8 \times 62{\cdot}5 = 85{,}250$ lbs. Taking the cohesive strength of iron at 56,000 lbs. per square inch, and one-third of this as a safe load, the sectional area required for the lower bolts is—

$$\frac{85{,}250 \times 3}{56{,}000} = 4{\cdot}56 \text{ inches};$$

$2\frac{1}{2}$-inch round iron has a sectional area of 4·9 inches.

For the middle row of bolts, the pressure on each is, $14{\cdot}5 \times 7{\cdot}5 \times 8 \times 62{\cdot}5 = 54{,}375$ lbs., and the required section is 2·91 inches; 2-inch round iron has a sectional area of 3·14 inches.

In the upper row of bolts, the pressure on each will be $7 \times 10{\cdot}75 \times 8 \times 62{\cdot}5 = 37{,}625$ lbs., and the required sectional area is 1·99 inch.

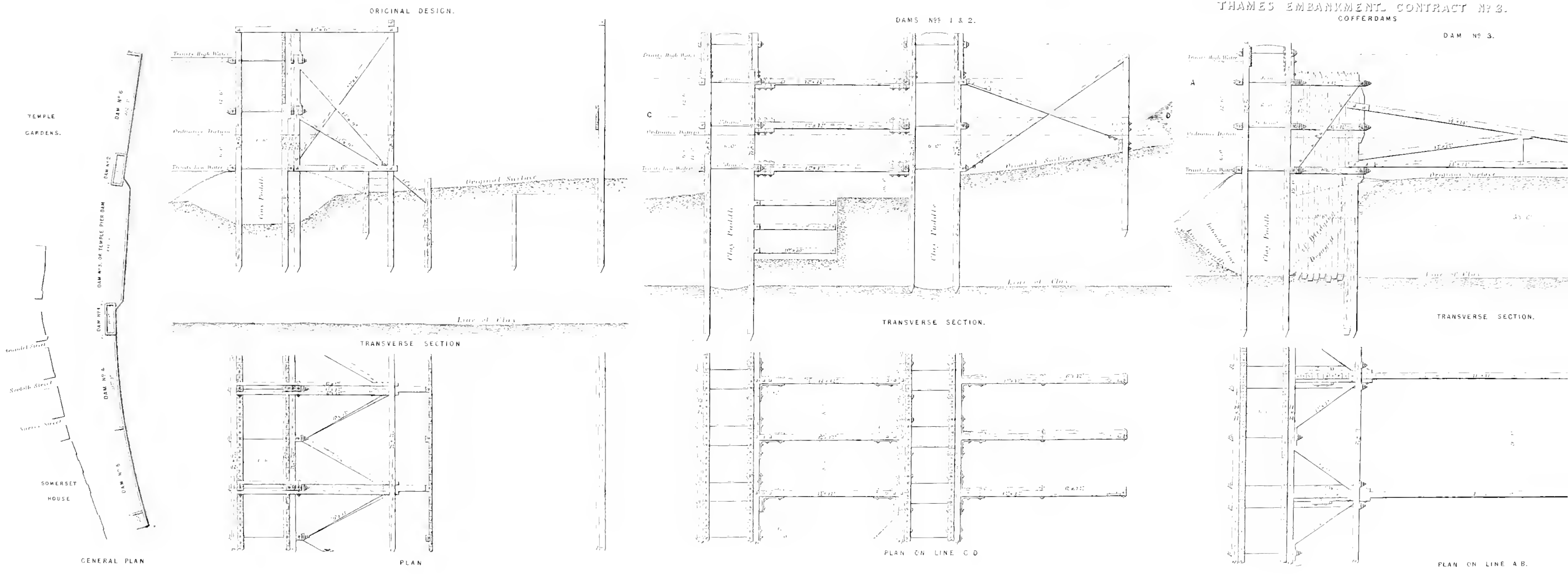
When the dams had served their purpose it became necessary to clear them away, and before the completion of the whole series the removal of those first constructed had been begun. As the piles stood so near the wall and reached to some depth below its foundation, the Contractor was not permitted to draw them, but was directed to cut them off at certain depths, all under low water. Those in front of the ordinary wall were to be cut off at a level of 3 feet under low-water mark, and those in front of the Temple Pier, where a greater depth of water was required for steam-boat traffic, were to be cut off 7 feet under low-water mark. The removal of the piles and puddle was effected in the following manner:

Upon the tops of the piles of each side of the dam half beams were fixed, and upon these rails were laid so as to form a road upon which the steam cranes and dredging machines to be used in the removal of the puddle could travel, and upon which the pile-cutter could also be moved. These machines were successively placed in position, and the work was begun. For the first 15 feet in depth the puddle was filled into skips, and hoisted by means of steam cranes. Below that depth it was dredged by the machines which had been used for excavating the trench. When the puddle had been cleared away to the requisite depth, the pile-cutter followed and performed its part of the work. This machine consisted of a platform upon a stout frame, resting upon four wheels, which travelled upon the rails before mentioned, and carrying a steam-engine with the requisite machinery for driving a circular-saw which was fixed at the lower end to an upright spindle, and adjusted to the proper level. The spindle was placed between the two rows of piles, and revolved in guides at the end of movable arms so arranged that it would shift to either side of the dam by turning a handle, and by the same motion it could be pressed towards the pile which was being operated upon until it was severed by the saw. Two piles were usually cut off on each side before the machine required to be moved backwards on the rails. When the way was clear for the pile-cutter, and a sufficient length of dam dredged, sixty piles could be cut off in a day, but the excavators could not keep pace with the pile-cutter, and the average number of piles actually cut off did not exceed thirty-six daily. The machine was devised by Mr. Charles Murray, of Loman Street, Southwark, and the Author, but the motion which regulated the position of the spindle was the invention of Mr. Murray alone, and was patented by him. The total cost of the removal of the dams was £1 4*s.* per lineal foot, made up thus: clearing out puddle, 13*s.* 6*d.*; dredging outside of dam, 7*s.* 6*d.*; cutting off piles, 3*s.* per lineal foot.

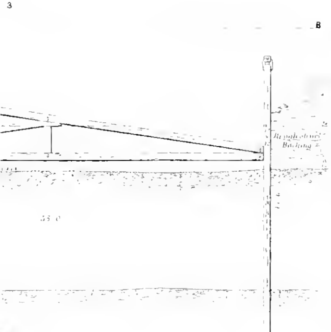
The communication is accompanied by five Drawings, from which Plate I. has been compiled.

[Mr. Vignoles

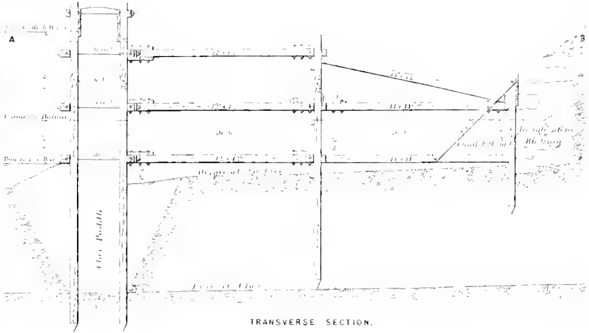
THAMES EMBANKMENT. CONTRACT No 3.
COFFERDAMS



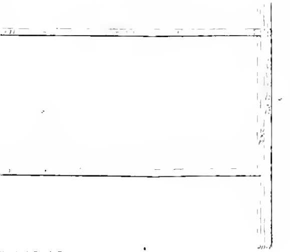
DAMS Nos 4 5 & 6.



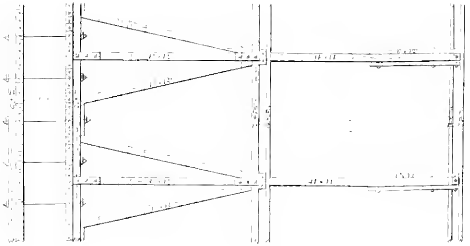
SE SECTION.



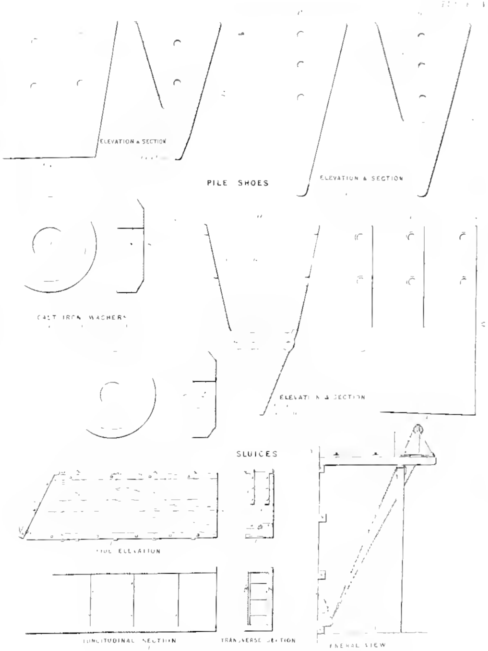
TRANSVERSE SECTION.



N LINE A B



PLAN ON LINE A B



DETAILS OF PILE SHOES WASHERS AND SLUICES IN DAMS

Mr. Vignoles, President, said the details of this work were extremely interesting, and he recommended the Members, and the Students in particular, to read the Paper at their leisure and compare it with the drawings. They were deeply indebted to the Author for giving these details. It reminded him how much the engineers of this country were indebted to the zealous scientific industry of the Contractors, who did, in fact, a great deal of the work which ought to be done by engineers, if they wished to understand their business, but which more frequently than not fell to the lot of the Contractors.

Mr. Hemans, Vice-President, said that though a cofferdam was apt to be looked upon as a temporary expedient of a rugged appearance in the progress of works, and as mere rubbish on their completion, yet it was, when conducted on a large scale, a work of engineering importance. For the last three or four years there had been a length of several miles of these cofferdams along the river Thames, which had been extremely important and successful. Having to sustain a tidal pressure of from 18 feet to 20 feet they were, and always had been, worthy of the study of engineers. From his experience in other places, he believed the contract price per lineal foot of such cofferdams could not be less than from £25 to £30. He had noticed this dam whilst it was in course of erection, and it did great credit to the Contractor's agent who designed it and carried it out, as the construction of a dam 50 feet deep, close to the work it had to protect, was a difficult operation. If the available width in the front of the work had been 25 feet, it could no doubt have been done at less cost. He knew the difficulty of driving piles from 45 feet to 50 feet long through gravel. They broke and tore up at the bottom, though the operation was successful in this particular case. It would be interesting to compare the cost of this plan with that of the wrought-iron caissons which had been adopted in the construction of No. 1 Contract of the Thames Embankment by another Contractor, Mr. Furness (Assoc. Inst. C.E.). The latter, it was true, involved a great deal of costly plant, and unless the work was a large one it would not pay to have such a quantity of wrought iron in use; but it was desirable that a careful comparison should be made between the cost of the pile system and that of the caisson system. He believed these caissons had been sunk, in some cases, to a depth of 40 feet. Where the bottom was unknown, and where borings could not be relied upon to show it, caissons were very valuable, as affording ready means of getting the water out, either by pumping or by pneumatic pressure, and actually allowing the strata to be seen.

He hoped some one would analyse the respective merits and cost of the old-fashioned timber dam, like that used by the Author and also by Mr. Webster (Assoc. Inst. C.E.) on the other side of the river, constructed with great rapidity and carried out with great success, and of the caisson dam at considerable depth and with a heavy head of water upon it.

Mr. J. Phillips agreed that, to make the Paper complete, a careful *résumé* of the cost per foot lineal of the dam should be given, including that of cutting off and removing the piles. The object of a Paper like this was to elucidate the proper means, and the most economical method, of carrying on a large contract, where a great length of cofferdam was required. There might be various forms and designs, and each might have some particular merit; but the most meritorious in the eyes of the Contractor would be that which was the most economical in the long run. He thought the Paper showed that there was a limit to the economical use of timber dams, and though this one had served its purpose admirably, the injury it was stated so many of the piles received during the driving, and the fact that, after all, ninety-four were found, on pumping out the water, to be bruised, and to have failed to penetrate to the clay, showed the system was not altogether satisfactory—that it involved great risk and in some cases a chance of failure. It was his opinion that, in the case of a long length like this, an iron dam would have been more economical than a timber one, as well as much safer, and of greater advantage to the permanent work.

He had been told that an iron dam could be not made so cheaply as a timber one, and he knew that length for length it was not always possible to do so. It was not a question of cost only, however, but how to utilize the materials over again. Cofferdams were often looked upon as a bore by Contractors. They were expensive, and the feeling was to get them made and out of hand as soon as possible, and hence sometimes that which was commenced hurriedly for cheapness proved in the end the dearest. In the present day, when contracts were cut very fine and close, the cost of everything had to be considered, involving that of all temporary works, which required to be made as much a study as though they were part of the permanent work. If an iron dam was used, it entailed careful consideration and carrying out, but he believed that in all cases like the Thames Embankment works, or others of a similar nature, he should be able to show at a future time, that iron must inevitably supplant wood for dams, as it had done in other matters.

Mr. G. B. BRUCE observed that when Contractors came before the Institution with the description of a work like this, they naturally might not care to give the detailed cost. He therefore felt a considerable amount of gratitude to the Author for stating the exact cost of driving the piles and of cutting them off, and if he added to that the cost of the dam as a whole, the Institution would feel doubly grateful to him. The kind of timber of the piles which had broken, twisted, or turned in the driving had not been mentioned. If it was ordinary Memel timber he was not surprised at the trouble it had given. About twenty-three years ago, when driving piles for the Royal Border Bridge over the Tweed,[1] under Mr. Harrison (V.P. Inst. C.E.), he had found it impossible to get the ordinary Memel timber into gravel similar to that in the bed of the Thames, and it was only by using piles of American rock elm that the difficulty was overcome. The description of shoes used was a detail of importance. He had found cast-iron shoes were the best in many instances, and preferable to the malleable iron that was generally used for the purpose. He was old-fashioned enough to believe that timber dams, in a case like this, would be cheaper than iron caissons. Where, however, there was a place for removing the iron to, and where it could be used again, the cost might be reduced considerably, and if often used be in the end cheaper than wood; but in nine cases out of ten, when an iron dam was done with there was no further use for it, whereas timber was always useful.

Mr. BRAMWELL gathered from the Paper, that the Author considered the cofferdam as competent to resist part of the pressure of the water by its weight, treating it as a wall; for he said that had this dam been 9·23 feet wide, in lieu of 8 feet wide, it would have been sufficiently strong to balance the pressure of the water without the aid of strutting. He thought this was an erroneous view. The sides of the dam were made of piles, which, from their great height, were competent to bend, and the interior of the dam was made of puddle, which was plastic. That being so, the dam must be looked upon not as a rigid wall, nor as a rigid trough filled with a weighty body, but as a flexible wall, and one both the sides and the interior of which were capable of bending under pressure. In his opinion any one who trusted to the resistance of the dam as a wall, to take a part of the strain, leaving the remainder alone to be supported by the struts, would find that the whole of the weight would come upon the struts, and that, if

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. x., p. 219.

the necessary provision for this state of things were not made in the calculations, and the struts were not prepared to support the whole of the weight, the dam would give way.

Mr. A. Ogilvie remarked that the dam in question appeared to have been well constructed. If he had been the Contractor, instead of having the outside piles of the full length of the rise of the tide, he would have had them about half-tide, and the inside piles of the dam caulked and a sloping wall of clay put on the top. It would have saved timber, and the Contractor's pocket too.

Mr. Beardmore said, if it was required to drive a dam accurately and economically in gravel, it was important previously to clear away the substratum, as it was commonly most difficult to drive through gravel or fine running sand; and the larger the scantling, or the closer the piles, the more difficult did this process become. The dam should be pitched in a trench made by bag and spoon, and the puddle should afterwards be filled outside the piles as well as between them, and the trench should be narrow compared with the width of the dam, as by that means the outside clay became wedged into its place. The first experiment with the Thames dredger did not answer so well, because too large a trench was scooped out, involving an unnecessary expenditure of clay and loss of holding, or anchoring, power in the deeper or main piles.

From a long experience in dams, where sands and gravels prevailed, he would never attempt driving the piles more than was absolutely necessary. The pile-heads and shoes were soon punched and rendered useless by the friction being greater than the strength of the timber could support.

In substituting a close row of whole piles for the north wing wall of the West India Dock entrance, which had failed in running sand, a Nasmyth engine was used for a long period with disastrous loss and failure. Rock elm piles were used when other materials failed, and it was at last found that part of the old brick wall, which had become buried in the quicksand, was being punched through, and the work had to be removed by divers. If the bag and spoon had been used before the driving was commenced, there would have been a great saving, and all obstructions might have been got out of the way. From the Author's calculations of the actual pressure in relation to the moment of resistance, it appeared that there was a pressure of 49 tons, and a moment of 37 tons, so that, in reality, there was an available force of 12 tons in excess of the resistance; but that was assisted by the solid piled counterforts or struts, and by the tenacity of the bolts, so that the work was safe. Nothing short, however, of the most careful strutting would have stood with so near a balance of the resistance to pressure as 15,709 to 15,125, where one-third was taken as the factor of safety. In such a case, including the danger of even a movement due only to elasticity, the dam was so near its final resistance that it was almost afloat; which, at extreme tides, was frequently the case with dams in the most critical period of their use. As to the bolting together of the two rows of piles; each through bolt was 2½ inches in diameter, and the resistance was 85,000 lbs., or 42 tons; but it was possible that more than this force might be thrown upon one bolt by the slacking of those adjacent. Hence it often happened that, unless great care was taken in bolting together with most efficient wales, the through bolts were liable to burst, and this inevitably caused the failure of the dam in a greater or lesser degree; and it must be recollected that a small failure often caused a disastrous loss.

In the case of the embankment dam at Hungerford Bridge a portion burst, and the tie-bolts were torn like threads. If in any given tier of bolts, some were a little looser than others, the bursting force and accumulated weight of puddle would be thrown upon one place, and the pressure due to three bolts was thrown upon one, it might easily give way; thus it was advisable to have a predominant size and strength for the bolts of a dam and for their heads, so as to prevent them drawing into the wales. It was generally found that the greatest strength was required where theory pointed out, viz., at one-third from the ground level.

An interesting point, well-known to practical men, was the tendency of puddle to settle and leave a gap under the bolt; this action made it impossible to apply internal wales in a coffer-dam. Many of these difficulties were avoided when a dam was made with only one row of piles. When shores could be easily applied, and the depth of water did not much exceed 20 feet, a single dam could be constructed with economy and efficiency. In this case ordinary caulking took the place of puddle, and stability was obtained by shoring, and afterwards strutting the work as it advanced. But although he had used this plan with great success, the late Mr. James Walker, Past President, told a story of one that a Contractor made across the West India Dock entrance which succeeded well at first, but suddenly lifted and floated away with a man, costing him his life. This accident happened at a high spring tide, but, although partly due to the floating power of the dam, it was probably due to the quicksand at that spot which gave so much trouble when the wing wall slipped in forty-five years later, as before referred to. He imagined that the calculations of strength, given in the Paper, did not take into account the tendency of the dam to float, taken as an empty area with fluid outside,—a tendency which was increased when once excited by an undue pressure or slight movement of the structure. Hence the necessity for guard piles and other contrivances, if a dam was liable to be struck by ships or other heavy vessels. This floating power was very evident in the case of sinking iron cylinders or caissons, such as those lately used in the foundations of Blackfriars Bridge, and other structures.

His own rule, in every kind of cofferdam, whether single or double, was to have a certain proportion of gauge piles, longer than those afterwards driven, so as to obtain a hold on the soil; for piles at 10 feet spaces were easily driven, and gave a stability to the finished work that could not be so readily obtained, as the piles drew closer together in a more advanced stage of the work.

Mr. Harrison, Vice-President, said Mr. Beardmore had mentioned one difficulty which the Author had also referred to, viz., the leakage which occurred where the bolts were put through the dam. He had been accustomed, from the earliest time, to a practice which quite overcame the difficulty of bolt leakage. The plan, which was not his, was this:—Take a plate of thin sheet-iron 12 inches to 15 inches square, and make a hole in it near the upper edge to fit the bolt which was to bind the two sides of the dam together. When the bolt was put in slip one, or in some cases two, of these plates along the bolt, so that the plates hung down, and when the puddle settled the flow of water along the bolt would thus be stopped. It would be interesting to be informed of the particulars of cost, in each case, of the different varieties of form that had been applied to the construction of the dams for the Thames Embankment. There was no doubt that engineers were in the habit of throwing the responsibility of the construction of dams, considering them as temporary works, entirely upon the Contractor; but at the same time the question of these dams was one of great importance, and as capable of scientific and practical solution as any other point of engineering.

The construction of river walls, such as the Thames Embankment, was, independently of the question of cofferdams, a subject of great interest. He had lately designed a quay wall, where there was a depth of 20 feet at low-water, and he had adopted the plan of building it entirely upon cylinders, and arches between the cylinders, with sheet piling of cast-iron at the back. He found he could by these means construct a wall without the liabilities which attached to cofferdams; and he believed the time was not far distant when quay walls would generally be constructed upon cylinder foundations without the use of cofferdams. Mr. Bateman was carrying out the same principle in the construction of quay walls at Glasgow.

So far as he was aware there was no instrument for driving piles that could be compared in efficiency with the Nasmyth hammer. Where they were being driven through gravel—which should be avoided if possible—or through sand, which was quite as difficult, it could be most effectually done with a Nasmyth hammer, striking sixty to eighty blows per minute; and he believed Contractors would find the Nasmyth hammer, in such cases as that which had been described, where a large amount of piling was involved, the best pile-driving machine that could be employed.

Mr. Redman remarked that one point had been incidentally touched upon, with reference to the comparative advantages of wrought-iron and cast-iron cofferdams and the old method of timber piling. It would add to the interest of the Paper, if statistics were given to show the reason for the introduction of the old-fashioned timber cofferdam in this great wall instead of the caisson system, which was used at the commencement. Thirty years ago a Paper was laid before the Institution by the late Mr. Grant Stair Dalrymple,[1] giving a description of the cofferdam in front of the first length of the Thames Embankment and the foundation walls of the Houses of Parliament, in many of its details similar to the one under discussion. Now these cofferdams could not ordinarily be constructed for less than from £25 to £30 per lineal foot, equal in many cases to the cost of the wall, so that it was evident that a less costly form of construction of cofferdams was a want in the practice of engineering. Undoubtedly the method adopted in the Thames Embankment, viz., using cofferdams and shutting out the tide, was the most practical for this special work, and spread over the vast cost of the wall itself, and considering the large areas reclaimed from the foreshore, was the most economical. But when wharf walls were required, with a limited frontage and area, the question of cofferdams would in some cases almost negative the work. The cost of the cofferdam would equal that of the wall itself. He had put in wharf walls in the lower reaches of the Thames by tide work without cofferdams; but in a large work like this that would have been to

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. i., Session 1840, pp. 18, 19.

some extent speculative and hazardous. A series of cylinders under the area of the wall base, supporting a platform formed by groined arching at the level of low water, would in most cases form a good foundation. Some years ago Mr. D. Stevenson had presented to the Institution a Paper upon the cofferdams employed in the Ribble,[1] and Mr. Neate had also given a description of the cofferdam employed by the late Mr. Rendel at the Grimsby docks.[2] In the latter case was first introduced the counterforts of sheet piling, and the difficulty of leakage by through bolts was overcome by driving three rows of piles with two trenches of puddle, and through bolts of the ordinary length, breaking joint with one another, so that there was no bolt hole through the dam. This, however, was a most costly expedient.

Mr. G. Furness said that Mr. Ridley had given the cost of driving the piles, puddling, and cutting off the piles, but had made no comparison between that and the caisson system. As shown by the borings supplied by the Metropolitan Board of Works, clay was reached at an average depth of 30 feet below the datum line, whereas it was not more than about 22 feet below that level, which made a material difference in the use of timber piling. Had the foundations of the dam been carried to the depth suggested by the borings, it would have been a grave question whether timber piling could have been used successfully for the dam, so as to exclude the water from the inside.

Mr. Ridley said that he agreed with Mr. Harrison as to the efficiency of the Nasmyth hammer in pile driving, but in a long wall such as this, where three or four such machines would have been necessary, the question of cost arose: the price of those machines was £1,800 or £2,000 each, and on the score of economy the cheaper pile-driving machines were used. With regard to the depth of foundation, he had driven two short experimental dams, each about 100 feet in length, and when he had found out the actual depth of the clay, he had proceeded with full confidence. As regarded the staging, it must be noted that the materials employed therein were used, again and again, for successive lengths of the dam. In the item for dredging the whole cost of the operation was not charged, allowance being made for the sand and gravel produced, the value of which repaid a considerable part of the outlay. With respect to timber, the Contractor was restricted to no particular description, and he was

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. i., Session 1841, pp. 81–85.
[2] *Ibid.*, vol. ix , pp. 1–23.

bound by no specification, either as to quality or scantling. The kind most extensively used was white fir from Dantzic and Riga, a small proportion only of Memel or Dantzic red fir being employed, and where the piles were exceedingly hard to drive American rock elm was used. The white-wood cost 65*s.*, the red-wood 80*s.*, and the elm 100*s.* a load. The average cost of the whole quantity, after making ample allowance for waste, was not more than 1*s.* 7*d.* per cubic foot. The cost of the labour on the piling, waling, and shoring, including the fixing of the tie bolts and other equipments, and covering all contingencies, was 7*d.* per cubic foot; and if to this was added 4*d.* for the use and depreciation of plant, the total cost to the Contractor was 2*s.* 6*d.* per cubic foot of timber in the finished work. The greater part of the clay used for puddle, and half of that employed for the filling against the outer and inner faces of the dam, were not only delivered to the Contractor free of cost, but he received a payment for the deposit.

The following were the details of the cost per lineal foot of all the dams, except that at the Temple pier :—

		£.	s.	d.
Stage for dredging and piling		1	4	0
Dredging		0	7	0
Timber	117 c. ft. at 2*s.* 6*d.*	14	12	6
Cast iron	143 lbs. „ 1*d.*	0	11	11
Wrought iron	59 „ „ 2*d.*	0	9	10
Puddle	9 c. yds. „ 2*s.* 3*d.*	1	0	3
Clay backing to outer and inner faces,	5 „ „ 1*s.* 2*d.*	0	5	10
Cost of dam per lineal foot		18	11	4
Removal of dams as per details previously given		1	4	0
		19	15	4
Deduct value of timber and iron removed		2	10	6
Total net cost of dam per lineal foot		£17	4	10

In the Temple pier dam, where more timber and iron were used than in the other dams, the cost was £20 15*s.* per lineal foot, and the average of the whole series was within a fraction of £18. On account of the return and cross dams, the cost of the total length constructed was about one-fourth greater than the actual length of wall built, and therefore the cost of the dams when measured by the net length of the wall was £22 10*s.* per lineal foot. The total average cost of all the work, including the wall, the filling of the foreshore, the sewers, and the dam, was £125 per lineal foot.

He was still of opinion that the cofferdams were cheaper, and were more expeditiously constructed, than the caisson dams, and he was confirmed in this opinion by the fact, that although the caisson dams were at first employed upon No. 1 Contract, cofferdams were afterwards resorted to, and were extensively used on that section of the Embankment. And on all the contracts subsequently executed cofferdams alone were employed.

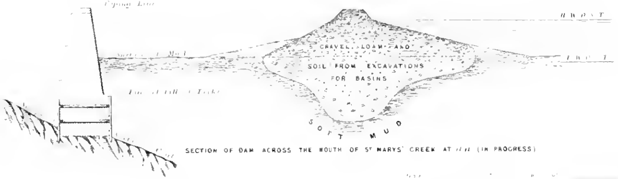
Lieut.-Col. Clarke, R.E., said, the question had occurred to him during the reading of the Paper, why were engineers dealing with first principles in relation to these works? When he went into the library, he found that there were no good books to put into the hands of students in regard to this subject, which was employing a great number of engineers; in fact, it had been entirely neglected. With the exception of a little book, by Mr. Dobson,[1] there was really no treatise on cofferdams. Plate 2, Figs. 1 and 2, represented part of the works in progress in the Medway at Chatham, where he had to construct two locks and entrances, and a portion of river wall. He first of all went into the calculation whether he should construct an ordinary timber dam for these works, somewhat similar to that described in the Paper; but he found the cost came out at £22 10*s.* 8*d.* per lineal foot. He then fell back upon the plan which Mr. Phillips had referred to, and had worked it out, when he found the cost would be reduced to £15 17*s.* per lineal foot. He was on the point of offering the contract when the local Engineer, Colonel Pasley, suggested that there were certain conditions, imposed by the Thames Conservancy, which did not apply to the Medway; such as that the pile dams in the Thames must be confined to a certain line so as not to interfere with the navigation, in addition to which the material to be dealt with was different; so that a comparison of cost in the Thames and the Medway could not be made. If the Thames Conservancy could have assented to the Metropolitan Board of Works being similarly unfettered by conditions, a great saving might have been effected by a modification of their dams, by carrying them more into the stream than they were. He had constructed the dam simply with the material taken out of the excavations of the basins at Chatham, its length being between 1,100 feet and 1,200 feet. Under ordinary circumstances, it could have been executed for £11 15*s.* 3*d.* per lineal foot, but it was costing far less, as convict labour was employed. He might state that the river wall at Chatham cost £35 or £36 per

[1] Weale's Rudimentary Series.

COFFER DAMS.

Fig 1

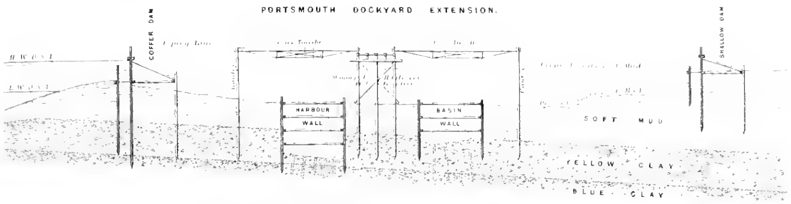
CHATHAM DOCKYARD EXTENSION.



SECTION OF DAM ACROSS THE MOUTH OF ST MARYS' CREEK AT a a (IN PROGRESS)

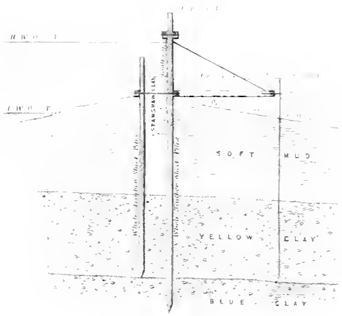
Fig 3

PORTSMOUTH DOCKYARD EXTENSION.



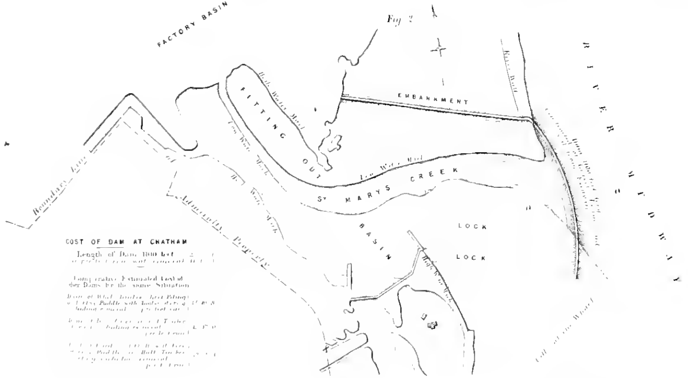
SECTION THROUGH COFFER DAM AROUND WORKS, AT a a

Fig 5



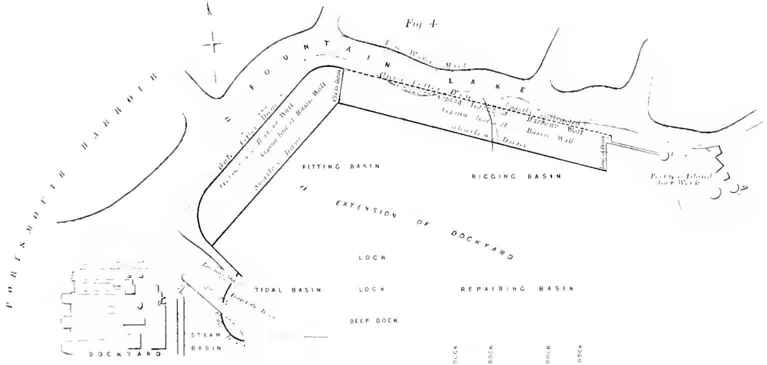
ENLARGED SECTION OF OUTER COFFER DAM

Fig 2



PLAN SHEWING POSITION OF DAM ACROSS THE MOUTH OF ST MARYS CREEK

Fig 4



PLAN SHEWING POSITION OF COFFER DAM AROUND WORKS

lineal foot. It was a suggestion worked out successfully, which he owed to the officer in charge of the work.

Besides the works at Chatham, there was another large dam constructed at Portsmouth. In this case there were 4,500 feet of dam, enclosing a space of 95 acres. It had been constructed for three or four years, standing in some parts in from 25 feet to 30 feet of water, and at spring tides the land within the dam was covered with a depth of 7 feet of water. No difficulty had arisen with regard to the gravel; but beds of cement stone had been encountered which were as difficult to deal with as gravel banks, though there was not the same underflow of water into the inner works as was the case with the works on the Thames. An area of nearly 9 acres was enclosed in that part of the dam in which the whole of the work had been done, and the water had been kept down with very little trouble. This dam was more expensive than that at Chatham. Taking the outer, the inner, and the cross dams, and spreading the cost of these over the whole work, it was, nevertheless, a cheap form of construction. The cost of the outer dam was about £27, and of the inner dam about £10 10*s.* per lineal foot. The outer wall cost about £50, and the inner basin wall £34 per lineal foot. From these facts might be computed the relative proportion of the cost of the dam to that of the permanent work constructed inside it. No dredging was required. The actual form of the dam was that of an inverted mushroom, and differed from that deduced from theory. The foreshore was fine silt and mud, and was always afloat; and the rise of the tide was 18 feet.

Mr. C. Neate said, that he believed the cofferdam at the Grimsby docks was the first instance in which counterforts of solid piling had been introduced, and he pointed to their particular value in that position where the dam, of which the length was 1,500 feet, stood much exposed, springing from the extremities of two embankments which projected five-eighths of a mile from the shore, and being entirely self-supported. He might mention that the shields to prevent leakage under tie bolts, described by Mr. Harrison, were used throughout at Grimsby, being made in this case of cast iron. The mass of the cofferdam was more than sufficient to withstand the pressure of the water, its width being about 18 feet. Altogether he believed it was one of the finest works of the kind ever carried out.

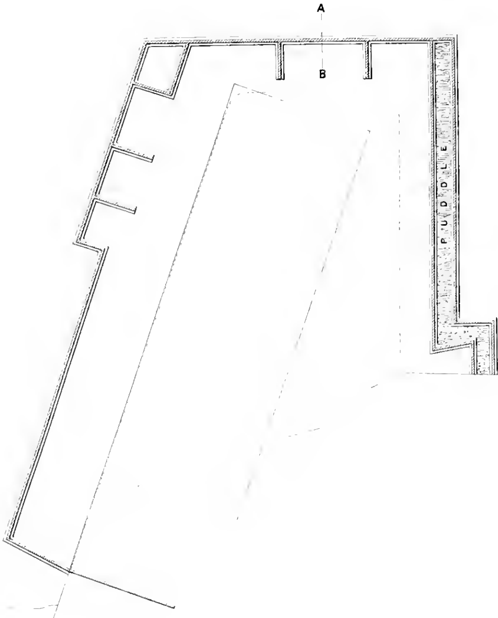
There were cases where it was not requisite to depend so largely upon weight as that just referred to, and where the structure, being literally a coffer—or box—dam, might be strutted across from side to side, or where it might be shored from some solid mass behind it. In such cases, and more particularly when the permanent depth of the water was considerable as compared with the rise and fall of the tide, he believed that the use of single-sheet piling, tongued and grooved, might be resorted to with advantage, as being cheaper, more expeditious and more convenient than a puddle dam. He had successfully applied the system under circumstances of the kind. On one occasion, in the construction of a quay wall, he had put down a length of 100 feet of single dam in juxtaposition with a similar length of ordinary puddle dam, and being satisfied with the result, had adopted it on another part of the same work for three sides of a cofferdam surrounding an old pier which had to be demolished and reconstructed. Here the length of single piled dam was between 200 feet and 300 feet, the bottom of the excavation being about 25 feet below low-water, and 30 feet below high-water. The ground was silty clay; the piles used were both of hardwood and of creosoted pine, and the tongues were of bar-iron welded up in one length (Plate 3). Although the work was not on a large scale he thought it worth recording, as an example of the successful application of this system. He was not aware, till Mr. Beardmore stated it, that the plan had been tried by Mr. Walker at the West India docks.

Mr. Abernethy remarked that local circumstances, such as depth of water, nature of soil, &c., were matters which affected the comparative cost of dams, and therefore the Paper was of little importance as affording practical information.

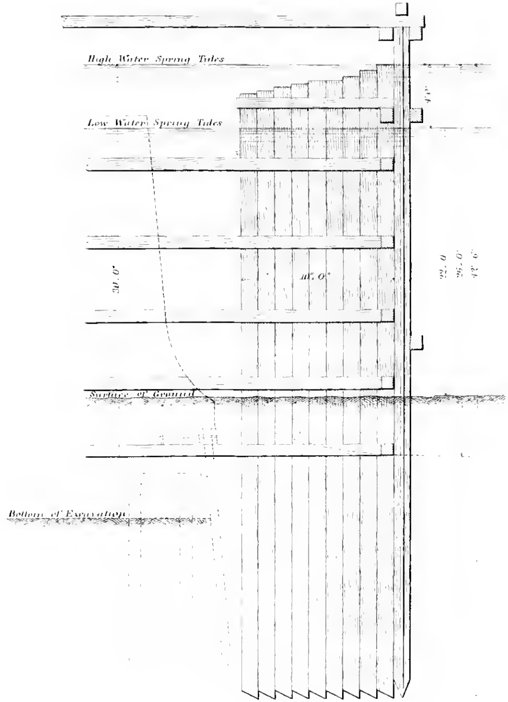
In the year 1846, he had to construct a dam in the centre of the harbour of Aberdeen, which would allow passages for vessels on each side. The dam was required for the lock works. It was 390 feet long and 150 feet wide, composed of two rows of whole piles, driven 15 feet into silty clay, these rows being 8 feet apart. After driving the two rows of piles, the space between them was dredged out with spoon and bag through gravel down to the silty clay. In the dam, sufficient sluices were provided for the ingress and egress of the water. When the four sides of the dam were completed, the water being level within and without, brace piles were floated on the surface of the water, and were secured to the piles at regular intervals within the dam. A powerful pumping engine was next employed to lower the water, and three other sets of braces were floated and fixed in the same way. Upon these braces, tramways were laid, on which travelling cranes worked, and the masonry was thus set from the braces. They afforded also great facility for driving the piles, as the whole of the foundation

COFFERDAM

OF SINGLE PILES AT CUSTOM HOUSE QUAY WORKS (1866) RIO DE JANEIRO.



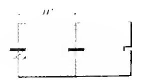
PLAN.



SECTION AT A B

Scale for Plan

Scale for Section



of the lock rested upon piles 5 feet apart from centre to centre, and 3 feet apart under the sills and floors. By adopting this plan, the use of the ordinary pile-driving machine was avoided, mere sliding frames being used. The dam proved successful, except a certain amount of leakage, caused by internal waling pieces, and the settlement of the clay puddle beneath the tie-bolts. It was a work which answered its purpose well, both as a dam and as a means for setting the masonry of the lock.

Mr. W. A. Brooks stated that he had constructed a dam at Northumberland dock, in the river Tyne, on what was considered a bad foundation, being nothing but sea-sand 60 feet deep. The dam was composed of two rows of 13-inch fir piles, 5 feet apart, filled in with clay puddle to the depth of about 4 feet below the level of low water of spring tides. That dam, with a head of water of 14 feet, on the receding of the tide, was perfectly tight. It was several thousand feet in length, and enclosed a dock of the largest description. He attributed its successful accomplishment to the great care with which the piles were driven by the Contractor, which piling, by compression of the sand, rendered the latter impervious to the passage of water under the head before alluded to. The coffer-piling was strengthened by deposits of clay and gravel, faced with rubble, forming slopes to the river and the dock, but, as they were only formed on the surface of Jarrow sand, they had no share in preventing a scour from taking place under the piling of the cofferdam. A careful record was taken of the depth to which each pile was driven, and it was found that none penetrated the sand to a less depth than 18 feet below low water of spring tides.

Mr. R. P. Brereton observed that in February, 1864, the contract having been taken in the previous month, the Contractor consulted him with respect to the description of cofferdam that would be appropriate for this particular work. At that time, a commencement had been made in the employment of iron dams in the adjoining portion of the Thames Embankment. After full consideration, he came to the conclusion that the circumstances in this instance were not such as to necessitate the use of iron for the dams; either timber or iron would have been feasible; the subject was one rather of comparative cost, convenience, and time of executing the work. If the foundation of the wall had involved a depth of 80 feet or 90 feet, such as at Chepstow or at Saltash, the use of timber would have been nearly impracticable. His own opinion, and that of Mr. Ritson's officers, appeared nearly to coincide in this respect, although Mr. Ritson's engineer—a man of experience, and an able mathematician—thought that a dam might be constructed of clay and piles sufficiently strong to stand as a wall, without requiring any large amount of shoring. In the Paper it was assumed, that if the thickness of the dam as executed had been increased only 1 foot 3 inches, it would have stood alone as a wall, without any assistance from struts or shoring, and that the piles alone would have been sufficient to resist the pressure of the water before being broken off at the ground line; and it might appear from this as if the Author was under the impression, that the use of heavy and expensive shoring might have been dispensed with. The risk that would have been incurred in such a proceeding had been shown in an instance where it had been attempted to construct a dam without shoring or similar precautions. A lock entrance had to be put in, and a dam constructed, in the deep water of a dock basin. The Engineer of the work had proposed that the dam should be supported by rubble slopes on either side to keep it up; but as this would have required a considerable length, and have occupied with its slopes so much of the half tide basin, and would have been expensive to remove after the work was completed, the Contractor adopted the plan which had been referred to as that which was employed at the Grimsby docks, viz., with deep counterforts of solid timber. These were placed at 18 feet apart, and were 14 feet in depth of solid timber piles, the width of base being, with the dam itself, 22 feet. The piles were driven 25 feet into the ground, or 18 feet into the silt below the original bottom of the basin. It was computed that this would be sufficient to support the dam. When the dam was being tested, and when only 15 feet head of water had been attained, the line of pressure still being within the puddle wall, the dam heeled over, and leaked so much that it became necessary to let the water in to save it, when it was found that the dam and counterforts had permanently turned over to the extent of 3 feet at the top. It was then well shored from behind to the old dock wall, the disturbed puddle was restored, and it served for getting in some of the outside apron work of the new lock. When the time arrived for putting in the entrance, and the old dock wall and shores against it had to be removed, other means of supporting the dam had to be devised. Being requested by the Contractor to advise as to the course that should be adopted, he suggested that a rubble slope against the inner side would have a tendency to turn the dam outwards when the water was let out of the basin, and that a rough rubble wall of masonry should be built behind the dam, between the timber counterforts. The wall was 13 feet thick, and concreted behind. The entrance had since been built, and no further mischief had occurred. He could not agree with the suggestion in the Paper, that if the dam had been a foot or two thicker it would have been sufficient, without shoring, for the purposes for which it was intended. He had heard nothing to alter his opinion, that the kind of dam which had been used was that best adapted for the work. The Contractor had himself large experience of works across bays of the sea for dock purposes, with dams such as had been described by Colonel Clarke for Chatham, viz., large earthwork embankments tipped without dredging or puddle walls. These had been remarkably successful. It was, however, not considered that this description of dam would have been appropriate to the circumstances of the Thames Embankment.

Mr. Vignoles, President, said it had been truly remarked by Colonel Clarke that there were at present no text-books upon cofferdams. He was afraid there were but few upon any branch of engineering for systematic adoption in tuition. Almost all experience, particularly of cofferdams, was what was gained by Contractors, who had not published it ; and each in succession was obliged to form his own conclusion as to the best mode of effecting his object. That was one of the misfortunes of the present mode of carrying on engineering. A work was first of all completed, and then rules deduced from it. This had often been considered and discussed, and it presented a remarkable contrast between the engineering practice of this country and the theoretical researches of other countries. It was clear, *à priori*, that no rules could be laid down for cofferdams, because the circumstances varied. It had been stated that one of the Members of the Institution was about to introduce the system of iron cylinders for the purpose of building walls upon, and it was further mentioned that Mr. Bateman, instead of having cylinders of iron, proposed to have them of brick. Probably centuries ago, the Hindoos built the foundations of river walls in that way, and brick cylinders, or wells, of about 3 feet diameter, had so been in use in India almost from time immemorial. He had no doubt the adoption of cylinders of iron or brick would eventually supersede the ordinary methods of forming cofferdams, and he believed would be universally adopted. The simplest contrivance for bad foundations was to surround the area with a ring of timber, sufficient to resist the ordinary floods; then to lay within a heavy bottom of concrete, and to build thereon up to the ordinary level of the tide. That he had seen practised, and had himself practised. He once had a triple cofferdam, of enormous size, carried away by floods, broken in two, and each part lodged upon a bank, like the wrecks of two large vessels. It was impossible to recover that dam, though he had saved some others by large barges filled with stones, which he brought upon the piles, and kept there while the dam was being repaired.

November 15, 1870.

CHARLES B. VIGNOLES, F.R.S., President,
in the Chair.

No. 1,255.—"On the Water Supply of the Town of Paisley, Renfrewshire."[1] By ALEXANDER LESLIE, Assoc. Inst. C.E.

IN the year 1835 parliamentary powers were obtained to bring in water, for the supply of Paisley, from the districts of Gleniffer and Harelaw lying to the south of the town, having respectively drainage areas of 624 acres and 166 acres. The works were executed under the direction of the late Mr. R. Thom, M. Inst. C.E., who made careful experiments, extending over a period of three years, to ascertain the amount of water flowing from the Gleniffer district, by means of which the quantity actually available was found to be 70,354,769 cubic feet per annum, which is equivalent to 31·06 inches, out of a depth of 46·13 inches of rain over an area of 27,189,063 superficial feet, leaving a loss by evaporation and absorption of 15·07 inches. The whole of the water from the drainage area was not available for the use of the town, as one-fourth was reserved for compensation to bleachfields situated on the natural water courses. This amounted to 22,267,735 cubic feet per annum, leaving 66,803,206 cubic feet available, being 183,022 cubic feet per day, or 127·1 cubic feet per minute.

The works consist of a reservoir at Harelaw, having a capacity of 14,248,000 cubic feet, with a conduit leading from thence to Stanely, where there are two other reservoirs, one, to act as a subsiding pond, capable of holding 28,340,000 cubic feet, the other for clear water, with a capacity of 7,194,000 cubic feet, with regulating sluices for turning the water into either. The open conduit

[1] The discussion upon this Paper occupied portions of three evenings; but an abstract of the whole is given consecutively.

between Harelaw and Stanely is the principal feeder for the Stanely reservoirs; in its course it intercepts the burns flowing from Gleniffer braes, which are almost all pasture ground.

There are self-acting compensation sluices at the outlet of the lower of these reservoirs, designed by Mr. Thom to insure an uniform delivery with a varying head of water in the reservoir, which are reported to work satisfactorily. From thence the water is conveyed by a masonry conduit, 2 feet by 1 foot 6 inches, to filters and a covered tank, on an elevation at the southern end of the town. The population in 1853 was about 50,000, so that the supply of water for each person, including manufactories, was $22\frac{1}{2}$ gallons per day.

The growing wants of the town rendered it necessary for the authorities to look out for increased supplies, and, after examining various sources, Parliament was applied to in 1853 for power to bring in the water of the Rowbank burn, which rises on the borders of Renfrewshire and Ayrshire, and which is one of the tributaries of Castle Semple Loch. The average height of the land selected for the reservoir is 500 feet above the ordnance datum, rising in undulating ridges to 700 feet at the water-shed. At this time the water works were the property of a company; but in the year 1855 they were transferred to the Town Council, and the claims of the company were settled by arbitration at an annual payment of $6\frac{2}{3}$ per cent. on the capital. The power obtained in the year 1853 having lapsed, another nearly similar Act was procured in 1866, with the privilege in addition to supply the town of Johnstone and the village of Elderslie, having together a population of nearly 10,000 persons.

The drainage area acquired by the Paisley Water Commissioners contains 1,220 acres, and is partly arable, partly pasture, and $\frac{1}{13}$th of the whole, amounting to 94 acres, is moorland, the water from which is rather mossy at times, but this will be diverted from the store reservoir, making it however available for compensation. Two rain gauges were placed in the neighbourhood, of which a careful register was kept. One of these was at Springside 540 feet above the sea, and the other at Muirhead at a height of 490 feet above the sea. Of the drainage area the rainfall of 700 acres is represented by the Springside gauge, and the Muirhead gauge represents the rainfall of the remaining 520 acres.

The following table shows the amount of rainfall registered in these gauges from December 1865 to December 1866:—

		Springside.	*Muirhead.*
		Inches.	Inches.
1865.	December . . .	9·89	7·75
1866.	January	10·64	8·30
	February . . .	6·13	4·75
	March	2·41	2·15
	April	4·05	3·40
	May	2·12	1·50
	June	2·75	2·10
	July	4·05	2·82
	August	5·30	4·80
	September . . .	7·15	6·00
	October	4·55	3·30
	November . . .	5·35	3·92
	Total. .	64·39	50·79

The quantity of rain falling on 700 acres, the depth being 64·39 inches, is equal to 163,614,990 cubic feet, and on 350 acres, the depth of rain being 50·79 inches, 64,528,695 cubic feet. The average depth of rain over the whole area is 59·86 inches, or 228,143,685 cubic feet, and subtracting the amount measured by the weirs, subsequently mentioned, 179,662,325 cubic feet, there remain for loss by evaporation, &c., 48,481,360 cubic feet, which is equal to 12·72 inches of the rainfall, leaving 51·67 inches available for the high ground and 38·07 inches for the low ground. There now remain 170 acres with a rainfall of 38·07 inches to be added, which yield 23,492,997 cubic feet of water per annum, raising the total to 203,155,322 cubic feet per annum. A store reservoir was constructed at Nethertrees on the Rowbank or Birkcraig burn, about 3 miles south-east of Lochwinnoch; the water area is 100 acres, and the greatest depth 35 feet. The pipes were constructed to carry 184 cubic feet of water per minute, and the compensation water was fixed by the Act of 1866 at 92 cubic feet per minute. Storage was provided for one hundred and eighty days, or six months of this whole quantity, being about 77,000,000 cubic feet.

To test the flow of water into the Rowbank reservoir four gauge weirs were erected on the tributaries, and the water flowing over them was measured every day. These gauges were constructed of battens and stakes carefully levelled and made watertight, with a free overfall, and with sufficient still water behind to prevent inaccuracy from initial velocity. No. 1 was 5 feet long, No. 2 was 2 feet 4 inches, No. 3 was 1 foot 4 inches, and No. 4

was 1 foot long. The depths being taken, were calculated by the formula $Q = 4{\cdot}904bd^{\frac{3}{2}}$, where

Q = cubic feet per minute.
b = breadth in feet.
d = depth in inches.

Discharge over Weir No. 1.

		Cubic Feet.
1865.	December . .	11,868.501
1866.	January . .	27,950,143
	February . .	20,615.473
	March . . .	5,295,857
	April . . .	2,806,319
	May . . .	2,437,603
	June . . .	682,548
	July . . .	1,395,731
	August . . .	12,935.485
	September . .	19,509,420
	October. . .	8,878,606
	November . .	15,108,363
	Total . .	129,484,049

Discharge over Weir No. 2.

		Cubic Feet.
1865.	December . .	3.663,190
1866.	January . .	8,362,040
	February . .	5,527,766
	March . . .	1,935,562
	April . . .	1,193,935
	May . . .	706,027
	June . . .	282,342
	July . . .	606,695
	August . . .	3,815,273
	September . .	5,452,188
	October . .	2,563,874
	November . .	4,250,171
	Total . .	38,359,063

Discharge over Weir No. 3.

		Cubic Feet.
1865.	December . .	724,632
1866.	January . .	1,573,001
	February . .	1,095,960
	March . . .	426,618
	April . . .	261,213
	May . . .	142,191
	June . . .	66,155
	July . . .	108,320
	August . . .	537,995
	September . .	753,857
	October . . .	528,143
	November . .	1,044,194
	Total . .	7,262,279

Discharge over Weir No. 4.

		Cubic Feet.
1865.	December . .	391,684
1866.	January . .	821,538
	February . .	558,440
	March . . .	208,062
	April . . .	135,557
	May . . .	86,192
	June . . .	58,152
	July . . .	104,125
	August . . .	695,941
	September . .	726,494
	October . .	313,352
	November . .	457,397
	Total . .	4,556,934

The total flow per annum over all the weirs was :—

	Cubic Feet.
For No. 1	129,484,049
" No. 2	38,359,063
" No. 3	7,262,279
" No. 4	4,556,934
Total . .	179,662,325

A conduit $6\frac{1}{4}$ miles long conducts the water to the Stanely filters.

whence it is conveyed to Paisley by a 16-inch pipe. A branch pipe leaves the main 3 miles west of Paisley to supply the towns of Johnstone and Elderslie; and a set of filters and a tank were constructed at Craigenfeoch for filtering the water supplied to these places. Another set of filters and a tank are placed on the high ground to the south of the original reservoirs at Stanely, with a branch pipe leading down to them, to make up any deficiency that may occur in the old works.

Store Reservoir at Nethertrees.

To impound the water it was found necessary to construct three embankments; the largest of which is situated across the bed of the burn. The first operation consisted in the formation of a bye-wash channel, to divert the water of the Reivoch burn from the reservoir during the construction of the bank, as it was from this burn that floods were apprehended; and it now serves for carrying the water of that burn past the reservoir, should it be at all impure owing to floods. When this was finished the outlet tunnel (Plate 4, figs. 5—7) was proceeded with. The purpose of this was, in the first place, to discharge the waters which would have accumulated during the construction of the bank, and to receive the two outlet pipes, one of which carries the compensation water, and the other the water for the town. The tunnel is 426 feet long, a length of 150 feet of which, at the lower end, was open at first, and afterwards covered in, the remaining 276 feet being tunnelled through rock. The interior dimensions of the tunnel are 5 feet 6 inches by 5 feet 6 inches. It has vertical side walls and a semicircular roof. The whole length of the arch was built of moulded brick. Where in open cutting the side walls were 15 inches thick, and the arch was built of the same thickness, set in mortar, with a rubble stone arch outside, 9 inches deep; and where in tunnel the side walls, for a length of 236 feet, were built of brick, and the space between the wall and the rock was filled in with close-packed rubble stone set in mortar. A length of 40 feet at the inner end, where the rock was friable whin, was built wholly of brick set in cement, the brickwork filling up the entire space to the rock. This portion had a brick invert varying from 9 inches to 15 inches in thickness set in cement, the remainder of the floor of the tunnel being natural rock, dressed off as smoothly as possible. The rock varied in quality from what is locally called Osmond, being like the hardest whinstone, to a soft grey, granulated, sedimentary substance, easily cut with a knife. It required blasting, and in some places the roof had to be supported until the building was finished. At the lower end of the tunnel is the sluice house (Plate 4, figs. 8—11), 10 feet square, with an arched roof 10 inches thick, and side walls 3 feet thick, in which are placed three sluices for directing the water into the town, or for diverting it into the burn. At the inner end of the tunnel is a horseshoe-shaped recess of masonry (Plate 4, figs. 1—4), in which is placed the iron upstand or sluice shaft. This recess is 10 feet 9 inches long by 5 feet 9 inches broad, with walls 2 feet 6 inches thick. Across the front are lintels 2 feet 6 inches by 1 foot 3 inches in section, and, again, in front of these is a groove for holding a wooden grating, which may be replaced by stop planks, when access to the sluices is required. Across the bottom, and 2 feet 6 inches above the floor, is a stone 3 feet by 1 foot 9 inches in section, on which stands the iron sluice-shaft, and below which passes the pipe conveying the compensation water. For a length of 17 feet at the upper end, the tunnel is of larger dimensions, being 7 feet 6 inches by 5 feet 6 inches, and tapering to 5 feet 6 inches by 5 feet 6 inches (Plate 4, fig. 12). This portion was filled up with masonry round the pipes, after the embankment was completed, to make it watertight; and round the upstand, up to the level of the ground, it was filled in with clay puddle and covered with pitching. Leading to this upstand is a channel 5 feet 9 inches wide (Plate 4, fig. 1), with side walls varying from 2 feet 6 inches to 3 feet 6 inches thick, with cross lintels 1 foot square to keep the walls apart, and the bottom is pitched with 9-inch pitching set on a bed of concrete 6 inches thick. The ashlar was procured from Shillford quarries, 4 miles south-east from the reservoir. Provision was made in the contract for filling up the tunnel with clay round the outside of the pipes, but this has not been required as the solid masonry at the upper end is watertight.

The greatest depth of the principal bank is 60 feet (Plate 4, fig. 19), and the length 500 feet along the top, which is 5 feet above high-water level, and is 10 feet broad. The slopes are 3 to 1 inside and $2\frac{1}{2}$ to 1 outside. The puddle is 8 feet broad at the top, and increases with a batter of 1 in 8 on each side down to the level of the ground, from which point it diminishes to one-half that width at the bottom of the trench. The puddle trench is 62 feet deep at the deepest part. To form a proper foundation, all soft material was stripped off the site of the bank, including a considerable accumulation of peat and silt at the bottom of the valley, which was excavated down to the clay or rock before the bank was commenced. The greatest depth under the surface of the valley was 17 feet on the outer, and 22 feet on the inner side. During the excavation it was found that the moss, on the inner side, was so soft that it would not stand even with a moderately flat slope; and it was also threatening to cause a leak in the temporary bank across the valley. To obviate this, and to enable the moss and silt to be readily cleared out, a row of piles was driven at the inner toe of the embankment. The broken nature of the rock forming part of the puddle trench rendered it necessary to excavate the hills on both sides to a considerable depth.

The material for the bank was found on the site of the reservoir, and consisted of clay, which, when mixed with the rock taken from the excavation of the puddle-trench, formed a good and substantial bank. To facilitate the work, a short tramway was laid from the north end of the bank to the place where the materials were procured. The wagons were worked by a small locomotive engine, and the stuff, having been tipped on a loading bank, was removed in common tip carts. The banks were then formed with a slope inwards towards the trench of 1 in 12. Care was taken to spread all stones and keep them separate, so that earthy matter might fill up the interstices. The layers, each 6 inches thick, were pressed and trodden down by carts and horses passing frequently over them, and were pounded with beaters where the carts could not work. No planks or rails were allowed in forming the banks, and in dry weather water was poured over the whole surface to make it settle.

The wagons for conveying the puddle were also worked by the locomotive engine. A staging, carrying rails, having been formed along the side of the trench, the wagons ran along it by their own gravity, and the clay puddle was tipped into the trench; it was then spread in thin layers, mixed with water, and properly cut and worked up by being tramped on by navvies. After undergoing this process, it formed a compact mass quite impervious to water. When the slopes of the bank had been made according to specification, and had settled, the inside slope was covered with a layer of broken stones, over which was laid pitching of hard blue whinstone. On the outer slope, and on the top of the bank, was laid a layer of stones 3 inches deep to keep out moles and rats, over which a layer of soil was dressed off, and sown with rye-grass and clover seeds. The natural slopes between two of the banks were pitched with rough pitching, set on a layer of broken stones.

The other banks were formed in the manner already described, but they were of smaller dimensions, one being 230 feet long, and 14 feet deep, and the other 815 feet long, and 18 feet in depth.

The waste weir at the south end of the large bank was 40 feet long, being at the rate of 1 foot in length for every 30 acres of drainage area. The side walls are 3 feet high on each side of the weir, and the channel, which has a gradient of about 1 in 6, has been cut out of the solid rock, with a width at the bottom of 10 feet.

It was originally intended to strip the entire surface of the inside of the reservoir, as the presence of vegetable matter was considered objectionable; but the cost led the Commissioners to dispense with the operation. The quality of the impounded water, however, has been decidedly deteriorated by the omission of this operation, though in the course of time the prejudicial effects of the presence of organic matter in the bottom of the reservoir will probably cease to be felt.

When the bank and waste weir were finished, two parallel lines of 21-inch pipes were laid through the tunnel; at the inner end one was connected to the bottom of the cast-iron upstand shaft, and the other passed under the stone carrying the sluice shaft, and was bolted to the sluice for giving out compensation water. The space under the stone was then built up. These pipes, which were in 12-feet lengths, were lowered by a crane on a bogie at the sluice-house end of the tunnel; a tramway having been laid through it, the pipe was then run up to the place required, and, when on the bogie, it was used as a ram to drive the preceding pipe tight home.

The sluice-upstand, in the horseshoe recess, is made of cast iron (Plate 4, fig. 1). It is 2 feet 6 inches in interior diameter, of $\frac{3}{4}$-inch metal, cast in five pieces, with flanges bolted together. It is about 35 feet high, and there are four sluices at different levels. The sluice openings are 17 inches square, and are fitted with double brass faces. The pipe for the town supply is connected to this cylinder, and at a lower level is the pipe for the compensation water, with the rod for working the sluice on the end of it, passing up in front of the iron cylinder. The sluices are worked by a moveable brass nut working on a $2\frac{1}{4}$-inch screw. The compensation water is discharged into the burn, across which is placed a gauge weir, to measure the amount of water. The water for the town is discharged into a cast-iron well with an overflow to take the pressure off the clay pipe, which leads from it towards Paisley.

Pipe Track.

The total length of pipe track from Rowbank reservoir to Stanely is 11,126 yards. For a distance of 3,872 yards this track has a gradient of 1 in 700, and is laid with 3,021 yards of 21-inch clay pipes, 76 yards of iron pipes in moss, with a few iron pipes at the burn crossings, and there are 765 yards of masonry aqueduct where the track is in deep cutting. The second portion of the track is supplied with cast-iron pipes, of which 3,986 yards are 18 inches in diameter, and 367 yards 16 inches. The third portion has 16-inch clay pipes for 2,700 yards, laid at a gradient varying from 1 in 140 to 1 in 70, and 200 yards of iron pipes. The portion from Stanely to Paisley, 2,895 yards in length, has 16-inch iron pipes. The pipe for supplying Johnstone and Elderslie leaves the 18-inch main near Craigenfeoch, and is 8 inches in diameter to the filters, from thence it is 10 inches to Thorne, from which place there is a branch to Johnstone 8 inches in diameter, and another to Elderslie 5 inches in diameter. The track for the pipes was excavated 1 foot wider at the bottom than the exterior diameter of the pipe, with slopes varying according to the quality of the material; opposite each faucet a clear space of 6 inches was left all round, to permit of the proper jointing of the pipes. When the cutting was in rock, the pipes were laid on a bed of earth 3 inches deep. Where the clay pipe track was through a porous material, the pipes were surrounded with clay puddle 12 inches thick. The clay pipes were jointed in the following manner:—Two strands of rope-yarn, steeped in thin cement, were wrapped round the spigot and caulked in after being inserted into the faucet; then the remainder of the faucet was carefully and closely filled up with cement, which was bevelled out from the end of the faucet along the outside of the pipe, with a slope of 1 to 1, and when practicable, as in the case of the 21-inch and 16-inch pipes, a boy was sent in to point the inside of the joint with cement.

Many engineers of experience have a prejudice against the use of clay pipes; but the successful results obtained in this, as well as in many similar places, warrant a word being said in their favour, wherever there is a constant fall and no pressure on the pipes. They should be found to answer the purpose well, provided sufficient care is taken in selecting those perfect in form and without cracks or flaws, especially at the neck where the faucet is fastened on to the body of the pipe, and where a crack is likely to be found. Care must be taken, too, that they are properly jointed, and that the thin cement is not shaken out of its place during the operation of refilling the track, a probable result if it is done before the cement has had time to set. Above all, they should not be laid in too deep cutting, as the superimposed material is certain to break and crush them; nor should they be subjected to any pressure from a head of water.

The great fault found in the pipes was a liability to crack at the junction of the faucet with the body of the pipe. A method was devised in order to test their soundness, when that could not be ascertained by ordinary inspection. The pipes were placed on a wooden platform, with the faucets downwards, and inserted in a thin bed of clay carefully worked so as to be water-tight. The pipes were then filled with water obtained from a pit close by. With a head of 3 feet of water some of them were found to leak, though the greater number were perfectly tight. The cracks in those which leaked were carefully pointed with Portland cement inside and outside. When the cement had set, they were again subjected to the water test, and for the most part they were now found to be water-tight; those that still leaked were rejected.

Where clay pipes were used in cuttings above 9 feet deep, a relieving arch of rough rubble was formed over them to protect them from crushing. Where the depth of cutting exceeded 12 feet, a masonry aqueduct was substituted for the clay pipe, the sectional area of which was 3 feet by 2 feet (Plate 4, figs. 16—18). The soles were of Thornhill pavement about 3 inches thick, which was set flush in mortar on a bed of levellings and well pointed. The sides consisted of parapet ashlar, procured from Shillford quarries, 9 inches broad, with the faces scabbled and the backs left quarry-faced; and the covers were of pavement from 3 inches to 5 inches thick, with a rest of 6 inches on each wall. Where part of the conduit was in treacherous ground, the soles and covers were checked, so as to keep the walls apart should there be any tendency to force them together. Great care was taken in filling the space behind the ashlar with clay and soft material, and a depth of 1 foot 6 inches to 2 feet of earth was placed on the top of the covers to protect them in filling in the cut, which in most cases was in rock. Where the track passes under streams an iron pipe is substituted for the clay pipe. This is built round with rubble, over which is placed hammer-dressed pitching 10 inches or 12 inches deep, and in the centre, over the pipe, pavement is laid of a thickness and extent depending on the size of the stream. One stream is crossed by a bridge of 16-feet span (Plate 4, figs. 13—15). The arch stones are 15 inches deep, and the side walls are tied together with bond stones with a hold of 12 inches at each end.

The clay pipes were provided by Messrs. Brown, Ferguslie Fire Clay Works, near Paisley, and were of the following dimensions, all being 3 feet long, exclusive of the faucet:—

Internal diameter.	Thickness.	Depth of faucet.
12 inches.	1 inch.	4 inches.
15 "	1 "	4 "
16 "	$1\frac{1}{4}$ "	$4\frac{1}{4}$ "
21 "	$1\frac{1}{2}$ "	$4\frac{1}{2}$ "

The faucets are of $1\frac{1}{2}$ inch greater diameter than the outside of the pipes, and are $\frac{1}{8}$ inch thicker than their body; the shoulder is $\frac{1}{4}$ of an inch thicker than the body of the pipes, and both spigot and faucet are grooved to make them hold the cement.

The iron pipes were supplied by Messrs. D. Y. Stewart and Co., of Glasgow, and were 12 feet long exclusive of the faucet. The principal dimensions and weights of these pipes were as follow:—

Interior Diameter.	Length of Pipe, exclusive of Faucet.	Length of Pipe, inclusive of Faucet.		Thickness of Body of Pipe.	Weight of each Length.		
Inches.	Feet.	Feet.	inches.	Inch.	Cwts.	qrs.	lbs.
5	12	12	4	$\frac{1}{2}$	3	0	15
8	12	12	$4\frac{1}{4}$	$\frac{9}{16}$	5	1	26
8	12	12	$4\frac{1}{2}$	$\frac{5}{8}$	6	0	15
10	12	12	$4\frac{1}{2}$	$\frac{5}{8}$	7	2	6
10	12	12	$4\frac{1}{2}$	$\frac{11}{16}$	8	1	13
16	12	12	$4\frac{3}{4}$	$\frac{3}{4}$	14	1	5
17	12	12	$4\frac{3}{4}$	$\frac{3}{4}$	15	0	16
18	12	12	$4\frac{3}{4}$	$\frac{3}{4}$	16	0	0
18	12	12	$4\frac{3}{4}$	$\frac{13}{16}$	17	1	15
21	12	12	5	$\frac{13}{16}$	20	0	22
22	12	12	5	1	26	0	18

The pipe-joints were, for the most part, turned and bored, and the pipes were laid in the following manner:—The spigots were wiped clean, and were coated with fresh Portland cement of the consistency of paint made up immediately before being used. They were then inserted into the faucets and the pipe driven home by repeated blows, in the case of the smaller pipes from a wooden mallet, and in that of the larger pipes with the next one slung as a ram, in which case a piece of wood was interposed to keep the iron from striking iron. The lead and yarn joints were made after the spigot was inserted, by caulking the faucet hard with sound rope-yarn up to within $2\frac{1}{2}$ inches of the outside, and filling the remaining space with melted lead, which was hard staved so as to be water-tight.

The pipes were tested with the pressure of a column of water, which for a pipe

5	inches	in diameter and	$\frac{1}{2}$	inch	thick, was	600	feet high.
8	,,	,,	$\frac{9}{16}$	,,	,,	300	,,
8	,,	,,	$\frac{5}{8}$	,,	,,	600	,,
10	,,	,,	$\frac{5}{8}$	,,	,,	300	,,
10	,,	,,	$\frac{11}{16}$	,,	,,	600	,,
16	,,	,,	$\frac{3}{4}$	,,	,,	300	,,
17	,,	,,	$\frac{3}{4}$	,,	,,	300	,,
18	,,	,,	$\frac{3}{4}$	,,	,,	300	,,
18	,,	,,	$\frac{13}{16}$	,,	,,	400	,,
21	,,	,,	$\frac{13}{16}$	,,	,,	300	,,
22	,,	,,	1	,,	,,	400	,,

While under pressure they were repeatedly struck with a hand hammer, and any pipes sweating or leaking were rejected. The uniformity of their thickness was also tested by calipers designed for the purpose.

Filters.

Two filters for the supply of Johnstone and Elderslie were constructed at Craigenfeoch, each 45 feet by 32 feet, and the tank was 50 feet by 26 feet and 13 feet deep (Plate 5, figs. 1—3).

The walls of the filters and tank have a foundation course 8 inches thick, and are built of good flat rubble bedded in mortar, and the face stones of the tank and of the filters above the level of the sand are of chisel-draughted ashlar. The tank walls are 3 feet 6 inches thick, at the level of the platform, and the filter walls are 3 feet thick; both have a batter on the inside of 1 in 12.

As the excavation consisted for the most part of porous rock, the whole area of the building was well grouted with mortar run into every crevice, and the floor of both filters and tank, including half way through the walls over the foundation course, was covered with a layer of clean gravel, 4 inches thick, grouted flush with Portland cement. The retaining walls were brought up with a void of 4 inches in the heart, with two dovetailed recesses to form a tie opposite each other 12 inches by 6 inches by 6 inches for every square yard of surface. These voids were filled with clean gravel in layers of 6 inches connected with the concrete of the floor, and each layer was grouted with Portland cement. The result was an excellent water-tight wall, the only objection being the cost, which amounted to from 40*s.* to 50*s.* per cubic yard. The floor of the tank was covered with pavement 3 inches thick, laid flush in mortar and pointed with cement, and an area of 6 square yards under the inlet pipe was laid with ashlar 9 inches deep, caulked on the joints with iron-rust cement. There are two semicircular wells at the outlet of the filters, with sluices for regulating the head of water over the filters during filtration. The filters have each a 12-inch clay pipe along the centre, with branches and 4-inch cross-pipes laid with open joints to admit the water, and with an iron air-pipe at the end of each. The filtering material consists of a bed 2 feet deep of coarse gravel, small enough to go through a 2-inch ring, but not through a $\frac{3}{4}$-inch ring; the upper surface is in ridges and furrows 6 inches deep and over that is a layer 6 inches deep of clean gravel which will go through a $\frac{3}{4}$-inch screen, but not through a $\frac{3}{8}$-inch screen; over this is a layer of slate chippings 6 inches deep, then a layer of coarse sand 6 inches deep, and lastly a bed 18 inches thick of fine clean sharp sand, dressed into the prescribed form of ridges and furrows (Plate 5, fig. 3). The water is admitted into the filters by feeding troughs along the side farthest from the tank, from which it passes through sluices and feeding chests into the feed pipe, and is delivered from a trumpet mouth at the level of the sand, which prevents any disturbance of the filtering material.

The roof of the tank is of wrought iron with T bar rafters and struts, and round tie and suspension rods, 6 feet apart, braced diagonally, resting on and bolted to a cast-iron wall plate, and having L lathes $8\frac{1}{2}$ inches apart for the slates. The slates, which are Welsh seconds, 20 inches by 10 inches, are fastened on by copper wire to the lathes, overlapping 3 inches.

The mortar employed was Arden lime well burned and ground, mixed in the proportion of two and a half parts of lime to two of sand and one of mine dust.

The high level filters and tank erected at Stanely are of the following dimensions: three filter beds each 90 feet by 60 feet, and a tank 138 feet by 38 feet and 14 feet deep. They are constructed on the same principle as those above described, the only difference being that the walls of the tank are 4 feet 6 inches thick at the top; all the walls inside batter 1 in 10, but for economy the concrete groove was dispensed with, and on the outside of the walls clay puddle was substituted for it.

Of the original plan, the low level filters below Stanely but higher than the present old filters at Calside, which are of insufficient area and on too low a level, have not been executed.

The Engineer for the works was Mr. James Leslie, M. Inst. C.E.

The Contractors for the reservoir at Nethertrees were Messrs. Alexander Wilson and Son, Granton, and the cost, inclusive of sluices and iron-work, was £17,433. The same Contractors executed the filters for Johnstone and Elderslie, and also the branch pipe from the main aqueduct, at a cost of £7,554. Mr. John Pollock, Bathgate, was Contractor for the pipe track from Rowbank to Stanely, which cost £15,414, and for the Stanely filters, which are not yet completed. Messrs. D. Y. Stewart and Co., Glasgow, made the iron pipes for all the pipe tracks, and laid that portion between Stanely and Paisley.

The total cost of the scheme, including price of land, way leave, parliamentary and engineering expenses, amounts to about £70,000.

The communication is accompanied by a series of Drawings, from which Plates 4 and 5 have been compiled.

APPENDIX.

Cost of Constructing Filters.

Craigenfeoch	320 yards, cost	£1,164 =	£3 12 9	per superficial yard.
Stanely . .	1,800 „ „	3,087 =	1 14 3	„ „

These were constructed with built walls. The Glencorse and Torduff filter-beds were banked without building, and belong to the Edinburgh Water Works.

Glencorse . . .	2,547 yards, cost	£3,063 =	£1 4 0	per yard.
Torduff	3,070 „ „	3,873 =	1 4 0	„
(Dundee) Stobbsmuir	3,350 „ „	4,150 =	1 4 9	„
Liverpool			2 10 0	„
Greenock			2 2 6	„

The cost of the service tanks was :—

Craigenfeoch	15,600 cubic feet, cost	£809 =	1*s.*	per cubic foot.
Stanely	68,544 „ „	2,537 =	$8\frac{3}{4}d.$	„
(Dundee) Lawton Reservoir	60,000 „ „	1,633 =	$6\frac{1}{4}d.$	„
Blairgowrie	7,605 „ „	380 =	1*s.*	„

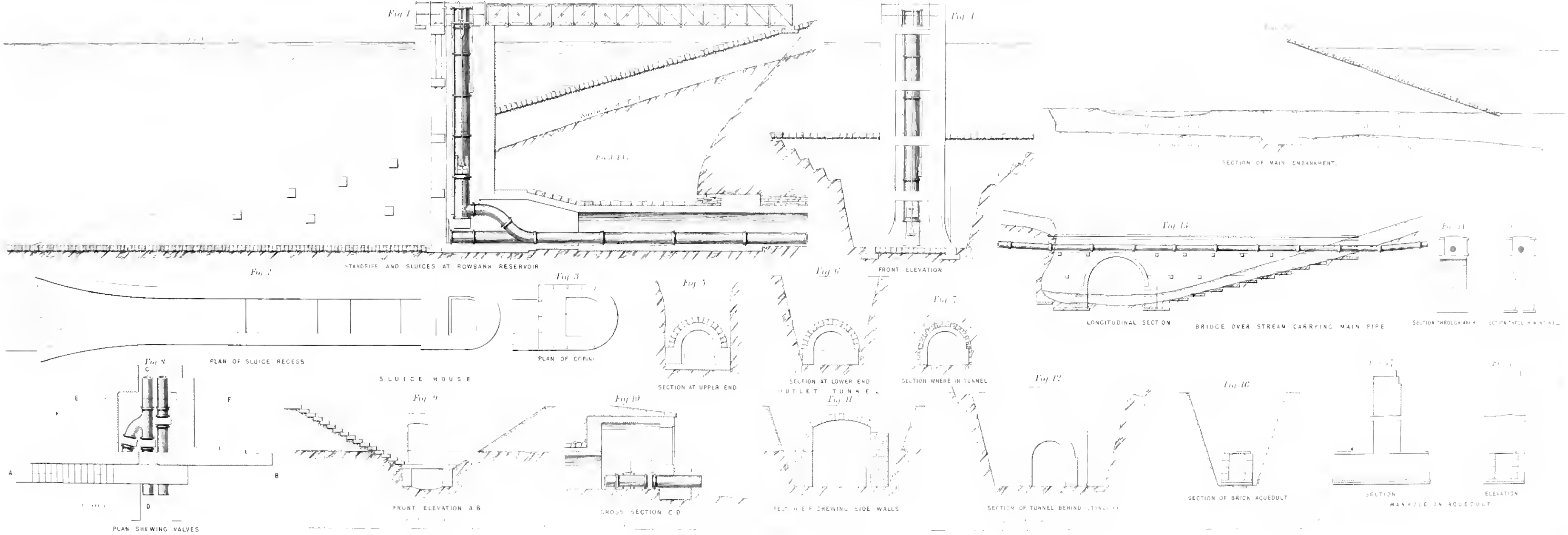
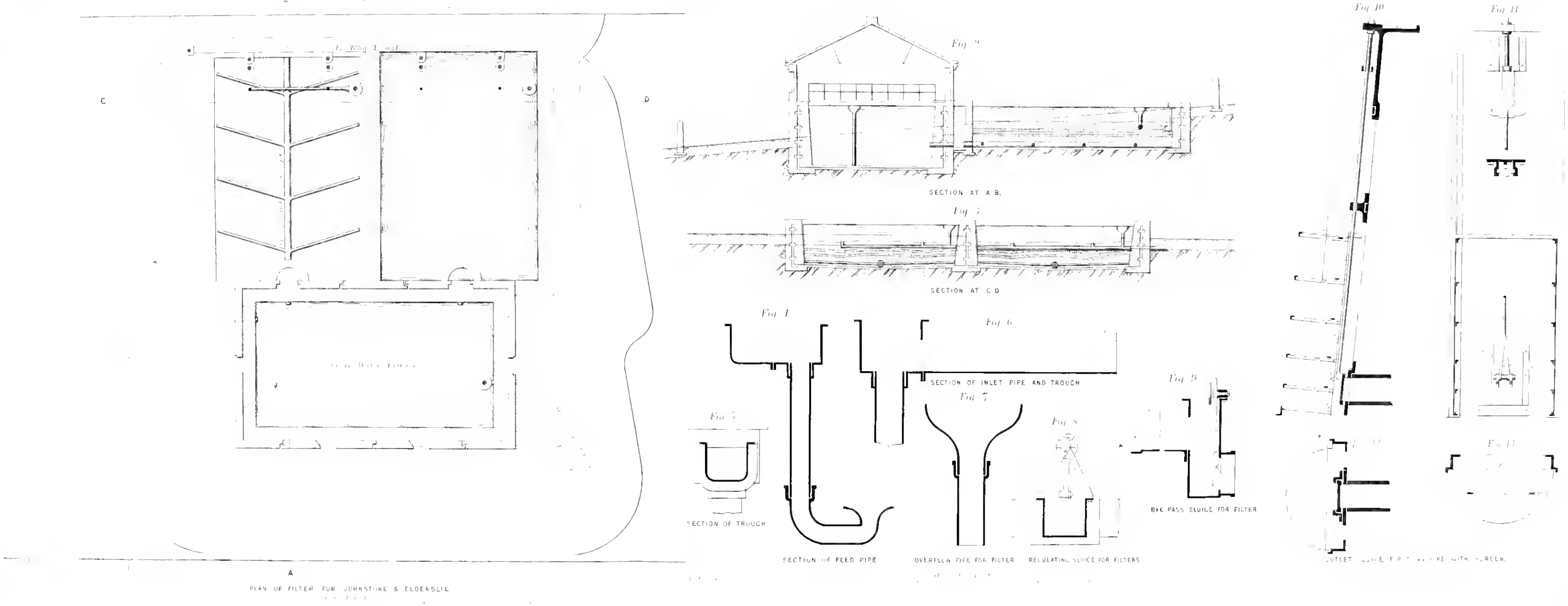


Fig 1
STANDPIPE AND SLUICES AT ROWBANK RESERVOIR
FRONT ELEVATION
SECTION OF MAIN EMBANKMENT.
Fig 13
LONGITUDINAL SECTION
BRIDGE OVER STREAM CARRYING MAIN PIPE
Fig 2
PLAN OF SLUICE RECESS
Fig 3
PLAN OF COPING
SLUICE HOUSE
Fig 5
SECTION AT UPPER END
Fig 6
SECTION AT LOWER END
Fig 7
SECTION WHERE IN TUNNEL
OUTLET TUNNEL
Fig 8
PLAN SHEWING VALVES
Fig 9
FRONT ELEVATION A B
Fig 10
CROSS SECTION C D
Fig 11
Fig 12
SECTION OF TUNNEL BEHIND
Fig 16
SECTION OF BRICK AQUEDUCT
SECTION
ELEVATION
MANHOLE ON AQUEDUCT

PAISLEY WATER SUPPLY.



The cost of filters, including all necessary works for storage, &c. :—

Craigenfeoch . . .	£3 16	per square yard.
Stanely	7 10	„

Sometimes reservoirs are estimated at so much per 1,000,000 cubic feet, or gallons impounded. A reference to the accompanying Table will show how uncertain and unsatisfactory any such mode of procedure must be :—

Contents.	Name.	Catchment.	Banking.	Puddle.	Total.
Cubic Feet.		Acres.	Cubic Yards.	Cubic Yds.	Cubic Yds.
25,000,000	Crosswood* . . .	2,113	51,856	29,739	81,595
32,000,000	Crombie Den† . . .	. .	62,031	22,671	84,702
70,000,000	Paisley	1,220	A 53,203	7,082	
			B 2,714	1,066	
			C 10,190	3,646	
			66,107	11,794	77,901
6,560,000	Dunfermline † . . .	370	23,864	6,971	30,835
26,000,000	Craigton, New . .	. .	142,977	31,544	174,521
18,100,000	Torduff*	868	153,397	17,915	171,312
10,500,000	Clubbie Dean* . . .	235	53,413	11,519	64,932
8,500,000	Bonally*	100	25,277	8,018	33,295
19,000,000	Logan Lea*	1,426	82,430	15,108	97,538
90,000,000	Harperrig*	4,217	38,587	12,464	51,051
136,000,000	Greenock, Upper . .	1,904			
65,000,000	„ Lower . .	796			
		2,700	128,605	29,817	158,605
27,000,000	Harelaw*	3,934			

* Torduff	Embankment,	one-fourth of contents.	
* Clubbie Dean	„	one-sixth	„
* Bonally	„	one-ninth	„
* Crosswood	„	one-eleventh	„
† Crombie Den	„	one-fourteenth	„
Rowbank	„	one-thirty-ninth	„
* Harperrig	„	one-sixty-fourth	„

Those marked (*) belong to the Edinburgh Water Works, and those marked (†) belong to the Dundee Water Works.

[Mr. J. Glaisher

Mr. J. Glaisher remarked that, although more or less familiar with the rainfall in every part of the country, yet he was best acquainted with that in the district of the Royal Observatory at Greenwich. He would in the first place direct attention to a table showing the annual rainfall at the Royal Observatory for each of the years from 1815 to the end of 1869:—

Year.	Rain.	Year.	Rain.	Year.	Rain.	Year.	Rain.	Year.	Rain.
	Inches.		Inches.		Inches.		Inches.		Inches.
1815	22·5	1826	23·0	1837	21·0	1848	30·2	1859	25·9
1816	30·1	1827	24·9	1838	23·8	1849	23·7	1860	32·0
1817	29·0	1828	31·5	1839	29·6	1850	19·7	1861	20·3
1818	25·7	1829	25·2	1840	18·3	1851	22·7	1862	26·5
1819	31·1	1830	27·2	1841	33·3	1852	34·2	1863	19·8
1820	27·7	1831	30·8	1842	22·6	1853	29·0	1864	16·8
1821	34·5	1832	19·3	1843	24·6	1854	18·7	1865	28·6
1822	27·7	1833	23·0	1844	24·9	1855	21·1	1866	30·1
1823	27·1	1834	19·6	1845	22·4	1856	22·2	1867	28·5
1824	36·3	1835	24·9	1846	25·3	1857	21·4	1868	25·2
1825	24·6	1836	27·1	1847	17·8	1858	17·8	1869	24·0

It would be observed that there was a great difference in the fall of rain at Greenwich between one year and another, and even of consecutive years, and the difference was relatively as great in all other parts of the country. In 1824 the amount registered was 36·3 inches, whilst in 1864 it was only 16·8 inches. The average fall of all the years was 25·3 inches, and at times, for several consecutive years, as from 1819 to 1824, the fall for each year was above the average; and at other times, as from 1854 to 1858, the fall for each year was below the average. Now, no reservoir had been made which would balance the rainfall for more than three years, so that the falls of rain in consecutive wet years could not be impounded under any circumstances; in fact, very heavy falls of rain were of no use, either to millowners or anybody else, as such rain took the shortest course to the sea. Evidently, therefore, the quantity to be dealt with must be less than the average. The safest principle appeared to be to treat with the amount of the water fallen in three consecutive years yielding the least. This amount was found to be nearly the average of all the years reduced by one-sixth. Thus at Greenwich the sum of the falls of rain in the three years ending 1857, viz., 61·4 inches, being smaller than in any other three years in the period, or 20·5 inches on an average of these years, the average, 25·3 inches, reduced by $\frac{1}{6}$th, or 4·2 inches, was 21·1 inches, differing from the observed by 0·6 inch. Taking this as a base, the capacity of storage reservoirs should be for two hundred days. But as water was growing more important and more valuable the need was becoming greater and greater for economising it, in order to supply large towns with continued service. He would refer especially in this respect to the town of Norwich. The late Mr. Brooke had told him that, many years ago, when the quantity drawn was equal to 40 gallons per head per day, the supply was insufficient, and the water was shut off from 8 P.M. to 6 A.M., and the company were obliged to send persons round at night to be in readiness in case of fire. The contrast at present was great indeed. In 1868 the city had a constant service, and the quantity of water used was only 16 gallons or 17 gallons per head, including what was consumed for trade purposes. This was entirely the result of economising.

He would next call attention to another table of each rainfall at Greenwich which had amounted to at least 1 inch per day during the last fifty-five years:—

Year and Date.	Amount of Rainfall.	Year and Date.	Amount of Rainfall.	Year and Date.	Amount of Rainfall.
	In.		In.		In.
1816 June 28	1·32	1831 Sept. 28	1·09	1853 July 14	2·63
1818 May 10	1·34	1832 Mar. 14	1·21	,, July 28	1·11
,, Sept. 26	1·30	,, July 12	1·01	,, Oct. 27	1·05
1819 Sept. 29	1·00	,, Oct. 7	1·47	1854 Aug. 3	1·40
,, May 5	1·10	1833 Aug. 30	1·14	1855 July 11	1·42
1820 Jan. 20	1·13	,, Dec. 23	1·10	,, July 26	1·15
,, May 16	1·10	1834 July 29	1·44	,, Oct. 30	1·06
,, July 31	1·51	1835 May 13	1·00	1856 June 20	1·00
,, Sept. 18	1·38	,, Oct. 25	1·14	1857 Aug. 14	1·12
1821 June 8	1·17	,, Oct. 30	1·00	,, Sept. 8	1·00
1822 April 16	1·11	1837 Jan. 26	1·10	,, Sept. 11	1·16
,, July 5	1·40	,, Aug. 23	1·10	,, Oct. 22	2·57
,, Oct. 19	1·16	1838 Sept. 27	1·10	1858 June 5	1·16
,, Nov. 16	1·12	1839 Nov. 27	1·20	1859 Sept. 26	1·26
1823 Jan. 29	1·07	1841 June 24	1·03	1861 May 11	1·07
,, July 25	1·05	,, Sept. 23	1·03	,, Nov. 13	1·29
,, Oct. 1	1·15	,, Oct. 27	1·03	,, Nov. 22	1·00
,, Oct. 31	1·15	1842 Aug. 10	1·10	1862 April 9	1·10
1824 Feb. 14	1·18	1843 Aug. 23	2·16	,, Aug. 17	1·27
,, May 16	1·25	1844 Oct. 15	1·38	1863 June 19	1·46
,, Aug. 14	1·63	,, Nov. 8	1·03	1865 Jan. 27	1·25
1825 May 13	1·40	1846 Sept. 23	1·18	,, May 23	1·03
,, Sept. 17	1·37	1848 June 12	1·43	,, June 30	1·39
1826 Mar. 7	1·00	1849 May 28	1·15	,, Aug. 23	1·79
,, July 24	1·97	1851 Mar. 15	1·45	,, Oct. 19	1·06
1828 July 22	1·21	,, July 23	1·44	,, Oct. 22	1·11
,, Aug. 8	1·00	1852 June 9	1·36	1866 Jan. 11	1·61
,, Aug. 14	1·21	,, July 25	1·99	,, June 4	1·34
1829 April 9	1·00	,, Aug. 15	1·08	1867 July 26	3·67
1830 June 3	1·38	,, Oct. 4	1·01	1868 Jan. 22	1·21
1831 Feb. 7	2·89	,, Nov. 26	1·00	,, May 29	1·08
,, Sept. 1	1·16	1853 June 13	1·15		

In this period there were eleven years in which the daily rainfall did not amount to 1 inch. The fall exceeded 1 inch, on five days in 1852 and on six days in 1865. Only one instance occurred in which the fall exceeded 3 inches, viz., on July 26th, 1867, when it reached 3·67 inches. The mean monthly fall of rain at Greenwich was least in February and largest in October, whilst at Aberdeen the least was in May and the greatest in November; and in fact the month of least rain proceeded from Greenwich going northwards in March, April, and May successively. The monthly fall of rain at Greenwich and at Aberdeen was—

Month.	Greenwich. Inches.	Aberdeen. Inches.
January	1·85	2·21
February	1·56 (Min.)	1·65
March	1·60	1·90
April	1·74	1.68
May	2·15	1·47 (Min.)
June	1·96	2·02
July	2·60	2·22
August	2·41	2·51
September	2·42	2·31
October	2·80 (Max.)	2·79
November	2·35	2·93 (Max.)
December	1·94	2·45

The distribution of rain at times differed greatly from the average. At Greenwich in the year 1868, in which there was an average fall, there were ten months of deficient rainfall, and the balance was made up by a great excess in the months of January and December.

By taking five-yearly means during the period from 1815 to 1869, he found a generally decreasing rainfall at Greenwich till the year 1859, but since then there had been an increase:—

						Inches.
In the five years ending		1819	the mean annual fall was			27·68
„	„	1824	„	„		30·67
„	„	1829	„	„		25·84
„	„	1834	„	„		23·98
„	„	1839	„	„		25·28
„	„	1844	„	„		24·74
„	„	1849	„	„		23·98
„	„	1854	„	„		24·64
„	„	1859	„	„		21·68
„	„	1864	„	„		23·08
„	„	1869	„	„		27·28

The deficiency to 1859 was therefore in no way attributable to an excess of drainage or clearance of trees, for the amount of drainage within the last few years was greater than at any preceding period.

Respecting evaporation, from all the experiments he had seen, it appeared to amount to from 13 inches to 15 inches per annum; therefore in the three driest years all the available water would be the difference between the mean of those years and from 13 inches to 15 inches, which would leave but little in one of those years to work upon.

Mr. G. J. Symons had been an observer of rainfall for many years, but had not specially inquired into the accuracy of the Greenwich register till a short time since, when, at the request of Mr. Dines, who had detected certain inaccuracies, he examined into the correctness of the early portion of the series of observations given in the table produced by Mr. Glaisher. His remarks applied to a period anterior to that at which Mr. Glaisher became connected with the Royal Observatory. Mr. Beardmore had referred to the early portion of that register in terms which were not altogether expressive of confidence.[1] The remarks of Mr. Beardmore referred to the total annual rainfalls, whilst the point which he had examined into was the daily fall. With regard to the table of heavy daily rainfalls at Greenwich, he might remark that the only way in which a list of that sort could be reliable, and useful, was where it was certain that the gauge was visited and emptied every day. Whenever that was done the numbers were comparable amongst themselves and with others. If there was any possibility of the fall of two days having been entered as one, the value of these numerical data was entirely destroyed. Upon the point which he was required to investigate were dependent a series of calculations made by Mr. Glaisher, also by Mr. Dines at Cobham, and by Mr. Chace in America, as to the influence of the moon's age upon rainfall. On examining into these early daily returns, it was found that the number of days on which rain was recorded to have fallen, during the ten years from 1820 to 1830, was about thirty-five per annum less than the average of the whole period. That might be due to something peculiar in the climate of those ten years. He had therefore compared this result with the registers of rainfall at Chiswick, at Cobham, at the Royal Society, and with Howard's register at Tottenham; and they all agreed in indicating a larger number of days of rainfall in those years than were recorded at Greenwich. Again, if a long-continued register like this of fifty-five years was taken, and the total rainfall during a month divided

[1] *Vide* "Manual of Hydrology," p. 282, foot note, 8vo. London, 1862.

by the number of entries, it would give the mean fall of each day on which rain was reported to have fallen. He applied that process to the Greenwich observations for the month of January during the fifty-five years, and the mean daily quantity during that period was about 0·15 inch; but during the period from 1820 to 1830 or 1831 the amount ran up to nearly 0·30 inch; consequently the average fall on each day of entry was nearly twice as great as at any other period. That implied that the small amounts during those periods were not registered separately. This did not show whether the rainfall was systematically taken at longer intervals, for instance, if the rain-gauge was emptied regularly once a week; but by tabulating the entries he found that that was not the case; consequently it was not an alteration in the rule that the rainfall should be measured every day, but negligent observance of it. One further test applicable to this case was the regular measuring of small quantities. Again, referring to the register of rainfall in January, there were on an average four or five days in the month on which a depth of 0·05 inch or less than that quantity was recorded; but from 1820 to 1829 or 1830 he found there was not an average of more than one day or a day and a half. These different investigations all tended to show, that the small quantities were not registered, but that they were allowed to accumulate till they became large ones; and, in fact, they proved what had been previously suspected, that the early records of rainfall at Greenwich were unworthy of confidence. He agreed with Mr. Glaisher as to the shifting of the epoch of minimum monthly fall of rain travelling north from London to Aberdeen; but the case was different in mountainous districts. In the hill districts of Cumberland or Wales, and even in Derbyshire, instead of the maximum rainfalls occurring in July and October, they occurred in December and January.[1]

He thought that a great deal of confusion had arisen with respect to the use of the term evaporation, as applied to the loss of water. A certain amount of rain fell on a gathering ground, and a certain amount was stored: the difference between the two was not necessarily due to evaporation from the surface, but arose to some extent from percolation.

Mr. J. GLAISHER said he had every reason to believe, from the records that were left, that every rainfall between 1820 and 1830 was registered at the Royal Observatory at Greenwich. The

[1] *Vide* "Report of the British Association, 1868," p. 135, *et seq.*, 8vo. London 1869

gauge was made by Troughton with the utmost care, and was as accurate in its construction as it was possible to be, and that was more than could be said of other gauges used at that distant date.

Mr. S. C. HOMERSHAM said, in the neighbourhood of the metropolis there were extensive districts, embracing an area exceeding 4,000 square miles, where all the rain that fell was absorbed by the chalk formation. Railway engineers could bear testimony, that large valleys in chalk districts were frequently crossed without there being any necessity for a single culvert. The rain was absorbed by the chalk as fast as it fell, though it might be as much as 3 inches in an hour. Over a water-shed consisting of 200 square miles, distant not 30 miles from London, not a drop of rain that fell appeared on the surface in the shape of springs or streams. The observations of Mr. Glaisher would not apply there because the whole of the water, whether in a wet year or in a dry year, was absorbed and stored under ground in the pores of the chalk. If there was a heavy rain, followed by very hard frost, then a fall of snow, and then a sudden thaw, water might flow off the surface in chalk districts; but such a concurrence of circumstances was very rare. The surface of the ground must be saturated, the water must be frozen, a fall of snow must cover the frozen surface, and sudden thaw take place with rain; then there were floods off those districts he was speaking of, otherwise there was no flow of water off the surface.

The question of the natural storage of subterranean water in the chalk, the lower green sand, the oolite, the red sandstone, and other similar formations, was one of great importance. Those were the formations which gave water of the best quality for domestic purposes, and with which surface water impounded in natural lakes or artificial reservoirs could bear no comparison. It was free from organic matters, and was of an uniform temperature the whole year through, and it could be artificially rendered as soft as any water that could be supplied to a town.

Mr. HAWKSLEY, Vice-President, said, before entering into the particular merits of the Paper and its very general subject-matter, which was far more important than the mere description of the works which had been made for the supply of Paisley with water, he would refer to the statement of Mr. Homersham. The Paper related to the supply of water from gathering grounds, by means of large storage reservoirs. Now the chalk district was not a gathering ground, and no storage reservoirs were ever made in such districts. The consequence was, though the observations of Mr. Homersham were perfectly correct in themselves, yet they had no bearing upon the subject-matter of the Paper. Excellent discussions might be raised upon the means of obtaining water from chalk districts, but that had nothing to do with mountainous water-sheds.

The first thing which presented itself to an Engineer, charged with the duty of obtaining a supply of water for a large community, was to ascertain the quantity of water which could be obtained from any of the gathering grounds in the neighbourhood. That was an important inquiry, and an inquiry about which many mistakes had been made. Formerly it was thought possible that, by means of reservoirs, the average of all the rainfall of a long period of years might be calculated upon for use by the community, with the exception of a small deduction to be made for what had been called 'loss,' but which was in general nothing more than evaporation. Now, in the course of years, it was found, in tho first place, that the allowance for evaporation had been considerably too small; and next, that it was impossible to store the floods of a series of wet years, or even, very frequently, of one year only. The endeavour had therefore been made to discover what were the real facts of the case, and what was the law by which Nature dealt with this important subject-matter. It was found, after long experience, and after laying down in curves the results of the observations over long periods—observations which had been afforded by the outlay of millions of money, in some cases successfully, but in the majority of cases more or less unsuccessfully—that in general, but not as an invariable rule, there were situations in which reservoirs could be made that would deal with the available rainfall of three consecutive minimum years, The next thing was to ascertain what was the law of the minimum; and this singular thing came out, which was true within the smallest possible fraction of all long series of rain-gauge observations extending over not less than twenty years,—that if the average of say twenty years was taken, and from that average one-sixth was deducted, the average of three minimum dry years would be obtained, within the fraction of a single inch. That quantity might then be relied on as the probable rainfall. The amount of loss or evaporation had then to be ascertained, according to the particular district. That loss varied in these islands from 10 inches per annum as a minimum to 18 inches as a maximum. The minimum occurred very rarely—indeed only in the case of bare precipitous mountains, consisting of nonabsorbent rock, such as slate or granitic rock. From that surface all the rain that fell could be gathered, with the exception of about 10 inches. But the case was very different where the surface was covered with soil and peat, where it became flat moor-land on the summit, and, more so, where the land was cultivated and thrown into the character of a sponge. In general, however, with mountain water-sheds, where the intermediate condition existed, the actual ascertained loss amounted to from 13 inches to 15 inches per annum, according to the situation and some local circumstances, and might be taken at a mean of about 14 inches per annum. As a practical illustration, he would take perhaps the most important one that had ever presented itself in this kingdom, the case of Sheffield. In that instance a population of a quarter of a million was entirely dependent upon mountain water-sheds. There were correct records for forty years, from which it was known that, upon the average, the fall of rain had been $39\frac{1}{2}$ inches annually. Deducting one-sixth, there remained about 33 inches; and taking from that 14 inches as the loss from what was called evaporation, there were left 19 inches, which was the actual quantity received into the storage reservoirs on a mean of three dry years. From that again had usually to be deducted, before the water could arrive at the town, a very large amount for the supply to the millers on the stream on which the reservoirs were made. Formerly, the legislature was in the habit of giving too large a proportion; but after longer experience, that had now been fixed at one-third of the available quantity. That, in the case of Sheffield, would have been $6\frac{1}{3}$ inches, leaving $12\frac{2}{3}$ inches for the use of the town. However, in consequence of the law of the deduction of one-sixth not having been known, and the loss by evaporation having been underrated—only a small deduction being made for loss—the legislature had given the millers 12 inches, thus reversing the proper figures, and leaving only 7 inches for the town. Under those circumstances it could not be wondered at that the town fell periodically short of water. He had been thus precise in dealing with this particular case, because it was typical of a great number of other cases, and because it was owing to mistakes upon this point that three-fourths—nay, five-sixths of all the large towns dependent upon gathering grounds had been left short of water, sometimes for many weeks together; and, at all events, their daily supply had been diminished for two or three months whenever the rainfall was less than the average.

After having ascertained, by a process of that kind, the actual, as distinct from the theoretical, quantity which could be depended upon from the rainfall, the Engineer had next to consider the magnitude of the basins which must be provided for impounding that supply. As was well known, rain fell very irregularly; so that if there were not large storage reservoirs, the floods would rush away at one period of the year, and the town would be left dry, with the exception of the little rills formed by permanent springs at another period. That was the condition in which, forty or fifty years ago, the towns in the mountainous districts of this kingdom really were. Engineers began with the idea that it was impossible a drought could last in England more than 100 or 120 days, so reservoirs were made capable of holding a supply equal to the requirements of from 100 to 120 days. That plan having failed, the reservoirs were increased in size so as to store a supply for 140 days, and subsequently for 160 days, when the failures, although not so numerous, still occurred in the majority of cases. An inquiry was then instituted to ascertain whether there was any law particularly applicable to the case; and, by dint of perseverance and of an extensive range of observation, it was found that the least proportionate storage was necessary in those parts of the kingdom where there was the greatest rainfall, and that the largest amount of storage was necessary in those parts of the kingdom where the least quantity of rain fell. The reason of this was—that where the greatest quantity of rain fell there was the greatest number of wet days, and where the least quantity fell there was the least number of wet days. Moreover, where the greatest quantity of rain fell, as a rule there was the least evaporation; and where the least quantity fell, as a rule, but not without some exceptions, there was the greatest amount of evaporation. Therefore it became apparent that a rule which was applicable to one part of the kingdom would not be available in another part of the kingdom. It was then discovered that when the rainfall amounted on the average, as at Sheffield, to 40 inches per annum, storage must be provided for more than 180 days' consumption, and then the reservoirs would be just run out at the end of the longest drought. Coming farther south, and taking Leicester by way of example, where the rainfall was somewhat under 30 inches, a supply for 180 days would not last; and it had been found by observation that there was no re-elevation of the surface of the water in these reservoirs for 250 consecutive days. In the east of England, where there was a still less fall of rain, a continuous supply of water to the towns could not be depended upon unless there was 300 days' storage. In places where the rainfalls exceed 40 inches, smaller reservoirs in proportion were required; while in the very driest parts of the kingdom, where the rainfall was only 22 inches, it was necessary to impound a supply for a longer period. The result was, it had been found, according to observation in England, that where the greatest quantity of rain fell a supply must be provided for 150 days; and where the least quantity of rain fell, 300 days' storage was absolutely necessary.

The determination of the quantity of water to be calculated upon as the use and waste of the town was one of the most serious of all the calculations, because that quantity had to be multiplied by 150, 180, 200, or 300 days, as the case might be, and the reservoirs must be made larger exactly in proportion to the quantity of water which it was probable would be consumed in the place, while, of course, there must also be a larger area of catchment ground. Now, the facts upon that question were most extraordinary. The consumption varied in large cities, where there was a constant supply, day and night, and every person drawing as much or as little as he pleased, between 15 gallons per head per diem and 100 gallons per head per diem, including in both cases the supply for manufacturing and sanitary purposes. No more water was wanted in the city where the quantity was 100 gallons per head per diem than in the town which was served with 15 gallons per head per diem. The cause of the difference was simply in the management of the undertaking. In many places the company or the corporation, or whoever might be the parties supplying the water, merely turned the water into the pipes, and left the care of the internal fittings and the mode of their application entirely to the consumer, or to the builder, or to the landord, as the case might be. The result was, as a rule, the worst possible character of fittings, and every cistern supplied with an overflow-pipe; and where there was an overflow-pipe, as a matter of course the ball-cock, which let in the water, was never attended to, because, whether it was right or wrong, nobody on the premises suffered the least inconvenience. The consequence was the ball-cock got out of order; it would not rise to shut off the water, and the water ran down the waste-pipe day and night. The same thing happened with regard to those abominations called soil-pans, and also to water-closets, the handles of which were propped up, under the idea of "doing good to the drains." The result was the water ran away without anybody being sensible of the loss. But when the consequences were considered, it would be at once apparent how important it was that those sources of waste should be suppressed; for, as a rule, every million gallons per day supplied to a city from a gathering ground cost in capital about £120,000, and ought to be capable of supplying 50,000 people. Now, there were places, particularly in Scotland, where the consumption amounted to 50 gallons per head per diem; so that, to supply the same number of people, an outlay would be required of £300,000. And it must be remembered that the taxation for the supply of water must, of course, be in proportion to the outlay. But there were still more important difficulties. It frequently happened that it was not possible to obtain in the neighbourhood of the town a sufficient supply to meet that amount of waste, and hence it had to be brought from a long distance. The area of the gathering ground must also be two or three times as great, and the reservoirs must be two or three times as large, or two or three times as many as would otherwise have been necessary; and altogether the affair became so onerous, that it was not surprising that in general there was great reluctance to encounter the expense, and still greater reluctance to support the taxation which that expense necessitated. All this could be remedied, but it was almost impossible to convince public bodies of this fact. At the present time many places were in great difficulty, by reason of the supposed want of a sufficient quantity of water, but where in reality there was plenty of water to effect every object which the law required, if only this extravagant waste was suppressed. There was still a further evil. In the majority of cases, where a constant supply was not enforced by law, the companies and the corporations, and particularly the local boards, refused to give the constant supply because of the enormity of the waste with which it was frequently attended.

In the metropolis the three millions of inhabitants were receiving more than 30 gallons of water per head per diem—a quantity far more than was necessary—and yet the water was rarely supplied for more than half an hour, and scarcely ever for more than one hour, out of every twenty-four hours. If the necessary care were taken, and improved fittings were applied, there might be a constant service during the whole of the twenty-four hours, and everybody might have all he wanted, although less than 30 gallons per head per diem would be used.

With respect to the tables which had been supplied by Mr. Glaisher, he regarded them as being of the utmost importance. One showed the average rainfall at Greenwich over a period of fifty-five years, and gave a good idea of the great variability of this climate; it also applied relatively to the case of the mountain gathering grounds in the north of England.

In reply to the question why the water taken into the houses should not be supplied by meter, rather than by the present defective system, Mr. Hawksley observed that the reason was a very plain and a very powerful one—the law did not allow it. It had been the policy of the legislature, ever since the formation of companies under legislative authority, to require that houses should be supplied for a payment proportionate to their rentals. It had been endeavoured, a great many times, to induce the legislature to adopt the system of supply by meter with respect to water, as in the case of gas. The legislature had invariably refused; and the reply always made to the application was, that it would not tend to the good and comfort of the people, or to their health, but that on the contrary it would be injurious to the poor if water were sold by measure. Water was allowed to be sold by meter for manufacturing and other non-domestic uses; but with regard to domestic purposes, the legislature was obdurate.

Mr. S. C. Homersham said there was no rule without an exception. Mr. Hawksley had said positively that the legislature would not allow water for domestic purposes to be sold by meter. Now he had obtained an Act in 1862 to construct works to supply a large district near London. The Act allowed the domestic supply to be by meter, and water had been supplied ever since by meter. The company charged according to rental, and if any person exceeded a certain quantity he was charged 2*s.* per 1,000 gallons in excess.

Mr. J. A. Longridge stated that he was anxious to get water by meter, and had applied to the Lambeth Water Works Company for this purpose in vain. They did not tell him that it was not allowed to do so by the Act, but they said that they would not. He thought if there was no Act of Parliament against it, and if water companies complained of the great waste of private consumers, there could be no reason why a man who was willing to pay for what he got should not be allowed to do so.

Mr. Beardmore expressed his doubt as to the practical success of meters for private houses. He thought they would cause dissatisfaction generally. The charge for supply to the poor would be greatly increased by the rental of the meter, and there would be perpetual quarrels as to damages; but they were used in the East of London and other places for manufacturing purposes.

Mr. Harrison, Vice-President, observed that when he was a member of the Royal Commission for reporting on the question of the water supply of London, no subject which had come before them was of greater importance than the general want of a good and sufficient water supply for the poor. From the returns of the large towns they found that wherever there was a superabundant supply of water there was large waste. In the case of Glasgow the consumption was 50 gallons per head per day; but in some places where the strictest economy was used the supply was under 20 gallons per head. He felt satisfied that, if strict economy were observed, which could only be effected by strict supervision, 20 gallons per head would be found to be a full and sufficient supply in nearly every case. But it was next to impossible for any private company to exercise that inquisitorial supervision of the supply, which was so necessary, in private houses, to ascertain where the waste took place; and thoroughly efficient supervision could only be exercised by placing the water supply in the hands of the public themselves.

Mr. Hawksley, Vice-President, said, the result of an immense amount of experience in the management of water companies was, that only in one was the water supply managed economically by a public body; while there were an immense number of cases in which they were managed very economically by private companies. At Norwich, where there was a private company, the supply of water was unlimited; yet the consumption did not exceed from 15 to 16 gallons per head per diem. That was only typical. Glasgow was in the hands of a public body, and there the consumption of water was 50 gallons per head per diem; and that was only typical again. Many public bodies were in the same position. The water ran to waste, through water-closets and overflow-pipes, for which there was no use; but public electoral bodies were exactly the people who dared not go into the houses of the electors to stop the waste which a company was able to suppress.

Mr. Beardmore said that the rainfall was as variable as the climate of England itself. Where the country was flat the fall was more uniform than where it was hilly, and generally in much less quantity.

There was perhaps no other science so difficult as meteorology whence to generalize from observed facts, and those who had to apply meteorological records to elucidate practical questions of water supply, drainage, and the economy of rivers, should be very cautious in accepting isolated facts. A long series of observations over wide ranges of country was required to develope the law of rainfall in respect to its periodical variations.

It was very doubtful, for instance, what provision should be made for floods, in carrying out engineering works—what was the maximum beyond which it was a waste of labour to provide; and where the line should be drawn between floods which might be controlled and extraordinary *débâcles* which overwhelmed all the works of man, and against which it was useless for the Engineer to contend. It was not safe to assume that the highest flood or the most severe drought in any man's experience would not be exceeded even in this country; and this was still more likely to be the case in India and Australia. Nor could he accede to the deduction of broad generalizations from the Greenwich series of rainfall observations, since there might be instances in the future when the results deduced from selected exceptional periods of the past might be nullified by conditions of which there was no present experience. For instance, in November 1852, there were nine days of constant rain in the Midland and South Eastern districts of England, and this rainfall was considered to be unprecedented. At that time the Great Northern, and some other neighbouring railways, had been but recently opened, and it was found that sufficient provision had not been made for the safe passage of the flood waters. Yet subsequent events proved that the magnitude of that flood might be, indeed had been, greatly exceeded in the districts served by those railways.

Intelligence had just been received that the Chey-Air bridge on the Madras railway had been carried away by a flood. The spans were large, and doubtless every reasonable provision was made; yet the experience of the past had been insufficient, and a time had come when the victorious waters had proved the peril of trusting to deductions from data extending over a limited period.

The time occupied in the fall of rain had an important bearing both on the replenishing of reservoirs and on the regulation of floods. There might be a year of abundant rainfall, yet, from its occurring in occasional heavy storms, and in summer, the water supply might still be deficient.

The years 1857 and 1858 were certainly the driest in modern times, taking into consideration the wide area over which the absence of rain was experienced. On the Continent the deficiency was chiefly felt in 1857, but in England in 1858; and the fall in some places did not reach 60 per cent. of the average. That drought extended over the whole of Central Europe, and was most severe over the area including the sources of the Weser, the Elbe, the Rhine, and the Danube. The foundations of a Roman bridge were laid bare on the latter river, the existence and presumed locality of which were only previously known from the writings of Pliny. And the springs suffered so greatly in Westphalia that they did not recover their full volume till three years after. Similarly the drought of 1868, which did not at the time seriously affect the flow of water in the English rivers, was probably one cause of the want of water in the past summer of 1870.

Seasons had a certain tendency to run in cycles, and one of about eleven years seemed to show more uniform maxima and minima than any other grouping.

The ridge of hills south of Paisley extended from the frith of Clyde opposite Arran to a point about 4 miles south of Glasgow, and these hills were subject to a very heavy rainfall, but this was confined to a limited area, probably not exceeding 2 miles in breadth.

Mr. HAWKSLEY, Vice-President, observed, that the apparatus which had been adopted for rendering the constant supply of water successful, by suppressing the waste, to which otherwise it would be subject, was distinct from the apparatus used for intermittent service. In constant service pressure was applied to the pipes during the whole twenty-four hours, instead of only during a very small portion of that time. Now, if a service pipe in the interior of a house would bear pressure for half an hour, it would bear the same pressure for half a year; but in the case of the constant supply forces came into operation which did not usually operate in the intermittent system, or only to a small extent; for where the supply was intermittent, the draught during the short time the water was on, owing to the majority of the ball-cocks in the houses being open, very much diminished the pressure; and besides that there were few or no shocks. But upon the system of constant supply the pipes were subjected to all the shocks occasioned by the rapid closing of the cocks, whereby the column of water was suddenly arrested when in rapid motion. That brought on a considerable amount of impulsive action which was unknown, or little known, in the case of an intermittent supply; and it was constantly found that the pipes leading into the houses, and distributed through the houses, although perfectly competent to bear the pressure of the intermittent supply, would not bear that of the constant supply when it became introduced in place of the intermittent supply. From this it followed that wherever the constant supply had been introduced, either voluntarily or by the pressure of the legislature, it had been found necessary to adopt rules and regulations for determining the magnitude and thickness of the pipes; also the mode in which the pipes should be united, and the kind of tap and ball-cock and water-closet apparatus to be used in connection with the constant pressure.

The rules and regulations now found to be necessary, and very generally adopted, were reduced to writing.[1] Formerly univer-

[1] The Rules and Regulations recently adopted by the Rochdale Corporation Water Works are as follow:—

"1.—The Corporation will, at their own cost, lay down and maintain all the sally, and still in most cases where the intermittent supply was used, a common plug tap was generally applied. This had the

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lead or other branches extending from their main pipe to the side of the public highway in which such main pipe is situate; and will, at their own cost, carry the pipe through the frontage wall (if there be one), and six inches beyond, or otherwise equivalently allow fifteen inches in length for the owner's or occupier's plumber to connect his work to.

"2.—The owner or occupier must, at his own expense, lay down and maintain all the pipes and apparatus upon his premises or for his use, and of the strengths and descriptions, and subject to the rules following, that is to say:—

"A.—Such pipes must, unless otherwise agreed, be of lead, and of not less than the following weight, namely:—

inch.	lbs.
$\frac{1}{2}$	7 per yard.
$\frac{5}{8}$	9 „
$\frac{3}{4}$	11 „
1	16 „
$1\frac{1}{4}$	$22\frac{1}{2}$ „

NOTE.—*Detective or warning pipes* may, if desired, be used of lighter weights than the foregoing.

"B.—The drawing (bib) stop and ball cocks must be strong and of hard brass, and the better to secure watertightness, of the kinds from time to time sanctioned and approved by the Committee; and unless and until due notification to the contrary, the drawing cocks must be of the best and most approved kind of those called 'screw down cocks,' and in principle as manufactured by Messrs. Guest and Chrimes, and in courts of houses and other exposed places, must be protected by an iron casing, and be made to open with keys. And the ball taps must also be of the best and most approved kind, and in principle as manufactured by Messrs. Lambert and Sons. Till otherwise notified, no other description of cock must be used without the previous and express permission of the Water Works Committee.

"C.—Every cistern must be absolutely watertight, and be provided with a ball cock, and proper means of access and inspection, *but must not have an overflow or waste pipe;* and if any such should exist, the same must be removed, or be effectually and permanently closed before the water is turned on; but nevertheless, as exceptional instances will occasionally occur in which it will be necessary to provide against the possibility of over-filling, the Corporation will, in such exceptional instances, allow a *detective or warning pipe* to be attached to the cistern, provided that in every such case a written consent must be first obtained from the Manager of the Water Works, stating the fact of such consent, and the position in which the *detective or warning pipe* must be fixed; and in every such case the work must be executed under the immediate superintendence of an officer of the Corporation, and in the manner stated. On no account whatever can the water of the Corporation be allowed to communicate with any cistern or place intended or used for the reception of *rain water.*

"D.—WATER CLOSETS.—Every *pan closet* must be provided with a full and complete apparatus, comprising a ball cock and a service cistern, fitted effect, on rapid closing, of suddenly arresting the column of water in the lead service pipe, and gave rise to two or three

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with a boot or division, to be carried as high as the top of the cistern, and capable of containing not more than one and a half gallons of water, when filled within three inches of the top, and two proper valves, so arranged as to let down not more than one boot or division full of water at each pull, or be capable of allowing the water to run to waste either by intention or neglect; and must also have a down pipe of lead from the cistern to the basin of not less than $1\frac{1}{4}$ inch in diameter, and weighing 9 lbs. to the yard run; and a proper basin, scatterer, weighted lever, pan, trap, and other appliances, needful to prevent such water closet from becoming a nuisance, and thereby inducing an undue consumption of water; and the valves must be worked by brass rods instead of by wires or chains. Every *self-acting, or pull-down water closet*, must be of a description approved by the Water Works Committee, and must have either a lead cistern similar to a pan closet, or a double valve cast-iron service box, of a kind approved by the Committee, and fitted with a proper cover to screw on, and internal apparatus in all respects similar to that of the boot of the pan closet above described, and a similar down pipe of lead or cast iron, and must have a proper wide-rim flushing basin and trap, of a kind approved by the Committee. No wire will be allowed to be used in the construction of these water closets.

"NOTE.—No pipe will be suffered, under any pretence whatever, to communicate directly or indirectly with the basin, or trap, or otherwise than with the *cistern* or *service box* of a water closet or soil pan, and the same shall be so constructed and used as to prevent the waste or undue consumption of water, and the return of foul air, and other noisome or impure matter, into the mains or other pipes of the Corporation.

"E.—Every bath must be constructed without an overflow or waste pipe, and must be provided with a well-fitted and perfectly water-tight apparatus, to prevent the water from flowing into and out of the bath at the same time. With the view to prevent damage from accidental overfilling a *detective or warning pipe* will be permitted, subject to the regulations and conditions hereinbefore-mentioned with respect to cisterns.—(See regulation C.)

"F.—No pipe must be laid through, in, or into any *sough, drain, ash pit, manure hole,* or other place, from which, in event of decay or injury to such pipe, the water of the Corporation might be liable to become fouled, or to escape without observation, or without occasioning the necessity of immediate repair. In every case in which any such sough, drain, ash pit, manure hole, or other place as aforesaid, shall be in the unavoidable course of the pipe, such pipe shall be passed through an exterior cast-iron pipe, or box, of sufficient length and strength to afford due protection to the water pipe, and to bring any leakage or waste within the means of easy detection.

"G.—Every pipe and apparatus laid and fixed by, or for the use of the consumer, must be inspected by an officer of the Corporation before it is connected to the works of the Corporation: and, if found not in accordance with the regulations of the Committee, must be forthwith removed or altered.

violent reactions. In time the lead pipe expanded into a sort of aneurism, and ultimately burst by a long slit, exactly as an

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"H.—Every meter (unless otherwise specially agreed) must be provided with a separate and distinct *inlet* pipe, leading from the main or other pipe of the Corporation—upon which *inlet* pipe no stop cock, except the stop cock (if any) immediately attached to the meter, branch, drawing cock, or other outlet, leading to or connected with the premises for the supply of which such meter is fixed, will be permitted.

"I.—No pipe will be suffered, under any circumstances whatever, to communicate directly with a steam boiler.

"3.—The water supplied must not be allowed to run to waste, either wilfully or by neglect; nor must it be used for any other purpose, or to any greater extent, than shall have been agreed for.

"4.—No pipe must be attached to the works of the Corporation, or to any pipe or apparatus connected therewith; nor must any alteration be made in any existing pipe or apparatus, without due notice being given to, and the consent of the proper officer of the Corporation being first obtained.

"5.—The supply and use of water for the purpose of trade and manufacture must be open to inspection and admeasurement whenever required; and such information must be from time to time afforded, as will be sufficient to enable the Committee to obtain a satisfactory account of the quantity of water actually consumed; and of the pipes, cocks, cisterns, and other apparatus, and conveniences for delivering, receiving, and using such water.

"6.—The Corporation will, if, and when so desired, execute all kinds of plumbers' work connected with the supply of water to their tenants, but are nevertheless desirous that the private business of the consumers of water shall be open to all the plumbers of the town; as, however, it is essential to the protection of the interest of the consumers, as well as of the Corporation, that such work shall be well and soundly executed, and that the Water Works Committee shall possess a full and satisfactory knowledge of the state of the undertaking in all its departments, it is announced that no plumber or other workman will be allowed to do or perform any work connected with the supply of water, till he shall have been admitted, enrolled, and published by the Committee, as 'an authorized water works plumber,' and shall have entered into a written engagement to conform to and comply with the rules and regulations of the Committee in relation to the construction and management of the works and fittings to which such rules and regulations shall from time to time apply; and all responsible master plumbers, on expressing their willingness to comply with such rules and regulations, will be admitted immediately on signing an undertaking to that effect. If at any time afterwards any such plumber shall be found guilty of wilfully breaking or evading the said rules or regulations, either by himself or his workmen, or shall refuse to communicate any information required of him in regard to any work done by him, or his workmen, or under his superintendence, or on his responsibility, his name shall be erased from the list of 'authorized plumbers,' and will be forthwith advertised as having been so struck off.

"7.—No person is to be employed in or about the Water Works, or any pipe or apparatus connected therewith, who has not been admitted 'an authorized plumber;' or whose name shall have been struck off the list as aforesaid.

artery under similar circumstances burst in the human body. Thus the introduction of constant supply led to the flooding of houses, damage of furniture, and destruction of property in many ways. But that had been entirely got over, and a cure established by the introduction of a screw-down cock in lieu of the old plug tap. This closed slowly against the pressure of the water, and prevented recoil. Also, by reason of the looseness of the face, the leather, which was interposed for the purpose of making a perfect valve, did not turn round on its face with the revolution of the screw; and consequently it was not ground or worn away as when the leather turned round with the screw. These valves did not lead to the bursting of the pipes, and were besides perfectly water-tight, which other valves were not; consequently the continuous trickle, which was often observed in other valves, and amounted to a serious quantity, did not occur. Moreover, the leathers could be replaced at about the cost of a penny, and so the cocks would last, with very little expense to the householder, for a considerable number of years. Probably, however, nine-tenths of the whole waste of water arose in the water-closets, and in every case where the constant supply had been attempted without a special apparatus to prevent the enormous waste which otherwise occurred in the water-closets, there had been a failure of the constant service system. In cases where there had not been total failure the leakage had brought up the supply from under 20 gallons per head per diem to 50 gallons or more; and in one town, with which he was well acquainted, the amount of water distributed and wasted through water-closets had amounted to 110 gallons per head per diem. Now an apparatus had been arranged, and was largely in

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"8.—The Committee will pay a reward of *twenty shillings* to any person who will give such information as shall lead to the conviction of any person who shall fraudulently attach any pipe or pipes to the pipes of the Corporation; or to any pipe, cistern, or apparatus connected therewith; or to or into which the water of the Corporation shall flow or proceed; or who shall fraudulently use or otherwise misappropriate the water of the Corporation, or who shall knowingly permit the same to be fraudulently used or otherwise misappropriated.

"The Committee will also adequately remunerate any person (not being interested therein) who will communicate timely information to their officers of any leakages or wastes of water, and whether the same be accidentally, negligently, or wilfully occasioned or suffered.

"10.—The Corporation do not permit their officers, servants, workmen, or agents, to solicit or receive any fee or gratuity whatsoever, and desire to be informed with respect to any infraction of this regulation; and also in respect to any act of incivility, or any neglect of attention on the part of such officers, servants, workmen, and agents, or any of them."

use, that had completely removed this difficulty. The vessel might be of any size, and of any material. The water was introduced into the vessel in the usual manner by means of a ball-cock; and the vessel was divided into two parts, one of which held a regulated quantity of water. By a particular contrivance two valves were worked, so that one valve always closed before the other valve opened. When the valve opened from the larger division, the water passed out of it into the smaller division, and was ready to be drawn for the use of the water-closet. On pulling the wire, the valve in the larger division descended, and closed the aperture; and no more water could therefore pass from the larger division into the regulating division. All this was done in an instant; but the wire being now pulled a little farther, the other valve began to open, and a flush of water immediately descended and cleared out the basin, leaving the regulating vessel empty. The down pipe leading to the basin was of considerable size to admit of a powerful flush of water, but the regulating cistern was only refilled when the pull-wire was allowed to return to its original position. The apparatus was not necessarily expensive, and might be made of iron for cottages at a cost of 35*s*. At Norwich, where this apparatus was in general use, the reduction of the expenditure of water had been from 40 gallons per head per diem to 15 gallons per head per diem; and no one had ever found fault with its action.

There was another apparatus suitable for supplying water to manufactories, railways, and numerous large establishments. The water in these cases was formerly supplied by contract; nobody knew the quantity that was taken; and, as a matter of course, owing to the waste pipes and to general negligence, much more water was allowed to run to waste than was paid for. Years ago, however, it became apparent that some means must be contrived for measuring the water. For this purpose numerous meters had been invented. Many of them resembled a small high-pressure steam-engine, and some of them, particularly Kennedy's meter, measured the water with considerable accuracy. But they were exceedingly cumbersome; they produced in many cases considerable shocks; and when they got out of order, often allowed the water to pass by without measurement. One beautiful instrument had been invented by Mr. Siemens, M. Inst. C.E., but that instrument was not strictly a meter, but rather an indicator; for it was made upon the principle of the turbine, and the quantity of water measured was determined by the number of revolutions of a wheel put in motion by the water itself. That meter was very useful for many purposes, but it failed more or less with particular descriptions of water, and with particular modes of drawing the water. The meter had apertures through it, which allowed the water to pass, whether the revolution of the wheel was accomplished or not; so long as all went right the meter indicated with tolerable accuracy the quantity of water which passed through it; but if, as happened in many cases, the instrument became furred by deposits from the water, the meter worked slower, and at last stopped. The water, however, did not stop; consequently the meter did not then register the quantity of water which passed through it.

The two meters he had alluded to were called high-pressure meters; they would act under any pressure of water, and they would, with a slight diminution of pressure, convey the water to any place that might be required. But there was another called the low pressure or Cressley meter—one of singular accuracy, and it had not the defects of which he had spoken. The principle on which it worked was that of the common gas meter reversed. In this meter the measurement was due to the number of revolutions of a partitioned drum. Below the drum was a kind of trough; and the water, entering the meter, was compelled to pour over the edges of the trough. The quantity of water propelled in one revolution was therefore determined by the depth at which the drum revolved in the trough. The adjustment was effected by raising or lowering the trough by means of a small screw; and when the accurate measurement was arrived at, it was soldered down, and became immovable. The meter was then sent from the maker to the water company or consumer, and constantly remained in the same state of adjustment. This meter had the apparent defect—for it was not a real one—of measuring the water only at a low pressure. The water must go through the meter into a cistern, the pressure would therefore be only that due to the elevation of the cistern, instead of the pressure due to the elevation of the reservoir of the water company. That was an advantage to the consumer, as well as to the company; for, in the case of a manufactory, the high pressure on the valves, the pipes, and other apparatus within the premises was so great, and the shocks occasioned by the sudden opening and closing of the valves were so violent, that the wear and tear were excessive. By placing this meter at the top of a manufactory, and leading pipes from it over the premises, these difficulties and inconveniences were avoided; and in the case of a dyer or other large consumer, it remained without much wear or tear for a long term of years; whereas on the other system the occupier of the manufactory had a plumber almost constantly on his premises. There was also a public reason why this should be used in preference to the high pressure apparatus, because with a high-pressure apparatus the water was let out, say, into a brewer's vat, with great rapidity, and then as suddenly shut off. The consequence was that the shock extended out of the factory, along the whole line of pipe, and was heard in every house in the neighbourhood. If the factory worked at early or late hours, the violent recoil had such a noisy effect that people were prevented sleeping in their beds. It was by means of these few arrangements that nearly the whole of the waste otherwise incident to the constant supply could be suppressed.

There had been much said of late years with regard to the virtues of soft and hard water. For towns in which water was used in large quantities for manufacturing purposes, and particularly where clothing materials were manufactured, or where, as in Lancashire, there was a good deal of dyeing, it was exceedingly desirable that soft water, and especially mountain water, should be introduced. But in other cases where it was not an essential, there could be no question that, in a sanitary point of view, moderately hard water was preferable to perfectly soft water, and the records of the Registrar-General established this fact, although he believed the result was not attributable solely to the kind of water used. However, the broad result proved that there was greater longevity in the places where moderately hard water was used, than where the water was particularly soft. Apart from that, water free from colour was a beverage generally preferred to water which was coloured: but coloured waters were for the most part those very soft waters of which he was speaking; for the soft waters had an extraordinary avidity for colouring matter, which they took up from peat and from heather, and other vegetable organic matters. That tinged the water brown and made it disagreeable to place on the tables of most persons. On the whole, there was no rule whatever by which it could be distinctly said whether hard or soft water ought to be adopted: the occupations of the people and their habits must be regarded, and in those cases in which soft water was wanted to enable them to obtain their livelihood, soft water ought to be introduced, while in other cases there was at least no objection to water of a harder class. There was, too, this important consideration—four-fifths of the land of the earth's surface, and far more of the populated part of the land, yielded hard water. It was therefore nonsense to say that water which Providence had supplied in such superabundance, while the other was only an exceptional water, should not be supplied for the use of man.

With respect to the construction of embankments for impounding water, there was an idea prevalent that they might be made with slopes, dependent on the nature of the material, but independent of the consideration of height. If an embankment 20 feet or 30 feet high would stand at a slope of $1\frac{1}{2}$ to 1, why should not an embankment 200 feet or 300 feet high stand at the same slope? Theoretically it would; and if no other circumstance than the mere question of slope entered into the consideration, there was no doubt the same slope might be adopted in the one case as in the other. But water-works embankments were usually made of necessity across valleys, which sometimes had a rather sharp fall downwards. As the embankment increased in height, so the weight increased: and the weight of the upper part of the embankment was much more considerable than the weight at the foot of the embankment. That, *per se*, would not make any difference: but it usually happened that the site of the embankment was more or less of a treacherous character, and that under the embankment there was some material or other of a compressible kind, or that would yield under a heavy pressure. The result of that was, and particularly if the base of the bank was liable to become wet, that the pressure at the centre compressed that particular stratum, and threw it out at the foot, the centre sank down, and perhaps the embankment was lost. That was provided against by flattening the slope or by adding a step at the lower part, which threw out a considerable amount of weight. That gave the necessary stability, and resisted the tendency to sink of the upper part of the bank.

The magnitude of floods was one of the most important things which the civil engineer had to consider—not only in the construction of water-works, but in the construction of bridges, and many other works. That subject, it was true, was dealt with empirically, but in this respect it was like nine-tenths of the rules which governed the actions, the contrivances, and the schemes of civil engineers. They were founded on long-continued observations, and if those observations were correctly arranged and properly plotted, a curve which represented the law in that particular instance could always be obtained. With regard to floods in this climate, it had been found that those which ran from any district were governed in part by the geological character of the district—for which an allowance could be made; by the more or less precipitate character of the dstrict, for which an allowance could also be made; and further, by the maximum amount of rain which fell in that district. In England the maximum falls of rain of short duration were nearly everywhere the same.

It was well known that from very small areas as much as 1 cubic foot of water per acre would come off in a second; from larger areas $\frac{3}{4}$ of a cubic foot would flow off; and $\frac{1}{2}$ or a $\frac{1}{4}$ of a cubic foot from still wider areas; for the water required a considerable time to descend to the point where the reservoirs were to be made. Now by making a horizontal line, which represented acres, and by plotting up the observed volumes of floods, which there were many opportunities of measuring on a great scale, it was possible to find points amongst which a curve could be drawn; and that curve would give the law which an engineer might safely, and, at all events, reasonably, apply with regard to the quantity of water which would flow over the weirs of reservoirs. This method had led to important determinations, which Mr. Hawksley promised to explain on a future occasion.

November 22 and 29, 1870.

CHARLES B. VIGNOLES, F.R.S., President, in the Chair.

The discussion upon the Paper No. 1,255, "On the Water Supply of Paisley," by Mr. A. Leslie, occupied both evenings.

December 6, 1870.

CHARLES B. VIGNOLES, F.R.S., President, in the Chair.

The following Candidates were balloted for and duly elected:—CRAWFORD JAMES CAMPBELL, JOHN JAMES CAREY, WILLIAM BELLINGHAM CARTER, WALTON WHITE EVANS, ALEXANDER FRASER, WILLIAM FREDERICK MARCH PHILLIPPS, JOHN ARTHUR PHILLIPS, ARTHUR POTTS, JOSEPH QUICK, jun., WILLIAM ROBERT ROBINSON, and EDWARD WELSH, as Members; CHARLES AUGUSTUS ALBERGA, Stud. Inst. C.E., JOHN PHILIP CORTLANDT ANDERSON, THOMAS ASHTON, ROBERT WILLIAM PEREGRINE BIRCH, Stud. Inst. C.E., JAMES BISSET, JOSEPH BOURNE, JOHN CHARLES COODE, Stud. Inst. C.E., CHARLES COWAN, Capt. ARTHUR EDWARD DOWNING, FREDERICK DRESSER, FRANCIS FOX, THOMAS WILSON GRINDLE, JOHN FALSHAW HOBSON, ARTHUR LUCAS, JAMES CHATBURN MADELEY, WILLIAM MATTHEWS, GEORGE PALMER, ALEXANDER RHODES, WILLIAM GEORGE SCOTT, PETER SOAMES, HERBERT UNWIN, THOMAS FINSBURY SEPTIMUS WAKLEY, WILLIAM THOMAS WALKER, EDWARD ORANGE WILDMAN WHITEHOUSE, CHARLES HENRY WILKS, JOHN HATTON WILSON, as Associates.

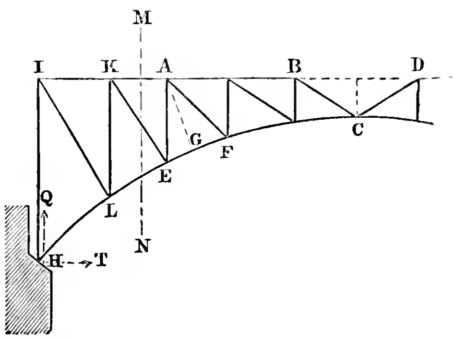
It was announced that the Council, acting under the provisions of Sect. III. Cl. VII. of the Bye-Laws, had transferred the following Associates to the class of Member:—HERBERT LOUIS AUGUSTUS DAVIS, THOMAS MANSON RYMER JONES, and HENRY SHIELD.

Also that the following Candidates, having been duly recommended, had been admitted by the Council, under the provisions of Sect. IV. of the Bye-Laws, as Students of the Institution:—ARTHUR TURNOUR ATCHISON, JAMES THOMAS ATCHISON, EDWARD KYNASTON BURSTAL, EDMUND EMSON, WALTER FREETH, CHARLES JOHN GOODMAN, ARTHUR GROSE, ARDEN HARDWICKE, FLETCHER JAMES IVENS, FREDERICK JACKSON, WALTER ROBERTS JONES, JOHN HERMAN MERIVALE, JOHN NOWLAN, WILLIAM PATTERSON ORCHARD, ERNEST EDWARD SAWYER, GILBERT STIFF, THOMAS SUGDEN, JOSEPH CALISTE GROSVENOR DU VALLON, and CHARLES EDWARD SABINE YOUNGHUSBAND.

No. 1,224.—"On the Theory and Details of Construction of Metal and Timber Arches." By JULES GAUDARD, Civil Engineer, Lausanne. (Translated from the French by William Pole, F.R.S., M. Inst. C.E.)

1. ELASTIC arches supporting a road or railway are connected to the horizontal platform by pillars and filling pieces, occupying the spandrels. The office of these parts is to transmit the load of the platform to the resisting arch; no other function is usually attributed to them, and the arch is the important member on which the whole rests. The spandrel, however, forming always a rigid filling or system, contributes powerfully to increase the resistance of the structure, so that it would be justifiable to consider as the chief member, not the isolated arch, but the framework, more or less complex, constituted by the arch, the spandrel filling, and the longitudinal horizontal piece placed at the level of the platform. Under this point of view, the arch would no longer be restricted to a rigid whole; it might be reduced to a chain of articulated segments. Now in all bridges hitherto constructed, the arches are rigid at the same time that the spandrels present a certain resistance; there is a superabundance of organs, and consequently an uncertain distribution in the functions assigned to each of them. The calculation is only rendered determinate by means of an arbitrary hypothesis of this distribution. Usually the spandrels are neglected, and the arch is assumed to resist without their aid. But there is nothing to prevent the system being assimilated to an articulated form. Hence the two modes of calculation which will now be examined.

Fig. 1.



2. *Articulated System.*—Suppose the form represented in Fig. 1. It is necessary to choose a single triangulation, where every piece

is essential; for otherwise, if, for example, two diagonals were introduced in each bay, the calculation would become indeterminate. At the summit C the two half-spans are joined by a simple point of articulation, like those of the other summits of the triangulation. The piece B D is useless in theory: and in practice it would be desirable to provide it with a free sliding joint, in order that nothing might impede the expansion of the fixed portion.

Under these conditions, the elements of statics furnish easily the stresses on all the pieces. If, for example, it be asked what is the stress t of the segment E F of the arch? Let fall A G perpendicular to E F, then $t \times$ A G is the *moment of resistance*, by virtue of which the piece E F prevents the pivoting round the point A. By making this equal to the *bending moment* relative to the same point, there will result an equation which gives the value of t. The bending moment in question comprehends the sum of the moments of the forces applied to the portion A E H I. If, for example, there is a weight = P applied to the point K, and if the reaction of the abutment H is decomposed into a vertical force = Q, and a horizontal thrust = T, the equation giving t will be

$$t \times \overline{AG} = Q \times \overline{AI} - T \times \overline{HI} - P \times \overline{AK}.$$

If the second number is positive, t is a tension; if it is negative the portion E F of the arch will be compressed.

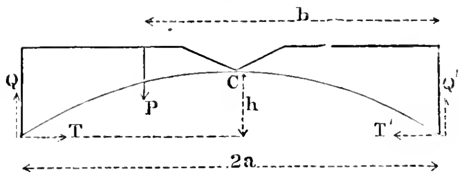
The calculations are the same for the longitudinal I B. For example, the piece K A ought to prevent the solid E K I H from pivoting round E.

3. When the stresses in the various parts of the arch or of the longitudinal are known, the stresses in the spandrel bars can be deduced from them by simple graphic decompositions, based on the equilibrium of the summits of the system. For example, the point E is subject first to two known forces exerted by E L and E F; construct the resultant of these two forces, and then decompose it between the given directions E A and E K; there will thus be obtained lines equal and opposite to the forces which the pieces E A and E K ought to exert upon the point E, in order to maintain it at rest.

These forces may also be obtained directly by calculation. Taking a vertical section such as M N, note down the equilibrium of vertical translation of the portion of the solid situated to the left of M N, introducing there the reactions proceeding from the segment to the right; that is to say, the stresses of the severed bars A K, K E, E L (of which the first has no influence, A K being horizontal). This amounts to saying that the vertical projection of the required stress of the bar K E, added to the projection of the known stress of E L, ought to equilibrate the *shearing force* $Q - \Sigma P$ (understanding by ΣP the sum of the various weights P applied between I and the section M N).

4. Hitherto the reactions Q and T have been regarded as known. In fact their preliminary calculation is easy, according to the equilibrium of the entire system. Using the notations of Fig. 2, and

Fig. 2.



designating by Σ the sums extended to the whole span, then $T = T'$; $Q + Q' = \Sigma P$; and $2\,a\,Q = \Sigma P\,b$. The fourth equation necessary will be furnished by the equilibrium of rotation of one half-span round the summit C, *i.e.*, $T\,h = Q\,a - \Sigma' P\,(b - a)$; the accent here put on Σ' indicates that this sum only embraces the terms arising from the half-span considered.

5. If the mutual thrust be required of the two half-spans at C, it can be as readily ascertained N′ and N″ being the horizontal and vertical components, it suffices, Q and T being already calculated, to write down the equations of equilibrium of translation of the half-span, *i.e.*, $N' = T$, and $N'' = Q - \Sigma' P$. The vertical component N″ will be null if there is complete symmetry of form and of load round the summit C.

The pieces requiring most strength are those near the summit. The Author would be inclined, in the execution of the framing before mentioned, to place the summit C notably lower than the line of the longitudinal, or to adopt a central portion of solid plate.

6. For a determined fixed load, there may be given to the arch the form of a *funicular polygon*, a figure of equilibrium such that the articulated chain may maintain itself in position without the intervention of the other pieces of the framework. In a bridge the load varies, but it is desirable that the figure of the arch should approach that of equilibrium corresponding to the complete load. This form would be the curve called the *catenary* for an arch of uniform section carrying only its own weight. It will be the *parabola* for an arch loaded uniformly per unit of length on a

horizontal line. This latter case is that of suspension bridges, and also of bridges with compressed metallic arches, for the weight of the arch and of the spandrels has but little influence, proportionally to that of the horizontal platform and its test load.

The theory of M. Yvon Villarceau furnishes differential equations for determining the figure of equilibrium, and the variable thickness of the joints, of an arch of equal resistance submitted to a fixed load. The reason why the circular shape is generally preferred to these theoretical forms is, in the first place, the simplification of the design and of the execution, and also the consideration that the theoretical figure only satisfies rigorously a single arrangement of the load, and not all those to which the arch may be exposed.

7. *Rigid Arches.*—The second mode of calculation referred to in Art. 1, consists in restricting the spandrels to simple supports for transmitting the load; the arch is then more strained, and ought to be rigid, for the funicular form only gives an instable equilibrium, corresponding to a particular state of the load. As soon as this state is changed, the arch no longer suffers simple compression, but is disposed to bend, *i.e.*, to change its figure.

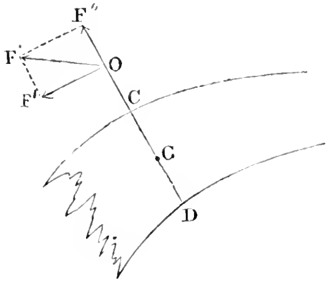
In an arch of masonry, the effect of the mortar is neglected, and the voussoirs are regarded as blocks placed in juxtaposition without adherence, having the power of pivoting one on the other, round the edge either of the intrados or of the extrados; that is what is called a *system of alternative articulation.* The *centre of pressures, i.e.*, the point of application of the resultant of the elementary reactions of the joint is considered for each joint. The locus of these centres, or the *curve of pressures,* ought not to pass outside the thickness of the arch, or the pivoting of certain of the voussoirs would take place; the curve ought even to keep within a zone more limited than that of the arch, for fear of endangering the crushing of the stones.

An elastic arch is subject to other conditions. If the connections are very good, as is (or ought to be) the case with plate iron, the arch forms an entire piece, suitable to resist both tension and compression. If it is treated as an arch by the curve of pressures, this curve will no longer be required to remain within the arch.

8. Let F be the total reaction on a joint C D (Fig. 3), and O its point of application, lying in the curve of pressures. Let O G = x the distance from this point O to the centre of gravity or of elasticity (mean fibre) of the section C D. The force F is decomposed into a normal pressure F′ and a shearing stress F″ which, acting in the direction C D, tends to shear the solid. The longitudinal force F′ may be applied at the centre of elasticity G,

provided that we combine with it a couple whose moment $= F' x$, which will be the bending moment. Then the fibre most strained,

Fig. 3.



i. e. C in the case of the figure, will be subjected to a pressure

$$R = \frac{F'}{\omega} + \frac{v\,F'\,x}{I}$$

per unit of surface: ω is the area of section, I its moment of inertia, and v the distance C G. The formula supposes that, under the stress mentioned, the section C D has slightly pivoted while remaining plane.

9. When the arch rests upon the abutments by an extended sustaining surface, being keyed by a range of wedges, there is nothing to prevent certain of these wedges being driven tighter than others. This will then modify the point of concentration of the thrust upon the abutment, and consequently also all the other points of the curve of pressures. Hence arises the uncertainty which generally attends the method of the curve of pressures, these pressures only being determined by arbitrary data in regard to the original keying-up, or the yielding of the materials. An infinite number of curves are possible. It is not known which is effectively realized, but suppose the material system to seek, in the first place, its equilibrium at the cost of the *least resistance* (Moseley's principle); then certain edges are compressed and oblige the curve of pressures to modify itself continuously, until it arrives at a stable condition. If no stable condition is met with, the system soon exhausts the series of conditions possible, and ends at the extreme limit, which is then the curve of the greatest resistances, beyond which the failure of the structure takes place. According to that a curve of any pressures whatsoever, arbitrarily chosen among all those compatible with the equilibrium, will give a strong presumption of stability, if it only submits the edges to

pressures offering all practical security; for, in the absence of any other stable condition, the system ought to stop at that, if the velocity acquired is insensible. Thus is justified the method of Méry, notwithstanding that it appears strange at first sight to base confidence on the existence of a certain curve, at the same time that there is no assurance that this curve will be effectively realized.

By these considerations, the safety is established, but it may be so in different degrees; and there is another consideration to bring into the question—that of economy. Now these two conflicting objects, the economy of material and the stability, lead the play or field open to the virtual oscillations of the curve of pressures to be confined within just limits. The amplitude of this play, if it is great, will give a superabundance of safety, and if it is small, it will testify to great boldness of design. M. Durand Claye has ('Annales des Ponts et Chaussées, 1868') indicated a method of appreciating the degree of boldness—a method of which the following résumé will present the essential features for the case of elastic arches, not excluding the action of tension.

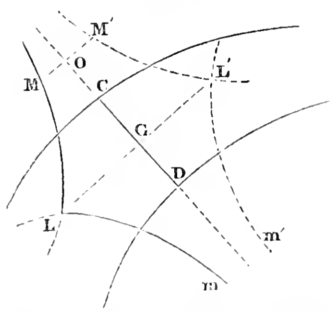
10. Two preliminary remarks may be offered:—

1st Remark.—In Art. 8, is stated the formula of resistance which gives the stress R on the fibre most strained in a joint C D. Suppose that this stress attains precisely the limit imposed by practical safety, it becomes a given datum, and the unknown quantity is the corresponding normal pressure F in respect of abscissa G O = x of its point of application. Now

$F' = \dfrac{R\, I\, \omega}{I + \omega\, v\, x}$, so that, considering the line representing the force

F′ as an ordinate, O M = y (Fig. 4), answering to the abscissa

Fig. 4.



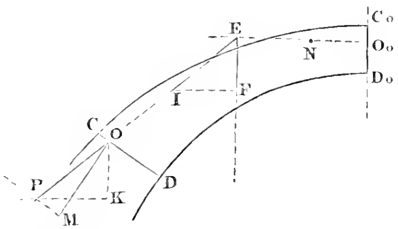
G O = x, the geometrical locus of the points M will be a branch of an equilateral hyperbola M L, of which D C, being prolonged, represents an asymptote. Beyond the point L, which corresponds to $x = O$, the first hyperbola ceases to apply, and is replaced by another L m; seeing that, when the centre O of the pressures passes to the other side of the centre of elasticity G, it is no longer the fibre C which is the most strained, but the fibre D.

For an arch capable of extension R and F′ may be admitted to be negative, which leads to a second hyperbolic contour M′ L′ m'; but this case rarely demands attention.

These different curves define generally the exigencies of the section C D. If a curve of pressures brings on this joint a resulting normal reaction F′, exceeding the ordinate, such as O M, or in other words, whose summit passes outside the contour M L m . . . m' L′ M′, this curve ought to be rejected, as involving a pressure, R, greater than the limit admitted.

2nd Remark.— In order to trace a curve of pressures, the voussoirs, such as C_0 D_0 D C (Fig. 5) are considered as departing from the ver-

Fig. 5.

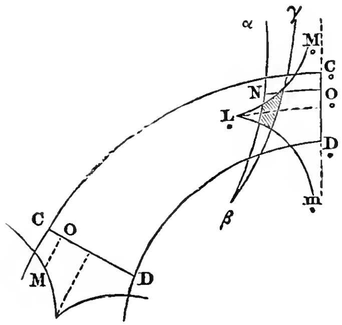


tical joint of the crown C_0 D_0 and stopped successively at various joints C D. Let the case of an arch be considered which is symmetrical both as to figure and load, so that the thrust at the crown is horizontal. Take the centres of pressure O_0 and O (arbitrary if they do not result from constructions already existing); through O^0 draw the horizontal O_0 E which will cut in E the vertical from the centre of gravity of the voussoir C_0 D_0 D C and of its load; draw a length E F equal to the weight, join E O, and draw F I horizontal. The three forces which maintain the equilibrium are represented by the three sides of the triangle E F I; the weight = E F, the thrust at the crown = F I (but applied at O_0), and lastly the reaction of the joint C D equal to I E, and applied at O. This force may be transferred to O P = E I, and the normal component of it O M taken, which is that called above F′.

Inversely, this pressure O M may be assumed *à priori*, and its point of application O, in order to arrive therefrom at the thrust on the crown. In effect, take O K vertical and equal to the weight E F; draw K P horizontal, and M P paralled to D C; then join the point of intersection P to O; prolong P O to E, where it encounters the vertical of the centre of gravity, and draw E O_0 horizontal. O_0 is thus known, and the value of the thrust at this point is measured by K P. It may be transferred to O_0 N = K P, and a point N will be derived from M. The constructions should be modified in the case of non-symmetry, for then the thrust at the crown is not horizontal.

11. Now, let there be chosen successively for M all the points of the hyperbolic contour M L m of Fig. 4. The points N corresponding to them at the crown, will describe certain curves $\alpha\ \beta\ \gamma$ (Fig. 6), which will represent at the crown the exigencies of any

Fig. 6.



joint C D. A thrust O_0 N will only be admissible if its summit N falls upon the contour $\alpha\ \beta\ \gamma$, or in the interval between the branches $\alpha\ \beta$ and $\beta\ \gamma$. But the joint $C_0\ D_0$ possesses, by the same title as C D, its own limiting contour $M_0\ L_0\ m_0$ formed of hyperbolic arcs. Severally, then, the summits N of the thrusts O_0 N admissible at the crown ought to fall in the interior of the space cross-shaded in Fig. 6, in order to satisfy the simultaneous exigencies of the joints C D and $C_0\ D_0$.

Repeat the constructions for the joints other than C D, transferring everything to the crown joint $C_0\ D_0$. The various cross-hatched spaces will contract more or less among themselves, and their common part alone must be taken into consideration, in order to obtain the thrusts at the crown, or the curves of pressure, compatible with the limiting resistance R imposed simultaneously at all the various joints.

If it is now desired to compare the boldness of two works, it will suffice to seek for each of them, by trial and error, the limiting stress R for which the various curves of pressure realizable become reduced to one only, or the cross-shaded space becomes reduced to a single point. The value thus found will be a well-characterized definition of the boldness sought. Such is the method of M. Durand Claye.

12. *Bridges with three pivots.*—A metallic arch might be provided with three pivots or hinges, one at the summit, the other two at the supports. The axis of these pivots would determine necessarily the centres of pressure at the crown and at the abutments, and then nothing would be arbitrary in tracing the curve of pressures, or in the methods of calculation. The calculations of the reactions, and consequently those of the resistances of the arch, would be simple and certain; the expansion would have free play, without straining the metal. These three pivots have been proposed, but hitherto, as far as the Author is aware, no one has ventured to apply them to large works, for fear that the too great mobility should favour the dislocation in certain parts. There have only been employed two pivots at the supports, an innovation introduced by M. Mantion in an iron bridge of the St. Denis Canal. Without doubt, for an isolated arch, a pivot at the crown would have the effect of a spherical knee, provoking the structure to turn over; but in a bridge sufficiently wide relatively to the opening, it would seem that the connection together of several arches might be sufficiently well arranged to realize in the whole a long joint, resisting effectually any tendency to bending, at least for an ordinary road, where the rolling weight is less than on a railway.

Bridges with two pivots.—With two pivots at the supports, the expansion is subject to some constraint, because it ought to augment a little the rise of the arch, on the hypothesis that the supports are rigidly fixed. This effect contributes in certain cases to increase the stress of the material; it is sought to lessen this by reducing the dimensions of the middle of the piece.

The reactions of the abutments can no longer be obtained by simple statics, as in Art. 4, for the last equation of this Article contains the rise h, which would be unknown, as it ought to be measured to the unknown centre of pressure of the middle joint. If no supplementary equation existed, the problem might be deemed indeterminate, and a choice be arbitrarily made of the centre of pressure of the summit, as, for example, the middle of the joint; this is the process of M. Méry.

In reality, however, the equation does exist; it depends on the deformation or on the yielding of the material of the arch, for this deformation must conform to certain conditions, as, for example, with respect to the primitive invariable situation of the points of support.

Now, when treating of an apparatus not homogeneous, the expression of such a delicate condition must be abandoned, and reliance must be placed upon M. Méry's process, notwithstanding its vague nature. Such is the case of an arch of masonry, a heterogeneous agglomeration of stones and mortar; and further, as there can be no question here of pivots at the supports, the same uncertainty prevails as to the point of concentration of the thrust upon the abutments. Such would also be the case with an arch of carpentry, the wood yielding much, being very sensitive to atmospheric influence, having a texture full of knots and other irregularities, and finally only allowing imperfect jointing. Arches of cast iron, even, are not rigorously homogeneous; they consist of several voussoirs fitted more or less exactly one on the other, and the material assumes irregularities of texture in the casting. A certain indecision seems, therefore, legitimate for such works, even in the case of two pivots at the supports. Further, it cannot be believed that a metallic arch would be menaced with failure by the simple fault of its having been calculated by the process of M. Méry; the great guarantee in construction always being the wise precaution of keeping at a respectful distance from the stresses of rupture.

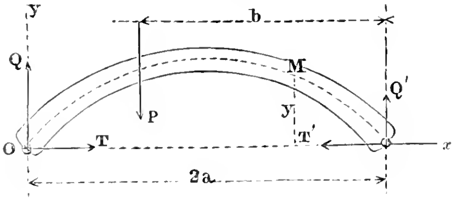
There only remain, therefore, arches in a single piece of homogeneous metal, to which the theoretical calculations of deformation may be applied with confidence. Arches in solid wrought-iron plate may be considered to belong to this category, the metal having been well worked, and the connections being as solid as the continuous parts. The theory of these arches has been the object of the researches of M. Bélanger, and subsequently of M. Bresse, who has entered into great details respecting them in his Treatise on Applied Mechanics (Stabilité des Constructions). His principal formulæ will be indicated.

13. The equilibrium of the entire system furnishes always, as in Art. 4, the three statical equations $T = T'$, $Q + Q' = \Sigma P$, and $2aQ = \Sigma Pb$, the P's expressing the weights or vertical forces (Fig. 7). The equation of deformation, which will express the invariability of the chord $2a$, is the equation following (see Art. 27 of the Author's Memoir on the Strength of Materials).[1]

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxviii., p. 561.

$$2\tau a + \int_0^{2a}\left(\frac{\mu y}{EI}\frac{ds}{dx} + \frac{N}{E\omega}\right)dx = 0.$$

Fig. 7.



The co-ordinate axes are Ox, Oy, the origin being taken at one of the supports; s expresses a length of the curved mean fibré, and ds its differential. The elastic arch may be more or less strained by expansion or keying; τ expresses the linear elongation resulting from these causes independent of the loads P. At the point M of the mean fibre, defined by the abscissa x and the ordinate y, let us imagine a transverse section: ω is the area of this section; I its moment of inertia with respect to the horizontal axis through its centre of gravity; μ is the bending moment, and N the total normal pressure, to which it is subject. The shearing stresses (parallel to the section), which have but little influence on the deformation, may be neglected. The letter E is the coefficient or modulus of elasticity; the section being supposed homogeneous, $E\omega$ is what is called the longitudinal spring (*ressort longitudinal*), and E I the moment of flexibility.

The three statical equations will give T′, Q, and Q′, then T, which alone remains unknown, ought to be derived from the last equation, where it enters implicitly. In order to show this, put $\mu = \mu_1 - Ty$, and $N = N_1 - T\frac{dx}{ds}$; then the moment μ_1 and the longitudinal stress N_1 are known, since they no longer take account of the unknown quantity T put aside; hence

$$T = \frac{2\tau a + \int_0^{2a}\frac{\mu_1 y}{EI}\frac{ds}{dx}dx + \int_0^{2a}\frac{N_1}{E\omega}dx}{\int_0^{2a}\frac{y^2}{EI}\frac{ds}{dx}dx + \int_0^{2a}\frac{1}{E\omega}\frac{dx}{ds}dx}$$

Such a formula is, indeed, alarming for practical application, especially when the calculation must be several times repeated for different cases of loading. However, by reducing it to the most essential terms, these may be taken approximately

$$T = \frac{2\tau a + \int \frac{\mu_1 y}{E I} d s}{\int \frac{y^2}{E I} d s};$$

and further, the two integrals or quadratures alone retained, and taken between the limits O and S (equal to the total length of the arch), may each be calculated by the approximate method of Thomas Simpson, whatever be the figure of the given arch whose stability it is desired to verify. Dividing the arch into an equal number of equal parts, the values of $\frac{\mu_1 y}{E I}$ and of $\frac{y^2}{E I}$, may be determined for each point of division, and Simpson's formula may be applied.

14. *For a symmetrical arch symmetrically loaded*, the axis of y may be transferred to the axis of symmetry passing through the summit of the arch, and then the general formula for T will be

$$T = \frac{\tau a + \int_0^a \frac{\mu_1 y}{E I} \frac{d s}{d x} d x + \int_0^a \frac{N_1}{E \omega} d x}{\int_0^a \frac{y^2}{E I} \frac{d s}{d x} d x + \int_0^a \frac{1}{E \omega} \frac{d x}{d s} d x}.$$

If such an arch is rigidly fixed by building in (*encastré*) at the extremities, the abutment will exercise reactions Q, T, which may always be regarded as applied to the point situated upon the mean fibre, provided there is combined with it a couple or moment of *encastrement* σ, destined to maintain invariable the angle between the crown joint and the joint at the origin. This couple will then appear in μ, from which it is disengaged by making $\mu = \mu_2 - T y + \sigma$. Then the theory of M. Bresse furnishes these two equations for determining T and σ, the origin of the co-ordinates being in the middle of the span.

$$\sigma \int_0^a \frac{1}{E I} \frac{d s}{d x} d x - T \int_0^a \frac{y}{E I} \frac{d s}{d x} d x + \int_0^a \frac{\mu_2}{E I} \frac{d s}{d x} d x = 0;$$

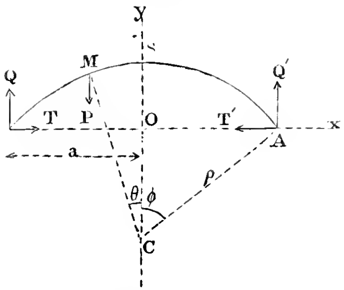
$$T\left(\int_0^a \frac{y^2}{E I} \frac{d s}{d x} d x + \int_0^a \frac{1}{E \omega} \frac{d x}{d s} d x\right) - \sigma \int_0^a \frac{y}{E I} \frac{d s}{d x} d x$$

$$= \tau a + \int_0^a \frac{\mu_2 y}{E I} \frac{d s}{d x} d x + \int_0^a \frac{N_1}{E \omega} d x.$$

15. *Circular Arches of uniform section.*—M. Bresse has further advanced the study of circular arches of uniform section, for which ω and I are constant, and $y =$ the ordinate of the arc of a circle. The following are the more important formulæ for the case of simple supports without *encastrement.*

Let ρ be the radius of the circle, $a = \rho \sin \phi$, the demi-chord, ϕ the half angular amplitude S C A, between the crown and the origin A, Fig. 8.

Fig. 8.



Let there be considered first the isolated action of a single weight P, applied to a point M defined by the angle S C M $= \theta$. The integrals will be calculated after having expressed μ_1 and N_1 which change on either side of M. The vertical components of the reactions of the abutments will be

$$Q = \frac{P}{2}\left(1 + \frac{\sin \theta}{\sin \phi}\right),\ Q^1 = \frac{P}{2}\left(1 - \frac{\sin \theta}{\sin \phi}\right);$$

and the horizontal thrust will be

$$T = T^1 = P \cdot \frac{2\,a^2\,A - r^2 \sin^2\phi\,(\sin^2\phi - \sin^2\theta)}{2\,a^2\,B + 2\,r^2 \sin^2\phi\,(\phi + \sin\phi \cos\phi)};$$

r designates the radius of gyration $= \sqrt{\frac{I}{\omega}}$ of the uniform section.

Further, for the sake of abridgment, let A designate the quantity

$$\frac{\sin^2\phi - \sin^2\theta}{2} + \cos\phi\,(\cos\theta + \theta \sin\theta - \cos\phi - \phi \sin\phi),$$

and B the quantity, $\phi + 2\,\phi \cos^2\phi - 3 \sin\phi \cos\phi$.

As a general form, the preceding thrust T may be expressed :—

$$T = P\,\frac{A}{B} \cdot \frac{1 - \lambda \dfrac{r^2}{a^2}}{1 + \lambda^1 \dfrac{r^2}{a^2}}.$$

Now, the principal coefficient $\frac{A}{B}$ is furnished ready calculated by the Table I. of M. Bresse's 'Mecanique Appliquée' for different values of $\frac{2\,\phi}{\pi}$ and of $\frac{\theta}{\phi}$. The Table IV. gives the coefficient of correction

$$\frac{1 - \lambda \frac{r^2}{a^2}}{1 + \lambda^1 \frac{r^2}{a^2}}.$$

for a series of values of $\frac{2\,\phi}{\pi}$, and of $\frac{r^2}{a^2}$. The factor λ is equal to $\frac{\sin^2\phi\,(\sin^2\phi - \sin^2\theta)}{2\,A}$, but differs little from $\frac{\pi^2 \sin^2\phi}{4\,\phi^2}$. And on the other hand, $\lambda^1 = \frac{\sin^2\phi\,(\phi + \sin\phi\cos\phi)}{B}$ may be replaced approximately by $\frac{15\,a^2}{f^2}$, f designating the rise or height of the arch.

16. Now, leave P aside, and only consider a simple dilatation τ per running metre. This will produce no vertical reaction, but a simple horizontal thrust

$$T = \frac{2\,a^2\,E\,I\sin^3\phi}{a^2 B + r^2 \sin^2\phi\,(\phi + \sin\phi\cos\phi)} \cdot \tau.$$

Writing $T = \frac{D}{1 + \lambda' \frac{r^2}{a^2}} \cdot \frac{E\,I\,\tau}{a^2}$, there will be found the principal coefficient D in the tables of M. Bresse. In the absence of tables, approximately $T = \frac{15\,E\,I\,\tau}{15\,r^2 + 8\,f^2}$, if the arch is sufficiently flat.

17. For the third case, a *uniform load p per metre of length of the arch* (such as the weight of the arch itself). Supposing it applied to the entire arch, of length = S, the vertical reactions will be $Q = Q' = \frac{p\,S}{2}$, and the thrust;

$$T = p\,\rho \times \frac{a^2\phi(\frac{1}{2} - 5\cos^2\phi) + \frac{9}{2}a^2\sin\phi\cos\phi - r^2\sin^2\phi(\frac{1}{2}\phi + \frac{1}{2}\sin\phi\cos\phi - \phi\cos^2\phi)}{a^2\,B + r^2\sin^2\phi\,(\phi + \sin\phi\cos\phi)}.$$

This expression is of the form $T = 2\,p\,\rho\,\phi \cdot D' \cdot \dfrac{1 - \lambda \frac{r^2}{a^2}}{1 + \lambda' \frac{r^2}{a^2}}$, and the coefficient D′ has also been calculated in the tables cited. In the absence of tables, the following approximate formula will suffice for a flat arch:

$$T = \frac{4\,f\,p\,\rho\,\phi}{7\,a}\left(\frac{7\,a^2 - 3\,f^2}{8\,f^2 + 15\,r^2}\right).$$

18 Finally, for a *uniform load p per unit of length on the horizontal line* (weight of the platform). As the moving load may only be partial, let θ_1 and θ_2 be the angles analogous to θ (Fig. 8) which limit the portion of the arch loaded. Then T will be given by the following equation:

$$\frac{T}{p\,\rho}\left[2a^2\,B + 2\,r^2 \sin^2\phi\,(\phi + \sin\phi\cos\phi)\right]$$

$$= a^2(\sin\theta_2 - \sin\theta_1)(3\sin^2\phi - 2 - 2\phi\sin\phi\cos\phi) - \frac{a^2}{3}(\sin^3\theta_2 - \sin^3\theta_1)$$

$$+ \frac{a^2\cos\phi}{2}(\theta_2 - \theta_1 + 2\,\theta_2\sin^2\theta_2 - 2\,\theta_1\sin^2\theta_1 + 3\sin\theta_2\cos\theta_2 - 3\sin\theta_1\cos\theta_1)$$

$$- r^2\sin^2\phi\left[\sin^2\phi\,(\sin\theta_2 - \sin\theta_1) - \tfrac{1}{3}(\sin^3\theta_2 - \sin^3\theta_1)\right].$$

When p extends to the entire length, make $\theta_1 = -\phi$, $\theta_2 = \phi$, and consequently we have

$$T = p\,a\,\frac{\frac{1}{6}\sin^2\phi\,(7\,a^2 - 6\,a^2\phi\cot\phi - 4\,r^2\sin^2\phi) + \frac{1}{2}a^2(\phi\cot\phi - 1)}{a^2\,B + r^2\sin^2\phi\,(\phi + \sin\phi\cos\phi)},$$

B designating, as above, the quantity $\phi + 2\,\phi\cos^2\phi - 3\sin\phi\cos\phi$.

This last expression for the thrust differs little from

$$T = \frac{4\,f\,p}{7}\left(\frac{7\,a^2 - f^2}{8\,f^2 + 15\,r^2}\right).$$

But it may also, without altering its value, be written under the form $T = 2\,p\,a\,D'' \dfrac{1 - \lambda \frac{r^2}{a^2}}{1 + \lambda' \frac{r^2}{a^2}}$, and D″ sought in the tables of M. Bresse. The vertical reactions are $Q = Q' = p\,a$ (in the case of the complete load). If the parabolic form for the arch had been adopted, the horizontal thrust would be, as with suspension bridges,

$$T_1 = \frac{p\,a^2}{2\,f}.$$

19. When once the determination of the reactions of the abutments has been arrived at, there is no further difficulty in applying the formulæ of resistance to any section whatsoever. The complete bending moment μ and the longitudinal force N are calculated from the known forces, then the formula $R = \frac{N}{\omega} + \frac{v\mu}{I}$ gives the stress R per superficial unit on the most strained fibre in the section (v = distance of this fibre from the neutral axis, ω = area of section, and $I = \omega r^2$ its moment of inertia). Considering successively various sections, the section will be found where the stress R attains the highest value. This *maximum maximorum* of stresses is obtained rapidly in the case of a load p per running horizontal metre on the entire length, by multiplying $\frac{pa}{\omega}$ by a coefficient β, which M. Bresse's Table V. gives ready calculated for a series of values of $\frac{r^2}{a^2}$, of $\frac{2\phi}{\pi}$, and of $\frac{h}{a}\left(\text{or } \frac{2v}{a}\right)$. This applies always to a circular arch of uniform section, this section being also symmetrical relatively to the neutral axis, which occupies the middle of its height h.

20. The best ratio $\frac{f}{2a}$ to adopt between the rise f and the opening $2a$ depends on $\frac{r^2}{a^2}$; it would be, for example—

For $\frac{r^2}{a^2}$ =	0·0001	0·0002	0·0003	0·0005	0·0008	0·0010	0·015
$\frac{f}{2a}$ =	0·124	0·150	0·158	0·176	0·198	0·212	0·2[illegible]1

Good sections to adopt for arches are those which make $\frac{r}{h}$ large, *i.e.*, which have a large radius of gyration or moment of inertia without too great height.

21. The tables prepared by M. Bresse simplify considerably the calculation of circular arches with a uniform section. The circular form is, indeed, that which is ordinarily chosen in practice; but the restriction to uniform section is more troublesome, for there is a double motive to vary the section. By diminishing the middle of the arch the play of the expansion is facilitated (in the absence

of a pivot at the summit), and by enlarging the section near the abutments the strength is increased where the pressures become greater. It is better to give to a structure the form which is practically most advantageous rather than that which most facilitates the theoretical calculations. If, then, in the case of a variable arch, either from want of time or of skilful aid, the general formula for T (Art. 13) be thought too complicated, the method of M. Méry may be adopted, by choosing arbitrarily the centre of pressure at the crown; but in this case it will be advisable to make say two calculations instead of one, with situations notably different from the arbitrary point. If these operations indicate advantageous results for the two hypotheses (repeated in the various cases of loading presumed to be dangerous), doubtless the safety of the work may be considered as sufficiently established.

If the joints of attachment at the abutments are keyed at several points, instead of being pivoted, it would be illusive, as in a masonry arch, to pretend to make a rigorous theoretical calculation; for it could not be ascertained that the keys were all driven equally in the first instance, or that in touching them afterwards, in case of derangement, the resistance of the structure had not been altered. Further, the wedging-up by numerous keys should be considered objectionable. If the centre of pressure of the extreme section, which is not adherent to the support, passes certain limits (*noyau central*) a key of the extrados or the intrados will become loosened, while the opposite one will suffer excessive strain. To avoid these evils it would be necessary to have recourse to bolting or *encastrement*, often difficult to apply, or of little utility.

Arches of Different Materials. Details of Construction.

22. Structures in timber are the most economical in many countries, but their durability is limited. This may be prolonged by successive renewals of the parts deteriorated; but such reparation is costly, and it tends to augment the general deformation or giving way, because every new piece introduced in a space shortened by previous alterations shortens it further when it takes its share of the work.

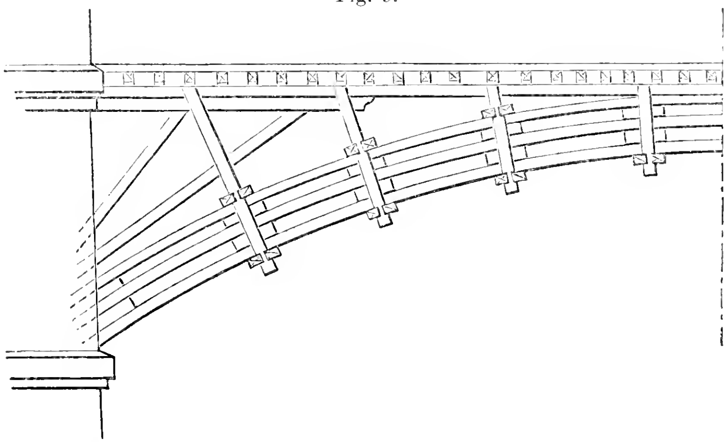
Metals, cast iron, wrought iron, or steel, promise generally a greater durability.

Arches in Timber do not form entire structures so well connected together as to admit of the application of M. Bresse's formulæ, nor of the adoption of pivots. It is, therefore, necessary to proceed nearly as for a vault of stone, by the curve of pressures. The difference only lies in the form of section of the arch, and in a different mode of distribution of the loads

As the wood alters and twists by hygrometrical variations, it is generally desirable to give a slight rise (camber) to the platform. Doubtless, at first, if the bridge has many spans, this camber may be slightly visible to the eye, and may appear as a series of convex curves breaking the continuity of the horizontal line of the platform; but this appearance is certainly less disagreeable than the hollows, signs of sinking, which, even while the solidity is undoubted, impress the public with an idea of danger.

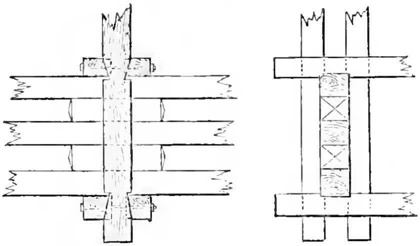
23. The principal type of timber arches appears to be that inaugurated at Yvry (Seine) by M. Emery, namely, that where the arches are composed of strong pieces of carpentry superposed to the number of three, for example. But it is preferred in many cases to leave open the intervals between the several pieces in order to allow better ventilation, and to prevent them from heating; and further, because this plan gives an increase of depth to the arch, and consequently a more ample field to the oscillations of the curve of pressures. The sketch (Fig. 9) will recall the type referred to.

Fig. 9.



The horizontal and the hanging members are connected together (Fig. 9*a*) by obliquely-cut joints, forming sliding planes, so that the bolts produce a reciprocal tightening of all the pieces brought together. With all surfaces in contact, moisture and heating are to be feared; but it is advisable to apply layers of coal-tar here as well as in other internal surfaces where the parts are fitted together. The iron-work should be as much as possible in the form of straps and stirrups, clasping the wood round without piercing it. In the butt-joints of the segments of the arch it is useful to interpose plates of copper, or of very hard wood, in order

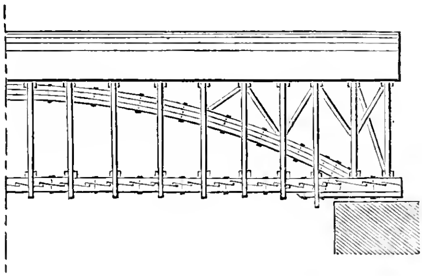
Fig. 9*a*.



to prevent the mutual penetration of the fibres. If it is absolutely necessary to use wood imperfectly seasoned, it ought not to be painted till a year or two after the construction.

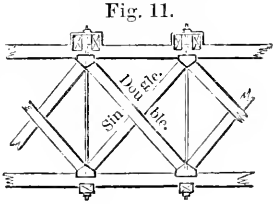
Certain timber bridges present a compressed arch, a tensile tie-bar, and vertical connecting-rods (Fig. 10). This system is allied to the bowstring form, which will be referred to hereafter.

Fig. 10.



The flat arches of Wiebeking (Bavaria) and those in thin layers of planks superposed (Emy) would appear only suitable for roofing purposes, being too subject to deformation for bridges.

Fig. 11.



Among the American forms of trellis framework for straight beams, capable of competing with arched openings, the system of Howe, having iron vertical tie-rods, deserves special mention (Fig. 11). Two observations may be made, by way of digression on this system.

A. It permits, within certain limits, of the correction of too much deflection by simply tightening the screws of the ties. In effect this operation shortens the rods which the flexure has elongated. Further, the tightening compresses the cross pieces in the diagonal which has distended, and may even, if pushed further, compress this simple diagonal more than the other, of which the section is double; these effects are contrary to those of the load, which has deformed the beam. In practice this resource is but limited, especially in bridges of several spans with continuous girders. A bridge on this system, constructed on the Rhone, near St. Maurice, in Switzerland, has suffered considerable deformation, resulting, at length, in requiring the application of thick packing pieces to level the rails; this work, built with green timber, and painted prematurely, has required almost complete reconstruction within three or four years after its first erection; the successive replacement of the pieces of timber, during the working of the railway, has not been possible without aggravating the deformation already produced by the load.

B. The association of two different materials, wood and iron, in one and the same resisting system, is not free from inconvenience. The tightened joints of the cross pieces occasion shocks under the vibrations of the trains; besides, it has been sometimes remarked, that the lowering of the temperature in winter may unduly contract and fracture the iron rods.

24. *Constructions in Metal.*—Steel is the material of the future for large bridges; for it sustains twice as much as iron, and the dead weight is diminished. It is, moreover, a material homogeneous and elastic, and little affected by vibrations. England has already for some time inaugurated the use of steel in bridges.[1]

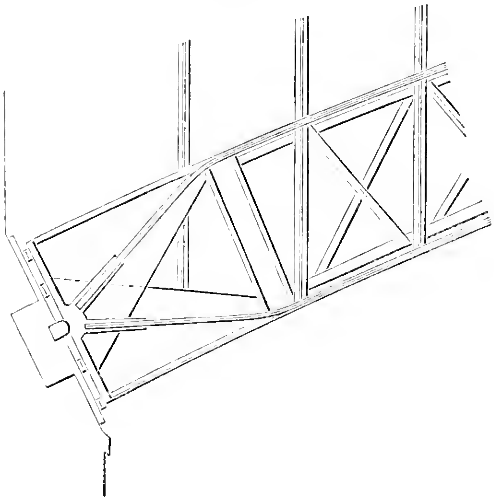
Cast iron resists compression well, but tension badly. It makes good arches in the cases where the curve of pressures does not pass out of a certain central zone, and where the flexure is nothing or insensible; and on the condition that there are no powerful vibrations, *i. e.*, that the dead weight is large compared to the moving load. This last consideration leads, in the case of railways, to the spreading of a thick layer of ballast on the platform, in spite of the increase of load resulting therefrom. This has been done especially on the bridge of Tarascon, over the Rhone, a great work of seven arches of 62 mètres (203 feet) each. Cast iron is less costly than wrought iron, for equal weights, and it may easily be used, under compression, up to a practical coefficient of 6 kilogrammes per square millimètre (3·8 tons per square inch) and

[1] This paragraph ought, I think, to be taken with some reserve.—Tr.

upwards. The stress on the mean fibre in works constructed, generally approaches 3 kilogrammes (1·9 ton), the voussoirs are applied one on the other by flanches rebated and bolted together. It is desirable to avoid thicknesses less than 2 centimètres (0·8 inch nearly), and also too abrupt variations in the thickness of the same piece, to avoid inequalities in cooling, and differences of texture.

Wrought iron is preferable to cast iron, in spite of its higher price, whenever there is reason to fear the effects of flexure in certain parts, or when it is desired to make a light structure, without loading of ballast. The rivetted joints render the piece as strong as if it were rolled entire;[1] there is the opportunity of adopting pivoted supports, and of calculating the thrust according to the theoretical deformation. However, this conclusion is not absolutely true for trellised arches, which it is necessary to use in the case of very large spans. Such, for example, is the bridge at Coblenz over the Rhine, consisting of three arches, each 96·70 mètres (317 feet) span (Fig. 12). In this bridge the pivot applied to the abutments is interposed between two cast-iron plates, one

Fig. 12.



bedded with cement on the masonry, the other provided with boxes or jaws to receive the feet of the arch, which are bent to converge

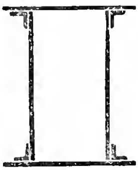
[1] Not always; rivetted joints are often badly designed, and worse made. —Tr.

to this point. The arch, however, has not been reduced in depth, the triangular parts have been preserved, and sustained by ordinary keying, in order to diminish the mobility. This may be justified for a work of this kind, with a trellised arch, but in the case of a solid plate web, the pivot alone without keys would appear better.

In general, a solid plate web is preferable to trellis-work for arches of moderate dimensions. In effect, the flexure is small, and the longitudinal pressure much predominates; the solid plate acts, consequently, more usefully than in a straight beam, which presents neutral fibres. Moreover, it will only be necessary to employ rivets at considerable intervals in the parts outside the joints.

To satisfy the advantageous condition named in Art. 20, *i. e.*, a large moment of inertia, without too great depth, it is desirable to adopt for the section a double T with large wings. For large openings, however, a box section (Fig. 13) appears to be preferred.

Fig. 13.



The lateral stiffness will depend essentially on the cross framing which connects together the different arch ribs of the same span. Where this resource is wanting, *i. e.*, where an isolated arch rib must maintain itself alone, the oval section, analogous to that of the arches of the gigantic bridge at Saltash, presents itself as one of the most favourable.

25. Wrought and cast iron may be associated in the same work. This combination has been adopted, for example, in a bridge of three arches constructed in the Park at Neuilly, near Paris. Fig. 14 indicates the general aspect of a semi-span of this work, many details of which have been suggested by the new Westminster Bridge. All the pieces essentially under compression, *i. e.*, the spandrels and the portions of the arch near the abutments, have, for the sake of economy, been made of cast iron, while the central portion of the arches and the longitudinal bearers of the platform are of plate iron. The summit of the arch is very flat, which exposes it to sensible flexure; at certain parts of the crown the stress of tension may attain 4 kilogrammes per square millimètre. The longitudinal bearer is, moreover, destined to act as an anchoring tie, in order to unite the three parts into a solid whole, securely fastened to the masonry; and this is extended to the abutments on either side. The resistance of such a structure is undoubtedly considerable, but it is difficult to estimate, with any certainty, the internal strains which may arise from expansion.

Arched bridges are, on principle, often more economical than those formed with girders; they admit better of an augmentation of the dead weight, for the purpose of deadening the vibrations and increasing the probable durability. But it is necessary that sufficient height should be available, and that the abutments should be able to resist the thrusts. When the work is low and the ground solid, it is easy to give the abutments the necessary stability, without too much expense. The compression, tending to close the molecules of the metal, appears to promise to such works a longer duration than would be due to pieces in tension, which threaten, after a long period, to be subject to enervation. These considerations in favour of arched bridges are, it is true, somewhat counterbalanced by the greater complication of the forms.

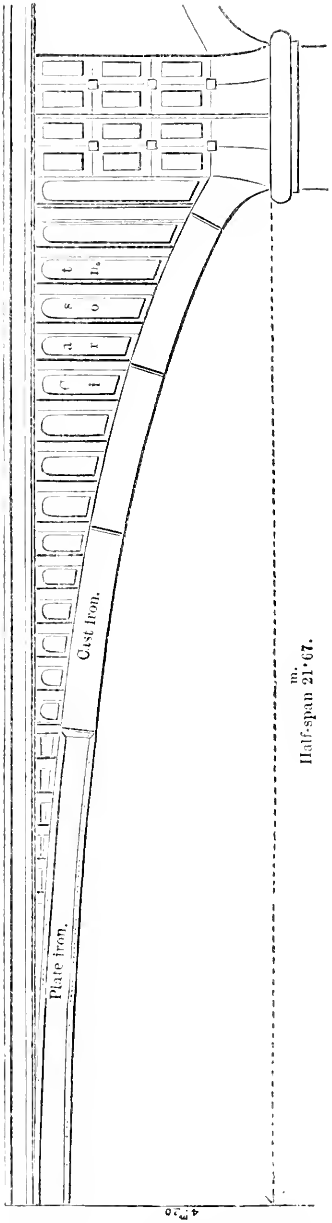


Fig. 14.

Bow and String Bridges.

26. The girders called by this name are, as the words imply, arches provided with a tie-rod which receives the thrust upon the extremities, without the intervention of supports; so that the abutments are freed from the effect of these thrusts, and only exert reactions in a vertical direction.

The most remarkable example is the bridge at Saltash, of two great spans of 139 mètres each.

The arch, being single, had to be kept at the two ends at the height above the roadway necessary to leave a free passage for the trains; and this led the eminent designer, Mr. Brunel, to adopt a curved tie, and to suspend the platform at a lower level.

When there are two arches, one on each side of the road, there is nothing to prevent their extremities descending to the level of the platform; the tie is then straight, and being strengthened in order to serve as a longitudinal bearer, it may support the roadway. An example of this disposition is the bridge of Audenarde, on the Scheldt. Between the arch and the tie, there is fixed a framework composed of vertical bars more or less strong, and of lighter diagonals. The office of these pieces is analogous to that of the spandrels in ordinary arched bridges, with this difference, that the tensions predominate, or are the only forces present. The attachment of the tie to the ends of the arch ought to be very strong.

When the bow or arch of such a structure is made very stiff, the vertical bars may be considered as simple suspending-rods, and the diagonals as stiffening bars bearing scarcely any strain. If, on the other hand, the arch requires to be stayed at intervals, then, for the sake of precaution, the trellis may be calculated as in the case where the arch might be made of articulated bars. This hypothesis does not entail so much expense as might be thought at first sight, inasmuch as certain practical exigencies otherwise lead already to the vertical rods being given a somewhat stronger section. As to the diagonals, they may remain slender, on condition that they are calculated for stresses of tension, *i.e.*, in the case where rods exist following the two diagonals of each interval, duty must only be expected from that which is brought into tension by the load considered. If the arch is parabolic, it maintains its figure of equilibrium under a uniform load per running mètre of the platform, extended to its whole length; then the diagonals only work under partial loads, extending over only a portion of the roadway. The vertical bars, like portions of the trellis opposed to the deformations of the arch, fulfil the office of compressed members, but at the same time they act as suspending rods under tension; it may be conceived, therefore, that the resulting stress on them will always remain as a stress of tension, and that in every case their accidental compression, if such occur, will be but small. Also, the stiffening of these pieces at the price of an increased section, would be determined, not so much by the chance of compression, as by the need of seeking in them fixed points of attachment for the stiffening or gusset-pieces necessary to prevent turning over.

In a work published in 1865 (Étude comparative sur les Ponts en Fer; published by Lacroix), the Author of the Paper has given formulæ (Chap. XII.) expressing the stresses on the various parts of bow and string girders, as well as the comparison between this and other systems of construction.

[APPENDIX.

APPENDIX.[1]

I. *Application to a Design for an Iron Arch Bridge over the Rhone, near St. Maurice, for the Western Railway of Switzerland.* (Plate 5^A.)

1. The Author had the honour a short time ago to submit to the Institution a memoir on Arch-Bridges in Metal and Timber. It may be objected to that essay that it is confined to the vague terms of general theory: and for this reason, the Author, having received a commission from the West of Switzerland Railway Company to design an iron arch-bridge, to be substituted for the temporary wooden structure at St. Maurice, believes the Institution would be interested to receive a *résumé* of the calculations he has made for this design. A particular application, in spite of its imperfections, or even by reason of its imperfections, may sometimes better show the difficulties of the problem, and better illustrate certain facts connected with it, than pure general investigations.

The accompanying plate gives the principal features of the proposed arch. The skew-opening[2] is 68·19 metres between the abutments: the chord of the arch is 69 metres, measured between the extremities of the mean fibre, these extremities being keyed on a nucleus of steel, shaped as a segment of a circle, to serve as a pivot, as shown in detail No. 6. By virtue of these pivots the bending moment will be always = 0 at the supports or extremities of the mean fibre.

The two arch-ribs are constructed as for a square bridge, and abut on normal supporting faces: but they are placed one in advance of the other, to provide for the skew. The cross-girders and vertical supports are normal to the axis of the work (see transverse section, Fig. 5), to avoid the complication of oblique joints.

2. The load on each of the two arch-ribs of the bridge, per running metre on the horizontal, is estimated as follows:—

		Kil.
Permanent dead load	p =	2,500
Live, or moving load	p' =	2,000
Maximum total load	$p + p'$ =	4,500

[1] This Appendix was received at different dates, after the original Memoir, and had not been translated or circulated previous to the discussion.

[2] The dimensions have been retained in their original form, as the metric system is now thoroughly understood and much used by English Engineers.—Tr.

The arch receives the load in a discontinuous manner by the vertical struts; but these being tolerably numerous, it is permissible to regard the distribution of weight as sensibly uniform. In principle, the only difference between a continuous load and a load applied at points some distance apart is, that in the latter case the locus of the bending moments is polygonal, instead of being curvilinear.

The mean fibre is an arc of a circle (Fig. 7) presenting a chord of 69 metres = $2a$, with a rise f = 7·575 metres. Consequently, its radius $r = \dfrac{a^2 + f^2}{2f}$ = 82·352 metres.

The vertical component Q of the reaction of the abutment, in the case of the complete load, is $Q = 4500 \times \dfrac{69}{2} = 155{,}250$ kil.

To obtain the horizontal thrust T, which depends on the deformation of the arch, recourse must be had to the following complicated formula, due to Messrs. Belanger and Bresse:

$$T = \frac{2\tau a E + \int \frac{\mu_1 y}{I} ds + \int \frac{N_1}{\omega} dx}{\int \frac{y^2}{I} ds + \int \frac{1}{\omega} \frac{dx}{ds} dx},$$

and in which the integrals are defined for the whole length of the arc or of the chord.

E = 14,000,000,000 is the modulus of elasticity of the metal, assumed homogeneous.

τ = elongation, per unit of length, due to causes independent of the load, such as expansion by heat, or an artificial wedging-up.

ω = area, and I = moment of inertia of the normal section of the arch at the point (x, y) of the mean fibre.

ds = elementary arc; $\dfrac{dx}{ds}$ is the cosine of the angle α of the normal with the vertical.

μ_1 = partial bending moment exerted round the point (x, y) by the weight and the reaction Q considered alone; or, in other words, the moment determined solely according to the vertical forces, abstraction being made of T, so that the complete moment μ should be $= \mu_1 - Ty$. The moment is regarded as positive when it tends to turn from $0x$ to $0y$ (*i.e.*, in a direction contrary to that of the hands of a watch), the portion of the solid situated to the right of the section under consideration.

N_1 = longitudinal force which stretches the section considered, always excluding the action of T. Thus the true total normal tension will be $N = N_1 - T \cos a$; that is, the sum of the components, normal to the section, of the forces acting between this section and one extremity of the solid. The term 'tension' is purposely employed here, for the formula is written under this hypothesis. It is therefore necessary to remark that in the bridge arches, where N acts by compression, this force must be given a negative value. This will be seen, indeed, by inspection of the formula; for if N_1 shortens the arch, it ought to diminish the thrust T. Similarly, τ would be negative if it were a question of contraction by cold, tending to loosen the arch from its abutments.

3. The expression for T comprises quadratures which will be approximately effectuated here by dividing, with this view, the arch into an arbitrary number of finite parts, not necessarily corresponding to the vertical struts, as the load is assumed continuous. If, for example, the semi-arch is subdivided into twelve parts of equal amplitude $= 2° 3' 50''$, the abscissæ and ordinates will have the value given in the annexed table (p. 101). For an angle a with the vertical, the abscissa, measured from the middle, is $x' = r \sin a$; and, reckoned from the extremity 0 (Fig. 7), it becomes $x = a \pm r \sin a$. The ordinate will be $r(1 - \cos a)$ starting from the tangent at the summit; it becomes $y = f - r + r \cos a$, measured above the chord, f being the rise or versed sine. The length Δs of the equal divisions taken on the arc is $= \pi r \dfrac{2° 3' 50''}{180°} = 2{\cdot}9664$ metres.

In this table will be found two kinds of abscissæ, those x' measured from the middle of the opening, and those x reckoned from the left extremity 0 (Fig. 7). In order to designate any particular point, the value corresponding to x' may be used, if treating of a symmetrical load; but in the case of a load unequal on the two halves of the arch, recourse must be had to the abscissæ x to specify, without the embarrassment of sign, which of the two symmetrical points is intended. The table establishes clearly the correspondence of x, x', y, and a.

The table also contains other columns, the mode of calculating which is as follows:

The bending moments μ_1 being estimated solely according to the vertical forces, neglecting the horizontal thrust T, are independent of the ordinate y; they have the same values as they would have for a straight beam. They may be obtained graphically, for whatever load, knowing that their geometric locus is the funicular polygon. For the present case of uniform and complete load, they

Angle α of the Section with the Axis.	Abscissæ. From the middle x'.	Abscissæ. From the Extremity 0 x.	Ordinate on the Chord y.	Moment μ_1.	Force N_1.	Half-thickness of the Arch $\frac{e}{2}$	Area of the Section ω	Moment of Inertia I.	$\frac{\mu_1 y}{1000\ I}$	$\frac{N_1 \cos \alpha}{\omega}$	$\frac{y^2}{I}$	$\frac{\cos^2 \alpha}{\omega}$
° ′ ″	m.	m.	m.		k.	m.	m²					
0	0	34·500	7·575	2678060	0	0·350	0·07282	0·0078728	2576800	0	7288	13·7
2 3 50	2·966	37·466 and 31·534	7·522	2658260	− 480	0·352	0·07290	0·0079405	2518200	− 6580	7126	13·7
4 7 40	5·928	40·428 and 28·572	7·361	2599000	− 1920	0·359	0·07318	0·0082693	2313500	− 26170	6552	13·6
6 11 30	8·882	43·382 25·618	7·095	2500560	− 4310	0·369	0·07358	0·0087515	2027200	− 58248	5752	13·4
8 15 20	11·825	46·325 22·675	6·722	2363140	− 7640	0·383	0·07414	0·0094013	1689900	−101980	4806	13·2
10 19 10	14·752	49·252 19·748	6·243	2188390	−11892	0·401	0·07486	0·0103938	1314400	−156290	3750	12·9
12 23 1	17·660	52·160 16·840	5·659	1976260	−17042	0·423	0·07574	0·0116117	963150	−219770	2758	12·6
14 26 51	20·546	55·046 13·954	4·971	1728270	−23067	0·419	0·07678	0·0131456	653510	−290930	1880	12·2
16 30 41	23·405	57·905 11·095	4·179	1445580	−29933	0·479	0·07798	0·0150446	401545	−368030	1160	11·8
18 34 31	26·233	60·733 8·267	3·285	1129700	−37604	0·514	0·07938	0·0174376	212820	−449040	619	11·3
20 38 21	29·028	63·528 5·472	2·290	782210	−46044	0·551	0·08086	0·0201790	88768	−532880	260	10·8
22 42 12	31·784	66·284 2·716	1·194	405050	−55202	0·592	0·08250	0·0236421	20456	−617270	60	10·3
24 46 2	34·500	69·000 and 0	0	0	−65039	0·636	..	..	0	−700880	0	9·8

are represented by the ordinate of a parabola, or by

$\mu_1 = \frac{p+p'}{2}(a^2 - x'^2) = 2250\,(1190{\cdot}25 - x'^2)$, the formula from which they have been calculated.

The longitudinal forces N_1 are calculated by $(p+p')(a-x)\sin\alpha$, which will always give negative values, regarding the angle α as negative when taken to the left of the vertical of the summit, in which case $x < a$. This expression may also be written

$$N_1 = -(p+p')\frac{x'^2}{r} = -\frac{4500}{82{\cdot}352}x'^2,$$

$$\text{or simply } -(p+p')\,r\sin^2\alpha.$$

Passing on to the calculation of the areas ω, and of the moments of inertia I of the sections: at present the arch has been only introduced by the figure of its mean fibre, and by a rough estimate of its weight, which enters into the valuation of the dead load p. But it is now desirable to give entirely, *à priori*, the dimensions of this member; the calculations only take the form of a verification *à posteriori*, not of a direct solution of the problem.

As a basis for the preliminary hypothesis, it may be considered in the first place what section would be necessary if the flexure were = 0 in the case of the complete load, *i.e.*, if the arch had the parabolic form. The thrust T would then be expressed by the simple formula of the suspension bridge—

$\frac{p+p'}{2f}a^2 = \frac{4500 \times \overline{34{\cdot}50}^2}{2 \times 7{\cdot}575} = 354$ tons, and the total oblique thrust against the abutment would become $\sqrt{354^2 + (4{\cdot}5 \times 34{\cdot}50)^2} = 386$ tons. These stresses would require a section at the summit = 0·0506 metres, and at the springing = 0·055, making the metal do a duty of 7 kilogrammes per square millimètre.

It will be necessary to adopt stronger sections, possessing a moment of inertia sufficient to resist the flexures due to the non-parabolic form, and particularly to the displacements of the load, and to the expansion by heat. It might be desirable perhaps to double the preceding sections, if the object were to realize a constant thickness for the webs or sole-plates. But it will be preferable only to add slightly to these dimensions; the calculations of verification may then probably discover some points which are too weak, and it will thus suffice to apply strengthening plates solely to the defective parts, in imitation of what is done with straight beams.

4. Let the section at the summit, drawn in Fig. 8, and whose area is = 0·0728, be tried. It is for convenience of execution that

this somewhat irregular form has been adopted; it will, however, be remarked that the mass of matter is distributed symmetrically above and below the neutral axis, in order that this axis may occupy the middle of the height. The part in surplus of the lower member is compensated for by an inequality of the horizontal plates, in such wise that the section may theoretically be assimilated to Fig. 9, which is perfectly symmetrical. Its moment of inertia will be

$$I = \frac{0{\cdot}9 \times \overline{0{\cdot}748}^3 - 0{\cdot}58 \times \overline{0{\cdot}7}^3 - 0{\cdot}258 \times \overline{0{\cdot}672}^3 - 0{\cdot}042 \times \overline{0{\cdot}5}^3}{12}$$

$$= 0{\cdot}0078728.$$

Departing from this section at the key, the mean fibre is traced with the radius $r = 82{\cdot}352$ metres, and the curve of the intrados with a radius $r' = 79$ metres only. These two curves are not concentric; their distance apart $\frac{e}{2}$, measured on the normal to the mean fibre, depends on the distance apart b of the centres of the two circles and on the variable angle a of the normal under consideration. It is calculated by the formula

$$\frac{e}{2} = r - b\cos a - \sqrt{r'^2 - b^2 \sin^2 a} = 82{\cdot}352 - 3{\cdot}002 \cos a - \sqrt{6241 - (3{\cdot}002 \sin a)^2}.$$

Where $b = 82{\cdot}352 - 79 - 0{\cdot}350 = 3{\cdot}002$ metres, on the understanding that the curve of the intrados of 79 metres radius is taken on the layer separating the angle-irons and sole-plates, so that the half-depth at the summit is $\frac{e}{2} = 0{\cdot}350$ metres (Fig. 8 or 9). The radical term differs but very little from 79; the greatest difference is at the extreme joint ($a = 24^\circ\, 46'\, 2''$), which gives $78{\cdot}990$. By laying down the variable half-depths $\frac{e}{2}$ above the mean fibre, the curve of the extrados is obtained.

The section from the summit to the springings only varies by the fact of the increase of e. The area ω only increases by virtue of the increase of height of the vertical plates. The successive moments of inertia I are calculated as above, by the aid of a table of cubes. It would be useful to have tables still more convenient, in order to obtain quickly the values of moments of inertia, the laborious calculation of which is inherent in all problems of resistance.

5. The elements are now prepared for the calculation of T. Having taken an even number of uniform divisions $\Delta s = 2{\cdot}9664$

metres, measured on the arch, with a view of applying Simpson's approximate formula of quadrature, all may be expressed in ds in the formula of T; that is to say, that it will be written thus, substituting $ds \cos a$ for dx:

$$T = \frac{2\tau a E + \int \frac{\mu_1 y}{I} ds + \int \frac{N_1 \cos a}{\omega} ds}{\int \frac{y^2}{I} ds + \int \frac{\cos^2 a}{\omega} ds}.$$

Table No. 2 contains the successive values of the quantities

$$\frac{\mu_1 y}{I}, \quad \frac{N_1 \cos \alpha}{\omega}, \quad \frac{y^2}{I} \text{ and } \frac{\cos^2 a}{\omega},$$

easy to calculate with the elements furnished by the preceding columns.

The arch being symmetrical and symmetrically loaded, it suffices to consider here one-half of it. According to Simpson's parabolic method of quadratures, every integral of the form $\int z\, ds$ will be the product of the third part of the equidistance $\left(\text{for example } \frac{\Delta s}{3}\right)$ by a sum compounded—

Of the sum of the two extreme ordinates $(z_0 + z_n)$;

Of the quadruple sum of the ordinates with an odd index $4(z_1 + \ldots\ldots + z_{n-1})$;

And of the double sum of the intermediate ordinates with an even index $2(z_2 + \ldots\ldots + z_{n-2})$.

Thus

$$\int \frac{\mu_1 y}{I} ds = 40476990000 \, . \, \frac{\Delta s}{3}, \quad \int \frac{N_1 \cos a}{\omega} ds = -9511976 \, . \, \frac{\Delta s}{3},$$

$$\int \frac{y^2}{I} ds = 115109 \, . \, \frac{\Delta s}{3}, \text{ and } \int \frac{\cos^2 a}{\omega} ds = 443 \, \frac{\Delta s}{3}.$$

Omitting, at first, the dilatation τ, due to the temperature or to the initial wedging-up,

$$T_1 = \frac{40467480024}{115552} = 350210 \text{ kil.}$$

The terms in $\cos a$ and ω might perhaps have been neglected without great error; for, without taking account of them, there would be

$$\frac{40476990000}{115109} = 351640.$$

The difference between this value and the former one is less than the unknown errors that might be produced by the expansion and the wedging. At any rate, the terms in $\frac{N_1 \cos \alpha}{\omega}$, which arises from the shortening of the mean fibre, has but small influence; without it, there would follow that

$$\frac{40476990000}{115552} = 350293.$$

It is to be remarked that the circular form has not much modified the thrust 351 tons, given by the parabolic curve, in Article 3.

6. Now let $\tau = 0{\cdot}0004$; this will be the linear variation due to an increase of temperature of 33° centigrade, or to a less increase, accompanied by a certain amount of wedging-up. This effect gives rise, in the numerator of the formula of T, to a supplementary term

$$2\,a\,\tau\,E\,\frac{3}{\Delta s} = 69 \times 0{\cdot}0004 \times 14000000000 \times \frac{3}{2{\cdot}9664} = 390777000;$$

it produces, consequently an increase of thrust

$$= \frac{390777000}{115552} = 3380 \text{ kil.}$$

The initial wedging-up is unknown, or arbitrary; the temperature varies. These effects, also, may very well be neutralized by an opposite cause; such as the settlement of an abutment, which may allow the arch to expand slightly while the dilatation recompresses it. Now write separately the bending moments $\mu = \mu_1 - T_1 y$ and the forces $N = N_1 - T_1 \cos \alpha$ due to the loading only, then the augmentations $-3380\,y$ and $-3380 \cos \alpha$ produced by the supplement of thrust 3380 kil., due to the admitted dilatation. These augmentations will then only enter into account for the sections whose conditions of resistance are aggravated by them.

Abscissæ, starting from the Middle x'	Effect of the Complete Load.		Supplements due to the Expansion T.	
	μ	N	For μ	For N
0	+25220	−350210	−25600	−3380
2·966	+23982	−350460	−25425	−3378
5·928	+21104	−351220	−24880	−3370
8·882	+15820	−352480	−23980	−3360
11·825	+ 9328	−354220	−22720	−3345
14·752	+ 2030	−356440	−21100	−3325
17·660	− 5578	−359100	−19130	−3300
20·546	−12622	−362200	−16800	−3273
23·405	−17947	−365700	−14125	−3240
26·233	−20740	−369570	−11103	−3204
29·028	−19770	−373780	− 7740	−3163
31·784	−13100	−378280	− 4036	−3118
34·500	0	−383040	0	−3070

7. In any section whatever, submitted to a moment μ, and to a longitudinal tension N, the work R, per unit of surface, on the fibres at the distance v from the neutral axis, is expressed by $R = \frac{v\mu}{I} - \frac{N}{\omega}$. By the convention, our values of N being compressions, are negative. For the maximum of the stress R, the fibres must be considered as the farthest removed from the neutral axis, for which $v = \frac{e}{2} + 0{\cdot}024$ metre (Fig. 9); and we may take v positive for the upper fibre, or that of the extrados, and negative for the intrados. The fibre which the flexure tends to break by extension is aided by the pressure $-$ N; on the opposite fibre, however, the two stresses μ and N act together to produce crushing; this dangerous point of the section is found at the extrados when μ is positive, and at the intrados if μ is negative. The value of μ is found to be positive in the central region, and negative in the external portions; the arch tends slightly to sink in the middle and to rise at the haunches, when not acted on by expansion.

The expansion, or a wedging-up of the arc, produces negative moments. Taking the intensity $\tau = 0{\cdot}0004$, these moments overcome those of the load, so that the central sinking disappears, and the arch rises throughout.

The section at the summit is strained at the extrados, when τ is nul; the maximum work, at this point, is then:

$$R = \frac{0{\cdot}374 \times 25220}{0{\cdot}0078728} + \frac{350210}{0{\cdot}07282} = 1198100 + 4809200$$
$$= 6007300 \text{ kil.}$$

per square metre, or 6 kilogrammes per square millimetre. The expansion above specified almost destroys at this place the effect of flexure, bringing back the centre of pressure upon the mean fibre. The special curve of pressure which would give the expansion considered alone, would be directed according to the chord of the arch.

When the expansion acts the greatest absolute value of μ is found at the abscissa $x' = 23{\cdot}405$, where we have $\mu = -17947 - 14125 = -32072$, with $N = -365700 - 3240 = -368940$. The maximum stress per unit of surface is, in this section, at the fibre of the intrados:

$$R = \frac{0{\cdot}503 \times 32072}{0{\cdot}0150446} + \frac{368940}{0{\cdot}07798} = 1042500 + 4731200 = 5773700.$$

At the preceding joint $x' = 20{\cdot}546$, the actions are a little less, and the section is a little weaker also.

In conclusion, the case of the complete loading of the bridge does not give any strains reaching the limit imposed of 7 kilogrammes per square millimètre.

But this is not the most dangerous test that the bridge has to submit to; it is necessary to consider the arch as loaded on one half only, and unloaded on the other half. The foregoing calculations are, however, not useless; for, according to a known theorem, the thrust estimated for the case of symmetrical loading will furnish, in a very simple manner, the new thrust when the load is unsymmetrically disposed.

Case where the Load ($p' = 2000$ Kil.) only covers half the Arch.

8. The dilatation τ, which will exert precisely the same effects as above, since it is independent of the load, may be neglected.

The arch supports a dead load $p = 2500$ kilogrammes per horizontal running metre, on the left half, *i.e.*, between the abscissæ $x = 0$ and $x = a = 34{\cdot}500$ metres (measured from the left abutment); then it carries $p + p' = 4{,}500$ kilogrammes on the right half, from $x = 34{\cdot}5$ to $x = 69{\cdot}0$.

If the symmetry were re-established by loading the left semi-span, it has been seen that the thrust would be $T_1 = 350{,}210$ kilogrammes.

If, on the contrary, the symmetry be re-established by unloading the portion to the right, so that the whole bridge be reduced to its dead weight p, the thrust T_0 would be obtained by reducing T_1 in the ratio of p to $p + p'$, *i.e.*, $T_0 = 350210 \times \frac{2500}{4500} = 194560$.

To return to the actual case of unsymmetrical load, it suffices to take the arithmetical mean of T_1 and T_0, that is, the horizontal thrust will be $T = 272385$.

The vertical component Q, of the reaction of the right abutment will be $Q = pa + \frac{3}{4}p'a = 138000$ kilogrammes, and that of the left abutment $pa + \frac{p'a}{4} = 103500$. Between $x = 0$ and $x = a = 34{\cdot}500$ metres, the bending moment will be expressed by $\mu = px\left(a - \frac{x}{2}\right) + \frac{p'ax}{4} - Ty$; then, beyond by

$$\mu = px\left(a - \frac{x}{2}\right) + \frac{p'ax}{4} - \frac{p'}{2}(x - a)^2 - Ty.$$

On the left half, the longitudinal force is $N = \left[p(a - x) + \frac{p' a}{4}\right]$ $\sin \alpha - T \cos \alpha$, and on the other half ($x > a$) it becomes

$$N = \left[(p + p')(a - x) + \frac{p' a}{4}\right] \sin \alpha - T \cos \alpha.$$

This force N is always negative (compression), taking α negative for $x < a$.

The terms in p may be deduced from the values of μ_1 and N_1 in the preceding calculations, reduced in the ratio $\frac{p}{p + p'} = \frac{5}{9}$. Or the operations may be abridged, as is done for straight beams, by the aid of diagrams or graphic delineations; after having calculated a limited number of moments μ and of forces N, they may be drawn as ordinates of a geometrical locus, the figure of which may be finished by hand, and which will serve afterwards for graphic interpolation.

Neglecting the term $- T y$, the expressions of μ are parabolic functions, of which the second differences are constant. This property may be made use of to obtain rapidly a large number of successive values corresponding to the values of x, increasing by equal steps.

More conveniently still, by cutting out a parabolic curved template in cardboard, having $\frac{1}{p}$ for semi-parameter, this curve will serve to draw, as at A O B (Fig. 10), the representative locus of the μ due to the dead load p. Afterwards, the additional values of μ due to p' will be represented in O D B by a rectilinear portion O D, followed by an arc tangent D U B of a parabola; this latter may be traced with a second template curve having a semi-parameter $= \frac{1}{p'}$, the axis of which is kept perpendicular to the abscissæ x. Then, for any abscissa O M, the moment μ_1, due to the vertical forces alone, is represented by the accumulated ordinate V U. It remains to deduct from it $T y$, a value proportional to the ordinate y of the mean fibre above the chord. This $T y$ may itself be obtained practically by reducing, in the proper ratio, by a reducing-compass, the ordinate of the arch on a large-scale drawing.

9. The values of μ and N, given below, have, however, been calculated directly. The last columns are obtained by adding algebraically to the former ones the supplements already known (No. 6) produced by the expansion.

—	Abscissæ starting from the Left Abutment x	Without Expansion.		With Expansion.	
		Moment μ	Force N	μ	N
Left abutment	0	0	−290690	0	−293760
Unloaded side	2·716	− 53350	−288600	− 57390	−291720
	5·472	− 91810	−286560	−102550	−289720
	8·267	−124570	−284580	−135670	−287780
	11·095	−143810	−282680	−157935	−285920
	13·954	−153170	−280890	−169970	−284160
	16·840	−153015	−279220	−172145	−282520
	19·748	−141075	−277680	−165175	−281000
	22·675	−126810	−276280	−149530	−279625
	25·618	−101460	−275050	−125440	−278410
	28·572	− 68270	−273990	− 93150	−277360
	31·534	− 28110	−273100	− 53535	−276480
Summit . .	34·500	+ 19620	−272385	− 5980	−275765
Loaded side .	37·466	+ 65420	−272350	+ 40000	−275730
	40·428	+101100	−272360	+ 76220	−275730
	43·382	+126080	−273250	+102100	−276610
	46·325	+141330	−274725	+118610	−278070
	49·252	+147240	−276780	+126140	−280100
	52·160	+144350	−279390	+123220	−282690
	55·046	+133530	−282530	+116730	−285800
	57·905	+115890	−286180	+101765	−289420
	60·733	+ 92310	−290300	+ 81210	−293500
	63·528	+ 64060	−294870	+ 56320	−298030
	66·284	+ 32960	−299820	+ 28924	−302940
	69·000	0	−305140	0	−308210

This table brings into view the dangerous points. Without expansion, the loaded side would be the most strained as far as the abscissa $x = 49{\cdot}252$, and this upon the extrados, as it is sinking (positive moment). Farther on, the danger is transferred to the intrados of the unloaded half span which swells out. Thus the point $x = 16{\cdot}840$ is exposed to the moment $- 153015$, while the symmetrical point $x = 52{\cdot}160$ is only exposed to the moment $+ 144350$, and the quantities N are nearly equal. The calorific expansion, or an artificial wedging-up of the arch between its abutments, ameliorates the condition of the loaded portion, but aggravates that of the unloaded half.

The greatest moment is realized at the section $x = 16{\cdot}840$. The stress is

$$R = \frac{0{\cdot}447 \times 172145}{0{\cdot}0116117} + \frac{282520}{0{\cdot}07574} = 6626800 + 3730100 = 10356900,$$

or more than 10 kilogrammes per square millimètre, on the fibre of the intrados. The extrados supports a tension $= 6626800 - 3730100 = 2896700$, nearly 3 kilogrammes per square millimètre.

At $x = 19{\cdot}748$ the forces are less, but the section is less also. The maximum stress is

$$R = \frac{0{\cdot}425 \times 165175}{0{\cdot}0103938} + \frac{281000}{0{\cdot}07486} = 6754000 + 3753700 = 10507700.$$

These amounts being too high, it is necessary to strengthen the arch. It suffices to apply a supplementary plate from the abscissa 5·472; for in this section, in the actual state of things, the stress, including expansion, presents itself under the admissible value

$$R = \frac{0{\cdot}575 \times 102550}{0{\cdot}020179} + \frac{289720}{0{\cdot}08086} = 6505200.$$

Similarly, the reinforcement may be stopped at 3 metres distance from the summit; for, at the abscissa $x = 37{\cdot}466$ metres, we find

$$R = \frac{0{\cdot}376 \times 65420}{0{\cdot}0079405} + \frac{272350}{0{\cdot}0729} = 6833800;$$

and the symmetrical point $x = 31{\cdot}534$ is less fatigued. It is clear, besides, that two symmetrical sections of the arch are exposed reciprocally to the same accidents when the load is moved.

The deficit of strength being considerable, the arch will not only be strengthened at the haunches, but its thickness at the key will be augmented 5 centimètres, in spite of longer calculations being entailed.

New Calculations, with Strengthening Plates at the Haunches.

10. The old theoretical section (Fig. 9) is now replaced by that of figure 13, where the height between the sole-plates is increased to 0·750 metres, and the thickness of the soles to 26 millimètres, instead of 24. In addition to this first general reinforcement, supplementary plates of 12 millimetres will be applied on the haunches, between the abscissæ 7·50 metres and 30 metres, and then between 39 metres and 61·50 metres (Fig. 11); so that in these parts the sole-plates may attain the thickness of 38 millimètres.

The depth e will vary according to the radius of the intrados, which we will take now = 78·50 metres, the radius of the mean fibre remaining always = 82·352 metres.

At the joint $x = 19{\cdot}748$, the half-depth $\frac{e}{2}$ will be = 0·433 metres, and the distance of the extreme fibre at the neutral axis will become

$v = 0{\cdot}433 + 0{\cdot}038 = 0{\cdot}471$; the area becomes $\omega = 0{\cdot}10134$, and the moment of inertia $I = 0{\cdot}017615$. If, then, the moment μ, and the reaction N, preserve values of the preceding case, the greatest stress would be reduced to

$$\frac{0{\cdot}471 \times 165175}{0{\cdot}017615} + \frac{281000}{0{\cdot}10134} = 7189400.$$

At this point the researches might be stopped, and the reinforcement made a little thicker, since this last figure slightly exceeds the limit adopted, of 7 kilogrammes per square millimètre. But the series of calculations will be taken up anew under the modified conditions indicated, reserving the intention of adding hereafter other smaller and shorter reinforcements at the points which betray weakness.

The strengthening mentioned above produces some increase in the dead load. Put it at $p = 2650$ kilometres per metre: the total load will be $p + p' = 4650$, or $\frac{1}{30}$ greater than that of the former calculations; so that μ_1 and N_1 themselves will have, in the case of complete loading, the values of the Table No. 2, augmented by $\frac{1}{30}$.

In general, a second calculation is always more rapid than a first trial, not only because the process becomes more certain, but because a great number of results already obtained are utilized again. With this view it is important to preserve the sketches and memoranda of the arithmetical operations, the logarithms of cos a, of y, etc., arranged in order. The student should not be too much afraid of extended calculations, made methodically; the labour becomes almost mechanical, and consequently rapid; and the accidental errors almost always betray themselves, on a glance of revision of the series of figures; or, if necessary, at their first differences.

In the present case, there are the sudden variations of section at the points where the additional plates begin and end. The theory assumes a gradual variation; it is not, therefore, absolutely correct to employ the same formula for the calculation of the thrust; but that approximation will doubtless suffice.

11. The following table is obtained for the complete load:

	Abscissæ starting from the Summit x'.	Half Depth of the Arch between the Sole Plates $\frac{e}{2}$	Area ω.	Moment of Inertia I.	μ_1.	N_1.	$\frac{\mu_1 y}{1000\ I}$	$\frac{N_1 \cos \alpha}{\omega}$	$\frac{y^2}{I}$	$\frac{\cos^2 \alpha}{\omega}$
	m.	m.								
	0 (Summit)	0·375	0·07742	0·009623	2767330	0	2178380	0	5963	12·9
	2·966	0·377	0·07750	0·009730	2746870	− 497	2123530	− 6409	5815	12·9
Strengthened portion.	5·928	0·384	0·09938	0·013843	2685630	− 1984	1539280	− 19912	4219	10·0
Strengthened portion.	8·882	0·396	0·09986	0·014722	2583910	− 4455	1245270	− 44352	3419	9·9
Strengthened portion.	11·825	0·412	0·10050	0·015940	2442220	− 7894	1029900	− 77733	2834	9·7
Strengthened portion.	14·752	0·433	0·10134	0·017615	2261340	−12290	801450	−119313	2213	9·6
Strengthened portion.	17·660	0·460	0·10242	0·019902	2042135	−17610	580667	−167940	1609	9·3
Strengthened portion.	20·546	0·490	0·10362	0·022617	1785880	−23836	392520	−222760	1092	9·0
Strengthened portion.	23·405	0·524	0·10498	0·025920	1493770	−30930	240836	−282480	638	8·8
Strengthened portion.	26·233	0·564	0·10658	0·030117	1167360	−38857	127329	−345590	358	8·4
	29·028	0·608	0·10834	0·026279	808280	−47580	70435	−410990	200	8·1
	31·784	0·657	0·11030	0·030947	418550	−57040	16148	−477070	46	7·7
	34·500 (Naissance)	0·709	0·11238	..	0	−67210	0	−543050	0	7·3

The use of Simpson's formula leads to

$$\int \frac{\mu_1 y}{I} d s = 27925604000 \frac{\Delta s}{3},$$

$$\int \frac{N_1 \cos \alpha}{\omega} d s = 7325136 \frac{\Delta s}{3}, \int \frac{y^2}{I} d s = 76735 \frac{\Delta s}{3} \text{ and}$$

$$\int \frac{\cos^2 \alpha}{\omega} d s = 342 \frac{\Delta s}{3}.$$

Consequently, the thrust due to the loads will be

$$T = \frac{27932929136}{77077} = 362410 \text{ kilogrammes.}$$

Now take the unfavourable case, where the moveable load of 2000 kilogrammes per metre is only applied on one half of the arch. The thrust will be a mean between the above thrust of complete load, and that of the arch unloaded, which is

$$= \frac{2650}{4650} \times 362410 = 206530.$$

It will therefore be = 284470 kilogrammes.

An expansion τ = 0·0004 will give rise to an increase of thrust

$$= \frac{390777000}{77077} = 5070 \text{ kilogrammes.}$$

12. The following table is calculated as in Article 9:

—	Abscissæ starting from the Left Abutment x.	Without Expansion.		With Expansion.	
		Moment μ.	Force N.	μ.	N.
Left abutment .	0	0	−303720	0	−308320
	2·716	− 54280	−301600	− 60330	−306280
	5·472	− 96400	−299410	−108010	−304150
	8·267	−126610	−297290	−143260	−302100
	11·095	−146120	−295270	−167300	−300130
	13·951	−155630	−293360	−180830	−298270
Unloaded side .	16·840	−155530	−291590	−184220	−296540
	19·748	−146580	−289960	−178230	−294950
	22·675	−129250	−288500	−163330	−293520
	25·618	−103850	−287210	−139820	−292250
	28·572	− 70610	−286100	−107930	−291160
	31·534	− 30400	−285190	− 68530	−290260
Summit. . .	34·500	+ 17350	−284470	− 21050	−289540
	37·466	+ 63130	−284410		
	40·428	+ 98760	−284470		
	43·382	+123690	−285400		
	46·325	+138880	−286940		
	49·252	+144740	−289070		
	52·160	+141880	−291760		
Loaded side .	55·046	+131070	−295000		
	57·905	+113580	−298770		
	60·733	+ 91270	−302940		
	63·528	+ 62460	−307710		
	66·284	+ 32040	−312820		
	69·000	0	−318280		

The last columns have not been finished, because the expansion is only prejudicial to the unloaded side.

On the loaded side the greatest stress will be found at the extrados, at the abscissa $x = 46{\cdot}325$; its value is

$$R = \frac{0{\cdot}450 \times 138880}{0{\cdot}015940} + \frac{286940}{0{\cdot}1005} = 3920700 + 2855120 = 6775820.$$

The preceding point $x = 49{\cdot}252$ gives $R = 6722640$.

On the unloaded side, the greatest stress, with expansion, is produced near $x = 19{\cdot}748$, where

$$R = \frac{0{\cdot}471 \times 178230}{0{\cdot}017615} + \frac{294950}{0{\cdot}10134} = 4765600 + 2910500 = 7676100$$

kilogrammes per square metre for the compression at the intrados; and $R = 4765600 - 2910500 = 1855100$ for the tension on the extrados.

The preceding point $x = 16{\cdot}840$ gives

$$R = \frac{0{\cdot}498 \times 184220}{0{\cdot}019902} + \frac{296540}{0{\cdot}10242} = 4609600 + 2895300 = 7504900.$$

At the point $x = 31{\cdot}534$ metres, deprived of the strengthening-plate,

$$R = \frac{0{\cdot}403 \times 68530}{0{\cdot}00973} + \frac{290260}{0{\cdot}0775} = 2838400 + 3745300 = 6583700.$$

13. It will be seen that it is necessary again slightly to strengthen the weak part of the haunches in order to limit the stress to 7000000, at least if we wish to be independent of the aid of the cross ties, which in reality maintain the arch against the flexure, and diminish consequently the calculated stresses.

An interesting question presents itself here: since it is the inequality of the load on the symmetrical sides which most threatens the stability, would it not be possible to ameliorate the conditions of the work by an addition of dead weight at suitable points? This is the case in some measure, as the following calculation will show.

Suppose two weights P, equal and symmetric, placed at the points $x = 19{\cdot}748$ and $x = 49{\cdot}252$ (*i.e.*, $x' = \pm\ 14{\cdot}752$), then let it be enquired what is the increase of thrust due to these weights?

For this special load it is necessary to prepare a table analogous to that of Article 11. The quantities I and ω will remain the same; μ_1 will here be $= P\,(a - x')$ between $x' = a = 34{\cdot}50$ metres and $x' = 14{\cdot}752$; then near the middle (for $x' < 14{\cdot}752$) they retain the constant value $19{\cdot}748$ P. As to the longitudinal forces N_1 developed by the two weights P, setting aside the thrust sought, they will be null in the middle part, and equal to $P \sin \alpha$ (with a negative sign) between $x' = 14{\cdot}752$ and $x' = 34{\cdot}500$. They may be omitted in the calculation of the thrust, which they influence very little. The result obtained is:

Abscissæ x' (from the middle) =	0	2·966	5·928	8·882	11·825	14·752	17·660	20·516	23·405	26·233	29·028	31·784	34·500
$\frac{\mu_1}{P}$ =	19·748	19·748	19·748	19·748	19·748	19·748	16·840	13·954	11·095	8·267	5·472	2·716	0
$\frac{\mu_1 y}{I\,P}$ =	15545	15267	11319	9517	8328	6999	4788	3067	1789	902	477	105	0

Simpson's formula gives

$$\int \frac{\mu_1 y}{I}\, ds = 212375 \cdot \frac{\Delta s}{3} \cdot P.$$

Consequently the increase of thrust

$$= \frac{212375\ P}{77077} = 2{\cdot}755\ P.$$

At the joint $x = 19{\cdot}748$ (where $x' = 14{\cdot}752$) found in the preceding article to be dangerous, the auxiliary load of the two symmetric weights P gives an additional positive moment $= 19{\cdot}748\ \mathrm{P} - 2{\cdot}755\ \mathrm{P} \times 6{\cdot}243 = 2{\cdot}553\ \mathrm{P}$, which will reduce the principal negative moment. This reduction, however, should be limited by the condition of not carrying the danger of rupture upon the symmetric section $x = 49{\cdot}252$, in the case of non-expansion. Leaving aside the quantities N, which differ little, the equality of the two dangers, on the two symmetrical points, will take place for $178230 - 2{\cdot}553\ \mathrm{P} = 144740 + 2{\cdot}553\ \mathrm{P}$; this equation gives approximately $\mathrm{P} = 6560$ kilogrammes. Then the total moment, at the point $x = 19{\cdot}748$, becomes $-178230 + 2{\cdot}553 \times 6560 = -161480$; the force N becomes $-294950 - 2{\cdot}755 \times 6560 \cos a = 312730$, and the maximum stress

$$\mathrm{R} = \frac{0{\cdot}471 \times 161480}{0{\cdot}017615} + \frac{312730}{0{\cdot}10134} = 4317700 + 3086000 = 7403700,$$

instead of the value 7676100.

This advantage would be realized by lowering the roadway-plates near the portions S T of the longitudinal section (Fig. 3), in order to increase the thickness of the layers of ballast. But it is evident that the advantage would be very small; it is based on pure hypotheses of expansion, and would change to a disadvantage if it should happen, on the other hand, that a settlement of the abutment caused the arch slightly to expand. And, finally, the bracing, though light, would probably have more efficacy in diminishing the theoretical flexures.

14. If, therefore, the intervention of the auxiliary weights P be given up, and if no account be taken of the aid of the bracing, it only remains to determine the new strengthening plates required.

Now, by Article 12, the sections $x = 13{\cdot}954$ and $x = 25{\cdot}618$ are found to have a stress on them, the first of

$$\mathrm{R} = \frac{0{\cdot}528 \times 180830}{0{\cdot}022617} + \frac{298270}{0{\cdot}10362} = 7100000,$$

and the second of

$$\mathrm{R} = \frac{0{\cdot}434 \times 139820}{0{\cdot}014722} + \frac{292250}{0{\cdot}09986} = 7048500.$$

The joint $x = 11{\cdot}095$ gives $\mathrm{R} = 6486400$. It will thus suffice to strengthen the arch between the abscissæ 13 metres and 26 metres, by the aid of additional plates of 5 millimètres, a thickness which ought to suffice, as may be verified by an approximate calculation. This thickness, however, would be weak when compared with that, 12 millimètres, of the first strengthening plate; and we will prefer to reduce the latter to 10 millimètres, and lengthen the 4 metres of the second portion to 7 millimètres thickness. New calculations of verification may be dispensed with, as the new addition further contributes slightly to cause the part to offer more resistance to expansion.

The weight of the arch is estimated at 210 tons, of which 130 are for the two arch ribs, and 80 for the other portions.

15. If it is desired to trace the curve of pressure, the operation will be easy when the calculations have been made of the moments μ and the longitudinal forces N; for the quotient $\frac{\mu}{N}$ gives the ordinate or distance of the centre of pressure from the mean fibre; above it if μ is positive, and below it towards the intrados if μ is negative. For example, with the values of the table, Art. 12, without expansion, the curve of pressures dotted on Fig. 11 is obtained. This curve emerges slightly from the arch at M and M′. For example, at M, the distance of the centre of pressure is $= \frac{146580}{289960} = 0{\cdot}505$, while the extreme fibre is only $0{\cdot}433 + 0{\cdot}038$ $= 0{\cdot}471$ metres from the mean fibre.

It may have been remarked, in certain parts of the application which forms the object of this Memoir, that some uncertainty has been expressed as to the unknown effects of the wedging-up, the expansion, or the settlement, which enter into the calculation of the strains in metallic arches. The adoption of a pivot, or free articulation at the summit, would eliminate these unknown quantities. The reason which has prevented the Author from proposing this in the present design is the smallness of the breadth in proportion to the length or span, a circumstance which might lead to a fear of lateral derangements.

The inconvenience, not ascribable to theory, just pointed out in regard to rigid arches, may be compared to that which is found in continuous straight beams of several spans, in which an unequal settlement of the various piers may seriously interfere with the conditions of resistance. The great wrought-iron viaduct of the Pandèze, near Lausanne, may be cited as an example. One of the abutments, situated near some subterranean excavations, underwent a settlement of more than 0·10 metre, causing a considerable curvature of the beams over the adjoining pier. It is clear that in such a case the iron must be subject to much higher

DESIGN FOR AN IRON ARCH BRIDGE O

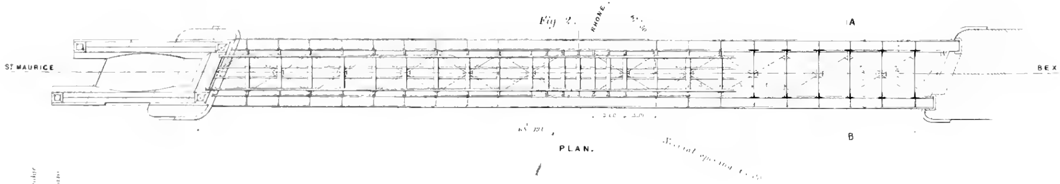
NEAR ST MAURICE, WEST SWITZERLAND R

(Not executed.)

Fig 1.

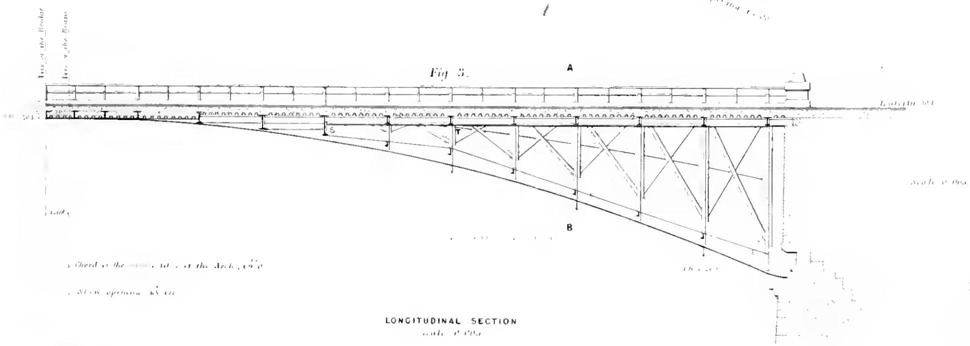
GENERAL ELEVATION.

Fig 2.

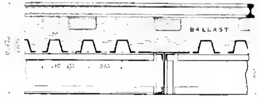


PLAN.

Fig 3.



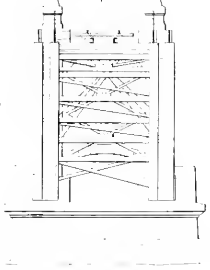
LONGITUDINAL SECTION



SECTION 1_2.

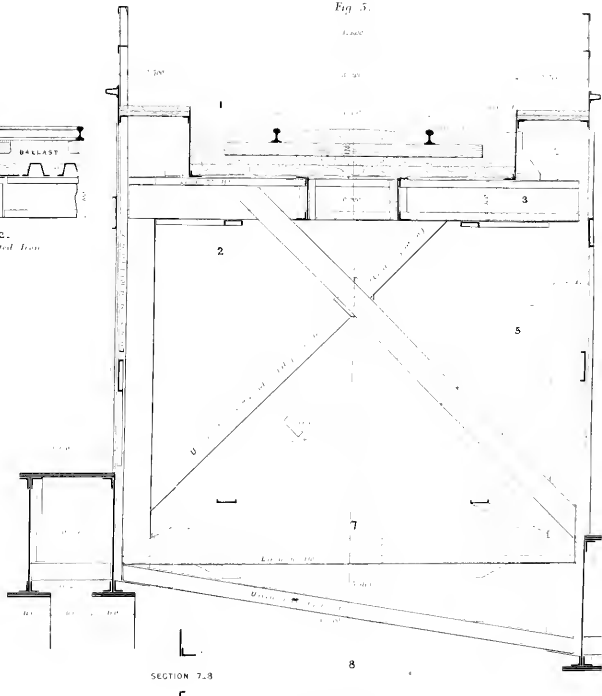
Platform Corrugated Iron

Fig 4



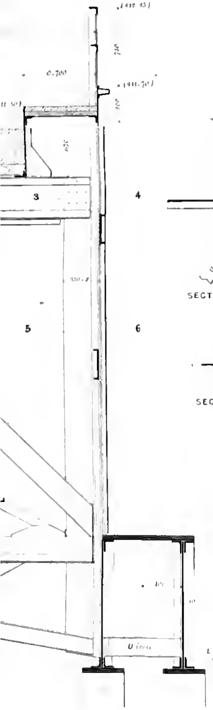
TRANSVERSE SECTION.

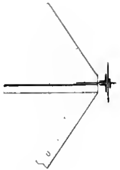
Fig 5.



NORMAL TRANSVERSE SECTION AT A.B.

CH BRIDGE OVER THE RHONE.
EST SWITZERLAND RAILWAY.
(executed.)





SECTION 3_4.

SECTION 5_6.

Data of the first Calculation.

Fig 7.

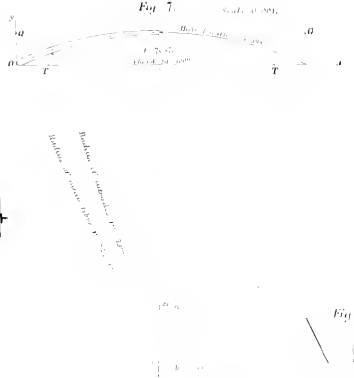
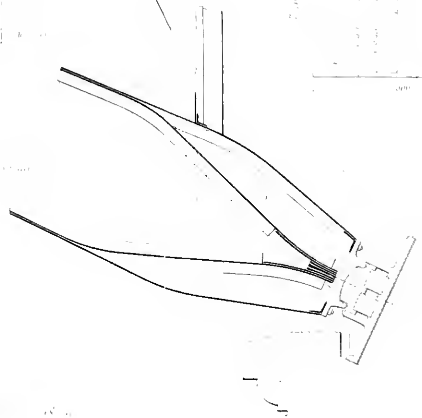
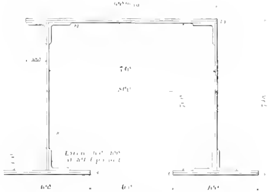


Fig 6



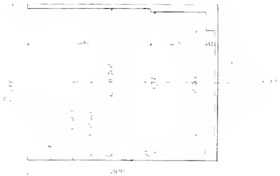
PIVOT.

Fig 8



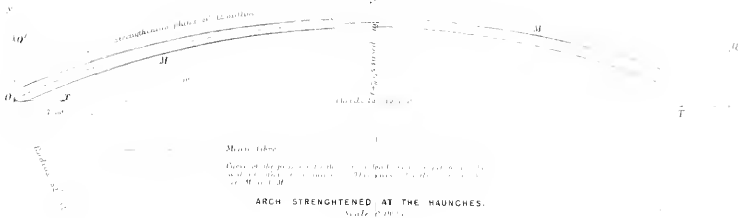
SECTION AT THE SUMMIT.

Fig 9



Second Calculation (No 10)

Fig 11



ARCH STRENGHTENED AT THE HAUNCHES.

Fig 10

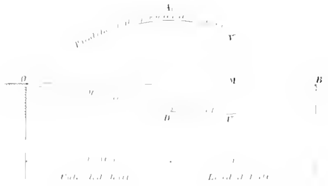
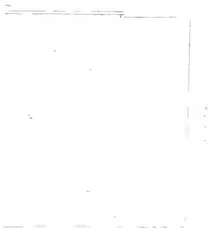


DIAGRAM OF THE BENDING MOMENTS AS FOR A STRAIGHT BEAM

Fig 14



SECTION STRENGHTENED AT THE SUMMIT

Fig 12

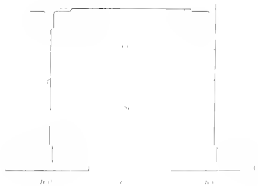


Fig 13

SECTIONS AT THE SUMMIT.

stress than was intended in the calculations of the design, and this trial was continued for a certain time before measures were taken for remedying the evil.

The calculations described in the foregoing Paper do not appear of an unreasonable length when large works are in question. For designs of less importance the labour may be much simplified by making the arch of uniform section. In this case, in fact, the tables of M. Bresse (" Mécanique Appliquée," 1st Part, " Sur la Résistance des Matériaux et Stabilité des Constructions ") furnish immediately the value of the thrust, so that it only remains to calculate the bending moments and the longitudinal forces at the sections whose resistance has to be verified. If the verification is not satisfactory, either another and stronger arch, still of uniform section, may be tried, or reinforcements may be added to the haunches, dispensing with a second calculation, for which the tables referred to would not suffice.

In the use of the formula for T in Art. 5, equal divisions on the arch have been taken. If it is preferred to take equal distances on the horizontal, with the view of giving values of x in round numbers, so as to simplify somewhat the calculations, the formula, for a circular arc, will be written in this form, in a function of the variable x:

$$T = \frac{2\,a\,\tau\,E + \int \frac{\mu_1\,y}{I\cos\alpha}\,d\,x + \int \frac{N_1}{\omega}\,d\,x}{\int \frac{y^2}{I\cos\alpha}\,d\,x + \int \frac{\cos\alpha}{\omega}\,d\,x}.$$

The term $\int \frac{N_1}{\omega}\,d\,x$ may be neglected without material inconvenience, as explained in Art. 5.

APPENDIX II.

II. *Further Remarks and Illustrations.*

1. This note forms a second Appendix to the Memoir which the Author has had the honour to address to the Institution on the subject of Arched Bridges. The first Appendix consisted of a detailed numerical application of the theory of rigid arches to a design for an iron bridge over the Rhone, near St. Maurice on the Western railway of Switzerland. Since that was written, the Author has prepared another design, with straight girders, for the same site. This new design has been found somewhat cheaper than the former one; it is estimated at the sum of 170,000 francs, while the arch bridge had been estimated at 220,000 francs. For this reason the girder-bridge has been adopted by the company, and is now in course of execution. The principal features of this bridge are represented in Plate 5^B, which will perhaps offer some interest as a comparative study. The arch design given in the first Appendix, although not carried out, does not on that account lose the utility it was intended to have as an example of calculation and as furnishing the opportunity of adding some general observations on the resistance of curved arches.

The economy found in favour of the new design does not imply, as a general rule, that the arch is more costly than the straight girder, for it will be remarked on comparing the two designs—

1st. That the normal water-way, allowed for the river stream, has been reduced in the later design to 60 metres, while in the earlier one it was 63 metres.

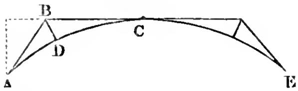
2nd. That the small shore openings have been made by light iron platforms, instead of by masonry arches, which would have been better architecturally, but more costly.

3rd. That the heavy loading of ballast, indicated in the first design with the object of lessening the vibrations, has disappeared in the final plan.

These various modifications might have been effected without abandoning the arch form, which, by its elegance and by the height of the free passage which it offered at the centre, would have been doubtless preferable for a navigable river, in the interior of a large city. But the Rhone, in this region, is not navigable; and, as at this remote locality questions of architecture were of very little weight, the economy of construction became of the first importance. The expense was found to be about the same in the two systems, when brought to parallel conditions; and hence it was considered preferable to adopt the plan which was most simple, in which the calculations were most certain, and the execution the most convenient, and where the expansion might take place with full liberty.

2. If the new design had been made according to the same type as the first, the Author would have sought to sustain the haunches of the arch by an armature of pieces A B, B C, B D (Fig. 15) entering into the system of the spandrel filling. Such an armature would resist with advantage the settlement of the rising of the haunches, either loaded or unloaded.

Fig. 15.



It is true the rigorous calculation would become more complicated and more doubtful than for the simple arch, but also the structure, being less left to itself, and being thus only able to take an imperceptible play, would probably not depart materially from the conditions of uniform pressure on each section, which would admit the use of simple approximate calculation.

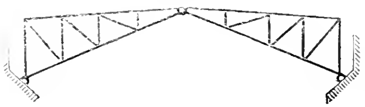
For example, the point D might be assimilated to a simple articulation, as is done, with no more exactness, in the lattices of straight beams. Or, in the first instance, the arch A D C E might be studied alone, without taking account of the armature, and making it sufficiently light to present a certain deficiency of resistance. Then, instead of covering this deficiency by strengthening plates added to the soles of the arch at the abutments (as proposed in the first Appendix), it should be done by the addition of the armature pieces. If, for example, the point D of the arch was found subject to a bending moment μ, for which the moment of resistance of the section was insufficient, B C might be given such a resistance that the moment of this with respect to the point D would be capable alone of equilibrating μ.

3. In the example treated in the first Appendix the load was regarded as transmitted from the platform to the arch by the intervention of the vertical risers, so that the weights retained their primitive direction. If it were a question of a bridge with radiating spandrel bars, analogous to the Victoria bridge, Pimlico, the arch would receive no more than the normal components of the load, these being decomposed between the direction of the radiating rods and the longitudinal bar of the roadway, which is subject to compression. If the load were further so arranged as to be distributed uniformly over the circumference of the arch, the strict figure of equilibrium would become circular, while it is parabolic in the case of vertical risers.

4. Some engineers propose to anchor the longitudinals to the

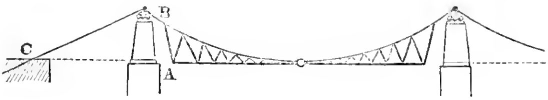
abutments, with the object of being able to consider the arch as composed of two isolated halves, disjointed at the summit, after the fashion of huge brackets or corbels, fixed (*encastrées*) in the masonry. This point of view, if it were followed out by really leaving a void space in the middle, would do away with the inconvenience of the expansion of the arch; but it would lead to a wasteful expenditure of metal, as the longitudinal would become as strong as the arch itself. Indeed, this is nearly the disposition adopted in certain swing-bridges in two halves, equilibrated at the back ends, but it is the dead weight alone which is thus balanced. When the bridge is shut, the two halves are rejoined together, in order to give the passing loads the benefit of the central butting joint. The abutment, therefore, cannot be dispensed with, except on the system of moveable pivots, if it is desired to render the expansion entirely free. Perhaps, in this case, there would be some advantage in substituting for the curved arch two triangular brackets (Fig. 16), whose points would be less slender. This is nearly the form of the swing-bridge at Brest. The calculations would be simple, and would follow the considerations of Arts. 4 and 5 of the original memoir.

Fig. 16.



Suspension-bridges, instead of presenting a multitude of joints, may be reduced to two links, or three joints to each span. The rigidity so often desired will then be obtained, still leaving liberty for expansion. It is stated that this is what has been done in the suspension-bridge at Frankfort. M. Bridel, Engineer-in-Chief of the Regulation of the Streams in the Jura (Switzerland), also proposes to adopt this disposition in a work on his district. It would seem that this is the true solution of the problem of rigid suspension-bridges; for each half-span may be latticed without inconvenience to the expansion, provided that a little play be left at A, Fig. 17,

Fig. 17.

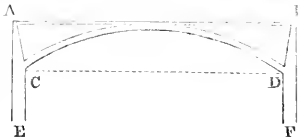


between the platform and the pier, in order to allow the expanded part to lower itself by moving on the central hinge. Rolling carriages B, at the summit of the piers, will keep the anchoring tie B C, stretched at all temperatures. In the case of several spans, the oscillation of these moveable saddles might be limited by the addition of bracing ties.

5. *Moveable abutments; Bridges of several arches.*—It may be desirable to consider a peculiar cause of perturbation in the state of equilibrium of bridge arches. The formula of thrust which has been given and applied to an example is based on the hypothesis of immoveable supports. This hypothesis being admitted, there is nothing special to be said on bridges of several arches, every one of these being similar to an independent and isolated span. But this invariability of the separating piers is not rigorously true, as is proved by the fact, determined experimentally, that the load of one span may produce an elevation of a neighbouring arch.[1]

The abutments even of a single-arch bridge may yield slightly, at least in certain particular cases. For example, the ceiling arches of the Palais de l'Exposition de 1867, at Paris, had for abutments only light metal pillars, A E and B F, Fig. 18, maintained against upsetting by an upper tie rod A B. In this combination, the chord C D does not remain invariable; it lengthens either by the stretching of the rod A B, which displaces the summits A and B, or by the flexure of the pillars, pressed laterally at C and D, and sustained only at their extremities. An analogous disposition occurs in the bridge of Szègédin, on the Theiss (Hungary), where the longitudinal member is utilized to hold together, by their summits, the metallic tubular piles.

Fig. 18.



6. The question of the mobility of the piles, under the influence of unequal loading of the contiguous arches, has been raised, and illustrated by a wood model presented to the Institution by Mr. E. A. Cowper.[2] It will be desirable to add here some theoretical considerations on the subject.

It is necessary to know, or at least to assume, by some hypothesis more or less plausible, the lateral resistance at the summit of the piers, *i.e.*, the quantity by which they are displaced under various intensities of resulting horizontal thrust. For piers which have no lateral resistance, as would be the case if the arches abutted on rolling saddles, the thrust of an arch would only be resisted by that of the adjoining arch; the point of support would be dis-

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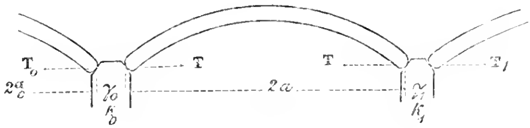
[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvii., pp. 66 and 74.

[2] *Vide* Ibid., p. 101.

placed until the opposing thrusts were equal, one arch resisting flattening, the other resisting a tendency to rise. This case would render inadmissible pivots at the summits, for then the arch that is crushed would no longer react with an increasing resistance; it would even tend to lose more and more a portion of its primitive resisting power, so that the other arch would thrust to falling. In the ordinary case, where the pier offers a certain degree of resistance, it will cease to move so soon as this resistance, added to that of the unloaded arch, has acquired a value equal to the thrust of the preponderating arch. This resists flattening under less favourable conditions than if the support was immoveable, and the contiguous span resists the former one by its resistance to rising, if it has no central pivot.

Consider a loaded arch (Fig. 19), of which the primitive

Fig. 19.



chord $2a$ has augmented by $\gamma_0 + \gamma_1$, by virtue of the accumulated displacement of the two piers. The left pier having moved γ_0, exerts, in consequence, a horizontal reaction k_0, connected with γ_0 by a relation which is either given, or may be estimated as best we may; for example, by assimilating the pier to an upright cantilever, which bends a quantity γ_0 under the lateral stress k_0. Similarly, the pile on the right side displaced γ_1, reacts by a resistance k_1, which co-operates with the thrust T_1 of the following arch. The equilibrium of the piers furnishes the conditions:

$$T = T_0 + k_0 = T_1 + k_1 \quad . \; . \; . \; . \; [1].$$

Further, the theory of the deformation of the elastic curved piece, instead of being based on the supposed invariability of the chord, will express on the contrary that this has increased by $\gamma_0 + \gamma_1$, a quantity which is a function of known form, $f(k_0, k_1)$, of the developed lateral reactions k_0, k_1. Thus, with the same notations as already employed in Arts. 2, 5, and 15 of the first Appendix:—

$$T = \frac{2\tau a - f(k_0, k_1) + \int_0^{2a} \frac{\mu_1 y}{E I} \frac{d s}{d x} d x + \int_0^{2a} \frac{N_1}{E \omega} d x}{\int_0^{2a} \frac{y^2}{E I} \frac{d s}{d x} d x + \int_0^{2a} \frac{1}{E \omega} \frac{d x}{d s} d x},$$

or,

$$T = \frac{2\tau a E - E f(k_0, k_1) + \int \frac{\mu_1 y}{I} d s + \int \frac{N_1 \cos \alpha}{\omega} d s}{\int \frac{y^2}{I} d s + \int \frac{\cos^2 \alpha}{\omega} d s},$$

or,

$$T = \frac{2\tau a E - E f(k_0, k_1) + \int \frac{\mu_1 y}{I \cos \alpha} d x + \int \frac{N_1}{\omega} d x}{\int \frac{y^2}{I \cos \alpha} d x + \int \frac{\cos \alpha}{\omega} d x}. \qquad [2].$$

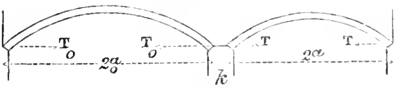
Any one of these equivalent formulæ [2], added to the double equation [1], connects the five unknown quantities T_0, T, T_1, k_0, k_1; and by writing out the analogous equations for the different arches of the bridge, the determination of the problem may be arrived at.

Three particular cases will be considered.

First Case. *A single Arch, bearing against moveable Abutments.*—The arch rests, acquiring the increase of the chord which developes in the abutments the required resistance. T_0 and T_1 are made null, so that there only remain the three unknown quantities T, k_0 and k_1, determined by one equation [2], and the two equations [1]. It results from this that $T = k_0 = k_1$, so that T is obtained immediately by the formula [2], by substituting in it $2 \phi T$ for $f(k_0\, k_1)$; the function ϕT expressing the recoil of one abutment under a stress T.

Second Case. *Bridge of two Arches, with immovable Abutments, and one moveable Pier.*—With equal spans equally loaded, the pier does not move, each arch preserving its chord unaltered. But suppose the thrust T_0 and T unequal (Fig. 20), then each span

Fig. 20.



gives rise to an application of the formula [2], and the displacement of the pier causes to one of the chords $2a$ the loss of the elongation gained by the other chord $2\,a_0$. Thus, by writing the abridged notations X, Y, instead of the sums of the integrals, the unknown thrusts T_0, T, and the supplementary reaction k, which is developed by the movement of the pier, will be given by the equations—

$$T_0 = \frac{2\,\tau\,a_0 - f\,(k) + X}{Y} \text{ and } T = \frac{2\,\tau\,a + f\,(k) + X'}{Y'},$$

joined to the equation of equilibrium of the pier $T_0 = k + T$. If k remained constantly null, *i.e.*, if the pier did not present any resistance to displacement, this displacement $f\,(k)$ would only be arrested at the value resulting from the condition $T_0 = T$. In this case the notation $f\,(k)$ would no longer be correct, and should be replaced by a single letter γ.

If the abutments themselves were moveable, they would introduce supplementary unknown quantities analogous to k; but their conditions of equilibrium would also furnish two more equations.

Third Case. *Three Spans; Abutments immovable; two flexible Piers.*—The five unknown quantities, T_0, T, T_1, k_0, k_1 (Fig. 19), will be determined by the equilibrium of the two piers, $T_0 = \mp\,k_0 + T$, $T = k_1 + T_1$, and the deformations of the three arches:

$$T_0 = \frac{2\,\tau\,a_0 \mp f\,(k_0) + X}{Y}, \quad T = \frac{2\,\tau\,a \pm f\,(k_0) - F\,(k_1) + X'}{Y'},$$

$$\text{and } T_1 = \frac{2\,\tau\,a_1 + F\,(k_1) + X''}{Y''}.$$

The upper signs of the ambiguous terms suppose the left span driving the middle one, and this one driving the third; the lower signs suppose the intermediate arch alone to be loaded, and to thrust the two others. If the two piers are in similar conditions of construction, the functions f and F will have the same form.

If k_0 and k_1 remained null for every displacement, the real movements $f\,(k_0)$, $F\,(k_1)$, (called in preference γ_0 and γ_1), would render equal the three thrusts T_0, T, T_1, and would be calculated by this condition.

Analogous considerations would apply to any number of spans.

7. *Numerical Example.*—Let us take the example of a bridge with three equal spans $2\,a = 69$ metres, perfectly similar to that of the design treated of in the first Appendix; and let us first consider the case where the arch of the left bank is the only one loaded. With the sections of the second trial, the calculations of

Art. 11 in the said paper will furnish, immediately, without expansion—

1st arch, loaded with 4650 kil. per metre, $T_0 = \frac{- E f(k_0 + 27932929136}{77077}$,

2nd „ „ 2650 „ „ $T = \frac{E f(k_0) - E F k_1) + 15918766067}{77077}$,

3rd „ „ 3 „ „ $T_1 = \frac{E F(k_1) + 15918766067}{77077}$;

also $T_0 = k_0 + T$, $T = k_1 + T_1$.

Admitting the same form of the functions f and F, it remains to assign this form. This is the delicate point, the uncertain element which renders the calculation but imperfectly applicable. The fault lies not in the theory, but in the variable, doubtful, or heterogeneous conditions of the foundation, and even of the substance of the piers. To complete the calculation, let us assume, for example, that the displacement of the pier, at the origin of the arches, provided that it remains very small, shall be proportional to the excess k of thrust, and equal to 1 millimètre per 5000 kilograms, for the portion of the pier acted on by the arch. This condition is expressed by replacing $f(k_0)$ and $F(k_1)$ respectively by 0·0000002 k_0 and 0·0000002 k_1. This being done, let k_0 and k_1 be eliminated, and take E = 14,000,000,000, then these three equations are obtained :

$$79877\, T_0 - 2800\, T = 27932929136,$$
$$82677\, T - 2800\,(T_0 + T_1) = 15918766067,$$
$$79877\, T_1 - 2800\, T = 15918766067;$$

from which we deduce T_0 = 357120 kil., T = 211640 T_1 = 206,710.

Thus, in consequence of the compressibility of the right abutment, the arch of the left bank, loaded alone, only exerts a thrust of 357 metric tons, instead of 362. due to the hypothesis of immoveable supports. As to the two following arches, unloaded, the last, which is slightly compressed, gives a thrust of 206·7 tons, instead of 206·5, and the middle one 211·6, instead of 206·5. The second pier moves scarcely a millimètre, while the first one moves 0·0000002 $(T_0 - T)$ = 0·029 metre.

8. Now suppose the same bridge, loaded only on the second half of the central arch. The thrust of this arch half loaded is the arithmetical mean between the thrusts unloaded and completely loaded, for the same state of the chord. Then—

$$T_0 = \frac{2800\,k_0 + 15918766067}{77077},$$

$$T = \frac{21925847600 - 2800\,(k_0 + k_1)}{77077},$$

$$T_1 = \frac{2800\,k_1 + 15918766067}{77077},$$

and $$T = T_0 + k_0 = T_1 + k_1.$$

From this is found—

$k_0 = k_1$; then $T_0 = T_1 = 209083$ kil., and $T = 279360$.

Thus the lateral opposing arches acquire an energy of thrust equal to 209 tons, instead of 206½ tons, which they exert on the immovable piers; and the thrust of the central arch, half-loaded, is restricted to 279 tons, instead of 284 tons. The movement of the supports of this intermediate span favours evidently the unloaded side, diminishing its upheaval, while it aggravates the sinking of the loaded half. If it is required to know to what extent this takes place, it will suffice to repeat the calculations of resistance similar to those of Art. 12 in the first Appendix.

For the point of the abscissa $x = 46{\cdot}325$ metres, this No. 12 gives a pressure of 6·78 kilograms per square millimetre. In order to know what the actual case would give, it is necessary first to calculate, at the point under consideration, the bending moment μ, and the longitudinal force N, according to the formulæ No. 8 (first Appendix). It will be found that $\mu = 173230$, $N = 281880$, and consequently the molecular stress

$$R = \frac{0{\cdot}450 \times 173230}{15940} + \frac{281880}{100500} = 7 \cdot 70 \text{ kil. per square millimètre.}$$

It would seem then that the extension of the theory of elastic arches to the case of abutments susceptible of yielding under the load is not difficult. The uncertainty which exists as to the mode of resistance is attributable, not so much to the processes of the theoretical solution, as to the doubtful nature of the data of the problem; to the initial compression of the arches, and to the degree of mobility of the piers.

9. In the example which has thus been treated, the displacement which well-constructed piers may suffer has probably been exaggerated. Such at least would appear from the following considerations on the deflections by the effect of the load.

The sinking at the summit of a flat circular arch, with a uniform section, by virtue of a load p' per horizontal running-metre, and under a linear expansion τ, is valued approximately by

$$\frac{25\,p'r^2 f}{2\,E\,(8\,\omega f^2 + 15\,I)}\left(1 + 0{\cdot}012\,\frac{\omega f^4}{I\,a^2}\right) \pm 1{\cdot}56\,\tau\,r,$$

r being the radius of the arch, f its rise or versed sine, ω the area of the section, and I its moment of inertia.

An arch entirely free, submitted to heat, would increase its chord by $2\,\tau\,a$ and its rise by τf; but the chord being retained at its initial value $2\,a$, the rise increases considerably, and M. Bresse estimates it at $1{\cdot}56\,\tau\,r$ for an arch sufficiently flat. Now, by analogy, it may be said that if the chord sustains an increase $2\,\tau\,a$ by the fact of the recession of the piers, the summit of the arc will be depressed by $1{\cdot}56\,\tau\,r$.

In the example already considered, $r = 82{\cdot}352$ metres, $f = 7{\cdot}575$ metres, and $a = 34{\cdot}50$ metres. If ω and I preserved at all points the minimum values at the summit $\omega = 0{\cdot}07742$ and $I = 0{\cdot}009623$, the depression would be $0{\cdot}0000016\,p' \pm 128\,\tau$. If, on the contrary, the section were retained constant with the maximum values immediately above the springing, *i.e.*, $\omega = 0{\cdot}1103$ and $I = 0{\cdot}030947$, the depression would be reduced to $0{\cdot}000001\,p' \pm 128\,\tau$. And for the variable section of our example, it may be assumed that the result will be intermediate between these two values. Thus, in the case of Art. 7 above, the first span would be depressed by a quantity comprised between 0·0032 and 0·0020, by virtue of the load $p' = 2000$ kilograms, and of another quantity $128\,\frac{0{\cdot}029}{69} = 0{\cdot}053$ metre, by the fact of the elongation of the chord. The second span, not loaded, but having the chord shortened by 0·028 by the mobility of the piers, would undergo an elevation at the centre of 0·052 metre.

Now, an elevation so considerable, by the sole effect of the load, has never, it is believed, been observed. On the Victoria bridge, Pimlico, the tests have only shown an elevation of 4 millimètres.[1]

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvii., p. 66. The bridge of Szègédin, with iron piers, did not give greater elevations than 5 or 6 millimètres according to the following information supplied by the "Nouveau Portefeuille de l'Ingénieur des Chemins de Fer," par A. Perdonnet et C. Polonceau. Texte, p. 366, 8vo., Paris, 1866.

"Lors des épreuves du pont, on a réalisé le cas le plus défavorable à la stabilité des piles, en chargeant chaque travée de 8000 kilog. par mètre courant, toutes les autres travées étant libres, et l'on a remarqué les lois suivantes.

"Toutes les piles fléchissent à la hauteur des naissances, en s'écartant de la travée chargée;

It is therefore probable that the movements of the piers are, in general, insignificant, and need hardly be considered in the face of the much more important effects of expansion. It is seen, really,[1] that the expansion by heat has produced an alteration of level amounting to as much as $1\frac{1}{2}$ inch, or 0·038 metre, twelve times as much as the elevation caused simply by the load on an adjoining span. It may be added, however, that the unknown part played by the spandrels and the longitudinals forbids us from affecting much precision in considerations of this delicate nature.

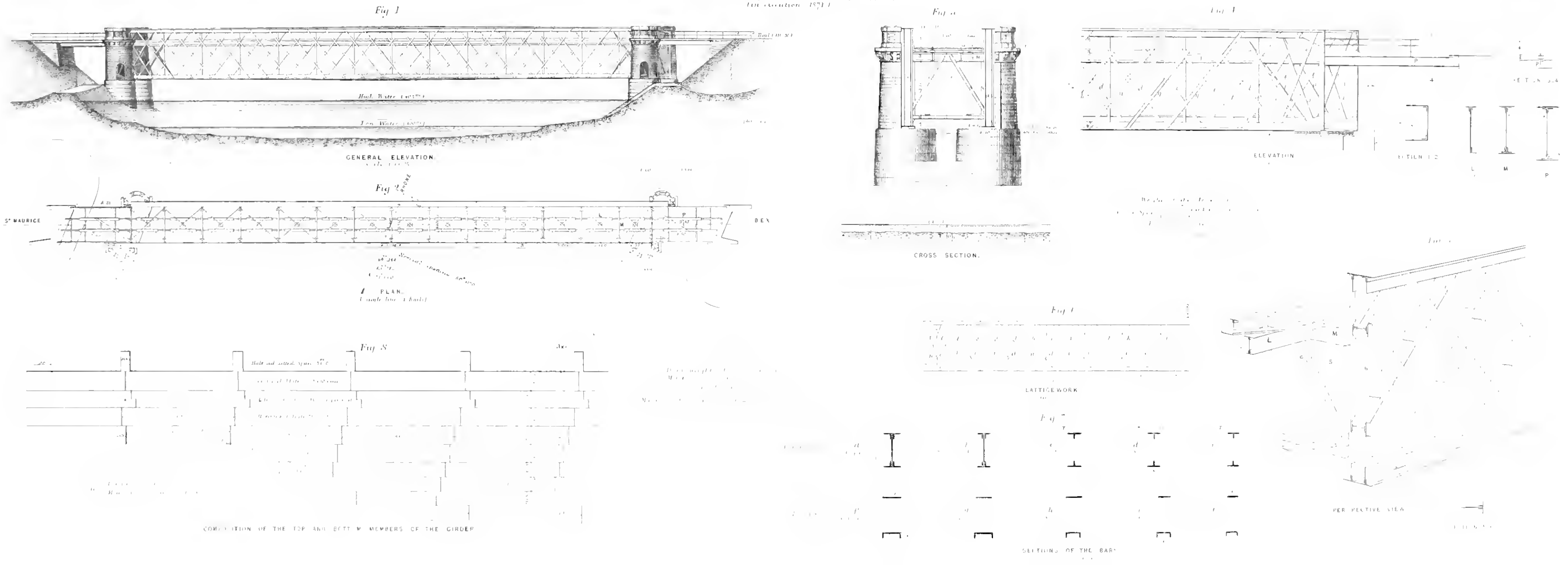
"Les deux piles adjacentes à la travée chargée fléchissent, en moyenne, de 4 millimètres; les deux piles situées à la distance d'une travée fléchissent de $1\frac{1}{2}$ millimètres: ces flèches diminuent rapidement quand on s'éloigne de la travée chargée; elles sont sensibles encore, quoiqu'on ne puisse les mesurer d'une extrémité à l'autre du pont.

"Les dépressions au sommet des travées, qui n'étaient que de 12 millimètres lorsque tout le pont était chargé, atteignaient 30 millimètres pour le travée chargée isolément, ce qui s'explique par l'augmentation de la corde; les deux travées adjacentes se relevaient de 5 à 6 millimètres, les suivantes de 2 millimètres au plus; plus loin, aucun mouvement n'a été observé."

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvii., p. 74.

IRON BRIDGE OVER THE RHONE,

NEAR St MAURICE, WEST SWITZERLAND RAILWAY.



Mr. Bindon B. Stoney observed, through the Secretary, that M. Gaudard very correctly described the usual method of constructing iron arches as one in which the fitness of the spandrels to sustain diagonal strains was altogether ignored, their duty being merely limited to prop up the load of the platform. In arches of this common type, the spandrels consisted of vertical pillars, and either the arch, or the longitudinal bearer, or both together, must be deep and strong enough to support transverse strains, whether arising from the normal curve of pressure produced by the permanent load passing outside the arch, or from some heavy moving load causing a temporary derangement of the same line of pressure. Though numerous bridges had been constructed on this principle, and were doing excellent service, they exhibited a rather lavish expenditure of material, and it seemed to be an unscientific and wasteful method of construction, for it was clear that an arched rib whose depth, regarding the arch as a girder capable of resisting transverse strain, was very moderate, must have much extra material to sustain occasional bending strains to which it might be subject, and which were additional to those of mere compression which it legitimately sustained if correctly formed to the curve of equilibrium for the permanent load. In fact, two calculations should, roughly speaking, be made for such arches. First, they should be made strong enough to support the permanent dead load together with the live load of maximum density, such as a crowd of people or a train of locomotives distributed uniformly over the platform. Secondly, the strains, regarding the arch as a bent girder of shallow depth and subject to transverse pressure from the live load in motion, should be calculated independently and the requisite material for this added to that already required for the permanent and uniformly distributed loads. If, however, the spandrel-filling were properly designed so as to act as bracing, and if what would otherwise be transverse strains in the arch and longitudinal bearer were thus converted into longitudinal ones, which was the *raison d'être* of bracing, the quantity of material in the arch need not be in excess of that required to withstand the longitudinal compression due to the total load uniformly distributed, and much greater liberties might be taken in giving the arch such an outline as might please the eye without risk of its being subject to dangerous transverse strains. Both Rennie and Telford seemed to have been aware of the proper function of the spandrel-filling, as evidenced by the Southwark and Tewkesbury cast-iron bridges. M. Gaudard then described the method of calculating the strains in an articulated system, that was, a braced arch, by the resolution of forces; this method was substantially the same as that already published by Mr. Stoney in 1866.[1]

There were also many points in the methods of M. Mery and M. Durand Claye which appeared to be common to the methods of calculating the stability of arches so fully described in two Papers read at the Institution in 1846, by Mr. William Henry Barlow and Mr. George Snell,[2] and which left little to be desired regarding the theory of stone arches. M. Gaudard also alluded to the plan of keying iron arches on the abutments by a range of wedges, and showed that the unequal action of these wedges might render rigorous calculations illusive. A better plan was to build the skewback up against the springing plate of the arch after the latter was in place. This could be readily done if the skewback was built of brickwork in cement. If, however, the springing plate was bedded on large ashlar masonry, he had adopted the plan, after the arch was in place, of pouring cast zinc into the irregular interval between the ends of the ribs and the springing plates. This made very solid work and gave a uniform bearing over the whole end surface of each rib. The joints of the cast-iron voussoirs of the bridge of Austerlitz in Paris, finished in 1806, were thus formed and a similar method had been adopted in America. Mr. Stoney had also applied cast zinc to make sound butt joints in wrought-iron girder-work under a variety of circumstances where accurate fitting would have been costly, if not impracticable.

In the application of timber to bridges M. Gaudard had described most of the recognised types of construction. It was, however, desirable to refer to that which was probably the simplest and certainly a very efficient form of timber girder for large bridges, namely, the A truss, which was capable of being extended to very considerable spans and, from its great simplicity of construction if properly designed, was suitable for localities where timber bridges were admissible. The design of the cast and wrought-iron bridge in three arches in the park at Neuilly seemed to possess some objectionable features, one of its characteristics being the *encastrément* or bolting of the ironwork to the masonry both of piers and abutments, and M. Gaudard regarded this arrangement with well-merited distrust, and observed that it was difficult to estimate with certainty the internal strains which might arise from expansion. A bridge of this character possessed

[1] *Vide* "Theory of Strains in Girders and Similar Structures," by B. B. Stoney, vol. i., p. 136. 8vo. London, 1866.

[2] *Vide* Minutes of Proceedings Inst. C. E., vol. v., pp. 162 and 439.

few features in common with the arch, but rather resembled a continuous girder whose points of inflexion were subject to movements of unknown amount. It was very prudent, therefore, to construct the horizontal bearer and the central parts of the arches of wrought iron, inasmuch as these parts might in winter be subject to unknown tensile strains, far exceeding the safe working strain of cast iron. M. Gandard evidently admired the gigantic oval tubular bow in the Royal Albert bridge at Saltash, and recommended this form of pillar for isolated arched ribs. Mr. Stoney's experience led him to take a somewhat different view of the merits of this form of pillar. Comparatively little was known about the ultimate crushing strength of cylindrical or oval plate tubes of such gigantic size; but it was reasonable to suppose that large tubes would follow the laws of smaller ones, and that the former, with sides having a small thickness compared to their diameter, would fail by buckling under a much smaller unit-strain than tubes of greater proportionate thickness. His researches led him to infer that the strongest form of tube to resist both flexure and buckling—two very different things, but apt to be confounded —was a rectangular tube in whose corners the great mass of the material was concentrated, leaving the sides chiefly to perform the function of a braced web, that was, to retain the angles in the line of thrust or, in other words, to prevent flexure of the tube. The mass in each corner prohibited them from buckling, and they would therefore be capable of sustaining a unit-strain of compression closely approaching to that which was the ultimate crushing strain of the material in short prisms. In such tubes as the Saltash bow the curvature at the flatter parts of the oval was so slight, that the plates there could derive but little stiffness from it, and notwithstanding that there were cross diaphragms 20 feet apart, these flat parts of the tube rather resembled isolated plates, which were certainly unsuited for resisting deformation; and though the Saltash bridge had apparently a large margin of strength, there was no experimental evidence to show that the crushing strain in the tube did not closely approximate to that which would cause buckling. M. Gandard alluded to the practice of spreading a large amount of ballast over the platform, or otherwise increasing the dead load on a bridge for the sole purpose of producing stiffness, without expressing either approval or the reverse of the practice. However excusable such practice might be in smaller structures, it could scarcely be regarded as desirable in such large bridges as that of Tarascon with seven arches of 203 feet span each; for unless there were something very exception-able in this particular example, it seemed inferior practice to load a bridge with a great mass of inert matter requiring a proportionate amount of costly material in the structure to support it, when the stiffness that was aimed at might be gained by a more skilful distribution of less material. However, the same practice might be found in some of the works of the eminent engineer who designed the Saltash bridge.

Professor W. J. MACQUORN RANKINE observed, through the Secretary, that the Paper was of great practical value as giving a comparative view of the different methods of treatment suited for problems respecting metal and timber arches under different circumstances. All such methods were more or less approximate; and the study of the Paper would enable an engineer to decide which method was best suited to a given case.

He thought the Author had made too strong a statement in Section 1, when he said that "usually the spandrels are neglected, and the arch assumed to resist without their aid." A method of treating the arch and spandrels as an articulated system, substantially identical with that described in paragraphs 2, 3, 4, and 5 of the Paper, had been used in Britain for eight years at least.

The method of M. Durand Claye, paragraphs 10 and 11, was sound in principle and very ingenious; and, so far as he knew, had not previously been published in Great Britain.

He had always been of opinion, that the proposal of M. Mantion, to construct arches with three hinges, at the two points of support and at the crown,[1] referred to in paragraph 12 of the Paper, held out the prospect of great advantages, in doing away with the straining effects, not only of heat and cold, but of the horizontal yielding of piers and abutments; which latter action was by no means to be neglected. He thought it was to be regretted that M. Mantion's proposal had not yet been tried in practice.

He referred to Papers which had appeared in the Civil Engineer and Architect's Journal for November and December 1860 as being worthy of attention in connection with the strength of arched ribs. They had been published anonymously, and he did not know who their Author was. The study of these Papers had been the means of suggesting to him the method followed in his own investigation of the strength of arched ribs, published in 1862; which, so far as he could judge from the information given

[1] *Vide* "Annales des Ponts et Chaussées," 3e serie, tome 20, p. 161. 8vo. Paris, 1860.

by M. Gaudard, had led to results substantially identical with those arrived at by M. Bresse. He had investigated two subjects not referred to by M. Gaudard; the effect of the horizontal yielding of the abutments, and the deflection of the rib.

The first equation in paragraph 13 of M. Gaudard's Paper could easily be modified so as to take account of the yielding of the abutments, by making the coefficient τ include a term varying proportionally to the horizontal thrust. He had met with cases in practice in which it had been necessary to ascertain empirically the value of that term.

It was desirable to have theoretical formulæ for the deflection, because the deflection as found by experiment was sometimes used as a test of the safety of the structure. He believed that the deflection was implicitly contained in M. Bresse's formulæ as given by M. Gaudard; and therefore by suitable treatment of those formulæ its explicit expression could be obtained.

There could be no doubt that the tables of M. Bresse for circular arched ribs were of great utility, and perfectly original.

Mr. G. H. Phipps said, that he did not think the part of the title of the Paper which referred to details was properly carried out. The Paper could not be called practical, as there were hardly any facts given for the guidance of practical men; and although different kinds of construction of arch bridges, formed of different metals, and combinations of metals, were alluded to, no decided opinions were offered by the Author, either in favour of, or against them. He believed that the main point to settle, in the consideration of the strains upon arched ribs, was the projection of the appropriate curve of equilibrium to any given load, and its relation at all points along the curve to the neutral axis of the rib. The strain at any point of the curve of equilibrium, taken into its distance from the neutral axis, constituted the bending moment of the section considered, and its moment of inertia, the resistance to bending; and thus, the amount of angular motion, or pivoting, as it was sometimes called, being known, as well as the square on pressure, these two combined gave the pressure on any filament of the section; on one side of the neutral axis, greater; and on the other, less, than the square on pressure. If the angular motion were sufficient, the strain on one side of the neutral axis might pass from compression into tension. From the above, it would be seen that the original square on pressure per square inch on any given section might be readily (and often was) more than doubled on the outside filaments at one margin of the rib, while upon the other margin the strain had become one of tension; and hence no doubt had arisen the very low measure of direct pressure allowed, or thought safe by engineers, on arched structures in general—about 2 tons on the square inch of the cast iron in Southwark bridge, $2\frac{1}{2}$ tons on the cast iron in Mr. Fowler's bridge over the Severn, 3 tons on the wrought-iron ribs of the Victoria bridge at Chelsea, and $3\frac{1}{2}$ tons on the wrought-iron ribs of Mr. Cubitt's new bridge at Blackfriars.

It being so important then to determine the true curve of equilibrium for any given arch rib, and load; and also, it being possible to draw an infinite number of such curves, starting from the same points at the abutments, but differing in altitude; the question was how that particular curve could be decided upon which alone would be the proper one?

He would state one or two general principles upon which the determination of the true curve of equilibrium, in an arch rib, depended:—

1. When any heavy elastic material, such as an iron arch rib, was supported by forces operating in any other direction than the vertical, the neutral axis became shortened by compression.

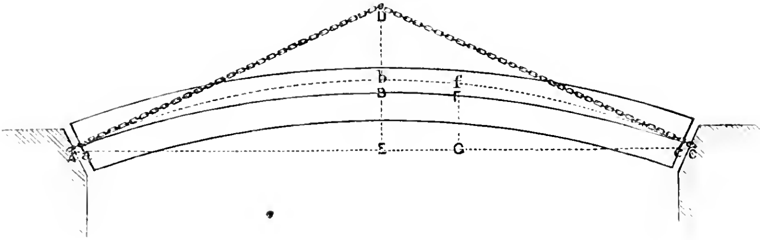
2. When the curve of equilibrium to any given load agreed exactly with the line of the neutral axis, the rib underwent direct compression only.

3. When the curve of equilibrium did not coincide with the line of the neutral axis, there would exist at any point of the latter a bending moment equal to the pressure at that point, taken into its distance from the curve of equilibrium, and there would be, at the same point, an amount of angular motion, directly, as the bending moment, and inversely, as the moment of inertia of the section. The amount of such angular motion would constitute an addition to the square on pressure at one extremity of the section, and a diminution at the other extremity, causing the original compression to pass into tension when the angular motion was sufficient.

Suppose a heavy arch rib, of which A B C, Fig. 21, was the neutral axis, and also the curve of equilibrium to the load, and the points A and C, the places on the neutral axis where the rib rested upon the skewbacks of the abutments. Now the first operation of the load was to compress the rib along the line of the neutral axis by some certain amount, regulated by the compressibility and strain upon the square inch of the material—suppose by the distances A *a* and C *c*, at the extremities of the rib.

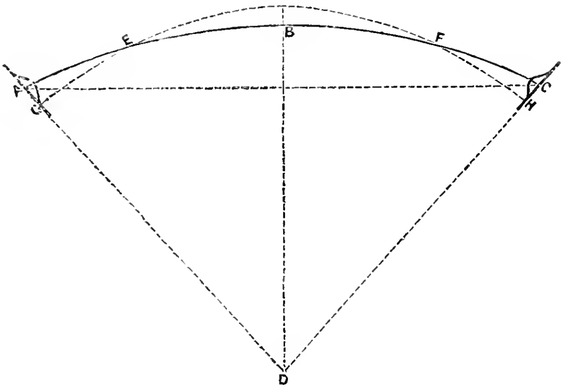
Next suppose for a moment such a rib, instead of resting upon immovable abutments, to be supported by two chains, A D and D C,

Fig. 21.



constantly tightened up, in proportion as the load was applied, it might be actually separated from the abutments by the above distances; but, inasmuch as when resting upon the skewbacks the extremities of the rib must reach to the points A and C, this could only be accomplished by the angular motion of all the various sections *b* B, *f* F, &c., pivoting about B and F, &c., as centres, and giving a quantity of horizontal motion, measured by the limbs B E, F G, &c. Thus, the curve of equilibrium, A *b f* C, must be raised as much above A B F C as would give the due amount of angular motion of every section, so as to throw out the points *a* and *c*, until they touched A and C.

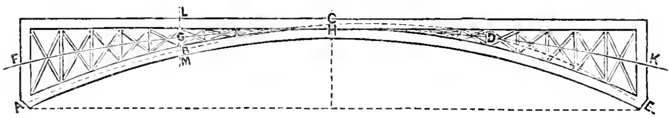
Fig. 22.



In the case just explained, the arch rib was supposed to touch the abutments at a single point only, where all angular motion was perfectly free; but where the rib had flat ends, resting upon plane skewbacks, as in Fig. 22, the relation of the curve of equilibrium, G E I F H, must be to the neutral axis, A E B F C, such, that while the balance of all the angular motion in favour of extension (due to E B F), and of that in favour of contraction (due to A E, and F H), was sufficient to bring the ends of the rib into contact with the abutments, the angular motion due to the distance between A and E must also be equal to that due to E B, for without this the flat ends of the rib could not apply themselves all along to the skewbacks.

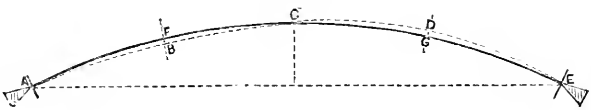
The previous remarks as to the cause of angular motion around any point in the neutral axis of an arch rib, led naturally to the examination of the proper form for its transverse section. He considered that, whether the rib consisted of one entire piece, unpierced by openings of any kind, or of an assemblage of an arch rib, a horizontal member at the top, and an efficient system of diagonal trussing between them, the axes of angular motion must equally, in both cases, be taken upon the neutral axis of the assemblage.

Fig. 23.



By way of illustration, he referred to two diagrams; one of a simple unperforated arch rib, as in Fig. 21, and the other, Fig. 23, composed by dividing the former rib into two equal parts, one of which retained its former position as an arch, while the other was placed horizontally above the arch; and both were kept at their proper distances by the system of bracing represented. If in both those cases, curves of equilibrium were drawn suitable to a loading extending over one half the rib, such as A B C D E, in Fig. 23, and relatively to the neutral axis of the rib, Fig. 21, such as A B C D E, in Fig. 24.

Fig. 24.



Also, F G H I K, representing the line of neutral axis in Fig. 23, and A B C G E, Fig. 24, the same for the rib Fig. 21, it would be seen that the bending moment at B, in Fig. 23, exceeded that at B, in Fig. 24, in the proportion of B G in the former, to B F in the latter, or nearly as 5 to 2. But the moment of inertia of the section in Fig. 23 was greater than Fig. 24, in the proportion nearly of 4 to 1; the result being that the angular motion of Fig. 24 was greater than Fig. 23, in the proportion of 4 to 2·5. Since, however, the total depth at L M, in the latter figure, was double the corresponding depth of Fig. 21, the outer fibres in Fig. 23 would be more strained than in Fig. 21, in the proportion of 5 to 4.

He therefore preferred the construction where the bulk of the material was placed in the arch proper; and considered the best form to be where the neutral axis of the arch corresponded with the curve of equilibrium to the fully loaded condition. Types approximating to this construction were to be found in the fine cast-iron single arch bridge,[1] of 200 feet span, and 20 feet rise, for conveying the Coalbrookdale railway over the river Severn, erected by Mr. Fowler; and in another[2] very elegant bridge of three arches, of 100 feet span, over the river Trent, erected by Mr. Vignoles, in 1839.

The Author of the Paper referred, he thought in rather an unsatisfactory manner, to the associating together of wrought and cast iron in the same arch rib. Referring to the bridge in the park at Neuilly, of which he gave the elevation, he said: "The summit of the arch is very flat, which exposes it to sensible flexure; at certain parts of the crown the stress of tension may attain 4 kilogrammes per square millimètre" (equal to $2\frac{1}{2}$ tons per square inch). The centre portion of the arch in this bridge was stated to consist of 'plate' iron, which meant wrought iron. Now he considered there must be some mistake in the statement as to the $2\frac{1}{2}$ tons on the inch tension. If the square on pressure upon the rib were taken at the very moderate quantity for wrought iron of 3 tons on the square inch, an amount of $2\frac{1}{2}$ tons per square inch tension could not be induced on either margin of the rib, without increasing the compression upon the other margin up to 8·5 tons on the square inch, a quantity which in this country, at least, would be deemed totally inadmissible. He could not understand the necessity for introducing wrought iron into the crown of a cast-iron arch, as in Westminster bridge and in the above-quoted instance, and he thought there was much misconception on this point, as well as on the comparative advantages of wrought iron and cast iron in arches.

Cast iron was objected to on account of its inability to withstand the force of tension; but wrought iron was incapable,

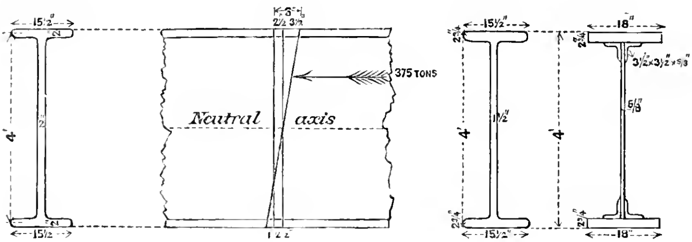
[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvii., p. 108.

[2] *Vide* Brees' "Railway Practice," p. 87, and vol. iv. Professional Papers Royal Engineers.

according to usual computation, of bearing safely a compressive strain of more than about $4\frac{1}{2}$ tons on the square inch.

Referring again to Mr. Fowler's bridge over the Severn, the cast-iron ribs were of the section shown in Fig. 25.

Fig. 25. Fig. 26. Fig. 27. Fig. 28.



The area of this section was 150 square inches; and if loaded up to $2\frac{1}{2}$ tons on the inch, the pressure stated by Mr. Baldry, the total pressure on each rib would be 375 tons. Referring to Fig. 26, if angular motion were produced in the above section by applying the gross load of 375 tons at a point removed 1·4 foot from the neutral axis, the effect would be to increase the compressive strain up to 6 tons on the inch at the one margin, and to convert it into a tensile strain of 1 ton on the inch at the other margin, both strains being well within the power of the cast iron to sustain; while, if applied to wrought iron of the same section, the material would give way under the crushing force. It might however be objected to this conclusion, that, with the same sectional area as the above, the iron, whether cast or wrought, might be disposed in a better form for resisting angular motion; and therefore he had examined other forms of section of the same area, namely, Fig. 27, cast iron, and Fig. 28, wrought iron. If these sections underwent angular motion about their neutral axis, Fig. 27, cast iron, would require the total pressure of 375 tons to be removed to a distance of 25·9 inches from the axis, instead of 16·8 inches, as before, in order to produce the same angular motion and strains at the margins; while for Fig. 28, wrought iron, a distance from the axis of 18·8 inches would give a compressive strain of 5 tons on one margin, and total absence of strain at the other. Thus, even with these improved sections, the wrought iron had no advantage over the cast iron. The chief difference to be remarked between the two materials was, not that the cast iron was any weaker than the wrought; but that the angular motion for that material was about double that for wrought iron;

which fact, however, was not of much importance in its bearing upon the subject of iron arches.[1]

As to the effects of temperature, in causing contraction and expansion, many of the older notions respecting the power of expanded metals, in oversetting the abutments of bridges, partook largely of the imaginative, but this power had recently been much more accurately measured. The only reason why an expanded arch rib should exert a greater power to overset its abutments than before its expansion, would be, if the depth of the arch were so considerable as that when the rib became expanded, and increased in length, the angular motion due to the increased rise of the arch should have the effect of depressing the centre of pressure at the crown, and elevating it at the springing, giving, in effect, a flatter arch. This motion, however, in practical cases was but very small. Through the greatest range of temperature in London, the centre arch of Southwark bridge rose and fell about 2 inches; and if the alteration in the virtual versed sine of the arch was calculated from this, the difference would only amount to a few inches.

Mr. J. M. Heppel said, in the first part of the Paper, M. Gandard drew a comparison between a system where the arch was flexible, and where the resistance to deformation was given by the spandrel bracing; and another, which he called the rigid system, where the resistance to deformation was in the arch principally. He also remarked, very justly, that in cases where both a rigid arch and rigid spandrels were introduced, it was difficult to tell exactly what the result would be; and in spite of the very clear explanations given by Mr. Phipps, he thought that, in the execution of an arch, although the utmost pains might be taken to set it in the position which should bring about the curve of equilibrium, which Mr. Phipps considered to be the only true and possible one, yet that it was far from certain whether it could be confidently affirmed that that was the direction in which the curve of equilibrium actually passed.

By way of illustration, what M. Gandard mentioned was a system of keys for adjusting an arch; but any inequality in driving them must distort the course of that line; and when a spandrel was superadded, which was understood to govern it in some degree by its rigidity, he thought the assemblage was such that it was difficult to say what the maximum strain on any portion of the material might be.

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvii., p. 88.

M. Gaudard's remarks were suggestive of a kind of construction which he did not now mean to propose as practically to be recommended, but which might be studied with advantage, because he thought it embodied, in the simplest form, the conditions which the Author assumed, and at least realized one desirable object—that of making known the mechanical conditions in which the principal parts of the structure were. The spandrel was taken to be to the arch what the stiffening girder was to the chain of a suspension bridge; in fact, so far from endeavouring to obtain the utmost amount of stiffness in the arch, the endeavour in this case would be to give it the opposite quality of flexibility, and rely entirely on the spandrel to resist its tendency to depart from its original form. Supposing an arch to be nearly a flat plane in transverse section, and in elevation a parabola—practically a circular arc—so as to be in equilibrium with an uniform load. Now the necessary consequence of that was, that the arch being perfectly flexible, whatever load it had, must be uniform, because under no other load could it retain its form. It would begin to depart from it, and, contrary to a chain, being in a condition of unstable equilibrium, it would immediately collapse. The way that might be cured was by means of a spandrel in the form of a bent girder; not that that was necessarily the best way of constructing it, but that it perhaps exhibited its functions in the clearest manner. It formed a continuous girder which simply sat upon the arch, just as a man sat upon a saddle, being however well bolted down at the two ends to prevent any rising there. The action was similar to that of the stiffening girder in the case of a suspension-bridge.

Taking the case of a half load instead of a whole one, the effect was to deflect the spandrel or girder which was under it to the extent of half supporting it. Suppose the load was 1 ton to the lineal foot; it would take up by deflection half a ton to the lineal foot, and would relieve the arch to precisely that extent. But the arch being itself a propagator of pressure in all directions discharged that half ton per foot on the opposite spandrel, and deflected that up; so that the deflection amounted to a resistance of half a ton: hence instead of 1 ton per lineal foot distributed over half the arch, there would be half a ton per lineal foot so far as the arch was concerned distributed uniformly over it.

He believed such a construction would be economical, because in an arch of the dimensions of the Victoria bridge he calculated that 50 inches of section would be sufficient to support mere compression, whereas on that bridge there were 80 inches of section, and that the stiffening girder would require only what was equivalent to an average section of about 50 inches. The horizontal member of the Victoria bridge had also about that section; so that he thought it would compare favourably in regard to weight. But if this girder were not made considerably stronger than was necessary merely to deal with tension and compression, its deflection would be considerable; and there would be a comparatively large rise at the haunch opposite to the half load and a corresponding fall at the other haunch. Supposing the girder were made of only sufficient scantling to take 5 tons to the inch, he believed that these alternate motions might amount to about 2 inches on either side of the mean position. If this were found objectionable, it might evidently be cured by adding to the strength of the spandrel girders, and would vary in about the inverse proportion; but no doubt a certain superabundance of material would be the consequence, as these would not then be strained up to their full capability.

This was just a case where, without intending now to go so far as to recommend it as a desirable construction, he thought he might venture to assert that it was one which, supposing it to be carried out, would possess the advantage of rendering absolutely certain the strain that was brought upon every member of it.

Mr. W. Airy said he was glad the Author had distinctly laid down the characteristics of a continuous elastic arch. This was done in paragraph 7, where it was stated that if such an arch were "treated as an arch by the curve of pressures, this curve will no longer be required to remain within the arch." This was essentially the property of a continuous arch; and he now noticed it because so much stress had been laid upon the importance of keeping the curve of pressures within the arch, that it would seem to be a general opinion that it was impossible for an arch to stand except this condition was secured. Such a condition was essential in the case of a voussoir arch, but was not essential in the case of a continuous arch.

The chief difficulty in solving the problem of the continuous arch was to ascertain the value of the horizontal thrust force at the abutment. This might be done in several ways—all of them depending upon the principle of invariability of the chord (as stated in par. 13), and he had adopted the following method:[1]

(1.) It was a well-recognised principle that the curvature impressed at any point of a beam by a bending moment was proportional to the bending moment that caused it.

[1] *Vide* "The Practical Theory of the Continuous Arch." 8vo. London, 1870.

(2.) It was therefore possible to express the impressed curvature at any point of an arch in terms of the forces that acted upon it, including the unknown horizontal thrust force at the abutment.

(3.) Having thus found the curvature impressed at any point, it was easy to calculate what would be the effect of this curvature in spreading out the feet of the arch, supposing they were free to spread; and the actual amount of spread might be calculated, still involving the unknown horizontal thrust force at the abutment.

(4.) The same might be done for every point of the arch by the process of integration; and thus the entire spread of the feet of the arch, due to the forces which acted upon it, would be ascertained, still involving the unknown horizontal thrust force at the abutment.

(5.) The spread of the feet of the arch thus ascertained was now put = 0 (since the feet were supposed to be prevented from spreading out); and this equation would afford the means of determining the unknown horizontal thrust force at the abutment (which was involved in the equation).

When the horizontal thrust force was thus ascertained, the problem of the continuous arch might be considered as solved, as everything else could be calculated with the greatest facility,

The Author, in paragraph 24, had referred to the advantage in certain cases of "spreading a thick layer of ballast on the platform (of railway bridges) in spite of the increase of load resulting therefrom." This remark had peculiar significance in the case of bow-string bridges, as would be seen by reference to his Paper[1] on such bridges. It was there shown that if the total stationary load, exclusive of the weight of the bow, were $5\frac{1}{2}$ times the weight of the single moveable load, none of the tie-bars would ever be in thrust. If, therefore, the stationary load were increased by a thick layer of ballast till the above condition was secured, the necessity for stiffening some of these bars would be entirely obviated, and the construction of the bridge would be considerably simplified.

With regard to the statement of the Author, towards the end of paragraph 24, that the "flexure (of arches of moderate dimensions) is small, and the longitudinal pressure much predominates," he held that this remark was only correct as regarded very flat arches. In the case of arches of large angle—as, for example,

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvii., p. 443.

arches of 120°, and semicircular arches—the strain on the metal, due to the bending moment, was far greater than that due to the longitudinal pressure. He did not, however, dissent from the opinion of the Author, that solid plates were preferable to trellis-work for the construction of arches.

In the last part of paragraph 26, which treated of the strains on the suspension ties of bow-string girders, everything was stated in the vaguest and most general manner. The ordinary arrangement of thrust and tension bars was assumed to be correct, and the strains were reasoned upon generally, without reference to calculation or experiment. The problem of these strains had been treated by him experimentally in the Paper which he had already referred to, and in that Paper would be found some positive numerical facts about these strains, which he merely referred to in order to notice that the bars liable to thrust were the diagonals, and these should be stiffened to receive the thrust: the vertical bars need not be stiffened, as the degree of thrust to which they were liable was insignificant. He was aware that this result was at variance with the method ordinarily adopted, and he admitted the convenience of the ordinary arrangement, but at the same time he considered that it was mechanically incorrect.

Mr. W. Bell described a method of drawing a curve of pressure for an arch acted on by oblique forces in which the forces acting in the space between the abutments A and B (Fig. 29, p. 144) were divided into a number of forces, each acting at its point of application; and to simplify the calculation these points of application were assumed to be at equal horizontal distances. All these were then to be resolved into a single force R, acting in the line R O, which cut the arch rib in a point C. A, B, C were to be considered as given points, through which the curve of pressure must pass. The forces acting between C and A were next resolved into a single force P, acting in the line P O, and those between C and B into a force Q, acting in the line Q O. These forces P and Q had the resultant R previously found, and must be balanced by forces whose directions must pass through the given points A and B. These directions would also converge to the same point in R O, in order that they might balance the resultant R of the forces P and Q.

If, therefore, any point R in R O were taken, and lines drawn from it to A and B cutting P O, Q O in the points E and F, the forces P and Q might be considered to be held in equilibrium by the frame A E F B; and the thrust along F E, which together with the force along A R balanced the force P, would be equal and opposite

Fig. 29.

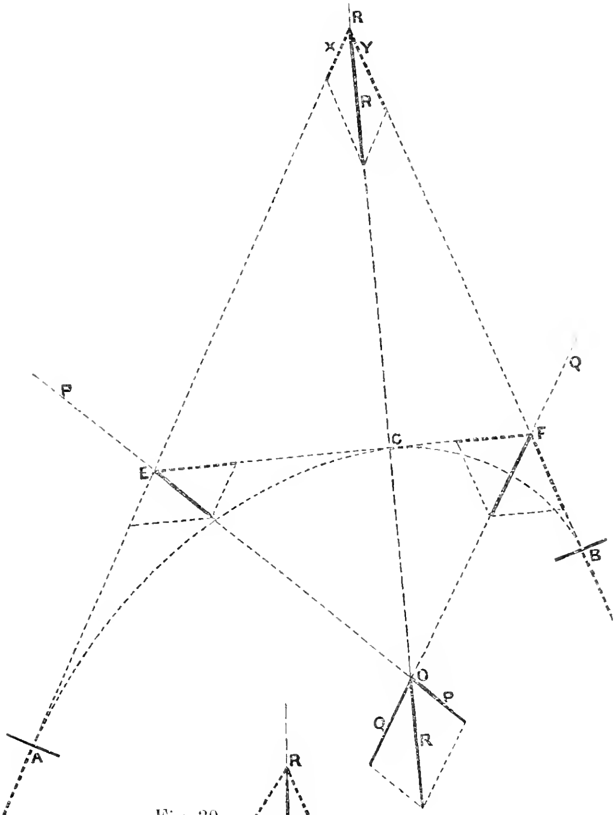
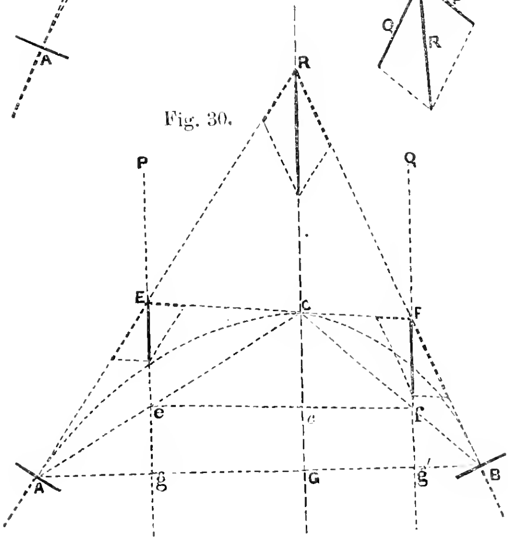


Fig. 30.



to the thrust along E F, which together with the force acting on the line B R balanced the force Q. For, calling X the force along A R, and Y the force along B R, these must have a resultant equal and opposite to that of the forces P and Q; and the four forces X, Y, P, Q might be considered as forming a four-sided figure, whose sides and diagonals were parallel to those of the figure R E O F, and of which one of the diagonals represented the resultant of P and Q, or of the equal and opposite one of X and Y. The other diagonal would represent the resultant of the forces P and X, and also of the equal and opposite resultant of the forces Q and Y, and therefore of the thrust along E F.

As the point R was shifted along R O the position of the line E F would change, and there was only one position of R where the line E F would pass through the given point C. The distance C R might be expressed in terms of the known quantities, but it was a very easy process to find the position of R tentatively. Having thus found the line E F, the force along it and the forces X and Y were found by drawing the force parallelograms. The force along E F, which passed through the point C, could then be combined with the separate forces between C and A, and the curve of pressure could thus be drawn, which must pass through the point A, and be there coincident in direction with A R. In the same way the branch of the curve between C and B might be drawn.

A similar method could easily be applied to the case of an arch acted on by vertical pressures, and loaded more on one side than on the other.

The line R O (Fig. 29) became the vertical R G (Fig. 30) through the centre of gravity of all the weights between A and B, and the lines P O, Q O (Fig. 29) became the verticals P g, Q g' (Fig. 30) through the centres of gravity of the weights acting between G and A and between G and B. The point R might then be found tentatively as before, or it might be determined by drawing the lines C A, C B, from the given point C to A and B, to cut P g and Q g' in e and f. Then let e and f be joined, cutting R G in c, and take G c : G C : : G C : G R.[1] This determined the point R, and

[1] Since R was the required point,

$$g\,e : g\,\mathrm{E} :: \mathrm{G\,C} : \mathrm{G\,R} :: g'f : g'\mathrm{F};$$

and from this it followed that E F and $e\,f$ produced must intersect the prolongation of the line $g\,g'$ on the same point; and, therefore,

$$g'f : g'\mathrm{F} :: \mathrm{G}\,c : \mathrm{G\,C}.$$

But by the first proportion

$$g'f : g'\mathrm{F} :: \mathrm{G\,C} : \mathrm{G\,R},$$

and

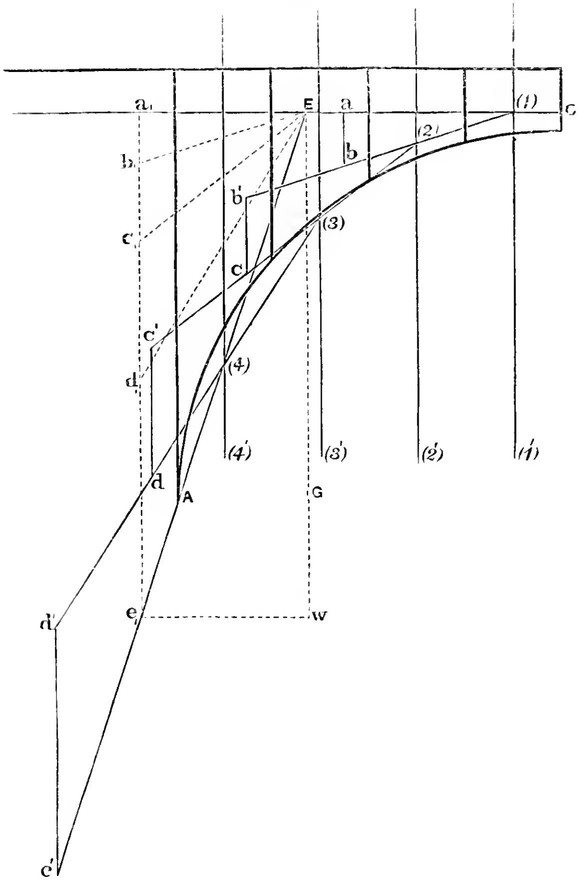
$$\mathrm{G}\,c : \mathrm{G\,C} :: \mathrm{G\,C} : \mathrm{G\,R}.$$

by joining it with A and B the line E F was found, after which the curve of pressure might be drawn as in the case of oblique forces.

This method of constructing curves of equilibrium for arches subjected to oblique forces was an extension of the following method of drawing these curves for symmetrical arches.

Each half of the arch being of the same form and weight, the thrust at the crown C would be horizontal (Fig. 31). Let the horizontal line C $a_,$ represent the line of thrust, then the vertical

Fig. 31.



line E G drawn through the centre of gravity of the semi-arch would intersect the line C $a_,$ in some point as E. Now if E were joined with the abutment A, E A would be the direction of the thrust at A, which together with the thrust at C equilibrated the weight of the semi-arch. If E w be set off on E G to represent this weight, and $w e_{,}$ drawn horizontally to cut the line E A in the point $e_{,}$, then $w e_{,}$ would represent the horizontal thrust, and E $e_{,}$ that of the thrust at A.

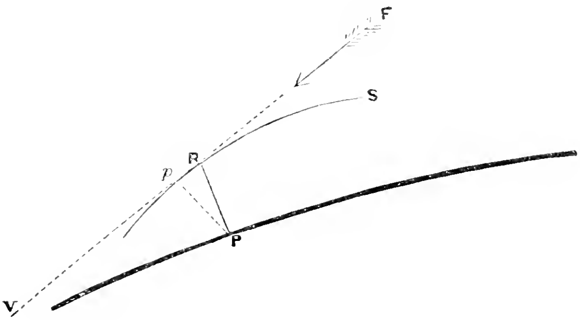
Supposing the semi-arch to be divided into a number of parts (say four), each of which it was convenient to make of the same horizontal length, and the vertical lines (1) (1′), (2) (2′), (3) (3′), and (4) (4′) were drawn through the centres of gravity of these parts, then from the point (1), the intersection of (1) (1′) with C E, set off (1) a on C E equal to the horizontal thrust $w e_{,}$, and draw $a b$ vertical and equal to the weight of the first portion of the arch whose centre of gravity was in the line (1) (1′). Now join (1) b, which would represent in magnitude and direction the thrust after it had passed the point (1), and had been combined with the weight of the first portion of the arch, supposed to be concentrated there. This line (1) b would cut the next vertical line (2) (2′) in a point (2) where the thrust had to be combined with the weight of the second portion of the arch. For this purpose set off from the point (2), on the line (1) (2) produced, a length (2) b' equal to (1) b, and draw $b' c$ vertical, and equal to the weight of the second portion whose centre of gravity was in the line (2) (2′). Then (2) c when joined would represent the resultant thrust after passing the point (2), and the line (2) c would cut the next vertical (3) (3′) in a point (3). From this point (3) a similar construction for the weight acting there would give the line (3) d, the resultant after passing the point (3), which line would cut the next vertical (4) (4′) in a point (4). From this last point, the direction (4) e of the resultant after passing the point (4) could be found in like manner, and this must coincide with the line E A already found, and would be a test of the accuracy of the several constructions.

The points (1) (2), &c., might also be found by means of a diagram E $a_{,} b_{,} c_{,}$, &c., where E $a_{,}$ represented the horizontal thrust, $a_{,} b_{,}$ the weight of the first portion acting in the line (1) (1′), $b_{,} c_{,}$ the weight of the second portion acting in the line (2) (2′), $c_{,} d_{,}$ the weight of the third portion, &c.: the total weight of the semi-arch being represented by $a_{,} e_{,}$. The points (1) (2), &c., were then found by drawing (1) (2) parallel to E $b_{,}$, (2) (3) parallel to E $c_{,}$, (3) (4) parallel to E $d_{,}$, and so on.

In the case of oblique pressures, the lines $a_{,} b_{,}$, $b_{,} c_{,}$, $c_{,} d_{,}$, &c., must be so taken as to represent the forces acting at the points (1), (2), (3), &c., and would not be vertical; but the diagram might be made by drawing these lines in the oblique directions of the forces which acted at the several points, and setting off the amounts of these forces on the respective lines. The lines which joined E with these points would represent the amounts of the resultants at the several points.

By dividing the arch into a sufficient number of parts, a series of points (1), (2) (3), &c., would be found, the curve drawn through which would be the curve of equilibrium, and the thrust at any point of it in the direction of the tangent. The position as well as the amount and direction of the resultant was thus known, and the transverse strain at any point of the arch rib was capable of being determined, since the moment of a resultant with respect to any point, and therefore with respect to the point of the rib to which it corresponded, was equal to the moment of the forces of which it formed the resultant.

Fig. 32.



P being a point of an arch rib, Fig. 32, and R S the curve of equilibrium, draw P R through P in the direction (vertical or oblique) of the force there, to cut R S in the point R. The resultant of all the forces acting on the arch rib up to the point P would be the force acting at the point R of the curve R S. Let F be this force, and F v its direction, tangential to R S. Draw P p perpendicular to F v, and the bending moment at the point P would be measured by F × P p.

Mr. E. W. Young explained a method of finding the strains on the diagonals of the spandrel of an arch, which he had used, and which he believed was even simpler than that of M. Gandard, while it had this advantage, that the calculations were easily checked.

The first step was to find the loads on the abutments.

Fig. 33.

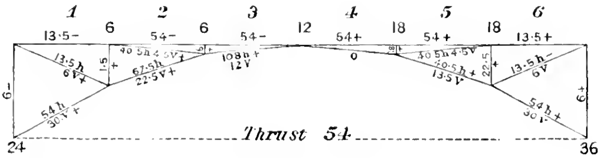
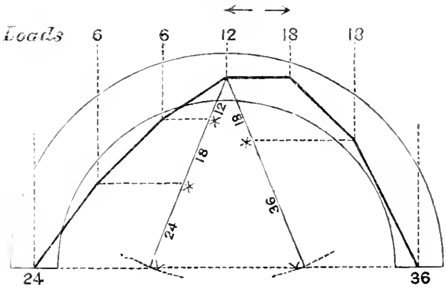


Fig. 33 represented an arch loaded upon half the span. The dead load was 6 per bay, and the live load 12. The loads upon the abutments were shown in Fig. 33, the proportion being 36 to the right-hand abutment, and 24 to the left. Now dividing the load on the bridge in these proportions, the loads of 6, 6, and 12 were assigned to the left-hand abutment, and the loads of 18 and 18 to the right-hand abutment. Then multiplying each load by its leverage there was a thrust of 54. Knowing the horizontal thrust to be 54, the vertical elements on the rib in bays Nos. 1 and 6 were found. Taking 54 as the horizontal element, the vertical was 30; but there must be a pressure on the right-hand abutment of 36, and on the left-hand abutment of 24. Consequently there must be a strain of 6 in compression on the vertical which rested on the right-hand abutment to make up a total of 36; and this necessitated a vertical downward pull of 6 on the diagonal of bay No. 6 to resist that upward push; and that vertical pressure represented a horizontal pressure of 13·5, which again required a strain in tension of 13·5 on the horizontal top member to meet it. Proceeding in this way in each bay the strains on the diagonals were obtained. In order to check the results, there was found in each bay the total vertical force which should be equal to the shearing force on that bay, and the total horizontal force should be 54 on each bay representing the thrust. Thus the shearing force on bay No. 3 was 12; accordingly there was a vertical force of 12 on this bay. Again, if there was added the vertical effects which acted in the same direction, and those acting in the contrary way were subtracted, the result was $13·5 + 4·5 = 18$ as the total vertical effect on bay No. 5, and $22·5 - 4·5 = 18$ the total vertical effect on bay No. 2. Knowing the shearing force on any bay, the calculations could be thus checked, or from the shearing force alone, these strains could be ascertained by working from the centre towards the abutments; but it was more convenient to proceed from the abutment to the centre when the diagonals were inclined as in

Fig. 33. When the diagonals were crossed his practice was to consider half the load as acting on the diagonals in the one way, and half acting in the other way, and putting the two together an approximation to the true strains was obtained, though the true strains would depend more or less upon the proportion of the sectional areas of the bars to the calculated strains.

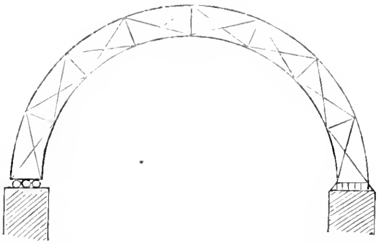
Fig. 34.



In Fig. 34 was shown the method of describing the curve of equilibrium which he generally adopted. He found, as before, where the load divided. He had taken the same loads in this case as in the former for the sake of convenience. Supposing he wanted to make the curve of equilibrium pass through a particular point, such as the centre of the arch at the crown, he took a length of any convenient scale representing the sum of these shearing forces—$12 + 18 + 24 = 54$. Taking 54 as a length and the desired point in the crown as a centre, he described an arc cutting the chord. Joining the centre and point of intersection at the chord, he marked off divisions 12, 18, and 24 along the line; and drew horizontal lines through the points in the line thus obtained to intersect corresponding vertical lines drawn through the points of application of the loads. The intersection of these lines gave points in the polygon of equilibrium. A similar operation would give the curve for the other side of the arch. It was also possible by this process, though more complicated, to draw the curve of equilibrium when the loads were distributed at irregular distances. For instance, if the distance between the two loads of 6 was double what it was in the other case, all he had to do was to make the corresponding length on the radial line 36 instead of 18 to obtain the curve.

Fig. 35 was a representation of what appeared to him to be a very complicated case. It was an arch in appearance; but yet it did not act as an arch. It represented the rib of an

Fig. 35.



arched roof which was fixed at one end and rested on rollers at the other end. Now, when this was exposed to the horizontal force of the wind from the left, the free end tended to approach closer to the fixed end. Again, when exposed to the force of the wind on the other side the free end was forced out. In this case two sets of calculations were required; it was a most difficult case, and he did not know of any work which showed how to treat it.

Mr. B. Baker said, that the Author, when treating on arches with three pivots, suggested the provision of a long joint at the centre to prevent bending. He assumed that the bending referred to was lateral bending, since the sole object of the pivots was to provide for a free vertical movement of the arch. That being so it appeared to him that an arch pivoted at the centre would be in the same condition as an ordinary continuous arch, for in both cases provision against lateral bending would be made by the introduction of horizontal bracing between the arched ribs, and as the strain transmitted through that bracing would never be sufficient to put tension on the arch at the centre, a simple butt joint or a pivot would be as effective as a rivetted joint. The real objections to pivots were that they afforded the designer far less freedom of treatment than a continuous arch; that the structure would be more sensitive to vibrations whilst at the same time the corresponding advantages were very problematical, since if material were saved labour would be increased.

The Author prefaced his formulæ with the remark that the conditions involved were so delicate, that they were applicable only to strictly homogeneous materials, and he excluded from that category timber, masonry, and, to a certain extent, cast iron. The question at once occurred, was wrought iron so far different to cast iron as to justify more than a partial acquiescence in the results of the formulæ in the one instance when they were avowedly inapplicable in the other. It was known that even two pieces of wrought iron cut from the same plate were rarely found to be of the same strength and elasticity. Cast steel was usually considered to be the most homogeneous of all materials; and yet a specimen of Krupp's manufacture, which was sliced into small pieces and tested by Mr. Kirkaldy, varied as much as 25 per cent. in strength, and 100 per cent. in extensibility; so that, even if an arched rib were made of one solid piece of steel it could not be considered as perfectly homogeneous; and when, as was the case in practice, rivets were introduced, the conditions were far more complicated. He did not clearly understand the action of rivets when they had to secure the three or more plates of a girder-flange. They did not, it was certain, make a mechanical fit, for, if a bolt $\frac{1}{16}$th of an inch smaller than the holes would pass through the rivet holes of a pile of plates as fixed for rivetting together, the work would be rather above than below the average. As the rivets were put in hot, it might be imagined that they would conform to irregularities in the holes, but such was seldom the case, for the head being struck off the rivet dropped out, possibly without having touched more than the two exterior plates of the pile. In well-executed work every rivet was strained up to the limit of elasticity, and it appeared that when four or five plates were rivetted together, the action of the rivets was merely to grip the plates together, and that the resulting frictional resistance constituted the strength of the joint. If that were so the joint could not be relied upon under strains exceeding the elastic limit, since the plates under the rivet heads would then drag out, the grip of the rivets would relax, and the frictional adhesion of the plates would necessarily cease. He could not reconcile any other method of action on the part of the rivets with the results of his experience as to the actual condition of rivets in ordinary girder flanges built up of from three to six plates. He had never seen a rivetted girder with more than two such plates broken. Some experiments of this nature seemed to be desirable, because it appeared but too probable, that reliance could not be put upon more than 10 tons or 12 tons as the ultimate resistance per square inch of rivetted girder-flanges made up of four or five plates; for unless the rivets exactly filled the holes in each plate, he could not conceive how they were to maintain their hold when the grip of the heads was relaxed. Some engineers, he was aware, considered that the strength of a joint was governed by the bearing area of the rivets. It was a question whether the bearing area ever had anything to do with the strength of rivetted work of the class dealt with by engineers in bridge building. He had seen many girders broken under test, but had never been able to trace a failure to deficient bearing area. He noticed in the recent report of the Committee of Civil Engineers on the strength of steel,[1] that the maximum strain per square inch upon the bearing area of the rivets in the joints tested was more than double that sustained by an average bar, and there was no evidence to show that the strength of the joint would have been diminished had the bearing area been still farther reduced. If the influence of bearing area were so important as to determine the strength of a joint, the same reasoning would apply to many other cases, and he should like to know how bearing area could be obtained in a chain cable, for instance, which was far more severely tested than girder work. It was obviously impossible to get bearing area equivalent in area to the section of the chain, nor was it necessary, for if it were, would mere knife edges sustain a load of 450 tons, as in Kirkaldy's testing machine? Whatever view, however, was taken of the action of rivets, it was obvious that their presence materially detracted from the homogeneity of structure required, as stated by the Author, to admit of the application of his formulæ.

In treating on continuous arches symmetrically loaded the Author based his formulæ upon the assumption that the angle between the joint at the crown and that at the springing would remain constant. He did not think this would be quite so in practice, as there would be some elasticity in the abutment itself beyond the arch rib, which would tend to distribute unequal stresses. If this were not so in an ordinary girder, resting at each end upon masonry, it would follow that when the girder was loaded it would deflect, and bear solely on the two inside arrises; hence if there were no elasticity in the abutment, fractures of bed-stones would be the rule and not the exception as they now were. He provided for this modifying influence of imperfect fixing in the case of arches by assuming, not that the abutment itself was elastic, but that the arched rib was more elastic near the springing than at the centre, and he fixed the amount arbitrarily by taking the modulus of elasticity at one half of the usual amount for

[1] *Vide* "Experiments on the Mechanical and other properties of Steel, made at H.M. Dockyard, Woolwich, by a Committee of Civil Engineers." Folio. London, 1870.

portions of the arched rib extending to one-eighth of the space from each abutment. The effect of this hypothesis upon the calculations was simply that the deduced sectional areas of the arched rib at different points would be more uniform. Thus, if in an arch rib, with a certain amount of rolling load, the section at the springing required, upon the Author's hypothesis, were one and a half time that at the centre, upon the hypothesis of elastic abutment and imperfect fixing the section at the centre would be increased about 5 per cent. whilst that at the springing would be reduced from one and a half time to about one and a quarter time the area at the crown. Probably no two arches in a viaduct were fixed under precisely similar conditions. A piece of felt interposed between the arched rib and the abutment would suffice to upset all exact calculations, and an error of but $\frac{1}{100}$th of an inch to the foot in the angle of the skewback would materially affect the strains; hence the results arrived at by any professedly exact method of investigation must be taken with due reservation. He agreed with the Author that, the thrust being given, there was no difficulty in finding the stress, but he could not admit the accuracy of the formula advanced for that purpose, because if that were correct a parabolic arched rib uniformly loaded would be submitted to only two bending strains, one arising from change of temperature, and the other from deflection; and the strains from both of these stresses would be diminished in proportion to the depth: it therefore followed that the smaller the depth the less was the strain, and that the best arrangement would be to give the rib no depth at all. But for an arched rib even uniformly loaded there must obviously be some depth which would be the most advantageous, and it appeared to him that it could only be arrived at by considering the condition of the arched rib as a column. Taking the most simple case by way of illustration—that of a parabolic arched rib pivoted at three points and uniformly loaded, in which the two arched ribs constituted in effect two columns with rounded ends, the line of thrust being parabolic, would follow the axis of the rib, and the latter, therefore, would be subject to the same conditions as two straight columns of equal length and with rounded ends; because in both cases the line of thrust would correspond with the centre line of the member resisting it, although in one case both these lines were curved, and in the other both were straight. The same conclusions held good with respect to continuous arched ribs, although in that case the effective length of column was considerably less than one-half of the span as in the previous instance. This condition further illustrated the questionable advantage of departing from the original type of square-ended arched ribs of uniform section. He did not consider that the determination of the most desirable proportions for arches was quite so simple a problem as might be inferred from an inspection of the table advanced by the Author. The proper rise for the arch, in his opinion, was governed principally by the weight of the spandrel-filling and its bracing as compared with that of the arched rib proper, and the depth of the rib depended upon a multitude of conditions. It would not be the same in England as in America, owing to the greater range of temperature in the latter country. Here it would be made generally from $\frac{1}{40}$th to $\frac{1}{50}$th of the span; in America from $\frac{1}{50}$th to $\frac{1}{60}$th would be a better proportion.

He thought the admission by M. Gaudard, that it was better to adopt what was practically most advantageous rather than that which most facilitated theoretical calculation, was a great concession to practical men. No man, however profound a mathematician he might be, could justly be styled an engineer unless his structures complied with many other conditions besides those relating to stress. Public opinion did not justify architects in putting up buildings of even the poorest class without some attempt at pleasing the eye: pilasters and engaged columns gave an appearance of increased stability even if in many instances they did not confer it, and he thought the engineer should consider such points when designing an arched bridge—the most expressive type of all bridges—and the one admitting of the noblest artistic treatment. To the engineer figures were indispensable, and admirable servants, but they should never be allowed to become the engineer's master.

Mr. G. J. Morrison observed that probably rivetted joints might be made nearly as strong as the plates which they connected; but he thought there were many cases where the strength of the joints was not more than 50 per cent. or 60 per cent. of that of the plates. Unless the joints could be made nearly as strong as the plates some of the formulæ of M. Gaudard would be of little use.

Mr. R. P. Brereton observed, that in paragraph 6 it was suggested as desirable that arches should be in equilibrium when fully loaded, and that the curves would be either the catenary for arches of uniform section carrying only their own weight, or the parabola when uniformly loaded on a horizontal line, the latter being the case for suspension or compressed metallic arches, considering the weight of the ribs and spandrels to have little influence as compared with that of the roadway and the load.

The Author assumed, however, here, and again in paragraph 21, that, in practice, the circular form of arch was generally preferred to theoretical forms, owing to supposed facilities and simplification in setting out, and in the execution of the work; and, besides, a theoretical figure would only be correct for a single condition of the load.

With arches, where the radius would be large, he had found greater simplicity in setting out the lines, and in the construction with the parabola, than with the circular arc; and that form, in most cases, approached nearer to the curve of equilibrium than the circle, and had been adopted. He considered, although desirable with large bridges, that the arch should be in equilibrium when fully loaded, that it should be also so when subject to its permanent strains, rather than when carrying the maximum rolling load, only occasionally applied, and but rarely remaining more than a few seconds. This, with a bridge of the greatest traffic, would probably seldom exceed in the aggregate a thirtieth part of its full life, and during the remainder it would be undergoing constantly continuing efforts at change of shape. In arch construction, varieties of forms became necessary as curves of equilibrium, varying considerably, principally at the haunches, according to the relative weights of the arch itself and loads, as well as the extent and mode of application of the load.

Taking a number of curves of the same span and height, there was, first, the parabola, the flattest at the haunches of the curves generally used in bridge-building. This could only be actually reached under a uniform horizontal load, the arch itself and spandrels being considered entirely without weight. Next, the common catenary, due to the weight alone of an arch or chain of uniform section. This was the curve seen in wire-rope suspension-bridges before attaching the floor and load, and in the festoons of a ship's cable. Next followed the catenary of equal strength, or curve formed by the weight of the arch alone, with section varying in proportion to the strain upon the different parts, as usually adopted in a suspension-bridge built with iron bars or links, and in arches well designed. After this came the circular segmental arc.

The catenary of equal strength was at the haunches, nearly midway between the circle and parabola; and the common catenary curve passed between that and the parabola. All additions of uniform horizontal load, if applied vertically, tended to departure from the circle and approached towards the parabola, increasing as the magnitude of the loadings. The same load, if applied through the spandrels in a radiating direction, affected materially the relative strains, or thrusts, on different portions of the arch, and the tendency of the curve of equilibrium would be reversed, approaching in the direction of the circular form, and increasing with increase of load. If, in addition to a uniform horizontal load, the spandrels or haunches were filled in with solid material, as often happened with masonry bridges, the curves of equilibrium passed greatly beyond the circle and partook more of an elliptic form. Taking, for instance, a span of 200 feet, with a rise of $\frac{1}{5}$th—a not unusual proportion, although many of the modern iron arches rose from $\frac{1}{8}$th to $\frac{1}{10}$th only—the difference at the haunches between the different curves would be considerable, the variation between parabola and circle nearly reaching to 2 feet, and between the parabola and ellipse to nearly 6 feet.

The following might be taken as approximately true:—

1st. With a uniform horizontal load, in proportion to the weight of rib or chain as 6 to 1, applied vertically, the curve would approach very near to the parabola.

2nd. With the same in the proportion of 3 to 1, the curve at the haunch would be about midway between the parabola and the common catenary.

3rd. With the same in the proportion of 1 to 1, the curve would be between the parabola and the equal catenary, or near the common catenary.

4th. With the same in the proportion of 3 to 1, applied through radiating spandrels, the curve would be between the equal catenary and the circle.

5th. With the same in proportion of 6 to 1 radiating, the curve would approach closely to the circular segment. The segment would also be nearly the curve of equilibrium for an arch of masonry with uniform horizontal load, in the proportion of 1 to 1, when the material over the haunches was lightened $\frac{2}{3}$rds or $\frac{3}{4}$ths by the introduction of spandrel walls or arches.

6th. With the same load and spandrels fully weighted, the curve would approach to the elliptic segment, and would depart, to the extent of nearly 4 feet beyond the circle, assuming the spandrel-filling to act vertically upon the arch; but, considering the uncertainty as to the direction in which the weight of the material used in the backing up of haunches really acted, the departure from a circular form might be much greater.

In the case of the Hungerford Suspension bridge of nearly 680 feet span, and a rise of $\frac{1}{13}$th, not intended for railway purposes, and with the floor not calculated to restrain the natural movement of the chains, which were heavy in proportion to the loads, four-fifths of the weight contributed to form an equal catenary with the constant load. And with the full moving load half contributed to the equal catenary and half to the parabola. The full load was probably realized on very rare occasions; the calculations for constructing the bridge were for a curve half way between the common and equal strength catenary.

With the Saltash bridge of 450 feet span, and aggregate rise of the two curves forming the bow and chain of $\frac{1}{8}$th of the span, half the whole weight contributed to the equal catenary and half to the parabola with the constant load; and with the full moving load one-third contributed to the equal catenary and two-thirds to the parabola. The arch was constructed as a parabolic polygon, and the difference at the haunches between this curve and the circle would be nearly 3 inches.

At the Windsor Railway bowstring bridge of 200 feet span and $\frac{1}{10}$th rise, one-third of the weight contributed to the equal catenary and two-thirds to the parabola with the constant load; and with the full load one-fifth to the equal catenary and four-fifths to the parabola. The ribs were also built as parabolic polygons differing from the circle about $2\frac{1}{2}$ inches.

In paragraph 7, there appeared to be a distinction between arches of masonry as rigid and inelastic, changing form, either by crushing of the stones or pivoting on their edges. But a material degree of elasticity was to be met with in the arches, as well as in the abutments and their foundations; and several instances had occurred of arches showing great elasticity during settlements, and from other causes. In the case of a large railway segmental arch of 120 feet span, with a rise of $\frac{1}{5}$th and thickness of masonry at the crown of 4 feet 6 inches, built in the year 1841, the arch had settled in the first ten years 21 inches, including the allowance originally intended, and in the next ten years a further 11 inches, being 32 inches in all, equal to $\frac{1}{8}$th of the original rise of the arch, and $\frac{3}{4}$ths of its thickness; the ballast at the crown having accumulated to more than double. There had been some trifling yielding of the abutments; but the settlement was mostly due to elastic change of form and compression of the masonry, the curve of pressure being nearer to one edge than the other, as in a metal arch. A few of the arch-stones had been slightly cracked, but without signs of pivoting or opening of the joints such as should have shown themselves with an inelastic structure. In 1860, some readjustment of the loading was carried out, since which year there had been no further movement.

With reference to arches with three pivots, he might observe that if a pivot at the crown were otherwise desirable, it could be obtained with less objection than had been assumed, since there was no necessity for a spherical knee-joint, the top and bottom members of large arches having width enough to admit of a sufficiently long cylindrical joint or hinge, which, when connected, would have no greater tendency to turning over sideways of the arch than was met with in the joints of king-post or polygonal trusses not depending for their existence on the adjoining ribs.

Fig. 12 in paragraph 24 represented a pivot construction at the abutments to avoid risk of imperfect bedding or keying up of the ribs upon the springing plate. With considerable spans and loads, assuming the small pivot to be the only bearing, the strain upon it and upon the bed-plate beneath must be excessive. Applying such a plan to the St. Louis bridge now building in America, where there was a direct thrust at the abutment of about 2,000 tons per rib, the strains by the converging of the top and bottom members to the pivot would be increased to 2,200 tons together, producing also a strain in tension on the cross-stay connecting them of about 500 tons, which would require alone 100 square inches of wrought iron, a failure of which or its connections would lead to the collapsing of both the members of the arch. More than double the quantity of metal would be required than would be otherwise necessary, besides the heavy bed-plate casting requisite for distributing such enormous weights upon the masonry; and with a pivot at the crown it would again be doubled. He believed it would be better if the two members of the ribs, each bedded on the abutment, were made to carry a proportionate share of the total loading, relying upon efficient trellising or web connection for distributing the strain during unequal loads.

He did not think sufficient justice had been done by the Author, in paragraphs 22 and 23, on the subject of timber bridge construction. It was assumed that, from decay, constant renewal of the parts was indispensable, and in doing this that there must every time be deformation or derangement of the structure. A skilful piece of carpentry or trussing should admit of any individual member being taken out and replaced, the proper initial strain of such member being maintained, without the remainder being subjected to derangement. The examples of timber bridges given in the Paper, of two or three types only, could not be considered satisfactory, and many of the railway bridges built in England had, he thought, been more so.

He did not think that keeping the timbers apart, as shown in Fig. 9, by blocks at intervals, would have any great effect in transmitting the line of pressure from one side to the other. Indeed the contrary was found to be the case; as experience had shown that the surfaces of the timbers, if properly preserved, should be kept as closely as possible together by bolting or otherwise. In this way, and by joggling to prevent the slipping of the surfaces in contact, the stiffness of beams placed one above the other might be increased to nearly double, or that due to a timber equal in depth to both; but when not so treated, the stiffness was but little greater than when placed side by side where, as in the construction cited, the three timbers would have been nearly three times as stiff, if they had been close together, one above the other, and efficiently connected.

The second type of arch, given in Fig. 10, had been frequently employed for railway bridges. When formed with layers of thin planks, and with timber not sufficiently preserved, it had not proved durable; but when built of substantial timbers, with the joints securely made, it had answered the purpose well. On the main line of the Great Western Railway there was a bridge of this kind across a river, with flat timber arches of 90 feet span, the ribs consisting of several layers of half timbers, 6 inches or 7 inches thick, bent to the required form, and well preserved by 'Kyanizing.' These arches had been in constant use about thirty years without requiring renewal. The class of timber rib, however, more frequently adopted for spans of about 100 feet, had been a system of double polygons placed one within the other, bolted together and breaking joint, by which means great stiffness had been obtained. A bridge of this description carried the Bristol and Exeter Railway across a river; the ribs were 102-feet span, springing from abutments intended for a masonry arch, and they were erected thirty years ago, of well creosoted timber; and these, too, were still in use.

On the subject of the preservation of timber, without which no degree of permanence in a structure could be expected, the Author was silent, beyond pointing out the desirability of using tar or paint. It must, however, be borne in mind that the influence of these was superficial, and that the latter was only advantageously attainable with wrought surfaces of carpentry, quite inapplicable to the enormous quantities of timber-work used on railways. He had had experience of about 10,000,000 cubic feet of pine timber in railway construction, all of which had been prepared by one or other of the different preserving processes. Amongst the earliest, in 1838, was the chloride of mercury, or corrosive sublimate—Kyan's process. Then came the sulphates of copper and of iron—Margary's and Payne's processes. Creosoting, or tar-oil—Bethell's process—succeeded, and also to a large extent, chloride of zinc—Burnett's process. The early Kyanizing, when efficiently done, had not been surpassed by subsequent methods; and permanent-way timber, laid thirty years ago, was in existence still. This process gave way to others, the material being expensive at the time, and difficulties existing in readily detecting fraudulent adulterations. It had, however, been resumed again about fifteen years ago, the material having become considerably cheaper. The sulphate processes had not been found so favourable; but the Burnett process, and creosoting had been largely used. He did not remember any instances of mischief from the combination of timber and iron, out of several hundred examples in England.

Pine timber did not contrast unfavourably with other materials, when used in compression, for large struts or arches. As compared with wrought iron, it was about $\frac{1}{12}$th as heavy, $\frac{1}{24}$th as costly, and $\frac{1}{20}$th of the ultimate strength; whilst as regarded extent of compression, or elasticity up to $\frac{1}{4}$th the breaking weight, it was found to be only about $\frac{1}{5}$th greater; and, as compared with cast iron, about $\frac{2}{3}$rds less. Timber, if substituted for a rib of ashlar masonry, would be about $2\frac{1}{2}$ times stronger, $2\frac{1}{2}$ times cheaper, and $3\frac{1}{2}$ times lighter.

In paragraph 24, cast iron was spoken of as resisting compression well, but tension badly, and as being ill-suited for arches when required to resist flexure. The latter might be true with metal that was brittle or imperfect; but good material might be made sufficiently elastic, and had been known to bend as much as 5 inches before breaking, in girders 30 feet long and 16 inches deep: the jointing also of the segments might be made, by bolting, as strong as other parts. As regarded resistance to compression of cast iron, as compared with wrought iron, although the ultimate strength was greater, yet, as used in practice up to $\frac{1}{4}$th or $\frac{1}{3}$rd of the breaking weight, the compression which led to deflection or derangement in a structure would be nearly double.

Mr. Stoney, in commenting on the Saltash bridge, and objecting to the form that had been used, as unfavourable to resist compression, had probably not been fully acquainted with the construction. The dimensions of the elliptic tube of the arch or polygon were 17 feet in width, and 12 feet in depth; the radius at the sides was about 5 feet; and there, in the line of the greatest strain, the plating of the skin was doubled. At the upper and lower parts the radius of curvature was 12 feet; and at these flatter portions of the tube some of the metal was disposed in several longitudinal ribs or webs, after the manner of the Britannia tube. Before erecting the bridge, the first span was entirely completed, and tested with a load of $2\frac{3}{4}$ tons per lineal foot, or 1,200 tons in all, in addition to its own weight, for the purpose of detecting any indications that might show themselves, of buckling of plates or other weaknesses. No such appearances were observed, nor any during the twelve years that the bridge had been subject to constant traffic.

As regarded the use of ballast upon railway bridges, none of those constructed under Mr. Brunel, including the Saltash bridge, had been so large as to necessitate the departure from what was considered a sound and useful practice, the nature of the designs admitting of great depth of trussing, which increased the stiffness and reduced the strains, even with some increased load of ballast, to greatly less than bridges of the same size constructed on other designs; and the cost of the extra metal necessary formed but a trifling proportion of the entire cost. It might be difficult to satisfy the rigid mathematician of the practical advantages of putting on a constant, or what might appear an unnecessary, load upon a structure. Theoretical calculations did not regard the results of trains getting off the rails, changes in the description of permanent way, fire, the bringing home of joints of ironwork or framing, preventing alternations of strains on different parts, the keeping in contact pins or bearings which diminished deflection, the reduction by the inertia of the mass of the effects of sudden jerks, blows, shocks, and vibrations of passing loads—always mischievous with rivetted boiler work. Besides, there was a great advantage in being able to reduce the load, if necessary, for repairs, or in the event of defective joints or workmanship being afterwards discovered.

Assuming the diagonals in bowstring bridges, as usually made of bars or rods fastened by pins or keys, and themselves incapable of compression, the condition of the vertical struts depended upon the proportion of the moving load to the constant load of the suspended floor. When one-half the bridge was covered with a moving load, and the above proportions were about 2 to 1, compression would be brought upon the vertical rods, and sooner if any of the reverse diagonals should have been too lightly keyed, or contracted in very low temperatures. A load of ballast was desirable in preventing alternation of the strains from tension to compression—a source of mischief to struts if made with pin connections at the joints.

Mr. Barlow remarked that M. Gaudard had given the solution of the polygonal arch and framing in connection with the spandrel, which was simple and easy to be understood, and there was no difficulty in applying it. But in respect to a continuous arch, although the form in which M. Gaudard had brought it forward could be easily handled by mathematicians, yet it presented considerable difficulty to the practical man. He thought it would have been better if the Author had adopted a more simple shape, because formulæ of such complexity had the effect of deterring many persons from entering on the subject. It was true that in a continuous arch the curve of pressure might pass considerably outside the arch without incurring the risk of failure, but in ordinary arches such a depth of rib was generally given, amounting in large arches to $\frac{1}{30}$th or $\frac{1}{40}$th of the span, that in fact the curve of pressure did not pass outside the arch. The arch of a bridge, too, never stood in the position of what was called a naked arch; it always received more or less support and restraint from the spandrels, and therefore the conditions on which the arch of a bridge had to be treated were not those requiring the elaborate investigation brought forward in the Paper.

The roof of the St. Pancras station of the Midland railway, which was a large arch unsupported by any spandrels whatever, was a naked arch, and he should have been glad if he had been in possession of any practicable formula which he could have used in designing that structure. He, however, was not in possession of such a formula, and therefore he was obliged to have recourse to other means to determine the subject. In relation to that arch, he had not only the condition of unequal loading, but that of lateral pressure brought on it by the wind. He was not able to find out from the Paper the mode of treatment of the condition of lateral pressure. The subject was one of importance, and he hoped the Author would continue his investigations, and endeavour to render them in a more simple and intelligible form.

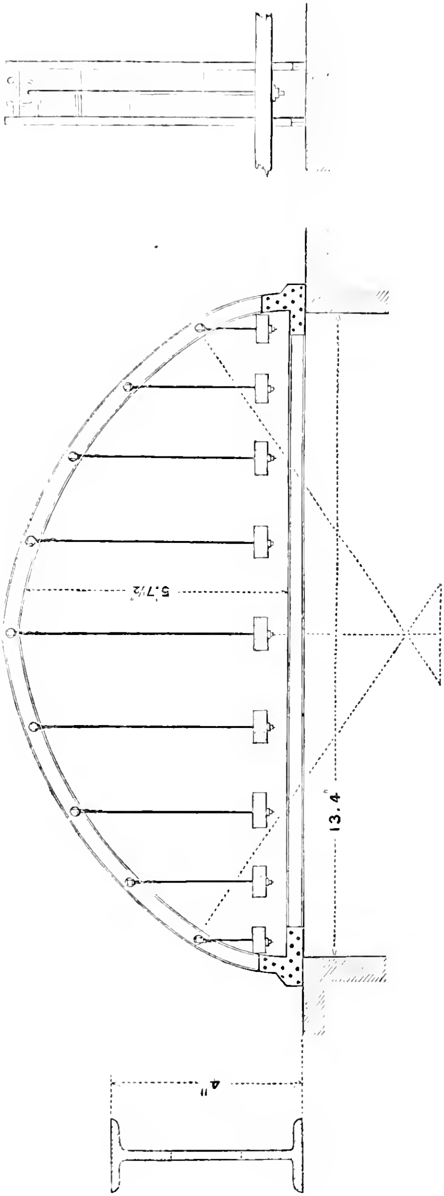
Mr. Phipps and Mr. Heppel had given views on the subject of treating bridges, which differed from each other. Mr. Phipps proposed, as the best mode of constructing a bridge, to put the whole of the rigidity into the arch itself, and to take no advantage whatever of spandrels. On the other hand Mr. Heppel had submitted, as a problem to be studied, an arch which in itself should be completely flexible, and which should obtain all its rigidity by means of the spandrel action, more or less in the form of a girder. This had been brought forward in the discussion on the Clifton Suspension bridge by Professor Airy,[1] who then pointed out that if there was a girder below a perfectly flexible suspension-chain, and the girder was simply capable of resisting with a given deflection the strain brought by half the moving load, that the structure would be of sufficient stability for that end, and the deflection would be that due to half the moving load upon the girder. He believed that was correct; the more so, because it was not a new principle, but was one which Mr. P. W. Barlow had experimented upon in reference to a suspension-bridge in Ireland. But there was a distinction to be drawn between a suspension-chain and an arch. In a catenary curve the suspension-chain was in a condition of what was termed stable equilibrium: it had a tendency to restore itself if disturbed. On the other hand, if it was turned upside down—and it was attempted to make it into an arch—it became in a condition of unstable equilibrium. If the form of the arch was disturbed, it had no power or tendency to restore itself. But arches were made with a given thickness, so that they possessed some elements of stability in themselves; and taking that property into account, the proposition of Mr. Heppel was a perfectly legitimate one; and he should be disposed to carry it farther, and unite the spandrel with the girder. It would follow in an arch so made, that if the lower member was sufficiently strong to resist the weight of the moving load in addition to the weight of the bridge itself as a whole, and then, secondly, if the half-arch was united into one piece with the spandrel, forming as it then would do a species of girder, and that girder was sufficient to carry the weight of half the moving load, there would be a condition of things consistent with perfect stability.

He would mention, with reference to the St. Pancras station roof, that he had made a rough test model $\frac{1}{18}$th of the full size of the roof, and, applying somewhat the same principle to it, had subjected it to two conditions of strain, which he would describe. The clear span of the model was 13 feet 4 inches, and there were two ribs, each of a width of 1·625 inch, and a depth of 4 inches, making a total sectional area of $3\frac{1}{4}$ inches. Railway-sleepers were suspended from the arch, as shown in Fig. 36 (page 165), and upon them pig-iron was placed.

First, a distributed load of 12 tons, 3 cwt., 2 qrs., 10 lbs., produced a strain of about $2\frac{1}{4}$ tons per inch at the haunches. Secondly,

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvi., p. 258.

Fig. 36.



The depth of the rib was 4 inches, and the rise 5 feet 7½ inches.

2 tons, 3 cwt., 2 qrs., of this load were transferred from one side to the other, which was equivalent to a distributed load of 7 tons, 16 cwt., 2 qrs., 10 lbs., and an additional load of 4 tons, 7 cwt., on one side of the arch. Now it would be observed that this difference of load of 4 tons, 7 cwt., if laid on one-half of the arch acting as a girder, would have been sufficient to create a considerable deflection; yet when applied on the half-arch in the manner described, the disturbance was so small, that the change of figure was barely appreciable. This was a rough mechanical way of treating the subject, but as far as it went it appeared to bear out the views of Professor Airy, Mr. Heppel, and Mr. P. W. Barlow.

With reference to M. Gaudard's reasoning upon the condition of the naked arch, which was really the main subject of the Paper, he had no doubt that it would be found practically applicable if the arch was formed of open framework; but if formed of solid pieces of metal, there were such differences between theory and practice, where conditions of flexibility were involved, that he thought it desirable to have some experiments, or a crucial test of some kind to determine the question.

The Author had spoken of putting in pivots at the springing and at the crown of the arch. He had seen one good illustration of the effects of pivots, if it was understood that a pivot meant a pin so placed that rubbing took place round it, and that was afforded at the time when the Hungerford Suspension bridge was taken down for the purpose of re-erecting the chains at the Clifton bridge. It was then found that the pins, which no doubt had been subject to that kind of action for a series of years under considerable strain, and with the London dust getting into the bearings, had been so cut away in the grooves, that it was necessary to replace the old pins and to substitute new ones of steel.

Dr. W. Pole stated, through the Secretary, that he congratulated the members of the British Engineering profession on the greatly increased interest now shown by the Institution in subjects of this class, as evinced by the remarks made in the course of the discussion that had taken place on M. Gaudard's Paper. The Author would, no doubt, be pleased to find how thoroughly his labours had been appreciated. The Essay on "The Practical Theory of the Continuous Arch," by Mr. Wilfrid Airy, Assoc. Inst. C.E., might in particular be mentioned, as showing that the mathematical theory of arches had not escaped the attention of English Engineers.

M. Gaudard, after receiving the notes of the foregoing discussion, communicated to the Secretary a Paper of remarks upon them, of which the following is a translation :—

The Author feels much honoured by his essay being so favourably received by the Institution, and submitted to such a searching discussion. Among the opinions put forward are some which invite a reply. As regards the general figure of the arch, the Author agrees with the opinions of Mr. Brereton, as to the facility of application of the parabolic curve. The equation of this curve is simpler than that of the circle; an advantage which will be found in drawing large plans, where the compasses can no longer be applied. Nevertheless the circular arch has in its favour not only ordinary usage or routine, but also the real advantage of a constant curvature, which admits of the different parts being verified with the same mould or template, or of the voussoirs being cast to one identical model, when they are of cast iron and of unvarying section.

Messrs. Baker and Barlow do not willingly accept the formulæ of the resistance of rigid arches: the first because he considers that practical instances depart too widely from the hypotheses of theoretical homogeneity: the second, because these formulæ are not readily applicable by practical Engineers. With reference to this second consideration, assuredly it is a perfectly legitimate desire for every scientific Engineer to be able to rely upon himself, in every case which may be submitted to him. It would seem that this desire for simplification would be sufficiently satisfied by the following method :—The arch, being considered parabolic (or nearly so), may be first calculated by the formula of the suspension-bridge, for the case of the full load applied over its whole length. This gives at first too little strength, because it is known that partial addition burden, affecting the piece by flexure, will further fatigue it; and besides, the nature even of the effort by compression exacts a rigidity not required for the stability of equilibrium of a stretched cable. Hence two modifications are necessary: first, to develop the calculated area in a form of section (such as the I) capable of resisting transverse flexure; the choice of this form may be left to the sagacity of the Engineer, assisted by good models of works already executed. In the second place, it is necessary to increase the area, or to support several points in the length of the arch by auxiliary pieces. Perhaps it may be convenient to double the section, if there are reasons to adopt light spandrels. But it may be better, in accordance with Mr. Heppel's advice, to resort to auxiliary pieces, by calculating upon a division of functions.

The arch will suffice for itself in the case of a load completely distributed. On the contrary, where the load rests only on one-half of the arch, or is subject to any other irregularity destroying the symmetry, the intervention of conveniently triangulated webs may be resorted to. The half truss should support the weight without appreciable deformation, by doing duty as a trellis girder, resting by its two extremities, which are, the abutment at one end, and the summit of the arch, or point of junction of the two half-trusses, at the other end. Meanwhile, it is to be observed that there will necessarily exist a very slight deformation, even in this strengthened system, so that it would be proper always to add a little to the strength of the section of the arch, although much less so than if it had been deprived of the support of the webs. Simple and easy processes are as much appreciated by theorists as by practical men, for cases of slight importance, and where time does not admit of complicated researches. But in the rarer cases of monumental works, where a considerable economy, as well as a more perfect assurance of strength, may be expected at the mere cost of more laborious calculations, it does not appear that the complexity of the method is a very serious objection. Hence the formulæ given in the Papers appear worthy of being generally known, and carefully preserved in the repertory of the profession. A fine large bridge cannot be constructed without the concurrence of several individuals, among whom possibly some one may be found who is sufficiently acquainted with calculations of this kind. The profession of the Engineer embraces, in our day, a knowledge of subjects so extended and so varied, that it becomes continually more and more impossible to find them all united, completely, in the same individual; specialities, and the division of labour, are an imperious necessity, inherent in the weakness of human powers. In particular there exists generally, not perhaps a natural antagonism, but a certain degree of practical incompatibility between the habits of laborious calculation and those of consummate practical skill. The Engineer who calculates designs has need of tranquillity; while he who has to carry them out must live in continual movement and excitement. It has always appeared to the Author, that the combination of the services rendered by these two classes of workers is effected very satisfactorily, and to their great mutual advantage. Indeed the Institution of Civil Engineers itself is a brilliant affirmation of the principle of co-operation of learning and speciality, and the conflict and discussion of different ideas result in substantial benefit to the interests of the Profession.

With respect to the remarks of Mr. Baker, the Author conceives that the use of the formulæ of the thrust of rigid arches offers sufficient approach to the truth to repay, at least in certain cases, the labour of its application. The ostensible objection is, that the construction is heterogeneous; but on further examination, it will be perceived that the real objection reduces itself to that already examined, of the complexity of the calculation; for, in truth, a straight bridge is just as heterogeneous as a curved arch, and yet the formulæ of straight girders commend themselves much more readily to all practical men because they are simple. In fact, the incontestable gulf between hypothesis and reality is bridged by the interval between the coefficient of practical work and the limit of rupture or elasticity. The ideal formula remains none the less the useful guide, which will serve to place a whole series of different works in nearly identical conditions of security.

The Author has also to offer some considerations on what has been said in regard to the comparison of the arch of wrought iron and the arch of cast iron. The following are the motives which have induced him to consider wrought iron as probably satisfying more nearly than cast iron the theoretical conditions of homogeneity of a single piece. In the arch of plate iron the elementary pieces are more numerous, thinner, intermingled and juxtaposed; the irregularities of texture would therefore appear to be more disseminated—more evenly spread about in the whole work—than would be the case of cast iron, cast in mass through the whole thickness of the section. In the case of wrought iron, the theorist cannot know what law to be guided by for the variation of the grain; he therefore concludes that the best hypothesis is that of homogeneity, the inequalities being completely comprised within the arbitrary limit, the practical coefficient. With cast iron the case is different, as the Engineer distrusts the hypothesis of homogeneity; it might be considered nearer the truth to rely upon the law of a skin of finer grain than the interior texture; but it is evident that the application of the formulæ under such conditions would only become more laborious still.

What has been said of the nature of the two metals will also apply to their combination. In the cast-iron arch we do not hesitate to accuse the joints of the voussoirs as very decided points of weakness, tending to gape slightly under certain conditions of equilibrium, seeing that the bolts uniting the ribs of contact are at some distance from the extreme fibres which are the most fatigued or the most distant from the neutral axis; fibres that therefore retain their want of continuity. It is true that this may be avoided by enlarging the seat of contact beyond the limits of the present section; but it is believed that that has hardly ever been practised save on the edge of the extrados contiguous to the web, and not on the lower edge.

In the case of wrought-iron joints it is more difficult to ascertain exactly the points of weakness, at least in a form as clearly defined as in a system of voussoirs; seeing that the joints of various juxtaposed pieces cross or overlap one another and are compensated by the application of covering pieces or joint plates. The weakening due to the rivet-holes is, again, multiplied and disseminated through the whole. Mr. Morrison reminds us, it is true, that plate joints often present a large deficit of strength in the experiments on rupture. Mr. Baker expresses the same opinion; adding to it an important observation, viz., that within the limits of practice, the rivets do not work by shearing, but support the whole by virtue of the pressure which they exert on the plates. This consideration seems to prove that the joint remains perfect as long as this pressure remains efficient, and prevents all sliding: such is the condition of the practical resistance of a solid structure where no movement of detail is produced beyond the elastic deformation of the whole. It is only under an exaggerated trial that the binding power of the rivets would be overcome and that then the joints would begin to make themselves felt as weak points, as is ordinarily proved by experiments on rupture. In fact, the formulæ of the Paper, like generally all ordinary formulæ for the strength of materials, are only good for small deformations, within the limits of practical security, and become defective for experiments bordering on rupture, or even for those going beyond a certain limit of elasticity.

The remaining observations which the Author has to add are not offered as controverting any opinions expressed, but rather as aiding in the discussion of the subject generally. The remarks of Mr. Phipps would lead us to think that some injustice has been done to the aptitude for resistance of cast iron in bridge arches; for the extension which may be developed in the arch remains notably inferior to the maximum compression, a condition appropriate to the character of the metal in question. With regard to the figure of 4 kilogrammes of tensile stress which the Author has indicated (No. 25) for the Neuilly bridge, this datum was taken from an article by M. Contamin in the number for February, 1868, of the "Annales du Génie Civil."

The solution of the case of perfect *encastrement*, mentioned at the end of No. 14, was noticed under the head of theoretical information rather than as a practical example; for according to the observation of Mr. Baker it is impossible in practice to secure invariability, in the angle comprised between the section at the crown and that at the springing. Hence the *encastrement* is scarcely ever adopted, a simple abutment being usually preferred.

The short table of the best proportion of pitch of the arch to the span, given in No. 20, is only applicable to the case examined by M. Bresse, of a circular arch with constant section, and considered as isolated or without spandrels; the heights given are those which would strain the metal least with regard to equality of section and of load. Mr. Baker was perfectly right in maintaining that such indications cannot be prescribed absolutely in bridge projects where other considerations are of influence; a diminution of the pitch is advantageous as diminishing the mass of the spandrels.

As regards the assimilation of the arch or the two halves of the arch to pillars whose resistance would augment when their extremities terminate with enlarged shoulders, instead of being rounded, it seems to the Author that the influence of temperature and of the arbitrary keying-up diminish the force of this view. When a straight pillar is dilated it thrusts back its points of support, never ceasing to apply itself thoroughly by the entire surface of the shoulder; but a curved piece will be liable to abut on an eccentric edge, a case where the pillar with shoulders works under worse conditions than that with rounded ends.

Mr. Barlow appears to fear for the pivots of abutment or of free articulation an effect from wear analogous to that which he instanced in the case of the chain gudgeons of old Hungerford bridge. The Author does not attach weight to this as regards a rigid arch, not liable to the large, sudden, and continuous oscillations of a suspension bridge chain. The pivot has no need to be an entirely cylindrical spindle; the shape of the pieces in contact would seem immaterial, provided it attains the object sought for, namely, a free abutment, or simple articulation, only allowing of an imperceptible mutual rolling, and which is only brought into action gradually by the change of temperature.

M. Probst, of the engineering firm of Ott, of Berne, has informed the Author that there exist at Berlin bridges with a central pivot, but that these works do not appear to comport themselves quite satisfactorily. The Author nevertheless thinks that it would be possible to obtain favourable results when the particular conditions of the work afford a sufficient degree of utility to the addition of the pivots.

The ingenious musical contrivance employed by Mr. Airy to measure, on a small scale, the strains of the lattice of a bow-string girder (Min. Proc. Inst. C.E., Vol. xxvii.) has given results different from those obtained by calculation based upon the hypothesis of articulated members. This ought not to be a matter for surprise; it is an additional proof that in these matters, when there are an accumulation of organs capable of affording mutual aid, calculation is insufficient to decide upon the real distribution of the efforts. Nevertheless, it does not on that account lose all its utility; for it indicates distribution, possible or virtual, if not effective. It ought to be generally admitted as a principle of resistance, that a construction sustains itself as long as it has not expended the whole of its possible conditions of stability; it suffices, then, to prove that a certain state of stability is realisable, in order to be assured that the work will not perish.

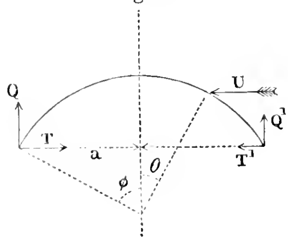
Appendix II. supplies, at least to a certain extent, the blank instanced by Professor Rankine relative to the effect of a small movement of the abutments and to the calculation of the lowering of the summit of the arch.

Mr. Barlow having manifested a desire for a solution of the resistance of a circular roof submitted to the horizontal pressure of the wind, the Author is induced to add a few words on this subject. If this particular question was omitted in the Paper it was because the Author had particularly in view bridges where the wind does not enter into consideration; and it was also owing to the dread of wearying the reader by placing before his eyes too numerous formulæ.

Let it be observed, in the first place, that the formulæ in No. 13 are general as regards the direction of the exterior forces, supposed only in the plan of the figure; oblique or horizontal forces, such as the impulse of the wind, will enter as easily as the weight or the vertical forces into the evaluation of the moments μ_1 and of the longitudinal forces N_1.

In the particular case of a circular arch with constant section, M. Bresse has given the following formulæ ("Mécanique Appliquée," first edition, Part I., No. 95), giving the reaction for the support occasioned individually by a horizontal force U, which is applied to the definite point by the angle θ of the following figure:

Fig. 37.



$$Q = -Q' = U \cdot \frac{\cos\theta - \cos\phi}{2\sin\phi},$$

$$T = \frac{T_1 + U}{2}, \quad T' = \frac{T_1 - U}{2};$$

the auxiliary T_1 represents the horizontal thrust, equal to two supports which there would be if the symmetry of loading were re-established, by the introduction of an equal and symmetrical force at U, on the other half of the arch. The value is:

$$T_1 = -2\,U\frac{2\,a^2\,A' + r^2\,\sin^2\phi\,(\theta + \sin\theta\cos\theta)}{2\,a^2\,B + 2\,r^2\sin^2\phi\,(\phi + \sin\phi\cos\phi)};$$

the letter A′ represents the following expression:

$$A' = \frac{\theta}{2} - \frac{1}{2}\sin\theta\cos\theta - \sin\theta\cos\phi + \theta\cos\theta\cos\phi;$$

A designates the value already indicated in No. 15 of the Paper of which the other notations are equally preserved; r is the radius of gyration of the constant section of the arch.

Every oblique force given can be resolved into a horizontal force and a vertical force, each producing stresses which can be calculated, and which superpose algebraically.

Mr. Young also has raised the question of the action of the wind; but adds to the problem a condition which, far from complicating, simplifies it.

He supposes that one of the extremities of the arch, the left, for instance, slides upon its support. There is in this case no longer any occasion to invoke the theory of the deformation of the arch to find the thrusts on the supports: the thrust T would be nil if the support were frictionless; it will be $T = f\,Q$ in virtue of a friction f by unity of normal pressure Q. Further, the equilibrium of forces, in horizontal projection, exacts $U + T' = T = f\,Q$, which determines the thrust T′ upon the right-hand support.

Although the definitive bridge over the Rhone (Plate 5B) has no direct reference to the discussion, seeing that it is a straight span, it is nevertheless thought advisable to add some information touching the execution of this work. In the first place the final disposition has to undergo a slight rehandling, by virtue of which the angle of the skew becomes equal to 69° 25′ instead of 67° 30′, which modifies a little certain oblique ribs.

The work has been adjudicated to Mr. Pilichody, Engineer, for the contract price of 205,000f. This sum exceeds the estimate of 170,000f. stated in Appendix II.; the addition is essentially owing to the fact that the state of the river and of the soil has led to the recurrence to the system of foundations by sheet-iron caissons worked by compressed air.

This system of foundation would appear to spread more and more, owing to the safety of its working and to the freedom from constraint in working at the time of low water. M. Croissette-

Desnoyers had laid it down as a rule for some years that the pneumatic system was too costly for depths less than 10 metres from the platform. We nevertheless see it here applied for a depth of 6 metres only. This shows not only that rules should not be considered absolute in the case of foundations, but also without doubt that the pneumatic process has now entered more fully into ordinary practice. Iron caissons always require careful construction. At one of the piers of the Rhone bridge, the caisson, executed by a sub-contractor, allowed the air to leak at several joints; and it was only by lining the interior with cement that the required tightness was obtained. The interior armatures of the carcase are stays of wood, capable of being taken away after the completion of the operation of sinking.

M. Cuénod, the Engineer of the company in charge of the works, has communicated the extract given herewith of the charges upon which the Contractor based his tender:—

		Fr.	Fr.
Piers and Abutments	Open air excavation	600	85,000
	Two sheet-iron caissons	20,000	
	Sinking with compressed air twice 6 metres, at 1,200 fr.	14,400	
	Masonry below low water mark (400 metres)	15,700	
	Masonry above water	34,300	
Superstructure	Wrought iron	100,000	112,000
	Cast iron (heel plates)	6,000	
	Wood	6,000	
Scaffolding		..	8,000
	Total of contract		205,000

There are besides some works of disposition on the banks, which will be executed by the company, as administrators.

Lausanne, Juin 26, 1871. J. Gaudard.

December 13, 1870.
T. HAWKSLEY, Vice-President,
in the Chair.

The discussion upon the Paper No. 1,224, "On Metal and Timber Arches," by M. Gaudard, was continued throughout the evening.

[Annual Meeting.

ANNUAL GENERAL MEETING.

December 20, 1870.

T. HAWKSLEY, Vice-President,
in the Chair.

Messrs. G. O. Budd, H. Hayter, C. E. Hollingsworth, R. C. May, and R. C. Rapier were requested to act as Scrutineers of the Ballot for the election of the President, Vice-Presidents, and other Members and Associates of Council; and it was resolved that the ballot-papers should be sent for examination at intervals of fifteen minutes, in order to expedite the labours of the Scrutineers.

The list of Members prepared for Council, together with the record of the attendances of the Members of Council, in Council and at the Ordinary General Meetings, was taken as read, and the Ballot was declared open.

The Annual Report of the Council, on the proceedings of the Institution during the past year, was read.

Resolved,—That the Report of the Council be received and approved; and that it be referred to the Council, to be printed and circulated with the Minutes of Proceedings, in the usual manner.

Resolved,—That the thanks of the Institution are due, and are presented to Messrs. Charles Hawksley and Hutton Vignoles, for the readiness with which they undertook the office of Auditors of Accounts; and that Messrs. Hutton Vignoles and Robert Charles May be requested to undertake the office of Auditors for the ensuing year.

Mr. Charles Hawksley returned thanks.

It was moved and seconded,—That the practice of inserting in the balloting list the attendances of the Members of the Council be discontinued.

On the motion being put from the Chair it was declared to be lost.

The Telford and Watt Medals, the Telford and Manby Premiums of Books, and the Miller Prizes, which had been awarded, were presented.

Resolved,—That the thanks of the Institution are justly due, and are presented to the Vice-Presidents and other Members of the Council, for their co-operation with the President, their constant attendance at the Meetings, and their zeal on behalf of the Institution.

Mr. Cubitt, Vice-President, and Mr. Murray, Member of Council, returned thanks.

Resolved unanimously,—That the cordial thanks of the Meeting be given to Mr. Vignoles, President, for his strenuous efforts in the interests of the Institution, for his extraordinary attention to the duties of his office, and for the urbanity he has at all times displayed in the Chair.

Mr. Hawksley, Vice-President, in the unavoidable absence of the President, returned thanks on his behalf.

Resolved,—That the special thanks of the Meeting be given to Mr. Barlow for his account of "The St. Pancras Station and Roof, Midland Railway."

Resolved,—That the best thanks of the Members be offered to Mr. Callcott Reilly, for his valuable Memoir on "Iron Girder Bridges."

Mr. Reilly returned thanks.

The Ballot having been open more than an hour, the Scrutineers, after examining the Papers, announced that the following gentlemen were duly elected to fill the several offices in the Council for the ensuing year:—

President.

CHARLES BLACKER VIGNOLES, F.R.S.

Vice-Presidents.

Joseph Cubitt.
Thomas Elliot Harrison.
Thomas Hawksley.
George Willoughby Hemans.

OTHER MEMBERS OF COUNCIL.

Members.

James Abernethy.
William Henry Barlow, F.R.S.
John Frederick Bateman, F.R.S.
Joseph William Bazalgette.
Nathaniel Beardmore.
Frederick Joseph Bramwell.
James Brunlees.
John Murray.
George Robert Stephenson.
Edward Woods.

Associates.

James Joseph Allport.
Major William Palliser, C.B.

Resolved,—That the thanks of the Meeting be given to Messrs. Budd, Hayter, Hollingsworth, May, and Rapier, the Scrutineers, for the promptitude and efficiency with which they have performed the duties of their office; and that the Ballot-papers be destroyed.

Resolved,—That the cordial thanks of the Meeting be given to Mr. Charles Manby, the Honorary Secretary, and to Mr. James Forrest, the Secretary, for their unremitting and zealous services on behalf of the Institution and of the profession.

Mr. Manby and Mr. Forrest returned thanks.

A vote of thanks to Mr. Hawksley, Vice-President, for his conduct in the Chair, was carried by acclamation.

ANNUAL REPORT.

Session 1870–71.

In conformity with the Bye Laws, it now becomes the duty of the Council, elected at the last Annual General Meeting, to lay before the members an account of the principal topics which have engaged their attention, and a statement of their management of the affairs of the Institution, during the past twelve months. This duty they perform with great satisfaction, as they feel that the members may be congratulated on the increased power and utility of the Society, as evidenced by its present condition and prospects, at the close of the fifty-third year of its existence.

The Minutes of Proceedings for the past session, which were issued during the recess at an earlier date than usual, will have enabled an opinion to be formed of the character of the Papers read at the Meetings, of the nature of the discussions upon them, and of the probable benefit to engineering science from their production. It will therefore only be necessary to allude to the different subjects brought forward at the twenty-five Ordinary General Meetings, to show that they were both varied and important. Two of the discussions were originated by Papers read in the preceding session, while fifteen new communications were selected from the great number presented to the Institution. These Papers related to the following subjects, viz., the present state of knowledge as to the theory of the Resistance of Materials; the Strength of Iron and Steel, with observations on the design of parts of structures consisting of those materials; the Public Works of the province of Canterbury, New Zealand; the line and works of the Saõ Paulo railway, in the empire of Brazil; the new Mhow-ke-Mullee viaduct on the Great Indian Peninsula railway, and the Pennair bridge on the Madras railway; the St. Pancras station and roof, Midland railway; the statistics of Railway Income and Expenditure, and their bearing on future railway policy and management; the maintenance and renewal of Railway Rolling Stock; the sluicing operations at the great Low Water Basin at the Birkenhead docks; the Wolf Rock lighthouse;

Ocean Steam Navigation, with a view to its further development; the conditions and the limits which govern the proportions of Rotary Fans; machinery for the Dressing of Lead Ores; the relative safety of different modes of Working Coal, and on Coal Mining in deep workings; and recent improvements in Regenerative Hot-blast Stoves for blast furnaces.

For some of these communications the following Premiums were adjudged:—

1. A Telford Medal, and a Telford Premium, in Books, to Edward Dobson, Assoc. Inst. C.E., for his Paper on "The Public Works of the Province of Canterbury, New Zealand."

*2. A Watt Medal, and a Telford Premium, in Books, to R. Price Williams, M. Inst. C.E., for his Paper on "The Maintenance and Renewal of Railway Rolling Stock."

*3. A Watt Medal, and a Telford Premium, in Books, to John Thornhill Harrison, M. Inst. C.E., for his Paper on "The Statistics of Railway Income and Expenditure, and their bearing on future Railway Policy and Management."

4. A Telford Medal, and a Telford Premium, in Books, to Thomas Sopwith, Jun., M. Inst. C.E., for his Paper on "The Dressing of Lead Ores."

5. A Telford Medal, and a Telford Premium, in Books, to James Nicholas Douglass, M. Inst. C.E., for his Paper on "The Wolf Rock Lighthouse."

6. A Watt Medal, and a Telford Premium, in Books, to George Berkley, M. Inst. C.E., for his "Observations on the Strength of Iron and Steel, and on the Design of parts of Structures which consist of those Materials."

7. A Watt Medal, and a Telford Premium, in Books (to consist of the second series of the Minutes of Proceedings, vols. xxi. to xxx. inclusive), to Robert Briggs, of Philadelphia, U.S., for his Paper "On the Conditions and the Limits which govern the proportions of Rotary Fans."

8. A Watt Medal, and a Telford Premium, in Books, to Edward Alfred Cowper, M. Inst. C.E., for his Paper on "Recent Improvements in Regenerative Hot Blast Stoves for Blast Furnaces."

9. A Telford Premium, in Books, to John Grantham, M. Inst. C.E., for his Paper "On Ocean Steam Navigation, with a view to its further development."

* Have previously received Telford Medals.

10. A Telford Premium, in Books, to Daniel Makinson Fox, M. Inst. C.E., for his "Description of the Line and Works of the Saõ Paulo Railway, in the Empire of Brazil."
11. The Manby Premium, in Books, to Emerson Bainbridge, Stud. Inst. C.E., for his Paper on "Coal Mining in Deep Workings."

In accordance with a rule long since adopted and acted upon, the Paper descriptive of the St. Pancras station and roof, Midland Railway, by Mr. W. H. Barlow, did not come under consideration in the adjudication of the premiums—the Author being a Member of Council. The warm thanks of the Institution are, however, justly due to Mr. Barlow for his valuable contribution.

It should be generally known, that Papers are willingly received from other than members of the Institution, whether natives or foreigners; and that the premiums are not limited to the subjects contained in the list issued yearly by the Council, such list being merely for the purpose of directing attention to questions of general interest, with the view of pointing out the kind of communications desired. The Minutes of Proceedings should, in the future, contain, as in the past, good Papers on the theory and practice of every branch of Civil Engineering, as one of the most obvious modes by which the reputation of the Institution and of the profession may be maintained. The Council feel confident that, in this respect, the record of the proceedings of the last official year may be referred to with gratification. The "Minutes" exceed those of any former year in the amount of matter and in the number of illustrations, and for the sake of convenience have been published in two volumes,—xxix. and xxx.,—each of 500 pages, and containing together 37 plates, instead of in one volume of 650 pages, with 26 plates, as in the previous year. Besides the articles already referred to, there will be found in these volumes, as usual, the inaugural address delivered in January last by the President, Mr. Vignoles, and a Memoir, by Mr. Callcott Reilly, explanatory of the principles on which two iron girder bridges, recently executed, had been designed, with the calculations on which the dimensions of the different parts had been determined. This Memoir the Council had great pleasure in accepting for publication, regarding it as an additional proof of the interest taken by the Author in the Institution. Although a larger sum than usual has been expended in publication, the Council consider that the cost is fully justified by the professional value of the additional matter, as the objects of the Institution are best promoted, and its advantages most widely extended, by the circulation of carefully edited and liberally illustrated Papers and discussions.

It has been decided to issue a General Index to the series of proceedings from Vol. XXI. to Vol. XXX., both inclusive, to be prepared on the same plan as the Index to the series from Vol. I. to Vol. XX., as the convenience and usefulness of that work have been fully appreciated.

The Council have awarded the following Miller Prizes to Students of the Institution for Papers read at the Supplemental Meetings:—

1. A Miller Prize to Robert William Peregrine Birch, Stud. Inst. C.E., for his Paper on "The Disposal of Town Sewage."
2. A Miller Prize to Henry Thomas Munday, Stud. Inst. C.E., for his Paper on "The Present and the Future of Civil Engineering."
3. A Miller Prize to William Walton Williams, Jun., Stud. Inst. C.E., for his Paper on "Roads and Steam Rollers."
4. A Miller Prize to Sidney Preston, Stud. Inst. C.E., for his Paper on "The Manufacture and the Uses of Portland Cement."
5. A Miller Prize to Edward Bazalgette, Stud. Inst. C.E., for his Paper "On Underpinning and making good the Foundations of the Irongate Steam Wharf, St. Katherine's, London."
6. A Miller Prize to Josiah Harding, Stud. Inst. C.E., for his Paper on "The Widening of the Liverpool and Manchester Railway between Liverpool and Huyton, and on the Construction of a Branch Line to St. Helen's."
7. A Miller Prize to the Hon. Philip James Stanhope, Stud. Inst. C.E., for his Paper on "The Metropolitan District Railway."

These prizes should operate as a stimulus to other Students to forward to the Institution detailed descriptions and accurate drawings of any work upon which they may be employed. The Council trust that the Students will avail themselves of every opportunity in the course of their professional engagements, to place on record observations on the progress of different works for communication to the Institution, in order that the Supplemental Meetings may be commenced at an early period each year.

The Council take this opportunity of tendering their sincere thanks to Mr. Fowler, Past-President, for the facilities he afforded the Students to examine the works of the Metropolitan District

Railway; to Messrs. John Bazley White and Brothers for the handsome manner in which they received and explained to the Students, at their works at Greenhithe, the whole process of the manufacture of Portland cement; and to Colonel Clarke, R.E., Associate of Council, for the equally liberal arrangements he made for the Students to inspect the Chatham Dockyard Extension Works. Visits to works in progress and to manufactories cannot fail to be productive of advantage, and it is hoped that similar privileges may be frequently extended to the Students.

One of the duties devolving on the Council under the Bye Laws is to arrange for the publication of such documents as may be calculated to advance professional knowledge. As an aid to this end, allusion may be made to the volume, lately issued, on "The Education and Status of Civil Engineers, in the United Kingdom and in Foreign Countries," compiled from original reports and statements by Engineers of eminence, and by the authorities of educational establishments, supplied to the Council in reply to their inquiry for particulars as to the system of instruction pursued in the training of Engineers. This collection of information, derived from so many sources, was circulated without the expression of any opinion on the part of the Council, it being deemed preferable, in the first instance, to put the members in possession of the particulars so obtained, as, owing to the nature of the subject, various and discordant ideas may be, and no doubt are, entertained regarding it.

The Library continues to receive considerable accessions, by presentation, by exchanges with other Societies, and by the purchase of all books which it is thought may prove useful. Every opportunity is taken to obtain copies of old and rare books on Engineering and on the arts specially allied to it, as well as to procure works of reference on general scientific subjects. The aim has always been to make the Library the depository of all treatises and documents directly or indirectly relating to engineering, and thus to accumulate a stock of information from which all may derive the greatest benefit. It is now more used than formerly for the purpose of research, by which its great value becomes better known, and deficiencies are sometimes pointed out. To show the rapid increase in the collection it may be mentioned that, in 1851, when the first edition of the Catalogue was issued, it contained 3,000 volumes and 1,500 pamphlets; in 1866, on the publication of the second edition of the Catalogue, it comprised 5,500 volumes and 3,200 pamphlets; whereas now there are in the Library upwards of 7,000 volumes and 4,500 pamphlets. The additions have been so numerous during the last four years as to necessitate the printing of a Supplement, which has extended to 160 pages, or about two-fifths of the size of the original catalogue. This Supplement has been prepared in precisely the same manner as the Catalogue, that is, in a form which has been found, by experience, best calculated to facilitate reference to the subjects particularly connected with the various branches of engineering.

The Conversazione given by the President in the rooms of the Institution at the close of the session was in every respect successful, and was never previously surpassed. A new feature was introduced—that of inviting the contribution of models and objects of engineering interest from the Continent. The collection was thus made more comprehensive and useful, while for a time it lessened the difficulty, which every year increases, of imparting freshness and novelty. The Catalogues of the works of art lent to decorate the rooms, and of the engineering models and instruments exhibited in the meeting room, have been reprinted in Vol. xxx. of the Minutes of Proceedings, as some acknowledgment to those who aided the President, and as a record for future reference.

The tabular statement of the transfers, elections, deceases, and resignations of the members of all classes during the years 1868–69 and 1869–70 (taking into consideration the names which have been erased from the Register) is as follows:—

Year.	Honorary Members.	Members.	Associates.	
1868–69.				
Transferred to Members	..	..	6	
Elections	1	30	82	113−72=41
Deceases	1	13	11	
Resignations	..	2	13	72
Erased from the Register	..	3	29	
Members of all Classes on the Books, November 30th, 1869	16	655	918	1,589
1869–70.				
Transferred to Members	..	..	19	
Elections	..	42	114	156−42=114
Deceases	..	17	20	
Resignations	..	..	4	42
Erased from the Register	..	..	1	
Members of all Classes on the Books, 30th November, 1870	16	699	988	1,703

This shows a net effective increase of 44 Members and of 70 Associates, being at the rate of $7\frac{1}{4}$ per cent. in the twelve months.

In the same period 55 Students have been admitted by the Council, 12 have retired, and 8 have been elected Associates; the increase has therefore been 35, which number added to 138 previously on the list raises the total to 173.

Ten years ago there were on the books 24 Honorary Members, 355 Members, 537 Associates, and 14 Graduates, together amounting to 930; now the gross number is 1876.

The Council have to regret the loss to the Institution by death of the following:—Peter Ashcroft, James Melville Balfour, John Braithwaite, Zerah Colburn, Frederik Willem Conrad, Samuel Dobson, Charles Caulfeild Fishe, John Harris, John Bernard Hartley, George Leather, Robert Morrison, George Paddison, Thomas Paterson, William Alexander Provis, Charles Sanderson, James Thomson, and William Weaver, *Members;* Henry Corles Bingham, William Thomas Blacklock, Robert Dunkin, Alister Fraser, William Gammon, *General* Sir William Gordon, Henry Hakewill, Conrad Abben Hanson, John William Heinke, George Houghton, *Major* Julian St. John Hovenden, Robert William Kennard, John Meeson Parsons, Joseph Pitts, George Selby, Gerrit Simons, George Henry Smith, *Sir* John Thwaites, George Barnard Townsend, and *Captain* James Vetch, *Associates.*

Biographical notices of some of those who have been thus removed will be given in the Minutes of Proceedings.

The following Associates have tendered their resignations in writing, and have been permitted to retire from the Institution:—*Admiral* George Elliot, *Major* Henry Hooper Foord, Rochfort Astle Sperling, and Henry Waring.

In laying before the Members "a statement of the Funds of the Institution and of the receipts and payments for the past year," the Council would remark that the accounts, as certified by the Auditors, show that on the 1st of December, 1869, there was a balance in the hands of the Treasurer of £268 9*s.* 9*d.*, and there has been received since (including the Appold bequests of £1,800), £9,653 10*s.*, making together £9,921 19*s.* 9*d.* The late Mr. John George Appold, Assoc. Inst. C.E., by his will dated the 7th of July, 1853, bequeathed to "the Treasurer of the Institution of Civil Engineers," for the use of the Institution, the sum of £1,000, payable on the decease of his wife. Mrs. Appold died on the 23rd of March, 1870, and by her will also liberally bequeathed a like sum to the Institution. Both these legacies have been paid to the

Treasurer, less £200 for the legacy duty. The disbursements have amounted to £6,583 6*s.* 10*d.*, while a sum of £2,968 15*s.* 6*d.* has been invested—in the purchase of £1,000 North Eastern Railway Company Four per Cent. Debenture Stock, of £1,000 Great Northern Railway Company Four per Cent. Debenture Stock, and of £1,000 Manchester, Sheffield and Lincolnshire Railway Company Four-and-a-Half per Cent. Debenture Stock—leaving a balance (after deducting £4 4*s.* 9*d.* due to the Secretary) of £369 17*s.* 5*d.* This statement does not represent the funds of the Institution in a sufficiently favourable point of view, as the amount of outstanding bills on the current expenditure is less than at the same time last year. Although a larger sum than usual has been expended in publication, yet out of the ordinary revenue a sum of £1,168 15*s.* 6*d.* has been invested, and the present cash balance exceeds the former one by £101 7*s.* 8*d.*

During the year William Matthew Coulthurst, Esq., the senior partner in the firm of Messrs. Coutts and Co., bankers, has been appointed, and has accepted the office of, Treasurer of the Institution, in the room of Sir Edmund Antrobus, Bart., deceased.

The ordinary Receipts have comprised :—

	1869.						1870.					
	£.	*s.*	*d.*	£.	*s.*	*d.*	£.	*s.*	*d.*	£.	*s.*	*d.*
From subscriptions, fees, &c., exclusive of Building Fund	5,734	3	6				6,199	12	6			
,, dividends on investments not in trust	328	14	3	6,062	17	9	293	7	4	6,492	19	10
,, Building Fund fees, on election . .	. .			586	19	0	. .			847	7	0
,, dividends on Trust Funds . . .	. .			382	3	5	. .			390	19	8
Total . .	. .			£7,032	0	2	. .			7,731	6	6

The Expenditure during the same period (exclusive of the new building) has been :—

	1869.				1870.			
		£.	*s.*	*d.*		£.	*s.*	*d.*
To disbursements, including Minutes of Proceedings (less received on that account) .	. .	3,650	16	2	. .	6,134	14	2
,, Premiums under Trusts . . .	. .	134	19	10	. .	242	17	1
,, Investment:— General account .	. .	. .			. .	2,968	15	6
Total . .	. .	£3,785	16	0	. .	£9,346	6	9

During the financial year ending on the 30th of November last, a sum of £66 17*s.* was paid on account of the new building and its accessories, making, with the sum of £18,210 2*s.* 4*d.* expended up to the date of the previous Annual General Meeting, a total outlay of £18,276 19*s.* 4*d.*

The Funds of the Institution consisted on the 30th of November last of

I. General Funds.

Institution Investments:—	£.	s.	d.	£.	s.	d.	£.	s.	d.
Great Eastern Railway Four per Cent. Debenture Stock .	3,650	0	0						
London and North Western ditto	1,162	0	0						
London, Brighton, and South Coast ditto . . .	1,000	0	0						
North Eastern ditto . .	1,500	0	0						
Great Northern ditto . .	1,000	0	0						
Manchester, Sheffield, and Lincolnshire Four and a Half ditto	1,000	0	0						
New Three per Cents. . .	1,344	1	8						
							10,656	1	8

II. Trust Funds.

	£.	s.	d.	£.	s.	d.	£.	s.	d.	£.	s.	d.
1. Telford Fund:—												
Three per Cent. Consols . .	£2,839	10	6									
Three per Cent. Annuities .	2,570	5	1									
				5,409	15	7						
Unexpended Income, Three per Cent. Consols	2,377	10	6									
Ditto, Annuities.	200	11	5									
				2,578	1	11						
							7,987	17	6			
2. Manby Premium:—												
Great Eastern Railway Five per Cent. Preference Stock							200	0	0			
3. Miller Fund:—												
Lancashire and Yorkshire Railway Four per Cent. Debenture Stock .	£2,000	0	0									
Great Eastern ditto . .	1,100	0	0									
				3,100	0	0						
Carried forward				3,100	0	0	8,187	17	6	10,656	1	8

II. Trust Funds—*continued.*

				£	s.	d.	£.	s.	d.	£.	s.	d.
Brought forward				3,100	0	0	8,187	17	6	10,656	1	8
Unexpended Income, Three per Cent. Consols	582	18	6									
Ditto, Annuities	248	19	8									
				831	18	2						
							3,931	18	2			
										12,119	15	8
Total Nominal or par Value of different Securities . . .										22,775	17	4
Cash in the hands of the Treasurer, Dec. 1, 1870							374	2	2			
Less Petty Cash due to the Secretary . . .							4	4	9			
										369	17	5
Together amounting to										£23,145	14	9

as against £19,775 17*s.* 4*d.* at the date of the last Report.

The Funds under the charge of the Institution were at a maximum on the 30th of November, 1867, when they amounted to £29,835 18*s.* Now they are only less by a sum of £6,690 3*s.* 3*d.*, although in the interval there has been expended on account of the new building and its accessories, as previously stated, £18,276 19*s.* 4*d.*

The following is a Summary of the different Securities in which the above Funds are placed :—

	£.	s.	d.	£.	s.	d.
Government Stocks :—						
Three per Cent. Consols	5,799	19	6			
Three per Cent. Annuities	3,019	16	2			
New Three per Cents.	1,344	1	8			
				10,163	17	4
Great Eastern Railway :—						
Five per Cent. Preference Stock	200	0	0			
Four per Cent. Debenture Stock	4,750	0	0			
				4,950	0	0
Lancashire and Yorkshire Railway :—						
Four per Cent. Debenture Stock				2,000	0	0
London and North Western Railway :—						
Four per Cent. Debenture Stock				1,162	0	0
London, Brighton, and South Coast Railway :—						
Four per Cent. Debenture Stock				1,000	0	0
North Eastern Railway :—						
Four per Cent. Debenture Stock				1,500	0	0
Great Northern Railway :—						
Four per Cent. Debenture Stock				1,000	0	0
Manchester, Sheffield, and Lincolnshire Railway :—						
Four and a Half per Cent. Debenture Stock . . .				1,000	0	0
Total Nominal or par Value . . .				£22,775	17	4

Notwithstanding repeated notices, the total amount of Subscriptions remaining due on the 30th November, 1870, including the current year, is somewhat in excess of what it was at the same date last year. Then the Arrears of Subscriptions were only £218 8*s.*; now they are as under:—

		£. s. d.	£. s. d.
For 1870.	From members of all classes residing abroad	55 13 0	
	Ditto, in the United Kingdom	166 8 6	
			222 1 6
For 1869.	From members of all classes residing abroad	5 15 6	
	Ditto, in the United Kingdom	27 16 6	
			33 12 0
For 1868.	From members of all classes residing in the United Kingdom	..	16 5 6
For 1867.	Ditto, ditto	..	13 2 6
	Total		£285 1 6

In the preparation of the balloting list for the election of officers for the ensuing year, the former Members of Council have been nominated for re-election. The attendances, both in Council and at the Ordinary Meetings, have been prefixed to each name, in accordance with special resolutions passed at previous Annual Meetings. A variation has been made in printing the list, by arranging the names of the Members of Council in the order of their election on the Council, instead of alphabetically as heretofore. To the list, in conformity with the Bye Laws, the names have been added of Messrs. John Coode and Charles William Siemens, *Members*, and of Messrs. John James Allport, James Timmins Chance, Sampson Lloyd, and *Major* William Palliser, *Associates*, all of whom have consented to serve if elected.

ABSTRACT

ABSTRACT *of* RECEIPTS *and* EXPENDITURE.

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RECEIPTS.

Dr.	£.	*s.*	*d.*	£.	*s.*	*d.*
To Balance in the hands of the Treasurer				268	9	9
— Subscriptions and Fees:—						
Arrears	143	17	0			
Current	5,023	14	6			
Subscriptions for 1871	39	7	6			
,, 1872	9	19	6			
,, 1873 (in part)	2	2	6			
Fees	491	8	0			
Life Compositions	309	4	6			
				6,019	13	6
— Building Fund				847	7	0
— Publication Fund				109	4	0
— Council Fund				50	0	0
— Publications:—Sale of Transactions				90	12	3
— Telford Fund:—						
Dividends, 1 Year, on £2,839. 10*s.* 6*d.*, Three per Cent. Consols	83	11	2			
Ditto, 1 Year, on £2,570. 5*s.* 1*d.*, Three per Cent. Annuities	75	12	8			
Ditto, 1 Year, on £2.377. 10*s.* 6*d.*, Three per Cent. Consols (Unexpended Dividends)	70	3	3			
Ditto, 1 Year, on £200. 11*s.* 5*d.*, Three per Cent. Annuities (Unexpended Dividends)	5	18	2			
				235	5	3
— Manby Premium:—						
Dividends, 1 Year, on £200, Great Eastern Railway Five per Cent. Preference Stock				9	16	0
— Miller Fund:—						
Dividends, 1 Year, on £2,000, Lancashire and Yorkshire Railway Four per Cent Debenture Stock	78	8	4			
Ditto, 1 Year, on £1,100, Great Eastern Railway, Four per Cent. Debenture Stock	43	2	8			
Ditto, 1 Year, on £582. 18*s.* 6*d.*, Three per Cent. Consols (Unexpended Dividends)	17	0	4			
Ditto, 1 Year, on £248. 19*s.* 8*d.*, Three per Cent. Annuities (Unexpended Dividends)	7	7	1			
				145	18	5
— Institution Investments:—						
Dividends, 1 Year, on £3,650, Great Eastern Railway Four per Cent. Debenture Stock	143	2	0			
Ditto, 1 Year, on £1,162, London and North Western Ditto	45	11	2			
Ditto, 1 Year, on £1,000, London, Brighton and South Coast Ditto	39	4	2			
Ditto, 1 Year, on £500, North Eastern Ditto	19	12	1			
Ditto, 1 Year, on £1,344. 1*s.* 8*d.*, New Three per Cents.	39	11	4			
				287	0	9
— Interest upon Deposit Account in the Union Bank				6	6	7
— Donations to Library				20	15	0
Carried forward				£8,090	8	6

from the 1ST DEC., 1869, *to the* 30TH NOV., 1870.

PAYMENTS.

Cr.	£. s. d.	£. s. d.
By House, Great George Street, for Rent, &c.:—		
Repairs	49 10 8	
Rent	618 19 6	
Rates and Taxes	49 12 9	
Insurance	20 11 6	
		768 14 5
— Salaries		900 0 0
— Clerks, Messengers, and Housekeeper		351 10 0
— Postage and Parcels:—		
Postage		50 14 11
Parcels		3 7 5
— Stationery, Engraving, Printing Cards, Circulars, &c.		167 15 0
— Coals, Candles, Oil, and Gas:—		
Coals	33 9 0	
Candles	0 1 0	
Oil	0 7 6	
Gas	50 18 6	
		84 16 0
— Tea and Coffee		43 7 0
— Library:—		
Books	141 10 7	
Periodicals	21 0 6	
Binding Books	50 15 8	
Council Gift	47 5 6	
		260 12 3
— Publication, Minutes of Proceedings		3,096 3 11
— Telford Premiums		160 1 0
— Watt Medal		1 10 0
— Manby Premium		14 11 8
— Miller Prizes		68 4 5
— Diplomas		39 6 6
— Manuscripts, Original Papers, and Drawings		3 10 3
— Annual Dinner		126 9 9
— Winding and Repairing Clocks		1 10 0
— Incidental Expenses:—		
Christmas Gifts	1 16 0	
Assistance at Meetings	22 4 6	
Ditto at Supplemental Meetings for Students	2 12 6	
Beating Carpets and Sweeping Chimneys	0 14 0	
Household Utensils, Repairs, and Expenses	60 18 6	
		88 5 6
— Engineering Education		182 11 6
— Indian Circular		51 11 2
— Legal Expenses, Locke Bequest		3 10 10
Carried forward		£6,468 3 6

ABSTRACT *of* RECEIPTS *and* EXPENDITURE

RECEIPTS—*cont.*

Dr.	£.	s.	d.	£.	s.	d.
Brought forward				8,090	8	6
To Benevolent Fund				31	11	3
— Bequest of J. G. Appold	900	0	0			
— „ Mrs. Appold	900	0	0			
				1,800	0	0
— Balance due to the Secretary				4	4	9
				£9,926	4	6

from the 1st DEC., 1869, *to the* 30th NOV., 1870.

PAYMENTS—*cont.*

Cr.					£.	s.	d.
Brought forward					6,468	3	6
By New Building :—	£.	s.	d.				
Gas Fittings	24	0	0				
Furniture	42	17	0				
					66	17	0
— Benevolent Fund					48	6	4
— Investments :—							
£1,000, North Eastern Railway Company Four per Cent. Debenture Stock	982	10	0				
£1,000, Great Northern Ditto	986	5	6				
£1,000, Manchester, Sheffield and Lincolnshire Railway Company Four and a half per Cent. Debenture Stock	1,000	0	0				
					2,968	15	6
— Balance in the hands of the Treasurer					374	2	2
					£9,926	4	6

Examined and compared the above Account with the Vouchers and the Cash Book, and find this account to be correct, leaving a Balance in the hands of the Treasurer of Three Hundred and Seventy-four Pounds, Two Shillings, and Two-pence.—Nov. 30th, 1870.

(Signed) CHARLES HAWKSLEY, } *Auditors.*
HUTTON VIGNOLES, }
JAMES FORREST . . *Secretary.*

December 2*nd*, 1870.

[Appendix

APPENDIX TO ANNUAL REPORT.

MEMOIRS.

SIR DAVID BREWSTER was the second son of James Brewster, rector of the grammar school of Jedburgh, who was well known and much esteemed as one of the best classical scholars and teachers of his day, and a man of sterling worth. David was born in the Canongate of Jedburgh on the 11th of December, 1781. The characteristic gifts which distinguished his later years can be traced from a very early age. Along with three brothers of excellent abilities, he always kept a high place in his classes, and was much looked up to by his schoolfellows. A dilapidated pane of glass in his father's house, carefully observed and experimented upon, paved the way for future discoveries and researches in refracted light, while the beauty of the scenery and the legendary lore amidst which he was reared produced the intense love of nature, art, and poetry which blended so remarkably with the "sterner stuff" of science.

Jedburgh and its neighbourhood were noted for the number of intelligent and scientific residents, many of them possessing inventive genius. David Brewster was not slow in availing himself of the advantages of such society, but the greatest help he received, which was indeed the foundation of his truest education, was from James Veitch of Inchbonny, a small but beautiful property, half a mile from Jedburgh. He followed the occupation of a wheelwright, which he could not be persuaded to relinquish; but threw into his ploughs and his wheels the skill and originality of his remarkable talents. The intervals of business, however, were spent in that wonderful process of self-education, which, as in many other cases, resulted in an amount of knowledge of almost every branch of learning and research, which made him universally esteemed, and brought him into scientific and friendly relations with many contemporaries eminent in science and literature. James Veitch's workshop was a gathering-place for all the young men in the neighbourhood who were athirst for knowledge, besides being a favourite resort for some of maturer years and well-earned distinction; and amongst others, of several intelligent and scientific French prisoners of war. Astronomy, mechanics, mathematics, and theology, were amongst their favourite topics, and David Brewster must have formed one of the quaint and varied group from a very early age, as it is recorded that at ten years old he had finished the construction of a telescope, under his friend's auspices, an occupation in which he delighted for many years. At the age of twelve, he went to the University of Edinburgh, where he pursued his studies with characteristic diligence, soon becoming the friend and companion of the eminent professors of his day—Playfair, Robinson, Dugald Stewart, and others; and at the age of nineteen, he became M.A. Destined for the Scottish Church by his father, he studied divinity with close attention, although his holiday time was spent in scientific research, of which there are interesting records in a long and close correspondence with his early friend at Inchbonny. He made his first scientific discovery in 1800, while studying his favourite science of optics. The year after, at the age of twenty, he commenced his independent literary career, by becoming the editor of the "Edinburgh Magazine." To all his other labours he added those of tuition, entering the family of Capt. Horsbrugh of Pirn, and afterwards of General Dirom. A tendency to nervous faintness, and a consequent dread of speaking in public, which was increased by overwork, caused him much suffering in preparing for the clerical profession. He however preached often, his ministrations were much liked, and he accepted a presentation to the living of Sprouton in 1808. This was in the gift of the Duke of Roxburgh, but being under litigation, Brewster withdrew his claim, rather than cause any disputation of his right. It was not till 1809 that he felt free to follow the bent of his own genius, and devote himself to literature and science. In 1810 he married Juliet, the youngest daughter of James Macpherson, Esq., M.P., of Belleville, translator of Ossian, and settled in Edinburgh, where four sons and one daughter were born.

The "Edinburgh Encyclopædia" was at this time, and for twenty succeeding years, the most arduous and anxious of his undertakings. Although commanding admiration for its intrinsic merits, the dilatory conduct of publishers and contributors marred its success, and taxed the editor's strength to the utmost, who had to make up for the negligence of others. Many painful circumstances and broken friendships took place; while in a pecuniary sense the undertaking was a complete failure, and till a compromise took place, a few years before his death, was a constant cause of anxiety and apprehension. He also continued to edit the "Edinburgh Magazine," under different forms, for many years. In 1811 he brought out a new edition of Ferguson's "Astronomy," contributing an Introduction and twelve supplementary chapters. In 1812 he wrote the article "Burning-glasses" for the Encyclopædia, containing the description of a polyzonal lens which he had invented the year before, when examining the experiments of Buffon. Soon after, he published a "Treatise on new Philosophical Instruments." Still later, he edited a translation of "Legendre's Geometry;" four volumes of Professor Robison's "Essays on Mechanical Philosophy;" and "Euler's Letters to a German Princess," with notes, and a life of the author.

In 1814 he travelled in France and Switzerland, becoming acquainted with many foreign *savants;* was received with distinguished honour by the French Institute, and made many interesting observations, nothing being lost on his enquiring mind. In 1815 he became a Fellow of the Royal Society of London, for which he had begun to contribute a long and important series of papers, principally on light; he received three of its best medals, and a prize from the French Institute, of which he afterwards became one of the eight Foreign Associates. Indeed, honours and rewards flowed so rapidly upon him that it is impossible to specify all, but the large book in which the letters, diplomas, burgess tickets, announcements of medals, &c., are collected is a remarkable one for size and interest. In 1816 Brewster invented the kaleidoscope; but by mismanagement he reaped no practical benefit from an invention which spread over Europe and America with a rapid *furore* which is now scarcely credible, but which was quickly pirated. In 1820 he was elected an Honorary Member of this Institution, in the objects of which he ever took a lively interest; and, in 1821, he was much occupied in founding the Royal Scottish Society of Arts, of which he was named "Director."

In 1823 he changed his residence to Allerly, a small property which he purchased not far from his early haunts, and became one of the remarkable circle which gathered round Abbotsford in the days of Sir Walter Scott. He met with a sore bereavement in the sudden death, by drowning, of a favourite son, and not long after, his early friend, Mr. Veitch, passed away from his useful and laborious life. Brewster's mind had been much impressed by the decline of science in England, and the various causes leading to this effect—a subject upon which he wrote and spoke with the utmost energy during his whole life. He was the first to propose a "British

Association for the Advancement of Science," a plan which was warmly taken up, and quickly resulted in a brilliant and successful meeting at York, in September, 1831. An annual meeting has taken place ever since in different towns of the United Kingdom, at one of which its originator made his last public appearance thirty-six years afterwards. Many of the advantages to science which he anticipated were worked out by this Association. In 1831 Dr. Brewster received the Hanoverian Order of the Guelph, and was shortly after knighted. In 1833 he offered himself for the vacant chair of Natural Philosophy in Edinburgh, and his undeserved failure was a serious disappointment, especially as his affairs had become extremely embarrassed. In 1836 the gift from Government of a well-earned pension, and, in 1838, his presentation to the Principalship of the United College of St. Salvator and St. Leonard, in the University of St. Andrews, relieved him from pecuniary difficulties. He threw himself with his accustomed ardour into the duties of his new office, and although incurring much unpopularity by the much-needed reformation of many old abuses, his residence of twenty-three years at St. Andrews was marked as a useful and happy era of his life. In 1843 Sir David Brewster took a prominent part in the disruption of the Church of Scotland; and in consequence of his adherence to the Free Church, an attempt was made to eject him from his Chair as Principal, which proved unsuccessful.

His busy pen had produced, at different times, a "Treatise on Optics," his first "Life of Sir Isaac Newton," "Letters on Natural Magic," a "Treatise on Magnetism," "The Martyrs of Science," besides many serial contributions; and in 1844 he became a regular contributor to the "North British Review" for twenty years. In 1850 Sir David Brewster lost his wife, and the following year his eldest son. In 1851 he was appointed a juror of the Great Exhibition in London, and he presided at the Peace Congress. The year after he was chosen President of the Working Men's Educational Union, and received from the Emperor of the French the decoration of the Legion of Honour.

In 1852 a visit to Ireland introduced him to the great telescope at Birr Castle, where he visited with delight its noble architect; making many astronomical observations, and never tiring of the mechanical wonders so peculiarly interesting to the early telescope maker. One of the many subjects which Brewster studied with his peculiar gift of absorbed energy was the "Plurality of Worlds,' writing a review of Dr. Whewell's celebrated essay for the "North British Review," which he afterwards expanded into a popular volume called "More Worlds than One; the Creed of the Philosopher, and the Hope of the Christian." His second and larger "Life of Sir Isaac Newton," after years of steady work, was published in 1855. He had unusual facilities for correct information, as the valuable collection of the Newton MSS. belonging to Lord Portsmouth was placed at his disposal, and his early and passionate admiration for the great master of science made it, more than any of his works, a true labour of love, and a successful vindication of an unjustly traduced memory.

In the same year Sir David Brewster fulfilled with much interest his duties as juror for the department of optical instruments in the Paris Exhibition. In 1856 he went to the South of France with his family; and in the spring of 1857 was united, at Nice, to Jane Kirk, second daughter of Thomas Purnell, Esq., Scarborough, by whom he had, in 1861, a daughter, Constance Marion. After his marriage he visited Rome, Florence, Padua, Treviso, &c.; and one of his greatest interests was following the traces of his favourite "martyr of science," Galileo Galilei.

While attending the meeting of the British Association at Aberdeen in 1859, he received by telegram the tidings that he was appointed principal of the University of Edinburgh. It was not, however, for some time that he could decide on its acceptance; but the pain of leaving his old St. Andrews home once over, he thoroughly enjoyed the resuming of old interests, and forming new friendships in his Alma Mater. From this time he resided part of the year in Edinburgh and the rest at his beloved home at Allerly, which was near enough to the University to permit of a regular attendance at college meetings, going and returning in one day—a practice which he continued till within a short time of his death. In October, 1859, he presided as principal at the first meeting of the General Council of the University of Edinburgh, where he declared his old college friend, Lord Brougham, duly elected as Chancellor, who in his turn afterwards appointed him Vice-Chancellor. Brewster's career in St. Andrews and elsewhere has been sometimes characterised by ruggednesses of temper and undue severity of judgment; but it is gratifying to find that the eight years of his connection with the University of Edinburgh were unclouded by any jars or misunderstanding, and his loss was afterwards felt by "each member of it as that of a valued and respected friend." His whole character became greatly mellowed, and after years of doubt and struggle on religious subjects, his acceptance with the heart of those gospel doctrines, some of which he had held intellectually all his life, was remarkably full and clear, holding "the faith once delivered to the saints" with the simplicity of the child and the reasonableness of the sage.

In 1860 he was made an M.D. of the University of Berlin, an honour which gratified him exceedingly, as it was in recognition of the services he had rendered by his discoveries "to the sciences auxiliary to medicine." He had previously been made a Chevalier of the Prussian Order of Merit. In 1864 a severe illness of a prostrating nature, arising from an organic disease of the heart of old standing, brought him very near death; but he again rallied, and being in London at the time, under the medical care of Dr. Sieveking, at that gentleman's request he presided at a meeting which resulted in the formation of the Edinburgh University Club, of which he was the first president. In the same year he was appointed President of the Royal Society of Edinburgh. Although the aged frame, in spite of the iron strength of his constitution, was now shaken by frequent illness, yet the energy and vigour of his mind were as unclouded as ever, while his habits of constant and persevering work were continued so near the last breath of eighty-six years, that it may literally be said that "he died in harness." It was with all the animation of youth that he came forward in 1867 again to vindicate the fame of his beloved master, in the well-known "Newton-Pascal Controversy." At the meeting of the British Association at Dundee in September of that year, he fainted in the first public meeting, and although well enough to read several Papers on that and various scientific subjects, yet he never again completely rallied. For some months the flame of the taper fluttered, but the light of faith grew ever brighter and steadier. He expired at Allerly, surrounded by his family, on the 10th of February, 1868, "at peace with all the world," as he touchingly said, and filled with "the peace of God." He was buried in the old churchyard of Melrose Abbey, and on his tombstone are graven the simple and suitable words, "The Lord is my Light."

Besides the books, pamphlets, and serial writings to which allusion has been made, the catalogue of his printed contributions to the different scientific societies and their organs of communication, which is preserved in the library of the Royal Society of Edinburgh, gives tangible proof of the passionate industry which, with the daily exercise of the most minute powers of observation, formed the secret of his successful researches.

His principal observations and experiments were in the demesne of polarized light, in which he made many original discoveries, although some of his theories, such as the red, blue, and yellow colours of the spectrum, have been disproved by subsequent researches. By patient observations he improved on the discovery of Wollaston and Fraunhofer, by increasing the 600 black lines already observed in the solar spectrum to the number of 2000. One of the subjects which most interested him in his later years was that of film forms, making many experiments on the tints of the soap bubble, and the beauties of decomposed glass, while an unfinished Paper on their nature was his latest legacy to science. He was well known as an inventor of philosophical instruments—the subject of his first book—amongst others, several kinds of micrometers, the lithoscope, the kaleidoscope, and the lenticular form of the stereoscope. There is also undoubted proof that the polyzonal lens now used in lighthouses, along with the lenticular apparatus called the "holophote," was invented by him, although afterwards independently discovered by M. Fresnel, who at once got it introduced into France. It was owing, moreover, to the persistency of Brewster's efforts that, after being thwarted for many years, it was at last adopted in the United Kingdom.

He was often himself unfortunate in missing the practical advantages and the deserved credit of his inventions, while he was too keen in their defence and too ready to take up scientific gauntlets; but he was ever warmly interested in the inventions and discoveries of others. The reform of the Patent laws, the introduction of scientific education into schools, and the recognition by the State of science and scientific men as mighty engines for the practical good of the country—as in France—were among his constant and not always unsuccessful aims. One of his latest efforts was to send a Paper on Scientific Education to the Journal of the Inventors' Institute, of which he was President, along with a few farewell lines, expressive of his warm interest in its objects and designs. Another was to petition Lord Derby on behalf of the widow and children of a prematurely deceased scientific *confrère*, the successful result of which arrived the day after the death of him who had made the appeal.

MR. JAMES MELVILLE BALFOUR, the youngest son of the late Rev. Lewis Balfour, D.D., was born on the 2nd of June, 1831, in the manse of Colinton, near Edinburgh, where his father was minister of the parish for the long period of thirty-seven years. He received his education at the High School and University of Edinburgh. He early showed a strong inclination and remarkable aptitude for mechanical pursuits, and was an earnest student of mathematics and natural philosophy. After attendance in different workshops in Edinburgh and Germany, with the view of gaining a knowledge of practical mechanics and of the proper methods of working materials, he entered the employment of Messrs. D. and T. Stevenson (MM. Inst. C.E.), by whom he was chiefly engaged in connection with the lighthouse department of their engineering practice. In the spring of 1863 he was appointed Marine Engineer to the province of Otago, New Zealand, and on the termination of a two years' engagement became Marine Engineer to the Colonial Government of New Zealand, and at once set himself to the task of designing marine works and of establishing the lighthouse system after the Scotch model. He took an active part in everything bearing on the welfare of New Zealand. In connection with the New Zealand Exhibition of 1865, he made an elaborate series of experiments on the strength, weight, elasticity, deflection and durability of colonial timbers. These laborious experiments were the first to put the colonists in possession of reliable coefficients for the strength of sixty-four different kinds of native timber. He was consulted on all matters relating to the improvement of harbours and rivers, the most important of his works of this kind being the Port Chalmers Dry Dock, estimated to cost £50,000. He completed the Dunedin Waterworks, which had been commenced under another engineer, and he also accomplished a great part of the marine survey of the west coast of the Middle Island. He also designed and had executed under his immediate superintendence various lighthouses, among others those at Tairoashead, Nugget Point, Dog Island, Cape Campbell, and Farewell Spit, the lanterns and apparatus for which were sent from Edinburgh. For the lighthouse department he prepared a set of uniform harbour regulations and signals, to replace the great variety of regulations and signals formerly in use. These arrangements have given satisfaction to all concerned. He took a lively interest in all matters connected with shipping, as evidenced by the elaborate instructions which he prepared for the proper adjustment of ships' compasses, and also the inquiries which he instituted as to the causes of shipwrecks on the coast. The whole lighting, buoyage, and beaconage of the colony was fast being brought into proper system at the time of his death.

He invented a "Refraction Protractor"—the first instrument of the kind that was constructed—and which is thus spoken of by Professor Swan, of St. Andrews, in his Paper "On new forms of

Lighthouse Apparatus:"[1] "I cannot too strongly express my obligations to that gentleman (Mr. Balfour) for the invaluable aid which I have derived from his ingenious instrument. Without its help I should scarcely have undertaken to protract the designs which accompany this Paper." His design for a pneumatic floating dry dock was novel, consisting of water-tight compartments, some of which were entirely closed so as to give a constant buoyancy which would barely allow the whole mass to sink. By forcing air in, instead of lifting the pontoon and allowing the water to run out, he proposed, on the principle of the 'camel,' to dispense with the cast-iron columns, hydraulic presses, girders, &c., which have hitherto been used to get the vessel out of the water.

Mr. Balfour was, unfortunately, cut down in the midst of usefulness at the early age of thirty-eight, being accidentally drowned by the capsizing of a boat in the heavy surf off Timaru, on the 18th of December, 1869. He was highly respected by the colonists for his unassuming manners and kindly disposition.

Besides his many printed official reports he has left, among others, the following publications:

"Description of a Refraction Protractor." Royal Scot. Society of Arts, 1857, vol. v., App. p. 34 (awarded the Society's silver medal and plate).

"Description of an Instrument for dividing circles on paper." Royal Scot. Society of Arts, 1859, vol. v., p. 149 (awarded silver medal).

"Description of a simple improvement on Reflectors for Lighthouses." Royal Scot. Society of Arts, 1863, vol. vi., p. 211.

"Experiments on the Strength of Colonial Woods," 1865. [Inst. C.E., Tract 8vo., vol. 155.]

"Description of a combined optical square and 'line finder.'" Royal Scot. Society of Arts, 1866, vol. vii., p. 319 (awarded silver medal).

"Instructions to licensed adjusters of the Compasses of Steam-vessels," 1869.

Mr. Balfour was elected a Member of the Institution on the 15th of May, 1866.

MR. JOHN CASS BIRKINSHAW was born in the year 1811 at Bedlington Iron Works, in the county of Durham, where his father was manager to the Bedlington Iron Company, and the patentee

[1] *Vide* Transactions Royal Scottish Society of Arts, vol. vii., p. 534.

of the malleable iron fish-bellied rails. He had no special education, other than a country lad received in those days, except in being sent to Edinburgh for a session, where he studied under Professors Leslie and Hope. His father and George Stephenson were at that time very intimate, and the latter used frequently to come over to Bedlington, where Mr. Longridge, then managing partner of the Iron Company, had employed him in making a railway to connect the iron works with a colliery about $1\frac{1}{4}$ mile distant. This was the first railway laid with the malleable iron rails. In his hours of play, young Birkinshaw made models of machinery, mills, forge-hammers, and electrical machines, as other boys did. To these tendencies of the imitative faculty, but principally no doubt to the intimate relations of his father with George Stephenson, must be attributed the reason of his becoming an engineer. But some other causes may have exercised an influence: the old timber bridge, which spanned the river Blyth opposite the windows of his house, was one day pulled down, and a new one erected in its place by the village mason; the dam across the river had become unsafe, and was rebuilt by the elder Rastrick while he was yet a boy; Sir W. Fairbairn, of Manchester, M. Inst. C.E., had spent some time at the works in his early days as a millwright repairing the old water-wheels; and young Birkinshaw had himself designed a small suspension bridge, which was subsequently put up by the Bedlington Iron Company across the Wansbeck at Morpeth.

But at all events it so happened that, on Robert Stephenson's return from South America, young Birkinshaw became, it is believed, his first articled pupil. At the Forth Street engine factory of Messrs. Stephenson at Newcastle he spent many pleasant days, sometimes visiting Bedlington to make a series of sketches of all the machinery in vogue there, furnaces, rolling-mills, and forges, for Mr. Stephenson's use, copies of some of which, elaborated into finished drawings, were afterwards published in the "Encyclopædia Britannica." After assisting to check the levels, and to test the general accuracy of a survey made by Mr. Giles, for the proposed Newcastle and Carlisle railway, which the Stephensons were retained to oppose in Parliament, Mr. Birkinshaw was next engaged upon the Leicester and Swannington railway for a year or more, as Resident Engineer. Then came, in 1830, the opening of the Liverpool and Manchester railway—the first line of any importance which had been made in England. On this occasion he was summoned to assist in carrying out the arrangements. This was no sooner over than he went to Canterbury, where, with

Mr. Thomas Cabry, he made the Canterbury and Whitstable railway, now a branch of the South Eastern.

The London and Birmingham railway was commenced in the year 1834, and Mr. Birkinshaw was appointed assistant engineer at the London or Camden Town end. But he did not long retain that position, for the contractor having become bankrupt, the works were carried on by the company, and Mr. Birkinshaw, as having already had much experience, was considered by Mr. Stephenson a proper person for their direction and management. The heaviest works were the Primrose Hill tunnel, made through the London clay, the open-cut tunnel at Kensal Green, and the Brent bridge, beyond which his portion of the line did not extend far. The works were well done, and elicited favourable remarks both from Mr. Stephenson and from the chairman of the company.

In 1837 Mr. Birkinshaw had confided to him the execution of what was called the Birmingham and Derby railway, but was rather the Derby and Hampton line, Hampton being a small village in the forest of Arden, at some distance from Birmingham, on the railway from that town to London. The works were of a moderately easy character in point of construction. The materials were the blue bricks of Staffordshire, sandstone from the neighbouring quarries, and Memel timber for the river crossings, of which there were several long ones but little elevated above the adjoining meadows. It is said that the permanent way of this railway has cost the present Midland Company less money for repairs than any other part of their system. Mr. Birkinshaw had a notion that the works were the best that had ever been done under his direction, and he went over them the year before he died to see what they were like then, after so many years had elapsed. The carriages and rolling stock were made under his own eye at Tamworth. As the line approached completion the general management was entrusted to him by the Directors, and Messrs. Allport, Assoc. Inst. C.E., and Kirtley, now the General Manager and Locomotive Superintendent of the Midland line, were his chief officers. On the completion of the Birmingham station and the works of the Tame Valley line in 1842, Mr. Birkinshaw resigned his situation; on which occasion he was presented with a gold snuff-box by the enginemen of the line, and a piece of plate by his friends and pupils. The year after this was, to all intents and purposes, an off-year, only enlivened by an application for the position of manager to the Edinburgh and Glasgow railway, which was unsuccessful. Subsequently Mr. Birkinshaw was appointed with Mr. Robert Stephenson joint Engineer to the York and Scarborough railway, which was begun in the summer of the year 1844, and was carried on with energy; the line having been staked out and much of the land got before the Act of Parliament was obtained. A single line was at first made, but it was doubled as soon as it was found what a large traffic was to be provided for. On the completion of the Scarborough line in 1845, surveys were made of the Seamer and Bridlington and the Hull and Bridlington railways. An Act had already been got by the Hull and Selby directors, and the line was placed under Mr. Birkinshaw's direction, and these two railways, when formed, made a continuous piece of road from Scarborough, by way of Bridlington, Driffield, and Beverley, to Hull. The Harrogate and Church Fenton railway was also started about this time; it is about 17 miles long, includes some heavy works, with a tunnel and viaduct over the Crimple beck. The Pickering branch of the York and Scarborough line was constructed for the purpose of communication with Whitby by the Whitby and Pickering railway, which, having been made for horse traffic only, had to be altered so as to render it available for locomotives.

From great practice Mr. Birkinshaw had acquired considerable facility in selecting the best line of country, whether by hill or dale, through which to carry any proposed railway, and a quick eye for detecting its advantages and disadvantages, as well as the faculty of imparting his ideas readily and with accuracy, so that his assistants had little trouble in finding their way through a country which he had once walked over, being merely required to level it; and, if it were asked if he did any one thing better than another, it might be said that in giving parliamentary or legal evidence he was not surpassed. These were valuable qualities at this juncture, when every engineer spent so much time in the witness box before Parliamentary Committees, or with lawyers preparing for the strife of partisanship. Many railways were projected, to which Mr. Birkinshaw was Engineer, but only a small number were destined to be made, among which may be mentioned the York and Beverley, in good part at least, the Selby and Market Weighton, the Knottingley branch, the Malton and Driffield, and the Thirsk and Malton, in which last he was associated with Mr. T. E. Harrison, Vice-President Inst. C.E.

Mr. Birkinshaw, although devoting himself almost exclusively to railways, sometimes applied himself to other things. There were the Harrogate waterworks and the Scarborough waterworks, and at the latter place he put up an engine to assist the water-wheel, which had pumped all the water required by the town previously, and otherwise extended the works. On the retirement of Mr. Hudson, Mr. Birkinshaw for a time gave up practising actively as an engineer; but it was not for long, as circumstances soon made it imperative on him to resume the profession which he had almost discarded. At the invitation of Mr. Leeman, he made an elaborate report on the Foss navigation, which it was proposed, on his recommendation, to do away with.

He now took up his residence in London, and made the plans of the Ware and Hertford railway, of the Luton and Hertford, of the Lymington branch of the South Western, and of the Sittingbourne and Sheerness railway, now a branch of the London, Chatham, and Dover railway, as well as designed a system of railways for the Isle of Wight. During a portion of this time (1860 to 1862) he was in partnership with Mr. Conybeare, M. Inst. C.E., but the connection was soon dissolved. In 1860 he went to Denmark, and made a report on the reclamation of about 25,000 acres of land on the west coast of Jutland, and on the cultivation of the land so reclaimed for the Danish Land Company. On returning he was appointed Consulting Engineer, and the works, or some of them, were let in 1863. It was in this last-named year, too, that he was employed for Mr. Fowler, in examining the Seine from Havre to the port of Rouen, in ascertaining by a series of measurements the depths of that river, and in collecting evidence as to its condition; and a "Report on the project of General De Brossard for the improvement of the port of Havre, and the navigation of the Lower Seine," was made by him. This was a work of considerable labour, of a tedious kind; in consequence of the impossibility of getting up a survey, day by day, of the port of Havre and the river Seine under the different aspects of high and low water, of spring and neap tides, and other circumstances. In 1863 he was engaged by Mr. Murray, who had the concession of the Turin and Savona railway, to go to Italy and report on the construction of the works. He spent the best part of a year on this expedition, taking levels down the valley through which the line runs, and which, like all mountain valleys, is subject to violent floods. He also made surveys, estimates, and reports of the Acqui and Cano, the Piacenza, Genoa, and Chiavari, and the Carmagnola and Turin railways, for the same gentleman.

He again went to Denmark, in the spring of the year 1865, for the Danish Land Company; and this time both as engineer and to superintend the work. Here he spent the summer and autumn, with very inadequate means in money and materials to carry on the works. From anxiety to see them well executed, he exposed himself to all weathers, which greatly increased a complaint he suffered from on his journey out. But the work went on, and the embankment was at length finished, when a violent storm of wind burst over the exposed shore, the sea rose higher than it had been known to do for twenty years, the water was seen to insinuate itself between the woodwork and the bank, and in a short time the bank was nearly all washed away. The advanced season of the year forbade any further action at the time with the embankment; and it was left to be repaired in some future summer.

There is not much more to tell of Mr. Birkinshaw. On his return to England he was engaged on various engineering works and arbitration cases, which occupied his attention until January, 1867. For the last ten or twelve years of his life he had been harassed by constant pecuniary embarrassments. An incurable malady had for several years been increasing in gravity, and the disease had now reached such a height that no skill or care could be of use. Gradually he became less able to endure the plodding of his daily avocations; till at last he succumbed, and died in March, 1867, in the fifty-sixth year of his age.

Mr. Birkinshaw was elected a Member of the Institution on the 2nd of March, 1847; but there is no record of his having joined in the discussions at the evening meetings, nor did he ever contribute a Paper.

Mr. JOHN BRAITHWAITE was the third son of the late Mr. John Braithwaite, whose ancestors had carried on a small engineer's shop and smithy at St. Albans from the year 1695, and was born at No. 1, Bath Place, in the New Road, London, on the 19th of March, 1797. In the year 1806-7 his father was operating with the diving bell on the 'Earl of Abergavenny,' East Indiaman, sunk off Weymouth, from which he recovered £130,000 of specie, as well as the general cargo, and successfully blew up the wreck with gunpowder. In this undertaking he was assisted by the subject of this memoir, who, young as he was, was entrusted with the signalling from his father, who was at work below, and who was frequently under water for from six to eight hours at a time. Mr. Braithwaite was educated at Mr. Lord's school at Tooting in Surrey. From the time he left school he was at home attending in the manufactory, and making himself master of the different trades pertaining to mechanical engineering; and he became a skilled draughtsman. In February, 1818, his father died, leaving the business to his sons, Francis and John. Francis died in 1823, and John Braithwaite carried on the business alone, having inherited a large connection with the London brewers, distillers, water-works companies, and being engaged in the manufacture of pumps, sinking wells, &c. He increased this business considerably in Scotland, Ireland, and the West Indies, and added to it the making of high-pressure steam-engines, many of which, from 1 H.P. to 30 H.P., are still working satisfactorily. Besides these works he had, in 1817, been called in by the then existing Committee of Engineers, held in the Strand, to report upon the Norwich steam-boat explosion before the House of Commons; and in 1820 he ventilated the House of Lords by means of air-pumps. In 1822 he made the donkey-engine, and in 1823 cast the statue of the late Duke of Kent. He was a great patron of inventors, rendering them much real service to enable them to develop their ideas and plans, and in a manner that secured him the esteem of all who availed themselves of his valuable practical experience. In 1827 he was introduced to Messrs. G. and R. Stephenson; and about the same time he became acquainted with Captain John Ericsson, who then had many schemes in view. In 1829 Messrs. Braithwaite and Ericsson constructed the locomotive engine, "The Novelty," for the "Rainhill experiments," which, as has been observed by Mr. Vignoles, President Inst. C.E., "if it did not command success, deserved it."[1] This engine was the first that ever ran a mile within a minute, as it did that distance in fifty-six seconds. At this time Mr. Braithwaite constructed the first practical steam fire-engine, which was ultimately destroyed by the London mob. It had, however, previously done good service, among other places, at the burning of the English Opera House, at the destruction of the Houses of Parliament, and had assisted largely in extinguishing the fire at Messrs. Bishop's distillery, for which service Mr. Bishop gave him £500. It threw 2 tons of water per minute, burnt coke, and got up steam in from twenty minutes to thirty minutes: but it was looked upon with so much jealousy by the fire brigade of the day, and such impediments were thrown in the way of its working—such as playing cold water upon the boiler, &c.—that he gave it up. He, however, constructed four others of larger dimensions; one for the municipality of Berlin, and one for Liverpool, which gave great satisfaction. In 1833 Mr. Braithwaite built the caloric engine in conjunction with Captain Ericsson. Next year he ceased to take an active part in the management of the engine works in the New Road; but began to practise as a civil engineer for public

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxix., p. 310.

works, and was largely consulted at home and abroad upon various railway and other undertakings and mechanical appliances, particularly as to the capabilities of, and probable improvements in, locomotive engines. Mr. Braithwaite was a strong advocate of four-wheel light inside-cylinder engines, and a supporter of the views of the late Mr. E. Bury, M. Inst. C.E., as to light engines and frequent trains, with duplicate power for heavy loads; but he afterwards altered several engines by adding another pair of wheels. In 1834 the Eastern Counties railway was projected and laid out by him in conjunction with Mr. Vignoles. The Act was passed in the year 1836, which incorporated the Eastern Counties Railway Company for constructing a line of railway from Shoreditch, London, to Norwich and Yarmouth, *viâ* Colchester and Ipswich, with a stock and loan capital of £2,133,000, and he was soon afterwards appointed Engineer-in-chief for its construction. At that time there was no gauge Act in operation, and engineers were free to select the gauges of their respectives lines. Mr. Braithwaite apparently considering the Eastern Counties isolated as to railway communication from the manufacturing districts, and likely to remain so, and well knowing by experience the ill effects of the then considered and admitted insufficient boiler and machinery space, for efficient and economic working, within the ordinary gauge of 4 feet 8½ inches, advised the adoption of a 5½-feet gauge, which was afterwards reduced to 5 feet, and upon that gauge the line was constructed as far as Colchester, a distance of 51¼ miles, to which place it was opened in March, 1843, the works being made wide enough for a 7 feet gauge. On the recommendation of Mr. Robert Stephenson, it was subsequently altered to the national gauge of 4 feet 8½ inches; and the change was effected on the Northern and Eastern, Eastern Counties, and Blackwall railways between the 5th of September and the 7th of October, 1844. In after years, Mr. Braithwaite advocated a still narrower gauge than that of 4 feet 8½ inches. He ceased to be officially connected with the Eastern Counties railway on the 28th of May, 1843. While Engineer to that company he introduced on the works the American excavating machine, the power of which was fairly tested. In a stiff clay the original and imperfect machine from New York filled the earth-wagons at a cost of 1½*d.* per cubic yard, all expenses included; and in America it is still used, particularly for dredging. In like manner Mr. Braithwaite endeavoured to utilise the American steam locomotive pile-driving machine, which failed, not by its demerits, as subsequent application has proved, but from disagreement among the patentees, of whom Mr. Braithwaite was one. He was joint founder of the "Railway Times," which he started in conjunction with Mr. J. C. Robertson, as editor, in 1837, and he continued sole proprietor till 1845. In that year he had been drawn into some commercial speculation, which, together with his undertaking the preparation of plans for the direct Exeter railway, and the panic of the period, necessitated the winding up of his affairs. Mr. Braithwaite had, in 1844, a share in a patent for extracting oil from bituminous shale, and works were erected near Weymouth, which, but for his difficulties, might have been successful. Up to this time Mr. Braithwaite had assisted Captain Ericsson in many costly experiments at the manufactory. Some years before, 1836-38, they had fitted up for Mr. John Robins an ordinary canal boat with a screw propeller designed by Captain Ericsson, which started from London along the canals to Manchester on the 28th of June, 1838, returning by way of Oxford and the Thames to London, being the first and last steam-boat that has navigated the whole distance on these waters. The boat maintained a speed of from 5 miles to 6 miles an hour on the canals, and upwards of 9 miles an hour on the river, when fully loaded. This was considered at the time a great success, but was abandoned on account of the deficiency of water in the canals, and the completion of the railway system, which diverted the paying traffic. In 1844, and again in 1846, Mr. Braithwaite was much on the Continent surveying lines of railway from Paris to Dieppe, the Eastern of France, and others; but he still found time for other business, such as surveying Langston harbour in 1850, and building the Brentford brewery in 1851. From that year, however, he was principally engaged in chamber practice, and acted as consulting engineer; advising upon most of the important mechanical questions of the day, for patent and other purposes, in which his opinion was much sought after and highly esteemed. Mr. Braithwaite was always kind and hospitable; his apprentices and employés were noticed by him and liberally treated. His conversation was lively, frequently instructive, and a vein of humour appeared in his remarks. He had no mean skill in painting and drawing, and his professional sketches were clear and explanatory. He was correct in his calculations, strict in his estimates, and his works on the Eastern Counties railway were characterized by solidity of construction.

Mr. Braithwaite was elected a Member of the Institution on the 13th of February, 1838, and at the time of his death he was one of the oldest members of the Society of Arts, having been elected into that body in the year 1819: he was also a life governor of seventeen charitable institutions. He died very suddenly on the 25th of September, 1870, and his remains were interred at Kensal Green cemetery.

Mr. GEORGE ROWDON BURNELL, after receiving a partial education as an architect, was in early life a partner in a large iron foundry, which he left in 1840. He then visited America, and on his return, after a short stay in Belgium and the North of France, was engaged under the late Mr. Joseph Locke, Past President Inst. C.E., in superintending the construction of a portion of the Paris and Rouen railway, and subsequently of the Rouen and Havre railway. On the completion of these works he was appointed superintending architect to the Havre Docks and Warehouse Company; but in the year 1848, like many of his fellow-countrymen, he found it expedient to return to England, as the cry was at that time raised "La France pour les Français." During the seven years that he resided abroad he contracted a great admiration for the talents of the French, and hence the strong colouring which pervaded most of his writings, in which he so constantly held them up for study and imitation. By degrees, however, after his return to England, he became more and more exclusively literary. He was as well known as a writer on architectural subjects as on those connected with engineering; and was one of very few who have united a Fellowship of the Royal Institute of British Architects with a Membership of the Institution of Civil Engineers.

Mr. Burnell though possessed of considerable engineering talent did not actively follow the practice of the profession, but was principally occupied in literary pursuits connected with it. He was the author of several rudimentary works, including one on limes and cements. In 1861 and afterwards he wrote "The Annual Retrospect of Engineering and Architecture." He edited for some years "A Builder's and Contractor's Price Book," and "The Engineers' and Architects' Pocket Book," was connected, too, for a long time with the "Journal of Gaslighting," contributed to "Brande's Dictionary of Science," the "Dictionary of Architecture," as published by the Architectural Publication Society, and wrote largely and constantly for the "Building News." In addition to these strictly professional works, he wrote occasionally on general literature for the "Eclectic," and other reviews. His incessant literary labour, from which he never rested, brought on the disease which so prematurely closed his career; an attack of paralysis of the brain in the summer of 1866 quite incapacitated him from any further labour, and after a tedious illness of two years, with no hope of recovery from the first, he died on the 25th of July, 1868, aged fifty-four, a great loss to his fellow-professional men, to whom his unbounded store of facts and universal information were always liberally open.

Mr. Burnell was elected a Member of the Institution on the 6th of February, 1866. He had previously, as a visitor, often attended the meetings and taken part in the discussions, besides contributing a Paper "On the Machinery employed in sinking Artesian Wells on the Continent,"[1] for which he was awarded a Telford premium in books. After his election as a Member he presented a Paper "On the water supply of the City of Paris,"[2] and for this also he was awarded a Telford premium in books.

Mr. ZERAH COLBURN was born at Saratoga, in the State of New York, in 1833, and was named after his uncle, the celebrated mathematician. His father died soon after, and his mother, very poor and infirm, removed to Hillsborough, New Hampshire, where, during his boyhood, young Colburn earned his living on a farm. His early means and opportunities for acquiring an education were limited to a few months' attendance at a district school, generally a week at a time, a short clerkship in a factory, and such books as he could find in a remote country village. But his industry and his wonderful memory more than made up then, and throughout life, for the want of early advantages. From an odd volume of the old "Penny Magazine" he gained a knowledge of the world and an inspiration to see and figure in it. From May to December, 1845, at the age of twelve, he was engaged in keeping the monthly accounts, invoices, pay rolls, &c., in the office of the Sugar River Manufacturing Company, at Claremont, New Hampshire, and in paying the hands, two hundred in number. His first sight of a city, and, what was a greater thing to him, a locomotive, was at Concord. The strong but hitherto in him undeveloped mechanical talent at that sight asserted its proper place, and the locomotive was ever after his chief study, and the subject of his best conclusions and ablest writings. At the age of thirteen he found his way to Lowell, Massachusetts, and

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxiii., p. 459.
[2] *Ibid.*, vol. xxv., p. 480.

was brought to the machine shop of Mr. L. B. Tyng, by Mr. Lovejoy, overseer of the Middlesex Corporation. In Mr. Tyng he found a friend for life. From that shop he passed in September, 1846, into the employment of Mr. A. L. Brooks, an extensive lumber manufacturer and dealer in Lowell, in the capacity of clerk. Here the intelligence and eagerness with which he studied machinery of all kinds, but especially stationary and locomotive steam engines, attracted the attention of his employer, who brought him to the notice of Mr. William A. Burke, at that time and for many years superintendent of the Lowell machine shop. The pen-and-ink sketches of young Colburn seemed to Mr. Burke so meritorious, that he at once employed the lad, then hardly fifteen years of age, in his drawing-office. Mr. Burke says of him at this period: "His entrance into a large machine shop, where a great diversity of machinery was being constructed, was to him like finding a new world, and a close attendance to his particular duties could hardly be expected in one of his genius, and certainly was not realised. But with this exception he was a favourite with us all, and the ease and readiness with which he comprehended and apprehended all the principles and details of machinery, were very unusual—I might say remarkable." In May, June, and July, 1850, it appears by his letters that he was employed by Mr. Tyng and Mr. Calvert, both well-known mechanics of Lowell, for occasional work, probably drawing. In March, 1852, he was engaged in designing machinists' tools for Mr. Tyng, and in May, 1853, accompanied his employer to Alexandria, Virginia, and was for a short time connected with the locomotive works there.

While his head-quarters were at Lowell, he was frequently resident in Boston. His first literary attempt was in verse for the Boston "Carpet Bag." His railway career commenced on the Boston and Maine railroad, under the late Mr. Charles Minot, then its manager, who was attracted by his brightness and practical ideas. He was also, about this time, engaged on various other railways leading out of Boston, especially the Boston and Lowell, in tabulating the particulars of their locomotives.

In a few months Mr. Colburn had mastered the details of the locomotive engine, tabulated the dimensions and proportions of those under his observation, and published a small, but excellent and still useful, treatise on the subject. He then got a subordinate position, and soon rose to the superintendence of the locomotive works of Mr. Souther, in Boston. Here he tabulated and committed to memory the dimensions of all parts of the then standard locomotive, and the cost of all the materials and labour employed in its construction. With the exception of a few months at the Tredegar Works, at Richmond, where, in connection with Mr. Souther, he started the manufacture of locomotives, Mr. Colburn now made New York his head-quarters until 1858. His more important professional work at this time was his superintendence, for a year or more, of the New Jersey locomotive works at Paterson, during which engagement he made some improvements, still standard, in the machinery of freight engines.

Although eminently fitted for the management of practical construction, Mr. Colburn had already found that the literature of engineering was his true calling. As early as March, 1847, some small pamphlets appeared from his pen, entitled, "Monthly Mechanical Tracts," which were published at Lowell. In 1850 these were followed by "The Locomotive Engine, theoretically and practically considered," published at Boston, and republished at Philadelphia in 1853. From 1851 to 1853 he contributed to "The American Railway Times," at Boston, including a serial treatise on the locomotive engine. And in the latter year he was introduced by Mr. Tyng to Mr. Poor, editor and proprietor of the "Railroad Journal," then the leading American publication in this department. Mr. Colburn immediately commenced writing for it, and in fact soon edited the mechanical department of it; and professional readers, recognizing the hand of a master, began to look for a new era in technical journalism. They were not disappointed. In November, 1854, Mr. Colburn started, in New York, the "Railroad Advocate," a weekly journal, devoted especially to the machinery of railroads, and addressed chiefly to the master mechanics and the more intelligent operatives. The next year he enlarged the "Advocate," and it is worthy of mention, as illustrating Mr. Colburn's extraordinary power of memory, that he kept no books for many months, but simply remembered when every subscription and advertisement fell due, and made no mistakes. In the summer of 1855 Mr. Colburn thought he saw, in his large acquaintance with railroad men, the way to a fortune in the business of railroad supplies. He therefore, in March, 1856, sold the "Advocate" to Mr. A. L. Holley, then draughtsman at the New York Locomotive Works, bought land warrants with the money, journeyed to Iowa and located his lands, and then returned to New York—but with another scheme. The frontier life had temporarily charmed him, and he got together an engine and machinery to set up a steam saw-mill in the far West. However, before his plans were completed, literature had resumed the mastery, and he again became a contributor to the "Advocate," and at the same time arranged his supply business. He engaged with Mr. Horatio Ames to introduce the Ames' tires; and with his knowledge, industry, and shrewdness, he assisted to build up a business which, unfortunately, the character of the tires did not maintain. But Mr. Colburn was not made for a merchant. He pined for larger professional observation and knowledge, and for a wider field. As suddenly as he went into trade he left it, and sailed for Europe. During a flying visit among the machine and iron works of England and France, whereof the story is recorded in the "Railroad Advocate," he had become again and finally wedded to literature. Returning to New York, he resumed a half share of this periodical, which was then enlarged and entitled the "American Engineer."

In the autumn of 1857, Messrs. Colburn and Holley were commissioned by several leading railroad presidents to visit Europe, to report on the railway system and machinery abroad; and in view of the financial troubles of 1857, they were advised to stop, at least temporarily, the publication of their paper, which was never resumed. Permanent way and coal-burning locomotives were found to be the most important subjects of the period, and in 1858 their report on these subjects, "The Permanent Way, and Coal-burning Locomotive Boilers of European Railways," was published, and circulated among American railway managers. Mr. Colburn wrote the report entirely, while Mr. Holley, besides sharing the expense, assisted in collecting information and in preparing drawings. Mr. Colburn's thorough and, to American readers, entirely new and startling analysis of the cost and economy of British railways was the foundation of many of the reforms that have since, although slowly, become standard in America, especially in the matter of improved road-bed and superstructure. The success of this book was such that its authors determined to continue their researches, and in the autumn of 1858 Mr. Colburn again visited London. Here his abilities attracted the attention of the founder and editor of "The Engineer," and, at that gentleman's request, Mr. Colburn wrote several articles, which were of so high a character that he was ultimately appointed to an influential position on the staff of that paper; eventually, for a time, occupying the post of editor in charge, while the responsible editor and proprietor was absent on the Continent through ill-health. The leading articles written by Mr. Colburn during this period have never been excelled in vigour, accuracy, and elegance of style, in any scientific journal. He at this time wrote the chapters devoted to American locomotives for Mr. D. K. Clark's "Recent Practice in the Locomotive Engine;" and in 1860 an essay on "Steam Boiler Explosions," working out the theory now known as the 'percussive theory.' Mr. Colburn then resolved to start another engineering paper in America. He left England in the 'Great Eastern' steamship, on her first passage in 1860, and soon selected Philadelphia, the principal seat of mechanical engineering in America, as the birthplace of his own "Engineer," which was commenced in August, 1860; but the time was not ripe, in America, for a publication of this kind, and in a moment of despondency he dropped his new enterprise, and sailed for England. In January, 1861, he again became the editor of the London "Engineer," which position he continued to occupy till November, 1864, and till the spring of 1865 he was an occasional contributor to its pages.

About this time he wrote several pamphlets on professional subjects. On the 5th of May, 1863, he presented a Paper to The Institution of Civil Engineers "On American Iron Bridges," for which he was awarded a Telford medal and a Telford Premium of books. In the same year he was the auther of "An Inquiry into the nature of Heat," and commenced, and subsequently completed, the first eight parts of "Locomotive Engineering;" and he also contributed a paper "On the Relation between the Safe Load and the ultimate Strength of Iron," to the Society of Engineers, of which Society he was President in 1865, when he read a further paper "On certain Methods of treating Cast Iron in the Foundry." In 1864 Mr. Colburn wrote a treatise on "The Gasworks of London," and gave a "Description of Harrison's Steam Boiler," to the Institution of Mechanical Engineers. On the 7th of February, 1865, Mr. Colburn was elected a Member of the Institution of Civil Engineers, and later in the spring of the same year read two papers before the Society of Arts, one "On the Ginning of Cotton," the other, "The Manufacture of Encaustic Tiles and Ceramic Ornamentation by Machinery." A paper "On American Locomotives and Rolling Stock," read before the Institution on the 9th of March, 1869, and for which he was awarded a Watt Medal and Telford Premium of books, and a paper "On Anglo-French Communication," read before the Society of Arts on the 1st of December, 1869, complete the catalogue of his contributions to learned Societies.

In January, 1866, Mr. Colburn started as his own property the well-known journal "Engineering;" and he continued its active management, and gave it the full benefit of his journalistic experience and of his talents as a writer until its success was firmly established. This done, however, his editorship became a nominal one, and at length, at the end of February, 1870, he ceased to be connected with that periodical in any way.

Naturally restless and exceedingly impulsive he went to greater extremes both in work and relaxation than most men, and his irregularities were attended with melancholy results. On his giving up the proprietorship of "Engineering," he proceeded to Paris, and subsequently to America, where he avoided all his old friends. They tried to follow him, but on the 25th of April he was found lying in an orchard at Belmont, near Boston, whither he had wandered a day or two before from New York, mortally wounded by a pistol-shot, fired by his own hand. He was taken in a dying state to the county hospital, where he expired a few hours afterwards. He was buried on the 4th of May at Lowell, Massachusetts; his funeral being attended by his family connections resident there, and by many members of the profession; among them Mr. Tyng and Mr. Burke, his early friends and instructors in mechanics. Mr. Zerah Colburn was a man whom the profession could ill afford to lose. His thoroughly practical education in the workshop, his extended observation of engineering works, his intimate acquaintance with professional literature, his remarkable quickness of comprehension, his more remarkable memory, and his mechanical talent and inborn engineering ideas, combined to give him the distinction of being the best general writer in the profession.

MR. SAMUEL DOBSON was the son of a farmer, and was born on the 28th of April, 1826, at Newton Hall, Horsley, in the county of Northumberland, and attended the village school at Ovingham. He was apprenticed as a colliery viewer to Mr. John Gray, of Garesfield, Durham, for three years; and at this time, finding himself somewhat deficient in education, he attended a night school at Crawcrook, near Ryton, kept by Mr. Craigie, a celebrated teacher of mathematics. At this school, Messrs. Nicholas Wood, George Elliot, M.P., John Nixon, Robert Anderson, C.E., and other men of note, received a considerable portion of their education. He afterwards acted for two years as an assistant to the late Mr. T. J. Taylor, of Earsdon, Northumberland. About the year 1848 he removed to South Wales, on being appointed, through Mr. Taylor's influence, mineral agent to the Clive (now the Windsor) estate; and subsequently he engaged in business on his own account as a mining engineer, and became mineral agent for many of the principal properties in the district. He had charge of the opening and working of some of the most important and extensive of the steam coal collieries of South Wales, amongst which Messrs. Powell's Duffryn collieries may be especially mentioned. He was also in extensive practice as a consulting Engineer in all matters relating to mining, and of late years had turned his attention to Civil Engineering matters. He projected the Penarth Harbour Dock and Railway, for which he and Mr. John Hawkshaw, Past-President Inst. C.E., were afterwards the joint Engineers. He was also instrumental in establishing several railways in South Wales, and reported upon experiments made by himself as to the comparative value of Welsh and North Country coals for marine purposes.

Mr. Dobson was elected a Member of the Institution on the 2nd of November, 1856. He was also a Member of the North of England Institute of Mining Engineers, as well as a Fellow of the Geological Society of London. He was devotedly attached to the profession, worked very hard, and in private life was a man of engaging manners, very sincere, and one who formed many lasting friendships. Mr. Dobson's health had been failing for some time, and he died in London, of consumption, on the 26th of July, 1870, in the forty-fifth year of his age.

Mr. CHARLES CAULFEILD FISHE was the second son of Commander Fishe, R.N., and was maternally descended from the noble families of Drogheda and Charlemont. He was born at Waterford, on the 22nd of October, 1821; and was educated at the endowed school, New Ross, and at Colonel Colby's engineering school, Phœnix Park, Dublin. In the year 1837 he was articled to Mr. Edward White, a Civil Engineer engaged on the Boundary Survey of Ireland; and in the following year went to Mr. William Jones, District Boundary Surveyor, and received the appointment of Boundary Surveyor from Sir Richard Griffith, Bart., M. Inst. C.E., then the General Surveyor of Ireland. On the completion of the Survey, in the autumn of the year 1843, he came to London, and through the introduction of friends obtained an appointment in the office of the late Mr. I. K. Brunel, V.P. Inst. C.E. In the year 1844 he was sent by Mr. Brunel to Ireland on the surveys for the Dublin, Wicklow, and Wexford railway, and in the ensuing year to Wales on the proposed railway from Worcester to Porth-Dyn-Uden, promoted by the Great Western Railway Company. Mr. Brunel's Parliamentary business increasing, he was recalled from this work to take the management of the office. On the death of Mr. Brunel, in 1859, his works were carried on by Mr. R. P. Brereton, M. Inst. C.E., with whom Mr. Fishe remained for five years. From 1864 he was principally occupied as Resident Engineer, first on the Blisworth and Stratford on-Avon line of the East and West Junction railway, and then on the extension of the line from Stratford to the town of Worcester. He also, in the year 1866, prepared the plans of the Teme Valley railway for Parliament, as Joint Engineer with Mr. Burke and Mr. Purchas, M. Inst. C.E. He subsequently gave considerable time to the Duchy of Cornwall office; and on the eve of marriage was nominated to represent the interest of H.R.H. the Prince of Wales, in a case of arbitration with the Crown. His engagements abroad compelled him reluctantly to decline this distinction, and he died at Rome during his wedding tour, on the 3rd of April, 1870, after three days' illness, of fever combined with congestion of the lungs. He had only been elected a Member of the Institution on the 4th of May in the previous year.

Mr. JOHN HARRIS was a native of Cumberland, and was born on the 16th of July, 1812. After completing his term of pupilage with the late Mr. Thomas Storey, Civil and Mining Engineer of St. Helens Auckland, in the county of Durham, he became Engineer to the Stockton and Darlington Railway Company, and was engaged both in the maintenance of the permanent way and works of that line, and in the construction of new works and branches in connection with it. Of the latter the principal were, the Middlesbrough Dock and its coal-shipping staiths and railway approaches; a bridge across the Tees at Stockton, to replace a suspension bridge which had failed to answer its purpose as a railway bridge; the Middlesbrough and Redcar railway, and an extension of the Wear Valley railway from Crook to Waskerley. In the construction of the Middlesbrough Dock and its appurtenances, he was associated with the late Sir W. (then Mr.) Cubitt, Past-President Inst. C.E., as consulting Engineer, and Mr. George Turnbull, M. Inst. C.E., as Resident Engineer, and in the design of the Tees bridge he had the advice of the late Mr. Robert Stephenson, Past-President Inst. C.E. He was one of the earliest to recommend and adopt wooden sleepers for railways in preference to stone blocks, which at that time (1839) were commonly used. In 1844 he became contractor for the maintenance of the permanent way and works of the Stockton and Darlington railway. He also constructed the Wakefield, Pontefract, and Goole railway and its branches, and the Kendal and Windermere railway, which had been designed and commenced by the late Mr. Errington, V.P. Inst. C.E. Besides these he was the contractor for the construction of the Middlesbrough and Guisbrough railway, the Stanley branch of the Stockton and Darlington railway, a large bridge across the river Wear, near Witton, for the Stockton and Darlington Railway Company, a description of which he communicated to the Institution,[1] and for various minor works. The last ten years of his life were passed without professional occupation.

He took an interest in public affairs, was a member of the Board of Health in Darlington, the place of his residence, and frequently attended public meetings. Mr. Harris was elected a Graduate of the Institution on the 14th of April, 1840, and was transferred to the class of Member on the 6th of April, 1841. He was of an open, genial disposition, and was universally respected by those under him.

He died at Kendal on the 20th of July, 1869.

Mr. ROBERT MORRISON was born in the parish of Moy, Inverness-shire, on the 14th of February, 1822. His father, David Morrison, was at that time tenant of the flour or meal mill situated at Kylachy, in the parish of Moy, where Robert, although young, most assiduously assisted him. Indeed, the most striking feature in Robert's character, when a boy, was his devotion to his books and his dutiful attention to his father, whom he helped late and early both before and after school hours; while he prepared his lessons, after the rest of the family had retired to bed, poring for hours over his books, by such light as could be got from the resinous chips from the roots of the fir tree. He resided for some portion of his school days at the house of the Rev. Dr. James MacLauchlan, who allowed him the use of books, which he read with avidity whenever he had spare time either in or out of doors; and Robert Morrison frequently, in after life, alluded in feeling terms to the good precepts instilled into his mind by the minister. When about the age of seventeen he was apprenticed by his father to a millwright in the same county named Reid, who during this time got orders from Sir George Munro, of Poyntsfield, Ross-shire, to supply and fix on that property a flour mill. Being sent to assist at this work, Robert Morrison's intelligent appearance, ac-

[1] *Vide* Minutes of Proceedings, Inst. C.E., vol. i. (1841), p. 138.

tivity, and business-like habits so favourably impressed Sir George, that he was offered the lucrative appointment of factor, or manager, over sugar-growing estates in Jamaica belonging to Lady Munro. His father, however, raised objections to this scheme, and Sir George, still wishing to encourage merit, sent him to Glasgow, with a strong recommendation to the late Mr. John Percy Henderson, of Polmont. Here he worked with various engineering firms, and acquired a practical knowledge of marine and other engines. After a time, having a strong desire to extend his knowledge, he determined to proceed south, going in the first place to Leeds, and subsequently to Manchester, where he got employment with Mr. Fairbairn, M. Inst. C.E., (now Sir William Fairbairn, Bart.).

At this time, 1841, his leisure hours after work were occupied in making sketches of the various parts of engines, which he executed with remarkable neatness. He was noted for his industry, being most scrupulous to his employer's interests, which brought him no small amount of ill-favour from other workmen; for, as chargeman, he insisted that those under him should commence work at the proper time and not wait for the appearance of the foreman; indeed it was partly on account of this feeling that he left Manchester and returned to Glasgow, where he obtained with Mr. Paton, locomotive superintendent of the Edinburgh and Glasgow railway, the situation of draftsman. In 1844 he went to Messrs. Hawthorn, of Newcastle-on-Tyne, as manager of their works. He was much valued and esteemed by the firm, and he had, in such a position, the advantage of getting more prominently under the notice of others who were able to judge and appreciate his merits.

In 1851 he married Miss Fleming, only daughter of Mr. John Fleming, solicitor, of Newcastle; and in 1853 he commenced business on his own account, at Ouseburn, as Robert Morrison and Co. The works at that time were comparatively small, but by his great mechanical skill, application and energy, he, in a few years, extended them so that they covered about 10,000 square yards of ground, and employed more than five hundred men, in the manufacture of marine and other classes of engines, as well as of an improved steam-hammer which he invented and patented. His hammers were extensively used by both the English and Russian Governments, by Sir William Armstrong, for his big guns, and by many large engineering firms in this country, as well as in America, where, by license from the patentee, they were manufactured by Messrs. Wm. Sellers and Co., of Philadelphia. Morrison's steam-hammer was adjudged the first medal and prize at the

Exhibition of 1862. The largest hammer he made was one of 40 tons, in 1863, for the Russian Government; its total weight when completed, in three parts, was 550 tons, and the diameter of the cylinder was 6 feet 6 inches, being, at that time, probably twice the size of any previously manufactured.

Pumping engines of considerable size were occasionally made at the Ouseburn Works, and of these one pair was erected for the Sunderland and South Shields Water Company, each cylinder being 5 feet in diameter. In the year 1866, in common with many others, he felt the effects of the commercial depression which then prevailed, and so severe were his losses that he was obliged nearly to close his works; but he settled handsomely, and beyond their expectations, with those to whom he was indebted. At this time he was also engaged as principal and manager, under the name of Morrison and Co., in developing iron-stone mines at Brotton, in the Cleveland district; and judging that little could be got, for some years at least, by following mechanical engineering, he devoted his time and energy to bringing as speedily as possible into complete working order these mines; and this was successfully accomplished, but only shortly before his death, which took place on the 20th of December, 1869.

He was a devoted husband, an indulgent and affectionate father, a sincere friend, and a dutiful son. For many years he occupied the mansion and grounds known as Shield Field House. He was much interested in the education of the poor, and had been but a short time at Brotton when, through his instrumentality, large and efficient school-houses were built there. Mr. Morrison was elected a Member of the Institution on the 28th of May, 1861.

Mr. GEORGE PADDISON was born at Louth, in Lincolnshire, on the 2nd of November, 1825. He received a general education at the Louth Grammar School, and his education as a Civil Engineer was subsequently completed at the Putney College for Civil Engineers. From his earliest years he was truthful, fearless, and brave. His daring may be imagined from the fact that at the age of thirteen he climbed, outside, to the top of the spire of Louth church, which is nearly 300 feet high. This feat had been but twice before recorded.

From 1846 to 1849 he was employed as an Assistant-Engineer on the York and North Midland lines of railway. In April, 1853, he was appointed Chief Assistant-Engineer to the Valparaiso and Santiago railway; and, for three years, till the temporary suspension of the line, in June, 1857, he occupied the position of Resident Engineer, in charge of one of the most important sections, and by his ability, practical knowledge, and energy contributed largely to the successful realization of a railway which ranks with the most difficult lines in the world. He, however, remained for nine months longer in Chili, during which he was engaged in the completion of a large reservoir and dam for irrigation at Catapilco, about 40 miles from Valparaiso; in the delineation and construction of an irrigation canal near Huasco, in the north of Chili; and in the delineation, survey, and estimates of the Coquimbo railway, with the construction of which he would have been entrusted as Chief Engineer had he remained in Chili. Mr. Paddison then proceeded across the continent to Paraguay, and in June, 1858, was appointed Engineer to the Assuncion and Villa Rica railway by the celebrated dictator, Lopez. He was at first quite alone, pending the arrival of the assistant-engineers; and explored, surveyed, and determined on 45 miles of line in a country of which no maps were to be procured, and he made the final plans for a considerable portion. The actual works were commenced in June, 1858, and the line was opened in September, 1861. Mr. Paddison left Paraguay for England in the following year, not on account of the works being finished, but owing to a disagreement as to the terms of a new contract. In July, 1864, he was engaged by Messrs. Peto, Betts, and Crampton, to proceed, as Second Engineer, to Peru, to survey a line of railway from the Pacific Coast, across the Andes, into Bolivia. The route lay from Tacna, in Peru, to La Paz, south of Lake Titicaca, and thence to Cochabamba. He suffered great hardships on this survey, and after the revolution of January, 1865, returned to Iquique; and as the constant revolutions made arrangements with the Bolivian Government impossible, he went again to Chili. Here he was soon entrusted, as Chief Engineer, with the survey and construction of a line of railway through a desert in the north of the country, from Flojo to Cerro Blanco, between 60 miles and 70 miles long. This work he accomplished with great economy, and much to the satisfaction of the company; its cost being less than £2,000 per mile, locomotives and trucks included. It was finished in December, 1867, and handed over to the directors of the company in January, 1868. On the termination of this engagement he was appointed by the Board of Directors of the Coquimbo railway to examine and report upon the works of the Coquimbo extension, previous to their being handed over to the company. This was a delicate commission, requiring much judgment and knowledge of the subject, as the line had been constructed by the contractor for a lump sum, on his own plans, without supervision, inspection, or intervention on the part of the company. Mr. Paddison, however, earned the gratitude of both the parties concerned, by the satisfactory execution of this commission. In January, 1869, he was appointed, by the Chilian Government, one of the Commissioners to report on the Public Works of the country. About the same time he accepted an engagement with Messrs. William Gibbs and Co., to survey some extensive nitrate-grounds in the south of Bolivia, and to examine into the means to be adopted for the manufacture of nitre, and into the facilities for its shipment. A new company, Milbourne Clark and Co., was established in Bolivia, for dealing in Bolivian produce, but more particularly for the manufacture of nitre. Mr. Paddison was appointed manager, and left Valparaiso in April, for the purpose of erecting the requisite works. He surveyed the harbour of Autofogasta, then called La Chimba, and fixed the locality of the port; erected one pier, and laid down the plans of another; constructed a road from the port to the nitrate-works, and completed a distillery for supplying fresh water. In November he was taken ill from enlargement of the liver; he arrived at Valparaiso on the 12th, but gradually sank, and died on the 24th of that month, leaving a widow and two sons, and a large circle of friends to mourn his loss. He was greatly esteemed as a friend, for his amiability, gentleness of manner, and generous character; while he was much respected as a man of high principle, unflinching courage, good ability, energy, and practical knowledge; and the most unqualified dependence was placed in him by all his employers. Mr. Paddison was elected a Member of the Institution on the 3rd of March, 1863. He highly valued the connection, and bore testimony to the standing it conferred on Civil Engineers in such countries as South America. His almost constant residence abroad prevented his attending the meetings more than once or twice, but he took part in the discussion on the Santiago and Valparaiso railway.[1]

MR. THOMAS PATERSON was born in Edinburgh on the 26th of December, 1830. He was educated chiefly at the High School in that city, and was subsequently a pupil of Mr. John Miller, M.P., M. Inst. C.E. On Mr. Miller's retirement from the profession, Mr. Paterson completed his pupilage with the late Mr. B. Hall Blyth,

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxiii., p. 399.

M. Inst. C.E. He continued with Mr. Blyth and the firm of B. and E. Blyth from 1850 to 1863, for many years acting as their principal assistant, and having charge of important works. He was Resident Engineer on the canal branch of the Great North of Scotland railway in 1853 and 1854, and left, on its completion, to assume the resident engineership of the Carlisle and Silloth Bay railway. In 1863 he was appointed, on the recommendation of the Messrs. Stevenson of Edinburgh, Engineer of Roads, Railways, &c., to the Otago Government, New Zealand, a post which he ably filled for two years, and then began business in Dunedin, the capital of Otago, on his own account, retaining the Government employment. In New Zealand he constructed several considerable bridges and other works, made extensive surveys, and prepared elaborate reports of projected roads and railways. Mr. Paterson's practice soon extended to other provinces, his professional advice being much sought after and relied on. He was employed by the Southland and Canterbury Governments; and when on his way from Dunedin to Timaru, to submit the plans of a bridge over the river Rangitata, one of the largest rivers in Canterbury, he was drowned on the 15th of December, 1869, by the upsetting of the mail coach when fording the river Kakanui while in flood.

He was elected a Member of the Institution on the 10th of April, 1866. His death was looked upon as a national loss in Dunedin, where he had established not only many sincere friendships, but a high character for uprightness, honour, and ability as a professional man.

Mr. WILLIAM ALEXANDER PROVIS was born at Wimpole, in Cambridgeshire, on the 5th of May, 1792. His father, Mr. Henry Provis, was one of the resident Engineers to the Grand Junction Canal Company, and in his office he entered as pupil at a very early age, and continued there until the year 1814, when he accepted an engagement as assistant to the late Mr. Telford. At the commencement of this engagement the improvement of the leading turnpike-roads in the kingdom had become very urgent, and a large portion of this work being put into Mr. Telford's hands rendered the efficient assistance he received from Mr. Provis highly valuable. Amongst the first duties assigned to Mr. Provis was to assist with the designs and drawings for the works of the Caledonian Canal then in progress, and in the survey for improving the mail-coach route between Carlisle and Glasgow: the line selected gave much satisfaction to Mr. Telford, and the improved road is now one of the finest in the kingdom.

In 1817 he assisted Mr. Telford in the examination and survey on which he was then engaged, for the improvement of the road between London and Holyhead, and during the execution of the work acted as Resident Engineer on the most difficult part of the road, that between Shrewsbury and Holyhead. The improvements included the making of large sections of new road, the thorough reconstruction of the old line where made available, and the building of several important bridges, and other works of a minor character. But the most formidable difficulty on this line of road was the necessity of bridging the Menai Strait. "It so happened," says Mr. Telford, in his Autobiography, "that in the year 1814 I had been called upon to consider the best mode of crossing the river Mersey at Runcorn, with a view of shortening the London road to Liverpool; and, under all the circumstances of the case, I recommended a bridge on the suspension principle." He then goes on to mention "several hundred experiments upon malleable iron" which he made on that occasion. In the conduct of these experiments Mr. Provis assisted; and when it was decided to adopt the suspension principle for the bridge over the Menai Strait, to Mr. Provis, as Resident Engineer, was confided the care of the work. In this capacity he laid "the first stone of this great work," on the 10th of August, 1819. In consequence of Telford's overwhelming engagements, the settling of many of the details of the bridge were left to Mr. Provis; and under his advice several alterations were made from the original design. The bridge was opened in 1826; and in 1828 Mr. Provis published an elaborate account of the work, with numerous engravings.[1]

During the progress of the works on the Holyhead road, Mr. Provis also superintended, under the direction of Mr. Telford, the improvement of the line of road from Chester to Bangor. This involved the construction of a bridge over the river Conway, in which the suspension principle was also adopted. Care was taken, in this design, to harmonize the bridge with the old castle of Conway, immediately beneath which the bridge crosses the estuary.

In the year 1825 Mr. Telford was consulted with reference to a project for improving the canal communication between Birmingham and Liverpool; and in the following year an Act was obtained for carrying into execution the scheme devised by him for this

[1] "An Historical and Descriptive Account of the Suspension Bridge constructed over the Menai Strait, in North Wales; with a Brief Notice of Conway Bridge, from the Designs of T. Telford, &c." By W. A. Provis. Folio. Plates. Lond. 1828.

purpose. "The Birmingham and Liverpool Junction canal" (as the line was named) leaves the Staffordshire and Worcestershire canal at Autherley, near Wolverhampton, and joins the Ellesmere and Chester canal near Nantwich. The system included also a branch from this line at Norbury to the Shrewsbury canal in the township of Wappenshall. A line was also laid out from Barbridge, on the Ellesmere and Chester canal, to Middlewich, on the Trent and Mersey navigation. Docks, warehouses, etc., were also to be constructed at Ellesmere Port, where the former canal joins the river Mersey. Mr. Provis was actively employed in the preparation of all these plans, in promoting the passing of the Act, in the preparation of the designs, drawings, and in setting out the lines for their formation. He was afterwards engaged in the execution of the work, which occupied his attention for some years.

In addition to the employments before enumerated, Mr. Provis was from time to time professionally engaged in the improvement of roads in North and South Wales, Cheshire, Derbyshire, and Herefordshire; in laying out lines of mineral tramways and railroads, the improvement of river navigations, drainage works, and other engineering business.

After the death of Mr. Telford, in 1834, Mr. Provis took the house formerly occupied by that gentleman, which continued to be his London residence until the close of his professional life.

About this time he prepared plans for a bridge over Poole harbour, which was erected in the years 1835 and 1836. A swivel bridge was necessary to allow the passage of vessels, and this, in the original design, was placed near the end of the structure, where the ground was sound, and where foundations for abutments of masonry would not have involved a great cost. In consequence of opposition in Parliament, the opening was removed to the centre of the structure, where the depth of water, and unsoundness of the ground, would have greatly increased the expense of the proposed piers of masonry, and a timber structure was therefore substituted for one of masonry with cast-iron arches.

During the passage through Parliament of the Act for the construction of the South Eastern railway, in the Session of 1836, Mr. Provis assisted in framing the estimates. He strongly represented to the directors the expediency of altering portions of the line so as to give a more direct route between the termini; and, although his advice was not taken, the soundness of his judgment has been confirmed by what has since taken place.

Towards the end of the same Session, Mr. Provis was appointed Engineer to a company which had adopted a project for a railway to Brighton, branching from the then authorized South Eastern line. The length to be formed was short and inexpensive, compared with others at that time before Parliament, and Mr. Provis was enabled to give such evidence in its favour as materially to assist in throwing out a line which had made much progress, and for which the promoters appeared likely to succeed in obtaining Parliamentary powers. After a thorough examination of the district in the following summer, he made several amendments on the former line (which formed part of the original South Eastern scheme, and had been laid out previous to his connection with the company), and he added a branch passing the town of Lewes to the harbour of Newhaven. In the line selected by Mr. Provis, the stations for the several towns, but most especially that of Brighton, were placed on more convenient levels than those which have since been adopted. Several competing lines were before Parliament in the following Session of 1837; and in a protracted contest a compromise was entered into, by which portions of the lines laid out by Mr. Provis and by Sir John Rennie were sanctioned; and in the following Session the arrangement was confirmed. Soon after this compromise, Mr. Provis's connection with the railway schemes to Brighton ceased. During the year 1836 he was also employed in laying out a branch railway from the South Eastern line near Tunbridge to Maidstone, which scheme was, however, deferred for a future Session. In the same year he was likewise engaged in a project for completing the railway communication between Edinburgh and Glasgow, by an extension of the Glasgow and Garnkirk railway to a junction with the Union canal near to Falkirk, and the conversion of that canal into a railway.

Mr. Provis's attention had been directed for some time to the improvement of the canal communication between Birmingham and Manchester. With this view, plans were prepared by him, and notices given for an intended application to Parliament in 1838, for a more direct line from the Trent and Mersey canal near Middlewich to the Duke of Bridgewater's canal at Timperly, near Altrincham. The scheme was strongly opposed by existing companies; and as canal projects were not favourably received by the public at that time, the proposed canal was not carried further than preparing and lodging the documents for proceeding with an application to Parliament.

In the great storm of January, 1839, the roadway of the Menai bridge suffered much injury. Its repair and improvement were put into the hands of Mr. Provis.[1]

[1] *Vide* Trans. Inst. C.E., vol. iii., p. 357.

In 1839 Mr. Provis undertook the execution of extensive works at Ellesmere Port, where the Ellesmere and Chester canal joins the tideway of the river Mersey. These works were, in fact, the completion of the original design made out and partially executed by him years before for Mr. Telford. They are admirably adapted for the large transhipments which are there effected between the Mersey and the canal.

In 1845 the proprietors of the Ellesmere and Chester canal, and the Liverpool and Birmingham Junction canal (under the title of the Shropshire Union Railways and Canal Company), entrusted Mr. Provis with the preparation of a scheme for the conversion of such parts of their canals as were suitable into railways; together with the laying out of such supplementary lines as seemed necessary for furnishing the district with a complete system of railway communication. His intimate knowledge of the canals, and of the adjoining country, eminently qualified him for this task. Accordingly, plans were prepared by him for several lines for which Acts of Parliament were obtained in the Session of 1846. In consequence, however, of the monetary crisis which shortly afterwards occurred, the execution of the work was delayed, and eventually an arrangement was entered into with the London and North Western Railway Company, which was subsequently confirmed by Act of Parliament, under which the proposed conversion of the canals was abandoned, and a guarantee was given to the canal companies, securing to them dividends of one half those paid by the London and North Western Railway Company. To this business Mr. Provis gave his unremitting attention for a long period; and the consequence of the anxiety and overwork was a sudden and severe illness, the effects of which induced him to decline any new professional engagements.

Having, however, formerly been consulted by the River Dee Company, as to the improvement of their navigation, he again became their adviser during their protracted disputes with parties who were interested in the navigation of that river, as to the correctness of the standard by which the Company measured the depth of the water they were bound to maintain under their Act. After several inquiries conducted by the Admiralty, the matter was at length settled by an Act of Parliament in 1851, under which an adjustment of the standard was agreed upon. With this contest Mr. Provis closed his professional engagements.

Mr. Provis, being chiefly engaged in carrying out designs and improvements in roads, bridges, and canals, his connection with this class of engineering subjects led to his being frequently employed as an opponent to railway projects in the early stage of those undertakings. This circumstance probably accounts for the small amount of railway work on which he was employed.

During the latter years of his life he spent most of his time on his estate—The Grange, near Ellesmere, which he greatly improved; and so long as his strength was sufficient, he took great pleasure in geological rambles, for he was a good walker, in the course of which he made a large and fine collection of fossils.

By his will he bequeathed a sum of £500 to the Benevolent Fund of the Institution, of which he had been a Member since the 6th of April, 1819. He died on the 29th of September, 1870, at The Grange, in his seventy-ninth year; and on the 5th of the following month he was buried in Kensal Green Cemetery.

Mr. CHARLES SANDERSON was born in Sheffield on the 21st of July, 1824. He was the third son of Mr. Henry Sanderson, Engineering Surveyor to Lord Fitzwilliam. The father was a man of remarkable mental capacity; and that he also possessed considerable professional ability is incidentally evidenced by a letter in which Lord Fitzwilliam recommends the son to the notice of one of the Commissioners for the Metropolitan Drainage. Mr. C. Sanderson had prepared a scheme for the drainage of London; and Lord Fitzwilliam writes:—"If the author of it has inherited the abilities of his father, who was in my employ and died the beginning of this year (1849), I am sure that anything he suggests will, at the least, be worthy of consideration." But the fortunes of the family were variable, and at the early age of sixteen Charles Sanderson left home to push his own way in the world.

He first found employment in the office of Mr. Fuller, of Reading, and subsequently with Mr. Moses Dodd, of the same town. In the latter appointment he, it is believed, projected, and certainly executed, a map of the country 10 miles round Reading. But as this was drawing to a close, in the year 1844, engineering work was offered him on the Great Western railway. To that system, accordingly, he attached himself, and for some years worked, in connection with Mr. Bertram, M. Inst. C.E., under the direction of the late Mr. Brunel, V.P. Inst. C.E., in every department of engineering construction. Several lines were selected and laid out by him in various parts of the country; and in the formation of various Great Western branches and extensions, more particularly the Berks and Hants, the Oxford and Rugby, and the Birmingham and Oxford, every practical detail passed through his hands and was executed to the satisfaction of his chiefs. Mr. Brunel testified to the promptitude, accuracy, and neatness of his work, and Mr. Bertram spoke of the great energy, intelligence, and efficiency in everything committed to his care.

It was while connected in this way with the Great Western railway that Mr. Sanderson was elected an Associate of the Institution, on the 6th of December, 1853. He was transferred to the class of Members on the 9th of April, 1867; and to the day of his death he took great interest in its proceedings.

The time, however, came when Mr. Sanderson felt it important to seek to make an independent position for himself in the engineering world. He had never been attached to the regular staff of the Great Western railway, but simply had a large amount of work, in connection with that company, put into his hands, with other general engineering business, such as the drainage of towns, &c.,—and he foresaw, that in the then position of engineering matters, and especially in the then position of the Great Western Company, this work might fail him. Accordingly, in the year 1857, he became concerned in the promotion of the Stratford-on-Avon railway, a short branch connecting the birthplace of the immortal dramatist with the Great Western railway at Hatton. After many delays and the surmounting of numerous obstacles, that line was completed under his direction, and opened for traffic. For this work he received the highest commendation from Colonel Yolland (the Government Inspector), a voluntary testimonial from Sir Robert Hamilton, of the most generous character, and an official testimonial from the Directors, couched in the handsomest terms.

Now it happened that, in the earlier part of his engineering career, Mr. Sanderson had occasion to refer to the late Mr. Robert Stephenson on some matter of business; and Mr. Stephenson, who had that consideration for younger men in the profession which was such a noble feature in his character, had evidently been favourably impressed; for in the year 1858 Mr. Sanderson received a note intimating that Mr. Stephenson and Mr. Berkley had jointly recommended him "as a fit person to fill an important engineering appointment in India," and fixing the time for an interview. Mr. Sanderson, however, was already committed to the Stratford project and very solicitous to complete it; and the negotiations accordingly, for the time, fell through. But in 1860, when the Stratford railway was opened, an offer was made to him of the Chief Engineership of the Bombay, Baroda and Central India railway, which he accepted in December of that year.

His position on the line was defined by the most explicit "Instructions" from Sir Charles Wood. But difficulties had already arisen in connection with the staff, and a short time sufficed to show that the task he had undertaken was a hopeless one; and in the earlier part of 1862 he resigned the appointment. In March of that year he returned to England to lay the state of things on the railway before the Directors, who voted him a certain sum of money in consideration of the early termination of his arrangements with them. It was a great satisfaction to Mr. Sanderson, in connection with that unfortunate business, that almost the entire body of the staff joined spontaneously in a vote of entire confidence in him as their chief, and forwarded the same to Colonel French, the Chairman of the Board of Directors, on his arrival in India.

In the year 1862 the so-called Cotton Famine was at its height, and manufacturers were casting about in every direction for substitutes for the precious fibre; and one eminent firm in the North of England was especially anxious to try the growth of jute in a congenial clime. Mr. Sanderson was strongly persuaded that it could be grown in the region of the Neilgherry Hills; and, being by this time much attached to Indian life, undertook to conduct the experiment there. Either his new occupation, however, or the climate of the Hills did not suit his health; or a hankering after his old professional life depressed him, for he was compelled, under medical orders, to leave Coonoor; and, an appointment on the Madras railway being offered him, he accepted it, and was once more in his right element. After fulfilling the duties of this position for a short time, the contractor of the north-eastern portion of the line, intending to return to England, made handsome proposals to Mr. Sanderson to become his chief agent in his absence. This position he occupied for some months, when circumstances arising that prevented the return to England of the gentleman whose representative he was to be, he resigned the now unnecessary office, receiving a liberal acknowledgment from his employer for so doing.

After being engaged for brief periods on various other works, Mr. Sanderson, finally, in April, 1868, received, from Mr. Berkley and Mr. Rushton, an appointment as Resident Engineer on the Great Indian Peninsula railway. In the service of that company he continued till his death, which occurred on the 20th January, 1870, at the house of his friend, Mr. Henry Conder, Traffic Manager G. I. P. railway, Bombay. He had never spared himself at work. Possessing as he believed a robust constitution, and being passionately devoted to his profession, he allowed himself altogether insufficient rest and insufficient recreation. He had, moreover, a highly sensitive and scrupulous constitution of mind which took everything to heart, and made things matters of conscience which many are able to regard as trifles. And these things co-operating with an Indian climate had broken him down in what might otherwise have been his early manhood. He received during his brief illness the most devoted attention from a large circle of friends; who also immediately upon his decease set on foot a subscription to erect a monument to his memory; members of his old staff, on the Bombay and Baroda railway, leading the way with their contributions. In private life—a man of the most refined tastes, of the simplest habits, of a most amiable disposition—he was beloved by all who knew him.

Often careless of his own personal interests, in professional matters his conscientiousness and caution were extreme. The interests of his employers he felt to be a solemn trust that he was bound religiously to discharge. Nothing that could be saved would he allow to be expended; nothing that could be suggested to give efficiency to the undertaking would he at any time withhold from them; no departure from the terms of contract would he ever countenance. It is also quite certain that not a single farthing of indirect emolument ever passed into his hands. This severe integrity sometimes caused him to be misunderstood, as was to be expected, by those with whom he had to deal; and occasionally made him enemies for the time; but it won him the respect of all upright men; and his unmistakable generosity and unselfishness generally converted even his temporary adversaries into ultimate admirers and friends. He lived and died in the fear of God, having nobly done his duty in that state of life unto which it had pleased God to call him; and those who were the most nearly related to him cherish his memory with reverent affection.

Mr. WILLIAM WEAVER was born at Beckington, in Somersetshire, in May, 1828; and was the youngest son of Mr. Henry Weaver, of that place, who died in 1829; and grandson of the Rev. Richard Weaver, sole curate of Chippenham. Having been for several years a pupil of Mr. J. Wilson, of Chippenham, he, at the age of fifteen, entered the office of his eldest brother, Mr. Henry Weaver, architect and surveyor, then practising at Chippenham and Southampton. In the latter part of the year 1846 he obtained an appointment in Mr. Brunel's office at Bath, under the immediate direction of Mr. R. J. Ward, M. Inst. C.E., who was engaged on the Wilts, Somerset, and Weymouth railway. He next joined the office of Messrs. Fuller and Gyngell, architects, of Bristol; and subsequently embarked, in conjunction with a Mr. Protheroe, in some coal-works at Lidney, Gloucester, which proved unsuccessful.

In July, 1850, he set sail for Sydney, New South Wales; where, by the aid of his uncle, the Rev. G. E. W. Turner, Colonial Chaplain, he obtained the appointment of Clerk of the Works in the Colonial Architect's office; and in 1854 was elected Colonial Architect for New South Wales, in recognition of a clever plan for a lighthouse on Gabo Island, being appointed over the heads of several senior officers on account of superior ability. He had charge of the roads and bridges of New South Wales at this time; and besides the Gabo Island lighthouse, he constructed the Government printing-office, Sydney, and several timber bridges, at Bathurst, Maitland, and other places. On a change of ministry he resigned his office, on account of interference on the part of certain members of Parliament with the officers under him. He then commenced private practice as an Architect and Engineer, in partnership with Mr. W. E. Kemp; and in the year 1863 was appointed to superintend the construction of a light railway from the Great Western railway at Richmond to Windsor. During this period he constructed the Oriental Bank, School of Arts, and several other public buildings at Sydney and elsewhere in New South Wales.

In April, 1864, Mr. Weaver accepted the appointment of Engineer-in-Chief to the province of Auckland, New Zealand, which province had just effected a loan for £500,000 for carrying out public works. In the following month he made a lengthy report upon the proposed Auckland and Drury railway, in which he pointed out the insufficiency of the available capital for completing the extent of work proposed, and recommended the postponement of the Auckland section of the line, with its heavy tunnel works; but his advice was not taken, the contracts were let, and the entire capital was expended without the completion of a mile of the railway; and when, in November, 1866, the line was put under his sole charge, he was unable to go on with it for lack of funds. Meanwhile, in August, 1864, he was requested to report on a scheme for supplying Auckland with water from the Waitakeri ranges. This he did in April, 1865, but the estimated cost, £82,000, or, omitting the filter-beds, £67,000, being far greater than had been anticipated, the works were not proceeded with. In the same year, however, he secured for Auckland, from the Domain at

Parnell, a supply giving a daily delivery of 40,000 gallons, for a section of the city, at a cost to the province of about £4,000. In November, 1864, he submitted a report and plans for the improvement of Auckland harbour, and for a wharf at Onehunga, on the west coast. These were carried out at a total cost of £57,500. In the same year he also furnished a design for a timber bridge over the Tamaki river. This structure was 576 feet long, with a roadway 21 feet broad; it had an iron swivel bridge at one extremity, resting on a stone abutment, and was carried out at a cost of £17,000. In 1866 Mr. Weaver furnished a detailed estimate, with plans, for a canal which had been proposed for connecting the head waters of the Auckland and Kaipara harbours, and estimated the cost at £60,000, recommending the adoption of a line of railway instead, when funds were available.

In 1867 he took over the telegraphs from the military authorities on behalf of the province, and organized a civilian staff of operators for its future conduct. The sunken coal hulk 'Marion' was also blown up by gunpowder, in the Auckland harbour, under his directions. In addition to these more prominent works, Mr. Weaver had charge of all the roads and bridges in the province; and with a very small staff of officers, viz., himself, an Assistant-Engineer, two inspectors, and two clerks, works were carried out during the four years 1864-67 to the value of £230,000, at a cost to the province for supervision of three per cent. only. In June, 1867, in consequence of the financial difficulties of the province, his department was broken up, but his services were retained at a reduced salary until the end of the year, and he was allowed private practice. Meanwhile, he was made district manager of the Colonial Telegraph Department, and became agent to the Royal Insurance Company's branch at Auckland. Later in the year, he was elected to the Provincial Council, as member for part of Auckland, but resigned his seat on accepting the appointment of Telegraph Engineer to the General Government at Wellington, in February, 1868. This he held for but a very short time, and after a few months left for Australia. He died at Geelong, in Victoria, rather suddenly, from effusion of blood on the brain, and in indifferent circumstances, in December, 1868, leaving a widow and family behind him.

Mr. Weaver was elected an Associate of the Institution on the 21st of January, 1851, and was transferred to the class of Members on the 4th of February, 1868. He possessed considerable talents, quick perception, and ready adaptation to circumstances, with no mean share of artistic skill. He was most kind-hearted and liberal, but a bad financier, and it was said of him that the worst friend he had was himself.

Mr. NICHOLAS WOOD[1] was born at Sourmires, in the parish of Ryton, on the south side of the river Tyne, on the 24th of April, 1795. Educated at the village school at Crawcrook, by Mr. Craigie, who had the reputation—a rare one at the time—of turning out lads clever at figures and well-grounded in the most useful branches of an ordinary English education, young Nicholas Wood proved himself to be a ready scholar, and soon did credit to his master. In April, 1811, Sir Thomas Liddell (afterwards Lord Ravensworth), having taken a fancy to the lad, sent him to Killingworth colliery to learn the business of a viewer. Here it was that Nicholas Wood made the acquaintance of George Stephenson, whose skill and ingenuity had already led to his being advanced from the position of a brakesman to that of an engineer, and, in 1812, to that of colliery engine-wright at the Killingworth High Pit; whose friend and confidant Mr. Wood immediately became, assisting in the construction of the "Geordie" safety lamp, and, it is said, being one of those who witnessed the testing of the lamp at a "blower" in Killingworth colliery. On the 15th of November, 1815, Mr. Wood explained the merits and details of the invention before the members of the Newcastle Literary and Philosophical Society, and he took a prominent part in the controversy with the advocates of Sir Humphry Davy's lamp. Mr. Wood also assisted in the early experiments in connection with the locomotive engine, maintained in the columns of the "Newcastle Magazine" for 1822 that it could be profitably employed as a tractive power, and in 1823 accompanied Stephenson to Darlington, when it was determined to proceed with the Stockton and Darlington railway. In 1825 appeared his celebrated "Treatise on Railroads," which has gone through several editions, and which materially assisted in the early development of the railway system. In the same year, as well as in subsequent sessions, until the Act was obtained, he gave evidence before Committees of both Houses of Parliament on the Liverpool and Manchester railroad bill. By this time Mr. Wood had acquired considerable fame as an engineer, but he retained his preference for the mining interest, and had already

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[1] In the "Transactions of the North of England Institute of Mining Engineers," vol. xv., there is a lengthy memoir of the late Mr. Nicholas Wood, by Mr. Doubleday.

entered into colliery speculations on his own account, and was rapidly extending his influence and position in the coal trade. When the British Association for the Advancement of Science met at Newcastle in 1838 he read an elaborate essay on the geology of Northumberland before the Geological Section, in which he endeavoured to show the probable identity of the red sandstone formations of the valley of the Tweed and those of the Cumberland plains. In 1844 he removed from Killingworth to Hetton, and assumed the management of the collieries belonging to the Hetton Coal Company, in which he was a partner. He took a prominent and active part in the investigations which ultimately led to the Mines Inspection Bill, which was passed in 1851; and in the organization of a society, proposed to be called "The North of England Society for the prevention of Accidents, and for other purposes connected with Mining," which was established at Newcastle on the 3rd of July, 1852. Almost immediately afterwards the name was changed to that of "The North of England Institute of Mining Engineers." Mr. Wood was elected the first President, and on the third of September in that year he delivered an inaugural address in which he defined the objects of the Institution to be—"First, by a union or concentration of professional experience, to endeavour, if possible, to devise measures which may avert or alleviate those dreadful calamities which have so frequently produced such destruction to life and property, and which are always attended with such misery and distress to the mining population of the district; and, secondly, to establish a literary institution more particularly applicable to the theory, art, and practice of mining than the institutes in the locality present, or which are within the reach of the profession in this locality." He retained the office of President to the day of his death, and he devoted all his influence, talent, and much of his time to promote its success, being a frequent contributor of essays on mining subjects. In 1855 the idea of a mining college for the cultivation, improvement, and teaching of that science, especially coal mining, was mooted in the North of England, and Mr. Wood, in conjunction with the late Mr. T. J. Taylor, took a prominent part in promoting the undertaking; but notwithstanding the support of the late Algernon, Duke of Northumberland, the project fell through; nor was a subsequent attempt, made under the same auspices, to induce the University of Durham to add mining science to their course of studies more successful. Mr. Wood appeared for the last time as an author on the occasion of the British Association for the Advancement of Science visiting Newcastle for the second time in 1863, when in conjunction with Mr. T. J. Taylor, Mr. I. L. Bell, Dr. Richardson, and others, he presented a Paper on the various industrial pursuits of the northern counties. Soon after his health failed and prevented him taking an active part in business, and he died in London, whither he had resorted for medical advice, on the 19th of December, 1865. Mr. Wood was of commanding height, portly form, and had a ruddy, good-humoured countenance, which bore no traces of the hard work he got through. He was an old Member of the Institution, having been elected on the 12th of May, 1829. Mr. Wood married Miss Lindsay of Alnwick, whom he survived some years, and by whom he left four sons and three daughters.

Mr. WILLIAM THOMAS BLACKLOCK, son of John Blacklock, calico-printer, of Kersal, was born in July, 1815. At the age of fourteen he was apprenticed to the late Mr. George Bradshaw, of Manchester, engraver and letterpress-printer, to learn the art of engraving. Before he had completed his apprenticeship he was offered a share in the business by Mr. Bradshaw, and from that period, about forty years ago, the well-known firm of Bradshaw and Blacklock dates its existence, Taking advantage of the opportunity offered by the introduction and extension of the railway system, they laid themselves out for and secured so much of the work required by the railway companies as to become popularly known as the railway printers. The merits of their "Railway Guide and Shareholder's Manual" are so well known to the members of the engineering profession, that no mention need here be made of the labour and exactitude with which it has always been prepared. In 1850 Mr. Blacklock was elected a director of the East Lancashire Railway Company, and in 1859, on the amalgamation of that company with the Lancashire and Yorkshire, a director of the latter, at the board of which he held a seat until his death. Upon assuming the responsibilities connected with an efficient discharge of his duties as a railway director, he retired from the firm in which he was the active partner, but soon after, finding that his energy demanded further occupation, he entered into partnership with Mr. George M'Corquodale, of Newton-le-Willows. He was also a county magistrate, a borough magistrate, a commissioner of taxes, treasurer to the Manchester and Salford branch of the British and Foreign Bible Society, treasurer to the Religious Tract Society, trustee of several churches, savings' banks, &c., and energetically exerted himself in the advancement of the education and social condition of the working classes. His death was very sudden, he being seized with apoplexy on the occasion of his youngest daughter's marriage, on the 29th of June, 1870. Mr. Blacklock was twice married, on the second occasion to Miss Lord, of Farnworth, by whom he left two sons and two daughters. He was elected an Associate of the Institution on the 7th of April, 1868.

Mr. LIDDLE ELLIOT was born on the 16th of October, 1807, at Trentham in Staffordshire. He was brought up as a land surveyor and articled to a Mr. Slater, who had been sent by Mr. Telford to improve the turnpike roads of North Staffordshire. On the death of that gentleman, Mr. Elliot succeeded him in this occupation and carried out many new roads besides. He was also engaged by the late Mr. Sneyd of Keele Hall in improving the roads on his estates; and gave assistance to several engineers in selecting and surveying lines for railways in Staffordshire. About the year 1845 the attention of the leading manufacturers in the Staffordshire potteries was drawn to the very inadequate supply of water, chiefly owing to mining operations draining the springs. Mr. Elliot was employed in conjunction with the late Mr. James Simpson, Past President Inst. C.E., to survey the country, and succeeded in finding an abundant supply of pure spring water at Wall Grange, near Leek, on the estate of the Duke of Sutherland, whose confidence he largely enjoyed. A company was formed, Mr. Elliot was appointed engineer, and successfully carried out the works for conveying the water a distance of 10 miles, to supply a population of about a hundred thousand inhabitants; and the Staffordshire potteries now enjoy an excellent supply of spring water. He joined the Institution as an Associate on the 3rd of December, 1850, and died on the 1st of March, 1869, at Newcastle-under-Lyne, of which place he was mayor in 1846, greatly regretted by a large circle of friends, and leaving behind him a numerous family.

Mr. ALISTER FRASER was a descendant of an old Inverness-shire family, and was born at Culduthel House, in that county, on the 6th of September, 1829. He was educated, first, at a private school at Elgin, then at the Inverness Royal Academy, and finally at King's College, Aberdeen. In the year 1846 he entered the office of Mr. J. Abernethy, M. Inst. C.E., who was then engaged in the construction of the Aberdeen harbour works, and on the completion of his apprenticeship, in 1849, obtained the appointment of Assistant-Engineer, under Mr. Abernethy, to the Swansea docks. In November, 1853, he entered the office of Mr. Abernethy in London, and in July of the following year he took charge of a section of the works of the South-Eastern railway of Switzerland for the contractor, Mr. Edward Pickering. He remained in Switzerland till January, 1857, and in September of the same year he was appointed to the staff of the Madras railway, and took charge of a district containing some difficult and important works, amongst which the chief one was a large bridge over the river Thoota. He left India on the expiration of his engagement, in July, 1861, the Madras Railway Company at that time declining to renew engagements, owing to a check in the influx of the requisite funds for carrying on works.

In May, 1863, he was engaged by Messrs. William and John Pickering to accompany Mr. Samuel, M. Inst. C.E., the Engineer of the Nicaragua Canal Company, over the proposed line of navigation, with a view to advising them in tendering for the execution of the works. He completed this commission in September of that year. In May, 1864, he was appointed by Messrs. Smith, Knight, and Co., to explore the proposed line of railway from Ismid, by Angora, to Sivas, in Asia Minor. He made rough surveys and estimates of the section from Ismid to Angora, but the project was abandoned, and he returned to England after an absence of four months. In September, 1866, he was appointed Chief Resident-Engineer of the Mexican railway—then the Imperial Mexican—of which Mr. Samuel was the Consulting Engineer. This appointment he held till the death of the Emperor Maximilian, in 1867, when he returned to England, passing through the United States and Canada. In January, 1869, he resumed his appointment in Mexico, but held it for a very short time, returning to England in ill health on the 20th of June, and on the same day in the following year he died at Edinburgh.

Mr. Fraser had been an Associate of the Institution from the 1st of December, 1863.

Mr. WILLIAM GAMMON, the son of Mr. Edwin Benjamin Gammon, was born on the 29th of November, 1841. He was educated at Hockley, Laleham, and Peckham; and in 1859 was apprenticed for three years to Mr. John Strapp, M. Inst. C.E., having previously, for nine months, been in business with his father. On the expiration of his indentures he was employed for one year as an assistant-engineer on the South Western railway. At the age of twenty-one he entered into partnership with his father, who was engaged in the execution of different railway and other contracts, such as building the locomotive and carriage sheds for the South Western railway at Nine Elms. The firm also carried out extensive works at Gillingham, for Government; and one of the contracts for the docks and sea-walls at that place was intrusted to them. He died on the 28th of September, 1870, from the effects of rheumatic fever, having been elected an Associate of the Institution on the 3rd of March, 1868.

Major-General Sir JOHN WILLIAM GORDON, K.C.B., was the eldest son of Colonel Thomas Gordon, of Harperfield, in Lanarkshire. This estate came to him while he was still young, at his father's death; and through his mother, Miss Nisbet, of Carfin, in the same county, niece of Andrew, last Earl of Hyndford, he not long after inherited Carfin and Maudslie Castle, formerly part of the Hyndford property. He was therefore born to such good prospects as would have indisposed most young men to steady exertion; but of his own choice he entered a hard-working profession, to labour thenceforward as though dependent wholly on it. His ample means he throughout life treated as a steward for others rather than an owner. From a private school at Bexley, in Kent, he passed the entrance examination—not very difficult in those days of nomination—into Woolwich Academy. During his cadet life he was remarkable chiefly for his physical powers, his carelessness of danger, and his steady application to work. To the latter almost entirely—for young Gordon was not gifted by nature with quickness of parts—he owed the prize he worked for, a commission in the Royal Engineers. The times were those of profound peace. In no part of the army did mere soldiership promise any special advantage, and perhaps least of all in the Engineers, whose war duties were almost ignored. Gordon passed from his first home station to North America, undistinguished from other subalterns; for the simple habits of life which were to him as a nature, prevented his being even known generally to be more wealthy than his fellows. He left Halifax after a long term of duty there, much regretted by a few friends who had discovered the sterling worth which was concealed by a reserved exterior, and learnt something of the kind deeds which he had already begun to practise the doing of in secret. But to the many he was known chiefly by his great height and by the endurance and activity which he displayed in the moose-hunts for which Nova Scotia was then noted, or for his avowed adherence to earnest, and to those not conversant with Scotch Presbyterianism, what seemed gloomy religious convictions. Promotion was of course in those days very slow in a seniority corps, and Gordon looked a middle-aged man when, in 1845, he was promoted, after sixteen years' service, to the rank of captain, and sent to Chatham to take charge of the 1st Company of Engineers, or Sappers and Miners as they were then called.

A neglected cold at this time brought out a predisposition to chest disease, and to those about him seemed to threaten his life; but happily his Company was under orders for Bermuda, and the change to that mild climate soon restored him to his natural vigour and the out-of-door habits in which he always delighted, though never allowing them to interfere with the duties of the desk. During the next five years he was constantly employed on the large works which were to create a Gibraltar of the West out of the sandhills that ages have solidified into Bermuda stone. His spare time, of which he allowed himself but little, was devoted wholly to manly exercises and to the good works which formed part of his daily life. Among these was a night-school kept by himself for the instruction of his men, and which he never allowed any engagement to interfere with. Frugal and temperate in his own habits, his ready hospitality was known to every passer-by who visited his station. Sparing in expenditure on himself, his liberality towards the poor near him, or in cases made known from any distance, was exhaustless. He not only gave, as a matter of course, to those that asked, if they deserved it, but his delight was to send help to those who deserved it and had not asked. The venerable bishop of the diocese has lately revealed the fact that Gordon maintained the private charities which he began at Bermuda for many years after he had left the island, and that his name is still familiar there among those who have heard it blessed by the aged and infirm whose special wants he had carefully ministered to. Meanwhile no case of distress or difficulty in his own corps, however far from him, but received instant attention when brought to his knowledge.

But it was not for his large-hearted charities that he became as well known at this time as for his marvellous physical powers and endurance. His theory was, that a soldier, to do his duty properly to his country, must keep his body in the highest perfection of its powers. Acting stringently up to this idea, he lived constantly, except in his exceeding temperance of diet, in such a state of regular training as few men ever reach even for a special purpose and a brief time. His work never slackened anywhere in consequence of this. It was confessed that no one ever saw so much labour got out of large working parties of soldiers or of convicts as Gordon obtained, and that without a harsh word. No office detail, however petty, was below his attention. A favourite fancy of his was the preparing of working drawings, which he might well have left to his subalterns but for his passion for labour; and after returning from a run of 12 miles, done within two hours, he would go straight to his high desk, without a moment's intermission, and fall to work with a steady hand in the standing attitude which he always used.

He returned to England about the time of the Great Exhibition of 1851, which was designed to usher in an era of universal peace. His reputation for strength and fearlessness and honesty of purpose went before him; but some of his comrades laughed at his theory of being ready for the active service which in their time could never come. Two years afterwards the nation was rushing into the Crimean war, and no department which had the choice would have overlooked such a born warrior and practical engineer as the subject of this memoir. Gordon was at once put under orders for the Crimea, being then a captain of some standing, and fifth in seniority of the Royal Engineers selected for service in the East. When the siege of Sebastopol was a month old, casualties had made of the captain the Commanding Royal Engineer of the army, and honours and rank were coming thick upon him. Gordon carried on his duties under the superintendence of Sir John Burgoyne, who had come out as adviser to Lord Raglan; and he acted afterwards as second to Sir Harry Jones, when Government sent that officer to take Sir John's place. To write the story of the duties of Gordon of Gordon's battery, and how they were performed, would be to write the history of the siege. His long-practised endurance now enabled him to do what no other man could in the way of personal attendance to the works in progress; and during one bombardment it is reported of him that he never slept nor sat down to take a meal for three days and three nights. His valour was not so much mere courage as a perfect indifference to danger, which became a proverb in the lines. "How do you manage to keep so cool under this fire?" said to him a regimental colonel noted for his gallantry, and beloved by his battalion, though his language was notoriously violent and coarse under excitement. "Colonel Y.," was the answer, given with much deliberation, "I am so cool because I read my Bible; you don't read yours." His gallant regiment buried Colonel Y. three months later, after the first assault on the Redan, but it is said that no man in it during that three months had heard an oath pass his lips. This is but one instance given from direct witness of a hundred that might be offered of the marvellous influence Gordon's pure life had on others. A severe wound received in the great March sortie, and much neglected afterwards, broke down his health just before the siege closed, and he was absent when the stronghold was surrendered, which, more than any other single man, he had contributed to make our prize. In the following year, being still regimentally a captain of Engineers, but by brevet a full colonel and A.D.C. to the Queen, he was called suddenly from a holiday in Scotland to become practically the military head of his corps as Deputy Adjutant-General. "It is a splendid appointment," he said, "but one I would rather not have, for the principal duty lies in refusing different men different things they want." With this somewhat morbid view of what discipline should be, it is not surprising that he was not as popular at the Horse Guards as his friends could have desired to see him; but his translation to the important charge of the great fortifications of Portsmouth, the largest military Engineer's command then in the world, which happened not long after, gave his zeal and energy and his natural kindliness better scope. His Sunday-evening entertainments, open to all his command weekly without special invitation, drew his young officers together once more as they had another generation of young officers fifteen years before, the survivors of whom warmly own the valuable influence these genial meetings had on them. With the design of the works of the Portsmouth district Sir W. Gordon (who received his knighthood while employed there) was not concerned. His duty was merely executive, and as an executive officer it may be fairly declared that he has never been surpassed. His command there was broken by a temporary call to Canada at the time of the 'Trent' affair; but the alarm over, he returned once more to the charge of the great works round Spithead, of the execution of which his old opponent, Todleben, after being escorted by him round them, publicly expressed his unalloyed admiration.

When Deputy Adjutant-General of Engineers he had become an Associate of the Institution, on the 3rd of February, 1857, and was a constant attendant at its meetings; but with his usual extremely retired manner shrank from taking any more active part than listening. In January, 1870, however, he rose to express his thanks for the mention made of the Royal Engineers in the President's Address, which pointedly alluded to himself, and he did so in a speech full of manly feeling and of sensible acknowledgment of what the education of the Royal Engineers owes to the civil branch of the profession, "their intercourse with which, he hoped, might, on all occasions, be as close and friendly as heretofore."[1] He had then not long been appointed by popular wish, as it were, no less than by royal choice, to the revived office of Inspector-General of Fortifications, which his friends thought to see him fill with the same dignity with which he spoke that night. Alas! disease, produced by the irritation of his severe Crimean wounds, acting on the nervous system, was even then preying on his brain. Traces of aberration of mind had been observed some time before by watchful and anxious friends, and a few weeks later he passed from among us by the saddest end a gallant soldier could know. In strength a giant, in modesty a maiden, in humility a child, so pure and noble a life never came to a more painful close, when, his mind losing its self-control, he suddenly laid violent hands on his own life, and died on the 8th of February, 1870, being at the time sixty-five years of age.

Left by his parents at the age of twenty-one the care of a younger brother and sister, he had discharged his difficult duties as though he had been the most loving and thoughtful of fathers. Of his practical benevolence let this one trait suffice. When defrauded by an agent he had implicitly trusted of several thousand pounds, he insisted on charging his own want of supervision as the chief fault in the temptation it had offered, and refused therefore to prosecute the offender. More than this, when he found the wretched man afterwards starving (who had robbed his employer only to fall into deserved penury), he ministered to the needs of the only living being who had ever done him serious harm. The sudden loss of such a hero may well have cast a gloom over the service which was proud of him, even had the circumstances been less painful; while to his personal friends, their bereavement would have been bitter in any case.

MR. HENRY HAKEWILL was born in Brook Street, Grosvenor Square, on the 11th of April, 1842, and was descended from a family of respectability in the West of England. He received his education at Goodenough House, Ealing, and showing at an early age a decided turn for mechanical pursuits, he was allowed to follow the bent of his inclination in choosing the profession of a Civil Engineer.

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxix., p. 319.

At the age of sixteen he was articled to Mr. Thomas Page, M. Inst. C.E., and was engaged on the works of Westminster bridge, after which he superintended the erection of a bridge at Brafferton, Yorkshire, designed by Mr. Page. From thence he went to Horsens, Denmark, in the employment of Messrs. Brassey and Betts, to assist in the construction of a railway between that place and Viele, which occupied his time for three years, and he was afterwards engaged on the line between Alborg and Randers till the completion of the works. He returned to England in November, 1869, where he remained until the following April, having in the mean time been elected an Associate of the Institution on the 1st of February, 1870, when he accepted an engagement with Messrs. Waring Brothers to proceed to Thorda, in Hungary, to commence a line of railway. While thus engaged he was attacked with inflammation of the lungs, which threw him on a bed of sickness for six weeks. After much suffering he reached London with difficulty, and in spite of the best medical skill he sank rapidly and died on the 9th of October, 1870, in the twenty-ninth year of his age, beloved and deeply regretted by a large circle of relations and friends.

Mr. CONRAD ABBEN HANSON, having been educated at a private school, commenced his professional career about 1837, under Mr. William Budden, secretary to Mr. Robert Stephenson, in his office on the London and Birmingham railway, and at Weedon; he then became secretary to Mr. G. W. Buck, at Manchester, on the Manchester and Birmingham railway. After that line was finished, in 1843, he entered the service of Messrs. Bramah, Fox and Co. (subsequently Messrs. Fox, Henderson and Co.) at Smethwick, near Birmingham, and soon became head of the estimating department. He remained with that firm till after the completion of the Exhibition of 1851, to the opening of which at the stipulated time his untiring energy and perseverance contributed.

After leaving Birmingham he was connected with Price's Patent Candle Company, and superintended much of the mechanical work executed on the premises. In 1858 Mr. Hanson became secretary to Bray's Traction Engine Company, and in 1860 he entered the service of Messrs. Waring Brothers as manager in the estimating department, where he had active occupation in practical matters, as well as in estimating, connected with the Pernambuco railway, the North London Extension railway, St. Pancras Railway Station, Hungarian railways, &c. Mr. Hanson was elected an Associate of the Institution on the 4th of February, 1862. He died on the

30th of December, 1869, aged fifty-two, greatly respected by all who knew him, and leaving a widow and six children.

Mr. JOHN WILLIAM HEINKE was born in London in 1816. His father, who was of Polish extraction, came to England after the conquest of Poland, and commenced business as a coppersmith, in which he was assisted by his son as soon as he was old enough to enter business. About 1845, in conjunction with his brother and father, he turned his attention to submarine engineering. Diving apparatus was at that time little understood, and not much used, but he saw that, properly applied, it would prove a valuable addition to an engineer's plant. The apparatus then in vogue was of very crude construction, and accidents were of frequent occurrence, so that there was a natural prejudice against the use of these machines, which had to be overcome. By the introduction of a valve, and other improvements in the helmet, dress, and air-pump, the diver was enabled to work under water with perfect ease and safety. It was, however, some time before he was able to command a sale for the apparatus; but by dint of great perseverance he established the business, and the apparatus obtained a first class prize medal at the International Exhibition of 1851.

In 1855 he made some trials with the apparatus at Paris, and again obtained a first class prize medal at the Exhibition. Here he had an interview with Prince Napoleon, who presided at a trial competition between three English and two French diving machines, when Heinke's was unanimously pronounced the best, both by the judges and by the competitors themselves, which led to a large business with France.

Soon after the Crimean War was terminated, he went to St. Petersburg, to arrange with the Russian Government for the removal by divers of the vessels sunk at the mouth of the Harbour of Sebastopol during hostilities. Mr. Heinke saw the Grand Duke Constantine on the subject, and having made the necessary arrangements, the vessels were blown up, and all impediments to navigation removed.

In 1858 he went out for Lloyd's to the Zuyder Zee, off Terschelling, to report upon the wreck of the 'Lutine' frigate, which was wrecked about seventy years ago, with between £1,000,000 and £2,000,000 in gold on board. Here he very nearly lost his life, whilst out in a small boat during a storm. In his opinion the treasure was not recoverable, as although the top of the ribs of the vessel were just visible above the sand, the body was embedded in a quicksand, which would have rendered it a matter of extreme difficulty to get at the hold, even had the diver been able to work uninterruptedly, which at the time was rendered impossible by the rough weather. On this report, the idea was abandoned; and a Dutch company, which was formed about the same time, with a similar object, also gave it up. On returning to England, he continued the business of a submarine engineer—a branch of the profession that was daily growing in importance. One curious instance of the use of the diving apparatus was given by him some time after this. When the famous watch robbery took place at the Messrs. Walker's, in Cornhill, a woman, who was known to be concerned in the burglary, being hotly pursued by the police, threw a number of watches over Blackfriars bridge into the Thames. The police consulted Mr. Heinke as to the possibility of their recovery, and he succeeded in obtaining ten of them. He also executed many other works and contracts by means of the diving apparatus, previously considered impossible.

About the end of the year 1868, he was attacked by congestion of the liver, and never rallied, the disease being augmented by great mental anxiety.

One of his last works was the sinking of a brickwork cylinder at Battisfield colliery, near Chester; but the work did not go on satisfactorily, as he was prevented by ill-health from attending to it personally.

Mr. Heinke was elected an Associate of the Institution on the 2nd of December, 1856; having in the previous March contributed a Paper "On Improvements in Diving Dresses and other Apparatus for working under Water,"[1] for which he received a Council premium in books. Subsequently, on one occasion, he spoke in reference to the same subject, as carried out at the Dover breakwater. It was often said by him that submarine engineering was quite in its infancy. He died on the 12th of April, 1870, aged fifty-four years, deeply regretted by all who knew him. He was much esteemed for his integrity, kindness, and benevolence.

Mr. EDWARD HOOPER was born on the 18th of November, 1822, at No. 1 South Place, Finsbury, where his father, Mr. John Hooper, was a prominent medical practitioner. His constitution was never robust, and he was subjected to no course of study beyond that of a classical education. This did not develop

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xv. p. 309.

to his friends an innate taste for mechanical pursuits of which he was himself sensible, and which gratified itself in such occupations as drawing steam-engines and making models of ships elaborately finished and rigged. Consequently his introduction to business commenced in the country with a miller, who combined with this the timber and coal business, and he was about nineteen years of age when he was allowed to enter upon the career of his choice. He was then articled for three years to the late Mr. Benjamin Cubitt, M. Inst. C.E., at that time local superintendent of the Brighton, Croydon, and Dover railway, and was chiefly employed in the mechanical department at the joint station, New Cross. He also took a prominent part for one in his then position in the experiment of the atmospheric railway from Croydon to London; subsequently he was engaged under other engineers in the making of the Jemappe and Louvain railway, on the Eastern Counties line, and on surveys in Wales and other places. He was thus engaged when the death of Mr. Cubitt in January, 1848, and the railway panic about the same time, combined to unsettle him in the pursuit of railway engineering. His father died in the autumn of this year, and in the following year he paid a visit to America. On his return the removal of the duty on bricks caused him to give his attention to that manufacture. The result was his establishment of brick-works at Exbury, 17 miles from Southampton, and his residence in that town, he at the same time pursuing his profession as an architect. This business he worked successfully till 1866, when it was superseded by the occupation of the Portland cement works.

The disease of which he died on July 2nd, 1869, had been slowly developing itself for two or three years, but during that time the influence of his life and conversation was also gaining upon his fellow-townsmen and friends, and he died much respected and beloved. The most marked feature in his character was submission to the Divine will, which after a life of unusual trial shone out to its close.

Mr. Hooper was elected an Associate of the Institution on the 25th of June, 1844; but, from living in the country, was unable to attend the meetings and take part in the discussions.

Mr. GEORGE HOUGHTON was born in London in the year 1841. He entered King's College in January, 1858, and remained in the Department of Applied Sciences until December, 1860, when he obtained an associateship of the college. From an early period he showed a decided predilection for mechanical and engineering pursuits. He was accordingly articled to Mr. G. B. Bruce, M. Inst. C.E., and entered upon his professional duties with such ardour that before the expiration of his pupilage he occupied the position of Assistant Engineer on the Tilsit-Insterburg railway, in East Prussia, where he was in charge of a length of 33 English miles, and subsequently he became Resident Engineer on the same work for eighteen months. After this he was appointed Resident Engineer of a length of 46 miles of the Berlin-Gorlitzer railway, between Gorlitz and Spremberg, which appointment he held for a year and a half. He was elected an Associate of the Institution on the 5th of March, 1867. In the following year he went to Hungary and acted as Secretary to the management, for the execution of the works of the Groswardein and East Hungarian railways, taking also as a portion of his work the control of the drawing office. In these capacities his general professional knowledge as well as his special experience in the construction of German railways was constantly exercised. In 1869 he was appointed first class Assistant Engineer on the Great Southern of India railway; but he had not been in India more than six months when he was attacked with cholera near Palamcottah, in the Madras Presidency, and after two days' illness died on the 24th of June, 1870.

Brevet Lieutenant-Colonel JULIAN St. JOHN HOVENDEN was born at Florence on the 24th of June, 1831. He was educated privately for several years, and afterwards at Brighton College. He joined the East India Company's Military Seminary at Addiscombe in January, 1849; and obtained his commission in the Engineers at the public examination in December, 1850, being gazetted a lieutenant on the 9th of that month.

After the usual course of studies at the Royal Engineer Establishment at Chatham, Lieutenant Hovenden proceeded to Bengal in the early part of 1853. At that time the Burmese war was in progress, and he was detained in Calcutta in readiness for service with the army in Burmah, in case additional Engineer officers should be required, being in the meanwhile attached to the office of the Garrison Engineer of Fort William.

In November, 1853, Lieutenant Hovenden was appointed an Assistant Engineer in the Public Works Department, and was posted to the Peshawur Division. He remained in this appointment until the outbreak of the Indian mutiny in May, 1857, when his services were required with the army. He was at first appointed Deputy Assistant Quartermaster-General of the Peshawur Division, and took an active part in disarming the disaffected and mutinous native regiments in garrison there and in the neighbouring forts; and he subsequently joined the besieging force before Delhi as a Field Engineer. At the assault of Delhi on the 14th of September he was in charge of a ladder party and was severely wounded. Colonel Baird Smith, commanding the Engineer Brigade before Delhi, in his official report to General Sir A. Wilson says: "Lieutenant Hovenden (severely wounded) led the ladder party of the Second Division with the same gallantry and intelligence which throughout the siege had made his services of so much value." Lieutenant Hovenden was then employed with a regiment of Sikh pioneers, which he for some time commanded. He served at the siege and capture of Lucknow (on which occasion he was again mentioned in despatches), and he was with the Commander-in-Chief's force in Rohilkund. In August, 1858, Lieutenant Hovenden was promoted to the rank of captain, and then immediately received the brevet rank of major for his services before Delhi, &c.

On the conclusion of active military operations, Major Hovenden rejoined the Public Works Department, and was posted to the Benares Division as Executive Engineer, and subsequently on promotion he was transferred in a similar capacity to Gwalior, his principal duty in both places being the rapid erection of temporary barracks for European troops. In 1862 Major Hovenden was appointed Deputy Consulting Engineer and Assistant Secretary to the Government of Bengal in the railway department, which appointment he held until May, 1867, when he visited England on leave of absence for six months. On his return to India in November, 1867, he was appointed Consulting Engineer and Joint Secretary to the Chief Commissioner of Oudh in the railway department, a new appointment formed when the Oudh and Rohilkund Railway Company were about to commence operations in those provinces. The duties of the officers of the Government railway department in India consist in exercising the Government control and supervision over the operations of the several railway companies whose capital is guaranteed by the State. In March, 1868, Major Hovenden was appointed to officiate as Consulting Engineer and Joint Secretary to the Government of Bengal in the railway department—the highest appointment in that branch of the public service in India. He held that appointment for nearly two years, and at the end of 1869 he left India on furlough to England, having been promoted to the rank of Lieutenant-Colonel in 1868. Under medical advice he spent the winter and early spring of 1870 in Italy and the South of France. He arrived in England on the 2nd of May, and died at Bath, of typhoid fever, on the 16th of the same month, at the early age of thirty-nine, sincerely regretted by his brother officers, and, indeed, by all who had known him, as he, in an eminent degree, obtained the esteem and affection of all with whom he was thrown in contact.

Lieutenant-Colonel Hovenden was elected an Associate of the Institution of Civil Engineers on the 5th of December, 1865.

Mr. JOHN MEESON PARSONS was the youngest son of Thomas Parsons, of Newport, Salop, where he was born on the 27th of October, 1798. He was educated first by the Rev. Richard Thurstfield, of Pattingham; secondly, by the Rev. Francis Blick, of Tamworth; and was for a short time in residence at the University of Oxford; but from too hard reading was seized with a violent inflammation of the eyes, which obliged him to give up all study. He then settled in London, and after some time became a member of the Stock Exchange. He very early in his London career took an interest in railways, was elected an Associate of the Institution on the 5th of February, 1839, and on the 9th of February, 1843, became a director of the London and Brighton Company, of which he was elected chairman on the 19th of the following June. In this office he was succeeded by Mr. Pascoe Grenfell on the 11th of April, 1844, and ceased to be director of the company on the 21st of August, 1848. He was also a director of the Shropshire Union railways from 1845 to 1849. For many years he resided at 6, Raymond Buildings, and spent much of his time in collecting pictures and works of art. He had amassed at the time of his death a valuable collection of pictures, principally of the German and Dutch schools, and of water-colour drawings by English artists. Having left by his will a power of choice to the directors of the National Gallery, they selected three—one an oil painting, "Fishing Boats in a Breeze off the Coast," by J. M. W. Turner, R.A., and two paintings by P. I. Clays, of Brussels. Nearly a hundred oil pictures, and about fifty water-colour drawings were left by him to the South Kensington Museum, where they are distinguished as "the Parsons bequest," and a number of fine prints to the British Museum. Mr. Parsons married a daughter of Mr. John Mayhew, but was soon left a widower with an only daughter, now the wife of Sir Charles W. A. Oakeley, Bart.

Mr. Parsons removed from Raymond Buildings in November, 1869, to 45 Russell Square, and died there on the 26th of March, 1870.

Mr. JOSEPH PITTS was born on the 8th of January, 1812, at Sticker Lane, near Dudley Hill, Bradford. The circumstances of his parents were such, that in early life he had to be made one of the bread-winners of the family; but through the kindness of a friend he had the opportunity afforded him of attending a day-school gratis. Here he soon became a good plain penman; quickly passed through the rudiments of an elementary education, and diligently profited by the opportunities for acquiring learning which came within his reach, though he had to work early and late, as well as between the school hours, to procure the necessaries of life. After a time he became a teacher in a small day-school at Horton, and a few months subsequently he obtained a similar but more advantageous position in a school at Horsforth. In the year 1834 the friend who first assisted him in his education obtained for him the situation of book-keeper to Messrs. Butler and Taylor, ironfounders of Stanningley. He proved to be an efficient, faithful, and indefatigable servant, and by the weight of his moral character and the foregoing qualifications, rose from one position to another, until he became, for several years before his death, the leading partner in the firm—a firm which, in his lifetime, rose from comparative obscurity to be one of the principal manufacturers of iron bridges for railways. He was twice married; by his first wife, whom he married in 1834, he had a large family; his second marriage took place in 1848. He was elected an Associate of the Institution on the 7th of April, 1857. He was a prominent member of the Methodist Free Churches, and took great interest in the schools and institutions of that persuasion, being a man of great benevolence and exemplary piety. For the last two years of his life he was subject to heart disease, to which he succumbed on the 17th of March, 1870.

Mr. THOMAS HITCHINS SMITH, the eldest son of the Rev. C. A. Smith, vicar of Macclesfield, studied mechanical engineering for two years at the Atlas works in Manchester, and afterwards civil engineering for three years under Mr. Samuel, M. Inst. C.E. He had charge of the construction of a railway bridge in Scotland before his term with Mr. Samuel had expired, and subsequently, in conjunction with another engineer, was entrusted with the surveys for a railway in Spain, in the neighbourhood of Seville. He was in Spain between two and three years, and, on his return to England, after some months, was appointed an Assistant Engineer on the Great Indian Peninsula railway. He reached Bombay on Christmas Day, 1867; but he fell a victim to the climate and excessive labour, and died in the May following. However, during this short term of service he won the respect and regard of all with whom he was associated, his fellow-engineers uniting in erecting a monument to his memory, and Mr. Brereton, M. Inst. C.E., his superior officer, bearing the strongest testimony at once to his professional attainments, and the elevation of his character. He was elected an Associate of the Institution on the 12th of January, 1864.

Mr. THOMAS HARDY TAYLOR was born at Manchester, and was articled to Mr. George Shorland, Surveyor to the Corporation of that city, from 1850 to 1855. He remained as an assistant in the City Surveyor's office until April, 1860, when he received the appointment of Surveyor to the borough of Ipswich. This he retained until the 22nd of May, 1862. After this he was employed by the Contractors of the West Riding and Grimsby railway. On the failure of Messrs. Smith, Knight, and Co. he entered the office of Mr. P. Pons, architect and surveyor of Manchester, and there he remained, being regarded as a steady, industrious, and efficient assistant, up to the time of his death, which occurred on the 13th of January, 1869, from consumption, of which he had been a sufferer for four years and a half. He was elected an Associate of the Institution on the 9th of April, 1861.

Mr. GEORGE BARNARD TOWNSEND was the third son of Richard Townsend, formerly of Doctors' Commons, by his first wife, daughter of Mr. John Garrard, of Olney, Bucks. He was born on the 17th of July, 1814, and was educated at Eton. Having chosen the profession of the law, he became a member of the firm of Hodding and Co., of Salisbury, and was well known as a solicitor and Parliamentary agent, especially in connection with railways. In conjunction with the late Mr. Joseph Locke, M.P., Past-President Inst. C.E., he took an active part in the struggle for the introduction of the narrow gauge system of railways into the

West of England. Among the various lines of railways, both at home and abroad, in the origination and construction of which he was instrumental, are the Salisbury and Yeovil, now a part of the London and South Western Company's undertaking, the Stockport, Disley, and Whalley-bridge, the South Eastern of Portugal, and the Great Southern of India. He also took much interest in the development of Mr. Fairlie's patents for improvements in locomotive engines and carriages. Mr. Townsend was elected an Associate of the Institution on the 1st of May, 1860, and he died on the 29th of August, 1870, at his residence, Gundimore, near Christchurch, Hants. Like that of Mr. Locke, his death was sudden, of apoplexy, caused by the breaking of a vessel on the brain. He was married in 1840 to Georgina, daughter of Mr. Daniel Eyre, of the Close, Salisbury, who died in 1846, and by whom he left one son—now a lieutenant in the Royal Horse Artillery—and three daughters. His life was distinguished by constant activity and energy, and by a kindliness of disposition and of manner which was felt by all who were connected with, or approached him.

CAPTAIN JAMES VETCH, R.E., F.R.S., was born at Haddington, N. B., on the 13th of May, 1789, and was the third son of Robert Vetch, of Caponflat, East Lothian. He was educated at Haddington and Edinburgh; and in 1804, having obtained a nomination from Lord Chatham, he joined the Cadet College at Great Marlow, from whence, in 1805, he was transferred to the Royal Military Academy at Woolwich. He left Woolwich in 1806, to join the Trigonometrical Survey at Oakingham, in Berkshire, as Assistant-Engineer under Mr. Robert Dawson, with whom he remained till the summer of 1807. He received his commission as second Lieutenant in the corps of Royal Engineers on the 1st of July, 1807; was promoted on the 1st of March, 1808, to be first Lieutenant; and on the 21st of July, 1813, to the rank of Captain.

After serving for three years at Chatham and Plymouth, he was ordered in 1810 to the Peninsula, to join the division of Sir Thomas Graham at the blockade of Cadiz; where, with some exceptions, he continued till it was raised in 1812. He took part in repelling the attack made on the town of Cadiz by gun-boats; and was employed in throwing up works to strengthen the fortifications of the place.

In the spring of 1811 he was employed with Lieutenant Wells, R.E., in superintending the construction and repair of roads by which the allied army was to advance from Tarifa to attack the enemy, and was present at the battle of Barrosa, where his zeal and judgment contributed in no small degree to the success of the action, as the following narrative will show: "On the 1st of March the allied Spanish and English army, commanded by the Spanish general, La Peña, proceeded in the direction of Medina Sidonia, and on the 5th took up its position on the south-east face of the hill of Barrosa. The Spanish general then sent down his own troops in successive divisions, to drive the French from their fortified position in front of S. Petri, and to establish communication with La Isla de Leon. Although little resistance was made, the Spanish troops were long in effecting their object, and General Peña ordered the English division to march through a thick pine-wood to the same spot. Sir Thomas Graham had no choice but to obey, though much against his will, for the hill of Barrosa was evidently the key to the position. Lieutenant Vetch, though a young soldier, was fully alive to the danger of this movement, and lingered on the hill to observe whether the French would attempt to occupy it. The British division had not advanced above a mile into the wood, when, as Lieutenant Vetch was preparing to follow it, a mounted peasant hastily approached, crying out, 'Where is the general? Where is the general? The French are advancing to occupy the hill!' Young Vetch directing the peasant to follow him, galloped after the British division, and falling in with Major Hare of the staff, was taken to Sir Thomas Graham, who, the instant he was apprised of this flank movement of the French, ordering each regiment to countermarch on its own ground, made all haste to get back out of the wood. The English emerged from the wood as the French were deploying on the top of the hill, and were immediately formed in line to right and left, with the batteries of artillery, consisting of ten guns, in the centre, on rising ground. The British infantry advanced slowly but steadily under a heavy fire up the hill, and when, supported by their own artillery, they began to close on the enemy, a spirited charge put the French to flight, and their artillery, waggons, and a great many arms fell into the hands of the British force."

Sir Thomas Graham was so gratified at the intelligence displayed by Lieutenant Vetch on this occasion, that he made him the bearer of his despatches to Gibraltar; and directed him afterwards to explore the surrounding country, and crossing to the coast of Barbary, proceed from Tangiers to Tetuan, with a view to reporting on the capabilities of those localities for furnishing engineer supplies. This duty was satisfactorily performed, and on his return he was ordered to proceed in command of a detachment of sappers and miners up the Guadiana to Elvas, where, on arriving, he was immediately employed in the trenches before Badajos. On the evening of the 6th of April, 1812, the town was assaulted, and Lieutenant Vetch received instructions to make a lodgment with three hundred men in the ravelin of San Roque; this he gallantly accomplished, and entered Badajos with the victorious army. After the capture of Badajos he returned to Cadiz, and remained there till the blockade was raised in September, 1812, when he was employed in various parts of the south of Spain until his return to England in 1814.

During the next six years, until 1820, Captain Vetch commanded a company of sappers and miners, and was stationed first at Spike Island, in Cork Harbour, and afterwards at Chatham. It was at this time that he devoted himself so earnestly to the study of geology and other scientific pursuits, for the thorough acquaintance with which he was, later in life, so well known. While employed in constructing the fort on Spike Island, it was found necessary to remove, among other obstructions, an old, strongly built tower, about 50 feet square, with masonry walls 8 feet thick. Captain Vetch, having paid considerable attention to military mining, undertook to blow it up bodily. In this he was completely successful, for the whole tower rose slowly and then fell into a thousand fragments,—so perfect were his arrangements, so well calculated and placed were the charges, and so simultaneous the shock; and it must be remembered that this was before the days of the application of electricity to mining.

In 1821, on the express recommendation of Colonel Colby, Director of the Trigonometrical Survey, the Duke of Wellington, then Master-General of the Ordnance, appointed Captain Vetch to the Ordnance Survey, and during this and the two following years, assisted first by Lieutenant Drummond, and afterwards by Lieutenant Dawson (the late Colonel Dawson, Assoc. Inst. C.E.), he conducted the triangulation of the Orkney and Shetland Islands, and the Western Islands of Scotland. The work was prosecuted by them with great zeal, and carried on so late in the year, that their tents, generally pitched on the tops of high hills, were surrounded with snow before they would allow themselves to suspend their labours for the winter months, a season occupied by Captain Vetch in attending courses of lectures at the Edinburgh University.

While employed on the Ordnance Survey, Captain Vetch already a Fellow of the Geological Society, contributed several valuable papers to this and other societies. He wrote an able Paper on the remains of a Mammoth discovered by himself near Rochester, also one on geological specimens from the Bermuda islands, and another on some terraces or ancient beaches in the island of Jura: he further contributed to the Memoirs of the Wernerian Society an account of Foula, the most remarkable of the Shetland Islands. In 1822 Captain Vetch submitted to the authorities an ingenious invention for throwing a line from the shore to effect a communication with a vessel in distress, which was very favourably received. In 1824, promotion being very slow, officers were encouraged to go on half-pay, in order to visit foreign countries and obtain professional information; and Captain Vetch, having been invited to take the management of some extensive silver mines in Mexico, availed himself of this opportunity, and obtained permission to retire on half-pay from the 11th of March, 1824. With the exception of a visit to England, for a year or two, he remained in Mexico till 1835; and during this time he not only devoted himself to the development of the Real del Monté, Bolanos, United Mexican, and other mining concerns of which he had the management, but by laying-out and constructing good roads, and by organizing efficient systems of transport, he paved the way for the more extended mining operations at present carried on in that country; in this work he was greatly indebted to the services of the late Colonel Colquhoun, R.A., whose co-operation he found most valuable. So conspicuous were Captain Vetch's disinterested endeavours to promote the welfare of the mining interest in Mexico, that they attracted the notice of Sir Henry Ward, the British envoy, who, in an official communication, passed the highest encomiums upon them.

During his residence in Mexico, Captain Vetch was much hindered in his work by the want of a reliable map of the country, and with his usual energy he determined to construct one for himself, and making use of the experience he had acquired in England on the Ordnance Survey, he accumulated a vast quantity of astronomical and barometrical observations, measured several short base-lines, and triangulated a large tract of country, a small part of which, on his return to England, he plotted for the use of the Admiralty; but it is much to be regretted that he never had sufficient leisure to arrange his materials, and construct from his voluminous observations and computations an accurate map of the eastern portion of the State of Mexico. While devoting himself so zealously to these scientific labours he did not lose sight of the vast wealth of the country in antiquarian remains, and on his return to England he presented the British Museum with a valuable collection, and contributed a most interesting Paper to the Royal Geographical Society, of which he was a Fellow, on "The Monuments and Relics of the Ancient Inhabitants of New Spain."

In 1836 Captain Vetch was appointed one of the Commissioners for settling the Irish borough boundaries; and, on completing this duty, he was for the next four years employed by the Birmingham and Gloucester Railway Company as their Resident Engineer for the construction of one half of that line of railway. He was then engaged for some time in Ireland and Scotland on matters connected with the reclamation of tide-lands and the formation of embankments; and was frequently consulted professionally both by the Commissioners of Woods and Forests, and by the Admiralty.

In 1842, at the request of the Town Council of Leeds, he designed a system of drainage for that borough, which was at once put into execution, and gave great satisfaction. His report on the drainage of Leeds was most favourably noticed in the House of Lords by His Grace the Duke of Buccleuch, and he was in consequence called upon to give evidence before the Health of Towns Commission. In 1843, his attention having been directed to the facilities for communication with India, he published a most exhaustive "Enquiry into the means of establishing a ship navigation between the Mediterranean and Red Seas," which attracted considerable attention at the time, and so far engaged public confidence in the proposal that the subject was widely discussed and advocated. The idea never again slumbered, and the execution and completion, by M. de Lesseps, of this great engineering work has but now been witnessed. It was in this year that Captain Vetch was associated with Sir Henry de la Beche in the preparation of designs for the drainage of the town of Windsor, and in the following year he was directed by Lord Lincoln, then First Commissioner of Woods and Forests, to design and carry out a scheme of drainage for the Royal Castle and Parks, and for the purification of the Frogmore lakes. These works, in which H.R.H. the late Prince Consort took great interest, were carried on partly by contract, and on the strong recommendation of Captain Vetch, partly by a detachment of Royal Engineers, under the command of Lieutenant, now Colonel, the Honourable H. F. Keane, R.E., whose skill and energy were officially represented by Captain Vetch to have rendered his assistance most valuable: the main portion of these works was brought to a conclusion in 1847. In the meantime, on the passing of the Assessionable Manors of the Duchy of Cornwall Act in 1844, Lord Lincoln was pleased to appoint Captain Vetch to be one of the three Commissioners to carry out the Act; the others being Mr. J. F. Fraser and Mr. J. M. Herbert, barristers-at-law. At the termination of the labours of the Commission in 1846, on the successful attainment of the objects of the Act, the Chancellor of the Duchy was desired by Prince Albert, the President of the Council of the Duchy of Cornwall, to express to the Commissioners the high sense entertained by His Royal Highness and the other members of the Council of the diligence, skill, and impartiality with which the Commissioners had conducted the enquiry; and the letter proceeded to say: "The Council are happy to believe that this opinion is shared by the great body of the landowners and others in Cornwall, whose interests have been submitted to your consideration, and that a general feeling prevails that questions so numerous, complicated, and difficult as have arisen under the Commission have seldom, if ever, been investigated by any tribunal with more care, or decided with more general acquiescence in the propriety of the adjudication."

During the years 1844-46 Captain Vetch was examined at some length before the Tidal Harbours, and Harbours of Refuge Commissions, by whom he was requested to send in a report with drawings and models, to show the advantages which he considered would be obtained by employing wrought-iron framework in the construction of piers and breakwaters. This report was subsequently published. So high an opinion had been formed by these Commissions of Captain Vetch's acquaintance with, and knowledge of, the subject of hydraulic engineering that, in 1845, he was directed by Sir Byam Martin to report on the various designs for a Harbour of Refuge at Dover, which had been submitted to the Commissions by the following eminent engineers: Sir John Rennie, Messrs. Walker, Cubitt, Vignoles, George Rennie, James M. Rendel, the late Sir Harry Jones, and Sir William Denison; the report of the Commissioners was in strong confirmation of Captain Vetch's opinions. In July, 1846, Captain Vetch was appointed Consulting Engineer to the Board of Admiralty on all questions relating to railways, bridges, and other works, which might interfere with, or injuriously affect the harbours, rivers, and navigable waters of the United Kingdom. In 1847 this appointment was abolished, and Captain Vetch was appointed a member of the new Harbour Conservancy Board at the Admiralty, the other members being Captain, afterwards Admiral, Washington, R.N., and Captain Bethune, R.N. Captain Washington was withdrawn from the Board at the end of 1849, and in 1853 it was broken up, and Captain Vetch was appointed sole Conservator of Harbours. From that date, till his retirement from office ten years later, at the advanced age of seventy-five, he continued to discharge its arduous duties, receiving most flattering testimony to his zeal and judgment from the late Sir Robert Peel, Sir J. Graham, the Duke of Newcastle, and other First Lords of the Admiralty. In a letter dated 1853, the late Duke of Newcastle writes: "I shall, at any time, have very great pleasure in testifying to the high sense I entertain of your services on the various occasions on which I had the good fortune to obtain your assistance."

In 1849 Captain Vetch was requested by the Government to become one of the Metropolitan Commissioners of Sewers, an office entailing a great deal of work and carrying with it no remuneration, but to which he devoted himself with much energy for four years. During this year he published a pamphlet on the question of an extended water-supply to the Metropolis, and in 1850 he proposed to the Metropolitan Commission of Sewers a complete system of drainage for Southwark, which was subsequently adopted. In 1859 he was a member of the Royal Commission on Harbours of Refuge, of which Admiral Sir James Hope was chairman.

During the sixteen years that Captain Vetch was employed at the Admiralty he was well known and esteemed by all the prominent members of the Civil Engineering profession. His position was one of great responsibility, and oftentimes difficult in the extreme; for he had to maintain unflinchingly the rights of the Crown and public benefit against countless attempts at encroachments or private aggrandizement, besides having daily to sit in judgment upon, and to control the plans of Civil Engineers of great eminence and reputation. The arduous character of his work may be gathered from a letter written by Admiral Washington, the late Hydrographer, in 1858, in which he says: "He (Captain Vetch) had at once thrust upon him, in the very first year of office at the Admiralty, one hundred harbour and railway bills, on which he was required to report to Parliament, and the work was such that, even Sir Francis Beaufort, with all his experience, shrank from it, and would have resigned his post had he not been relieved of it. Captain Vetch had also almost annually to report on works connected with Portsmouth, Cork, Rye, Ramsgate, Portpatrick, Wexford, Isle of Man, Table Bay, and other harbours independently of the common routine of business."

Captain Vetch retired from the Admiralty in 1863, when his office was abolished, and the duties transferred to the Board of Trade. He devoted the last years of his life to the interests of the various public companies of which he was a director.

He was elected a Fellow of the Geological Society in 1818; of the Royal Geographical Society and the Royal Society in 1830; and an Associate of the Institution on the 26th of March, 1839. He contributed a "Description of a Bridge built of blue lias limestone, across the Birmingham and Gloucester railway at Dunhampstead," in the year 1841;[1] and occasionally attended the meetings. In the year 1850 he was also elected a member of the Société Française de Statistique Universelle; and, in 1842, a Corresponding Member of the National Institution of Washington. For his services in the Peninsula he received the War Medal and two clasps, for Barrosa and Badajos.

He died at the advanced age of eighty, on the 7th of December, 1869, leaving a large family and a wide circle of friends to mourn their loss.

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[1] *Vide* Minutes of Proceedings Inst. C.E., vol. i. (1841), p. 156.

SUBJECTS FOR PREMIUMS.

Session 1870-71.

The Council of The Institution of Civil Engineers invite communications on the Subjects comprised in the following list, as well as upon others; such as—1. Authentic Details of the Progress of any Work in Civil Engineering, as far as absolutely executed (Smeaton's Account of the Edystone Lighthouse may be taken as an example); 2. Descriptions of Engines and Machines of various kinds; or, 3. Practical Essays on Subjects connected with Engineering, as, for instance, Metallurgy. For approved Original Communications, the Council will be prepared to award the Premiums arising out of special Funds devoted for the purpose.

1. On the Strength and Resistance of Materials, practically and Experimentally considered.
2. On the Theory and Practical Design of Retaining Walls.
3. On Steam and Hydraulic Cranes, and on the Application of Steam Power in the execution of Public Works.
4. On the Different Systems and the Results of the use of Road Traction Engines.
5. On Land-slips, with the best means of preventing, or arresting them, with examples.
6. On the Gauge of Railways.
7. On the Principles to be observed in Laying-out lines of Railway through mountainous countries, with examples of their application in the Alps, the Pyrenees, the Indian Ghâts, the Rocky Mountains of America, and similar localities.
8. On Peculiarities in the Systems of Construction adopted for Railways in different Countries.
9. On the Systems of Fixed Signals at present in use on Railways.
10. Descriptions of Modern Locomotive Engines, designed with a view to cheapness of construction, durability, and facility of repair.
11. Description of Continuous Breaks which have been extensively employed on Railways, and the general results.

12. On the Principles which should be observed in laying out the Streets and Thoroughfares of Towns, or of the successive extensions of large Towns and Cities.
13. On the most suitable Materials for, and the best mode of formation of, the Surfaces of the Streets of large Towns.
14. On the Advantages and Disadvantages of Subways, for Gas and Water Mains, and for other similar purposes.
15. Accounts of existing Water-works; including the sources of supply, a description of the different modes of collecting and filtering water, the distribution to the consumers, and the general practical results.
16. On the Theory and Practical Design of Pumps, and other Machines for raising Water; as well as of Turbines, and of Water Pressure Engines.
17. On the Principles applicable to the Drainage of Towns, and the Disposal of the Sewage.
18. On the Employment of Steam Power in Agriculture.
19. On the Theory and Practice of the Modern Methods of Warming and Ventilating large Buildings.
20. On the Supply of Gaseous Fuel in Towns for Heating Purposes.
21. On the Design and Construction of Gas Works, with a view to the Manufacture of Gas of high illuminating power, free from Sulphur compounds, especially Sulphide of Carbon; and on the most economical system of distribution of Gas, and the best modes of Illumination in Streets and Buildings.
22. On the Theory of Heat applied to Steam Engines.
23. On the Theory of Condensation in Steam Engines, and the total effects upon the efficiency of a Steam Engine of the various modes of producing condensation.
24. On the practical employment of Heated Air as a Motive Power.
25. Description of successfully applied Gas Engines.
26. On the Maintenance, by Sluicing, of the Harbours on the Coasts of France, Belgium, and Holland.
27. Description of the Sea Works at the mouth of the River Maas, and the effects produced thereby.
28. On the Construction of Tidal, or other Dams, in a constant, or variable depth of water; and on the use of cast and wrought iron in their construction.
29. On the arrangement and construction of Floating Landing-Stages, for passenger and other traffic, with existing examples.

30. On the different systems of Swing, Lifting, and other opening Bridges, with existing examples; and on the theory and Practical design of Machinery for working opening Bridges.
31. On the present condition of knowledge relating to the Friction of Vessels passing through Water at different velocities, with suggestions for future research, either theoretical or experimental.
32. On the design and details of construction of Ships of War, having regard to their Armour, Ordnance, mode of Propulsion, and Machinery.
33. On the Design and the Materials for the construction of Land Fortifications.
34. On the measures to be adopted for protecting Iron and Iron Ships from Corrosion.
35. On Steel, and its present position as regards production and application.
36. On the safe working strength of cast and malleable Iron and Steel, including the results of experiments on the Elastic Limit of long bars of Iron, and on the rate of decay by rusting, &c., and how far vibration or prolonged fatigue affects the Strength of railway axles, chains, shafts, &c.
37. On Modern Progress in Telegraphic Engineering, including a notice of the theoretical data upon which that progress has been based; as well as a description of the improvements in the construction of land and sea lines, and in the working instruments.
38. On the Methods of Producing Artificial Cold and Ice by the Conversion of Mechanical Force.

The Council will be glad to receive, for the purpose of forming an "Appendix" to the Minutes of Proceedings, the details and results of any Experiments, or Observations, on Subjects connected with Engineering Science or Practice.

The Council will not consider themselves bound to award any Premium, should the Communication not be of adequate merit, but they will award more than one Premium, should there be several communications on the same subject deserving this mark of distinction. It is to be understood that, in awarding the Premiums, no distinction will be made, whether the Communication has been received from a Member, or an Associate of the Institution, or from any other person, whether a Native, or a Foreigner.

The Communications must be forwarded, on or before the 1st of February, 1871, to the house of the Institution, No. 25, Great George Street, Westminster, S.W., where copies of this Paper, and any further information, may be obtained.

CHARLES MANBY, *Honorary Secretary.*
JAMES FORREST, *Secretary.*

THE INSTITUTION OF CIVIL ENGINEERS,
25, *Great George Street, Westminster, S.W.*,
October, 1870.

EXTRACTS FROM THE MINUTES OF COUNCIL FEB. 23rd, 1835.

The principal Subjects for which Premiums will be given are:—

"1st. Descriptions, accompanied by Plans and explanatory Drawings, of any work in Civil Engineering, as far as absolutely executed, and which shall contain authentic details of the progress of the work. (Smeaton's Account of the Edystone Lighthouse may be taken as an example.)

"2ndly. Models or Drawings, with descriptions of useful Engines and Machines; Plans of Harbours, Bridges, Roads, Rivers, Canals, Mines, &c.; Surveys and Sections of Districts of Country.

"3rdly. Practical Essays on subjects connected with Civil Engineering, such as Geology, Mineralogy, Chemistry, Physics, Mechanic Arts, Statistics, Agriculture, &c.; together with Models, Drawings, or Descriptions of any new and useful Apparatus, or Instruments applicable to the purposes of Engineering or Surveying."

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The Communications should be written in the impersonal pronoun, and be legibly transcribed on foolscap paper, about thirteen inches by eight inches, the lines being three-quarters of an inch apart, on the one side only, leaving a margin of one inch and a half in width on the left side, in order that the sheets may be bound.

The Drawings should be on mounted paper, and with as many details as may be necessary to illustrate the subject. Enlarged Diagrams, to such a scale that they may be clearly visible when suspended on the walls of the Theatre of the Institution, at the time of reading the communication, should be sent for the illustration of any particular portions.

Papers which have been read at the Meetings of other Scientific Societies, or have been published in any form, cannot be read at a Meeting of the Institution, nor be admitted to competition for the Premiums.

Notice.

It has frequently occurred that, in Papers which have been considered deserving of being read and published, and have even had Premiums awarded to them, the Authors may have advanced somewhat doubtful theories, or may have arrived at conclusions at variance with received opinions. The Council would, therefore, emphatically repeat, that the Institution must not, as a body, be considered responsible for the facts and opinions advanced in the Papers or in the consequent Discussions; and it must be understood, that such Papers may have Medals and Premiums awarded to them, on account of the Science, Talent, or Industry displayed in the consideration of the subject, and for the good which may be expected to result from the discussion and the inquiry; but that such notice, or award, must not be considered as any expression of opinion, on the part of the Institution, of the correctness of any of the views entertained by the Authors of the Papers.

[Original Communications,

ORIGINAL COMMUNICATIONS,

DRAWINGS, PRESENTS, &c.,

RECEIVED BETWEEN DECEMBER 1st, 1869, AND NOVEMBER 30th, 1870.

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January 10, 1871.

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The following Candidates were balloted for and duly elected :—JAMES EDWARD DAY and FRANCIS JONES, as Members; Lieut.-Col. GEORGE CHESNEY, R.E., JOHN WILLIAM INGLIS, and CHARLES MARSHALL POOLE, as Associates.

It was announced that the Council, acting under the provisions of Sect. III. Cl. 7 of the Bye-Laws, had transferred EDWARD GOTTO from the class of Associate to that of Member.

Also that the following Candidates, having been duly recommended, had been admitted by the Council under the provisions of Sect. IV. of the Bye-Laws, as Students of the Institution :—HENRY CHARLES BAGGALLAY, CRAWFORD PETER BARLOW, FRANK STUART COURTNEY, THOMAS DUERDIN, CHARLES ELWIN, GEORGE CHARLES GILMORE, ARTHUR CHARLES GOTTO, RICHARD HAMILTON, WILLIAM EDWARD HORN, JOSEPH TINTORER, and FREDERICK THOMAS YOUNG.

No. 1,229.—"An Account of Floating Docks, more especially of those at Cartagena and at Ferrol." By GEORGE BANKS RENNIE, M. Inst. C.E.

FLOATING docks are designed to enable the submerged part of the hull of a ship to be examined, repaired, and painted, for which purpose, in many localities, they are found more advantageous than other ways of performing those operations.

The earliest method of effecting repairs was by beaching, or careening the vessel; then by excavating a cavity for it on the shore; and lastly, by making a floating structure capable of receiving the ship, and discharging the water from the space between the inside shell of the floating structure and the outside of the ship. Each of these different plans has been much improved upon. The former method of beaching vessels, by hauling them up on a gravelly or sandy shore, or, as in the case of the war galleys at ancient Carthage, on ways or slips, with sheds over them, surrounding a basin, has been superseded by the more efficacious plan of Morton's patent slip, with its improvement of hydraulic hauling-up gear. Careening, now nearly abolished, was brought to perfection towards the end of the last century by the Dutch and others. 'The excavated dock,' from a mere excavation in the bank of a river, open to the flow and ebb of the tide, the sides of which were supported with timber, has given way to the more massive construction of granite docks, introduced by the late John Rennie. And lastly, the wooden "floating docks," which existed on the river Thames and elsewhere at the commencement of the present century, have been replaced by iron structures capable of lifting vessels of the largest class. It is this latter system, namely, that of Iron Floating Docks, which is treated of in the present Paper.

Before describing the Iron Floating Dock at Cartagena, an outline of the general introduction of floating docks, with such examples as have come under the Author's notice, will be given. But those commonly known as screw or hydraulic docks will not be included, as they have been already discussed in Mr. Edwin Clark's Paper on "The Hydraulic Lift Graving Dock," read before the Institution in 1866.[1]

The earliest examples resembled a ship having one of the ends fitted with a pair of gates to admit the entrance of a vessel, the water being subsequently removed from the space between the dock and the ship. Docks of this kind formerly existed at Portsmouth and on the river Thames; the latter, the Author was informed by an eye-witness, was broken up about fifty years ago. A similar arrangement may still be seen in some of the French ports, and on the river Garonne, opposite Bordeaux. On an examination of this six years ago, it was observed that balance chambers, or tanks, were used for keeping the dock level on the water: one end was built up, and the other inclosed by a pair of wooden gates. A small steam-engine was used for working the pumps.

The 'sectional dock' and the 'balance dock' of Mr. Gilbert, both made of wood, are principally adopted in America. The former, as its name implies, is divided into as many sections as are required for the particular vessel to be docked. Each section consists of a rectangular wooden box made water-tight, and in the ends of these there is an open wooden frame-work of a height somewhat greater than the depth to which it is proposed to sink the dock. Within this frame a wooden water-tight box slides up and down, which can be fixed by means of a rack and pall to any required position. These boxes or tanks serve the purpose of keeping the base or lower part of the dock steady, water not being allowed to enter therein. Thus a complete dock consists of a series of eight or ten independent compartments below, with two moveable air-chambers to each; and, although there are certain

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxv., p. 292.

timbers connecting the different boxes, they are not constructed so as to enable any box to support the adjoining ones. The Author has been informed that the floating dock at Callao owed its destruction to this fact; for, while lifting a ship, some of the boxes, having more strain on them than others, were too suddenly lowered; the keel of the ship then got off the blocks, the vessel canted over, and sunk with the dock.

The balance floating dock, also introduced by Mr. Gilbert, and considered to be an improvement upon the sectional dock, consists of one compartment, divided internally into separate chambers (instead of a series of independent compartments). Docks on this system have generally been successful, and most of the dock-yards of the United States Government have been supplied with them. They are used both with and without caissons at the ends; the heavier class of vessel, when the length does not exceed that of the dock, requiring caissons.

In 1858, Mr. Gilbert was employed by the Austrian Government to construct one of his timber balance docks for the new arsenal at Pola. The dock was made at Venice, under the superintendence of Mr. Gilbert, and was towed to Pola when completed. It was of a similar construction to those previously made by him in America; but he dispensed with the end caissons, and gave the basement sufficient breadth and capacity to raise the largest vessel required to be docked. This dock has answered well, and done good service. The largest vessels which have been lifted are the 'Kaiser' screw line-of-battle ship, of 3,225 tons, and the 'Ferdinand Max' iron-clad, of 3,066 tons. In Appendix A a list is given of the different vessels docked by it up to the end of 1867.

The Author having been at Venice during the construction of Mr. Gilbert's wooden floating balance dock, and having observed the great quantity of timber required to give the necessary strength and stiffness to support a first-class ship, came to the conclusion that a structure in iron would offer many advantages, and in the year 1859 he carried this idea into effect.

The Spanish Government having found great difficulty in constructing stone graving docks at their naval arsenals, General Quesada, Director of Engineers, desired the chief of the Spanish Naval Commission in London to communicate with the Messrs. Rennie, relative to the construction of an iron floating dock for the naval arsenal of Cartagena, which should be capable of raising the class of iron-clad ships then about to be added to the Spanish navy. The weight required to be lifted was stated to be from 5,000 tons to 6,000 tons, which represents that of the 'Numancia,' the 'Vittoria,' and other vessels. The dock proposed by the Messrs. Rennie, and eventually adopted, was in principle somewhat similar to that constructed by Mr. Gilbert, at Venice; but as it was of iron instead of wood, it was necessary to introduce certain modifications. In the wooden structure, in order to sink the dock sufficiently, it is not only necessary to allow water to run into the lower chambers, but water must be forced up into the top compartments at the sides, to overcome the buoyancy of the material of which the dock is constructed. In the iron structure, on the other hand, provision had to be made to prevent the dock sinking when the lower compartments were filled with water. To accomplish this, the upper part of the side walls was divided into compartments, forming permanent air-chambers, or floats, of a capacity sufficient to maintain the decks of the side walls from 6 feet to 8 feet above the level of the surrounding water. The Author considers these air-chambers essential to the safety of iron floating docks, and that the dock at Cartagena was the first practical example of carrying out an iron floating dock. Up to the present time no other dock has raised so large a vessel as the 'Numancia,' of 5,600 tons weight; and this vessel has actually been supported on it for a period of eighty days.

Within twelve months after the commencement of the Cartagena dock, one similar in principle, but of somewhat larger dimensions, was ordered for the naval arsenal of Ferrol; but although completed and sent from this country, no decided measures have yet been taken to put it together.

The following is a list of floating docks, with their dates of construction, dimensions, and material, which have come under the Author's notice:—

Locality.	Country.	Date.	Length.	Breadth.		Lifting Power.	Material.
			Feet.	Feet.	in.	Tons.	
Portsmouth .	United States .	1851	350	105	4	8,070	Wood.
Pensacola .	United States .	1851	350	105	4	8,070	,,
Pola . . .	Istria . . .	1857	300	108	0	10,182	,,
Havannah .	Cuba . . .	1858	300	79	0	6,433	,,
Cartagena .	Spain . . .	1859	324	105	0	11,500	Iron.
Ferrol . .	Spain . . .	1860	350	105	0	13,040	,,
Sourabaya .	Java . . .	1861	250	70	0	3,500	,,
Saigon . .	Cochin China .						
Callao . .	Peru . . .	..	300	100	0	..	,,
St. Thomas .	West Indies .	1866	300	100	0	8,357	,,
Bermuda .	West Indies .	1866	381*	123	9	16,700	,,

* But only 330 feet within the caissons.

With the exception of the Bermuda dock, which approaches an elliptical or U-shaped form, all the docks mentioned in the foregoing table are of a rectangular section, and have a large proportion of breadth to depth of immersion, varying from about $\frac{1}{8}$ to $\frac{1}{10}$, whereas the U-shaped form has a depth of immersion of about $\frac{1}{3}$ of the breadth. For a given displacement, the length being constant, as a rule the stability of a floating body varies in direct proportion to its breadth; but as the stability is materially influenced by the height of the centre of gravity above the line of flotation, other sections approaching a semicircular form may, by lowering the centre of gravity, have a stability equivalent to the rectangular form. This requires the ends of a U-shaped dock to be closed in, either by gates or caissons, after the vessel has entered, as the keel of the vessel would be considerably below the line of flotation of the dock; whereas in a rectangular dock, as at Cartagena, the same stability may be obtained without the inconvenience of closing up the ends, and usually there is less water to discharge. In the former case the volume of water occupying the space between the outside of the ship and the inside of the dock has to be disposed of; in the latter merely the quantity of water representing the weight of the dock and of the ship. It will be seen from this that in the rectangular form the amount of pumping will vary as the weight of the ship, but, in the other case, the smaller the vessel the greater will be the quantity of water to be discharged. It is on these considerations that the rectangular form of dock was adopted for Cartagena.

In the docks Mr. Gilbert made in the United States, it was arranged that, after the dock with its ship had been about one-half raised, the ends were closed by caissons, and the remaining water surrounding the ship was run into the lower compartments of the dock. This was only required for the heaviest class of ships, but such was the inconvenience, that when Mr. Gilbert built the dock for Pola, he increased the breadth by a few feet, to render the stability greater, and he abandoned the caissons altogether.

The depth of the basement, or lifting chamber, of a floating dock on the rectangular system, like that at Cartagena, mainly depends on the lifting power required; for it is generally found that after the breadth to give sufficient stability has been determined, the depth required to obtain the necessary displacement will allow of ample strength being imparted to the base to resist the transverse strain, due to the weight of a ship, without increasing the thickness of the material in the shell of the dock. From the difficulty, however, of discharging water from a flat surface, a greater depth is sometimes given than is necessary, either for displacement or strength.

It will be observed that the buoyancy of the empty chambers produces a strain on the basement, while supporting a ship on its keel, similar to that of a beam resting on a central fulcrum with a uniform pressure on the lower surface acting upwards, which is equal to the weight of the ship supported. Although the strength of the dock transversely is calculated to resist this strain, the shores and bilge blocks when in use to a certain extent diminish the strain at the centre. Considering, however, the uncertainty of the pressure, it seems best to rely upon the strain being wholly borne by the metal in the centre. In the Cartagena dock the shell of the basement is made of a single thickness of $\frac{5}{8}$-inch plates; this in the centre for a breadth of about 6 feet is doubled, giving a thickness of $1\frac{1}{4}$ inch of metal for resistance to the strain. For a vessel such as the 'Numancia,' weighing 5,600 tons, the strain would be 1·32 ton per square inch, and for a vessel weighing 20 tons per lineal foot 1·5 ton per square inch. But, when the dock has no ship in it, the pressure upwards, due to the buoyancy, merely supports the weight of the side walls acting on the extremities of the beam, besides the weight of the basement which is acting nearly uniformly over the whole surface.

It will be observed, on reference to the transverse sections of the Cartagena and the Ferrol docks (Plate 6), that arrangements are made for supporting the ship laterally by means of shores, such as are commonly used in graving docks, for which purpose there are two rows of altars for horizontal shores above what is usually called the 'broad altar;' but this is here replaced by a gallery, beneath which are eight rows of small altars or shoring steps. Besides these there are nine moveable 'bilge blocks' on the floor of the dock, which are worked by means of chains in communication with the upper deck.

Having described generally the form and construction of floating docks, the question naturally arises, How are these docks to be placed in the required locality, and how are repairs, cleaning, and repainting of the submerged parts to be accomplished? First, the dock may either be sent in pieces or be towed to its destination, for it is questionable whether it would be advisable to provide a floating dock with sufficient self-contained power to be navigable by itself, unless it were found more economical to fit it temporarily with machinery, which might be removed on arrival at its destination. The plan adopted for the wooden floating dock of Pola was to build it at Venice, and then to tow it to Pola when completed; that for Havannah was also towed when completed from New Orleans on the main land to Havannah. The dock recently made for Alexandria was towed from France; those for Cartagena, Ferrol, Sourabaya, Saigon, Callao, and St. Thomas's, were sent out in pieces and erected at the respective ports; but the Bermuda dock, which was launched in the river Thames, was towed out to its destination.[1] Secondly, the necessary repairs, painting, or cleaning, can be performed in various ways, viz., by canting or careening, beaching where there is sufficient rise and fall of tide, raising the submerged part out of water by pontoons, or lastly, by floating the dock into a shallow excavation and closing the entrance. The former plan is intended to be used for the Bermuda, the latter has been adopted at Cartagena.

Two plans were proposed for that of Cartagena—one, lifting it by means of pontoons; the other, by floating it into a shallow basin, out of which the water could be pumped. The latter was eventually adopted, the dock being put together in the basin, and floated out instead of being launched. The plan of the basin originally proposed was rectangular, of a length somewhat greater than that of the dock, and of a breadth sufficient for the construction of three horizontal ways or slips, for hauling ships from the dock on to the dry land by means of hydraulic machinery.

The most convenient locality for the receiving basin was on the site of some old timber-ponds, at the south-west corner of the arsenal basin; and it was considered advisable to make the end nearest the arsenal basin only of a breadth sufficient for the floating dock to enter, the other end being spread out in a fan-shaped form, wide enough for the three lines of ways before mentioned. Messrs. Rennie having proposed this plan, and having determined the dimensions of the basin, as far as the dock was concerned, the details of construction of this part of the work were intrusted to Don José Baldasano, a Spanish engineer of highways and canals.

At Ferrol it was decided merely to excavate a basin of sufficient size for the erection of the dock, without reference to any ways or slips, and to close the entrance by means of a cofferdam.

Having described, generally, the introduction and form of

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[1] *Vide* "Narrative of the Voyage of the Floating Dock Bermuda, from England to Bermuda, &c." By One of those on board. 8vo. Plates and Map. London No date.

floating docks, a more detailed description will now be given of the docks designed by the Author, and carried into execution by Messrs. J. and G. Rennie.

Description of the Docks at Cartagena and at Ferrol. (Plate 6.)

As these two docks are of the same construction, the description of one will answer for both :—

Outside Dimensions.

	Cartagena.	*Ferrol.*
Length	324 feet . . .	350 feet.
Breadth	105 „ . . .	105 „
Height of outside . .	48 „ . . .	50 „

The basement or lifting chamber is, at Cartagena, 324 feet long, and at Ferrol 350 feet long, 105 feet broad in both cases, and 11 feet 6 inches, and 12 feet 6 inches deep respectively. It is composed of $\frac{5}{8}$-inch boiler plate, and is divided longitudinally into two equal parts, by a plate-iron bulkhead $\frac{5}{8}$ inch thick, with angle irons 4 inches by 4 inches on either side. Each of these parts is divided by transverse bulkheads into ten equal compartments at Cartagena, and eleven such compartments at Ferrol. Thus the basement is formed at Cartagena of twenty, and at Ferrol of twenty-two water-tight and separate chambers. Each of these chambers is again divided into two equal parts, by a bulkhead parallel to the centre one, formed of plates $\frac{3}{8}$-inch thick. This is not water-tight but perforated, so as to permit a partial flow of water from one side to the other, though not to such an extent as to cause a rush of a large volume of water, in the event of the dock suddenly listing over.

Between each transverse bulkhead there are five lattice or trussed girders, at equal distances apart and parallel to these bulkheads, formed of T-irons, back to back, $4\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{5}{8}$ inch. These cross each other diagonally, at an angle of about 42°, and are attached to plates, 15 inches deep and $\frac{5}{8}$ inch thick, both at the top and at the bottom, and these again are united to similar ones at right angles, by angle irons $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches.

Between the main girders, and uniting them, are angle iron frames 2 feet $7\frac{3}{4}$ inches apart, forming floors for strengthening the outside plating of the basement against the pressure of water when the dock is submerged. The frames on the bottom of the base have in addition 'floor plates' 6 inches deep fixed to them. Every sixth frame is carried down vertically and united to the lower frame. They are then united in a longitudinal direction by means of diagonal braces. The basement is thus strengthened by nine elements in a longitudinal direction, viz., the two sides, the centre bulkhead, two intermediate bulkheads, and four lines of diagonal truss-work.

The side walls run the whole length of the dock, one on each side of the basement, with which, however, they have no communication, and they are divided by transverse bulkheads similar to those in the basement. They are also divided longitudinally by horizontal bulkheads, thus separating the wall into two parts, the upper of which serve as permanent air chambers into which no water is allowed to enter, the lower being used when the dock is sunk to an increased depth for a large ship. The inner wall of the lower part is made in a sloping direction towards the centre of the dock, on which are angle-iron steps, to which are bolted balks of timber, serving as altars for the shores to abut against when supporting a ship. The outside plating of the side walls is $\frac{9}{16}$ inch thick at the bottom, and is diminished to $\frac{3}{8}$ inch thick at the top.

The arrangement of the pumping machinery and distributing pipes to the various compartments differs considerably from that adopted by Mr. Gilbert. In his docks pumps are placed in each lower compartment; a line of shafting runs the whole length of the dock, and is worked by a steam-engine placed in the centre of its length. Each pump is driven from this shaft by a second motion, and can be connected and disconnected by means of clutch gear, so as to pump out any or all of the chambers, as may be required. Considering the nature of the material composing those docks, this was probably the most convenient plan; but it has the objection, that the machinery for regulating the quantity of water in each chamber is too widely distributed, extending as it does along the whole length of the dock. In designing those for Cartagena and Ferrol, it seemed desirable that the machinery should be as much concentrated as possible, so as to be under the control of one man, and at the same time to enable a uniform duty and speed of the engines to be maintained. The arrangement adopted was that of a pair of horizontal engines, working two pairs of lift pumps immediately under the engines, to draw water from a common pipe, communicating with all the chambers. On the ends of these pipes are fixed the sluices of inlet for filling the chambers, and on the sides smaller sluices and pipes in communication with each chamber, so that by opening all the sluices the chambers are filled, and on shutting the inlet sluice, with the engines at work, one or any number of chambers may be discharged. Thus the whole engine-power may be employed in pumping out any one compartment, if it is found desirable to do so, in order to balance, or level the dock.

It will be understood, from the form of the dock, that this applies to each side, each side having distinct pumping arrangements. The man in charge of the dock communicates to the opposite side by a simple arrangement of signals, the quantity of water in each compartment being indicated by floats.

The dimensions of the cylinders and pumps are as follow :—

The engines were intended to make sixty revolutions at full speed, the pumps being reduced by gearing in the proportion of 2 to 1.

	Cartagena Engine and Pumps.	Ferrol Engine and Pumps.
Diameter of cylinders	14 inches	18 inches.
Stroke of piston	18 „	18 „
Diameter of pumps	20 „	24 „
Stroke of ditto	2 ft. 9 ins.	3 feet.

The inlet sluices are four in number, $40\frac{1}{4}$ inches in diameter. There are twenty distributing sluices and pipes, 18 inches in diameter, for Cartagena, and twenty-two for Ferrol. The maximum height which the pumps have to lift is 20 feet. The barrels of the pumps, as well as the buckets and seats of the foot and delivering valves, are of gun metal, the valves being of india-rubber or leather. The compartments between the air chambers and basement or lifting chambers, termed the middle chambers, can have water admitted into them by four separate inlet sluices, $40\frac{1}{4}$ inches in diameter, which may be regulated independently. The spindles, with the columns and hand wheels of all the sluices, are carried to the upper deck within the engine house, and can be manipulated in a space not exceeding 15 feet square. A compartment is made in the 'middle chamber' and basement chamber containing pumps and sluices, which are water-tight, so as to enable them to be examined under all circumstances, by means of ladders leading from the engine houses.

The air pipes are 6-inch cast-iron socket pipes, carried up from each basement and middle chamber to the upper deck, for the exit and entry of air during the filling or discharging of water into or from the chambers.

On the outside of the dock are fixed about a hundred wrought-iron ring bolts, 8 inches in diameter, for fastening small vessels or boats alongside.

The floor of the dock is covered with teak planking, 3 inches thick. This planking does not rest immediately upon the iron plating, but upon teak bearers of solid timber about 2 feet square. These are placed 16 feet apart, one over each transverse bulkhead and an intermediate one; the teak planking is in part made so that it can be taken up for the purpose of cleaning, and to give more room for driving long bolts under the bilge of the ship. The transverse teak beams are adapted for the use of bilge shoring blocks, which can be drawn inwards to the central line by means of chains passed through sheaves, cast-iron racks with palls being fitted for the purpose of holding the blocks in their place. The keel blocks are such as are frequently used in large graving docks, and are made of teak with cast-iron wedge pieces; these are placed at distances of about 5 feet apart, one over the centre of each transverse girder.

Each dock is provided with eight of Brown and Harfield's powerful capstans, four on the basement and four on the 'top-sides,' with the necessary bollards, &c. Besides these are four large towing bollards, 5 feet high and 18 inches in diameter, made of plate iron; these are for the purpose of towing the dock from place to place when the distance is too great to use the capstans. The sides of the dock are of yellow pine, $2\frac{1}{2}$ inches thick; the engine and boiler houses are also of the same material.

The sides and ends of the dock are protected by fender pieces, running vertically the whole height of the dock, about 8 inches square, and between these, horizontal pieces of the same scantling are bolted to the vertical pieces, at intervals of about 5 feet. The fender pieces are not bolted through the iron plating, but to 4-inch angle-irons rivetted to the plating, which are permanently fixed, so that the wood-work can be renewed without danger of causing a leak by the removal of the bolts.

The iron, wood-work, machinery, &c., as well as the capstans and other parts forming the floating dock, were prepared and put together in England, and then taken to pieces and sent out by sea to Cartagena, to be erected in the basin which had been in course of preparation during the construction of the dock in England.

The town and naval arsenal of Cartagena are situated at the end of a spacious bay, about 1 mile in length and $\frac{3}{4}$ths of a mile in breadth, the entrance being $3\frac{1}{2}$ cables wide. The arsenal basin is situated in the western corner, at the end of the bay, and is well protected by the high lands on the west, south, and east; but from the north it is exposed to the strong winds which blow from that quarter. The basin is rectangular, and measures in a westerly direction 1,120 feet, and 1,800 feet in a northerly direction, giving an area of about 46¼ acres. It was originally from 30 feet to 40 feet deep, and was excavated by a powerful dredging machine to a depth of 50 feet where the dock is moored, and the dredging machine is still in operation for deepening the whole of the arsenal basin. This basin (Plate 7) was made towards the end of the last century, and is a fine specimen of engineering works constructed during a flourishing period of the country. The quay walls are of masonry, standing 4 feet 3 inches above the usual water level; the different store-houses, dry docks, building slips, sheers, and cranes are placed on convenient sites round the basin. At the south-west corner there formerly existed several timber ponds, which in recent times ceased to be used for their original purpose. It was therefore determined to appropriate this site for the shallow basin, or dock receiver, with its three lines of ways or slips. The basin is of a uniform depth of 16 feet 6 inches from the top of the quay wall; the entrance is 120 feet 9 inches wide at the bottom, and 126 feet wide at the top for a length of 106 feet. It is provided with two grooves, an inner and an outer one, for the purpose of closing the entrance with an iron caisson.

The basin is 382 feet long on the north side and 345 feet on the south, and it has a curvature at the end of 320 feet radius, the chord of which is 200 feet. From this end run three lines of horizontal ways or slips radiating from the centre, from which the end of the basin is described. Each of these is 725 feet long and 45 feet broad, constructed with two altars, 5 feet 9 inches in breadth and 10 inches in height, one above the other, thus making the 'floor' of the ways or slips 1 foot 7½ inches below the surface of the ground. Each is laid down with four lines of timber-ways, about 10 feet apart, with keel blocks to correspond with those in the floating dock, and is intended to receive vessels, after they have been raised by the floating dock, from the dock on to the slip, by means of hydraulic power. From the length of the slips, it is estimated that six vessels may be building, or be under repair, at the same time, besides one on the floating dock. But this arrangement has not yet been fully carried out, though a somewhat similar disposition is in use for the dock at Pola.

The foundations were made by excavating the ground to about 7 feet in depth, and driving in piles at intervals of 3 feet 7½ inches. On these were put horizontal timbers, longitudinally and transversely, about 2 feet above the bottom, above which was laid hydraulic concrete from 5 feet to 7 feet in thickness. The concrete was formed of two parts of hydraulic lime, Portland or Theil, from Ardèche, France, four of sand from the beach at Portus, and six to seven parts of scoriæ from the lead works at Escombrera. It was then mixed with sea-water in a large trough, conveyed in baskets to the required spot, and thrown into wooden frames, where it quickly hardened. The concrete was laid on the clay bottom as found upon the rock at the base of Mount Galeras; but nevertheless sea-water forced itself through in some places.

The piles are of Spanish and Italian pine, some of them having been driven about one hundred years ago, when the arsenal basin was constructed, and, from their perfect soundness, were again used in the new works.

The three top courses of the basin, the bottom, and the piers, are of squared stone from Alicante. The stone for filling in was obtained from the quarries in the adjoining Mount Galeras.

The Iron Caisson for the entrance was made by the Messrs. Rennie, and is somewhat of the form of a ship, of nearly similar size and design to that constructed by them in 1858 for the basin of the floating dock at Pola. The length is 126 feet at the top, and 120 feet at the bottom, and the extreme breadth 18 feet. The water lines are elliptical, that form being very suitable for transmitting the pressure of water on the sides to the ends and bottom. The caisson is provided with two sluices, 24 inches in diameter, for filling the basin, and two, 18 inches in diameter, for sinking the caisson. The caisson was sent out in pieces, and put up in the entrance between the cofferdams, so that it could be floated and tried in its place previous to water being let into the basin.

The construction occupied about eighteen months in England, but the work was not shipped for some time afterwards, and was many months lying at Cartagena before a start was made to put it together. This, combined with the difficulty of getting men to execute the work, the many festas, and official routine, retarded its completion. Particular instructions were given to prove all the chambers with water pressure, somewhat in excess of the greatest strain they might ever be subject to. The engines and pumps were also tested, by admitting water to the basement compartments and pumping it out, so that the whole dock was in working order before being floated.

On the 2nd of June, 1866, water was let into the receiving basin, and the draught of the dock was found to be 4 feet 7 inches. This gives a displacement of water equal to 4,400 tons, the weight of the dock complete. It is made up as follows :—

	Tons.
Plates, angle, T-iron and rivets	3,770
Machinery, viz , engines, pumps, sluices and capstans . . .	160
Small forgings and castings, bolts, including handrails, mooring rings, keel block, wedges, &c. &c.	48
Calculated weight of teak and fir timber	422
Total	4,400

When the dock had been hauled out into the arsenal basin, orders were given to open the immersion valves, and in an hour and twenty-five minutes the base was submerged 37 feet. However, the vessel not being ready, the dock had to remain in that position until the following morning, when at 9 A.M. the vessel was placed between the walls of the dock, pumping was commenced, and at 1·45 P.M. the floor was above the level of the surrounding water, enabling the shipwrights to fix the shores under the bilge. The actual time that the engines were at work was under three hours: the dock was only once, for twenty minutes, out of level, which was caused by one of the valves being opened without orders.

This first experiment gave the engineer so much confidence in the working of the dock, that vessels of a larger size were afterwards raised, including iron-clads of from 4,400 tons to 5,600 tons.

In Appendix B a table is given of the weight and draught of water of the vessels taken on and off the floating dock at Cartagena to the end of July, 1868.

On more than one occasion, the efficiency of the air chambers has been fully tested, by leaving all the valves open, until the dock was buoyed up by the air chambers in the side walls, when it was found that the greatest immersion was 40 feet, leaving the decks of the side walls 8 feet out of the water. It will be understood that besides the actual air chambers, the pump and sluice chambers are also water-tight, giving a total buoyancy equal to 3,550 tons, leaving about 850 tons for the displacement due to the iron and wood-work forming the dock.

As previously stated, the largest vessel which has yet been docked at Cartagena is the 'Numancia,' iron-clad. Although this vessel was undergoing repairs for a period of eighty days, the dock sustained no straining, or damage whatever, from the great weight remaining so long a time in it. This is the more satisfactory from the 'Numancia' being one of the class of vessels for which the dock was originally designed. The operation of docking was performed with great facility, although from the weight of the ship it was deemed prudent to raise her very slowly, the engines only making 40 revolutions per minute. The draught of water of the dock, with the 'Numancia' upon it, was 11 feet 3 inches, with a depth of water of 7 feet 6 inches in the basement, and of 7 feet 2 inches in the middle chambers, amounting to a weight of 800 tons. This added to the weight of the dock (4,400 tons) gives 5,200 tons, and, deducted from the total weight due to the displacement of the dock at 11 feet 3 inches draught, gives 5,600 tons as the weight of the ship, which corresponds with that calculated from the lines of the ship at 24 feet 1 inch—the draught at the time of docking. The height of the keel blocks was 4 feet above the dock floor, making the centre of gravity of the ship and the dock about 16 feet above the line of flotation.

The dimensions of the 'Numancia' are as follow:—

Length between the perpendiculars . .	316 feet.
Extreme beam	57 feet.
Displacement at load draught	7,420 tons.

The operation in docking this and other vessels has proved the dock in every way efficient, and from the arrangement of the distributing valves, it can be managed with facility, either in sinking or in lifting.

The *personnel* of the dock consists of one chief engineer, one master boiler maker, four assistant engineers, four firemen, four valvemen, one clerk, and one storekeeper, in all eighteen men; with this number everything goes on regularly and without trouble.

Don Thomas Tallerie was the engineer in chief of the arsenal, Mr. John Fenwick had charge of the erection of the iron floating dock, and Don José Baldasano superintended the construction of the masonry work.

The communication is accompanied by a series of drawings, from which Plates 6 and 7 have been compiled.

APPENDIX A.

Names and Tonnage of Vessels lifted on the Wooden Floating Dock at Pola.

Class of the Ship.	Name.	Number of Guns.	Tonnage, B. M.	Date.
Line-of-battle ship .	Kaiser	91	3,225	February, 1860.
Frigate	Donau	31	1,840	
Ditto	Novara	50	2,268	
Paddle steamer . .	Triest	..	730	
Corvette	Friedrich	21	1,267	
Paddle steamer . .	Elisabeth	6	1,103	
Frigate	Venus	30	1,071	
Ditto	Adria.	31	1,840	
Paddle steamer . .	Greif	..	1,100	
Sloop	Möve	4	360	
Frigate	Novara	50	2,268	
Ditto	Radetzky	31	1,826	
Caisson of the basin				
Sloop.	Möve	4	360	1861
Line-of-battle ship .	Kaiser	91	3,225	,,
Corvette	Dandolo	21	1,280	,,
Battery	Fewerspeier	..	..	,,
Sloop	Kerka	6	524	,,
Ditto	Narenta	6	524	,,
Gunboat	Sansego	4	280	,,
Ditto	Gemse	4	280	,,
Paddle steamer . .	Greif	..	1,100	,,
Frigate	Schwarzenberg . .	51	2,313	,,
Ditto	Ditto	51	2,313	,, Drawn on shore.
Corvette	Friedrich	21	1,267	1861
Gunboat	Grille	4	280	,,
Ditto	Wall	4	579	,,
Ditto	Seahund	4	579	1862
Ditto	Dalmat	4	590	,,
Frigate	Radetzky	31	1,826	,,
Ditto	Adria	31	1,840	,,
Ditto	Schwarzenberg . .	51	2,313	,, Drawn into dock.
Paddle steamer . .	Sta. Lucia	8	931	1862
Ditto	Curtatone	6	560	,,
Ditto	Fantasie	..	292	,,
Ditto	Vulcan	6	483	,,
Ditto	Taurus	6	359	,,
Gunboat	Hum	4	590	,,
Ditto	Reka	4	579	,,
Paddle steamer . .	Elisabeth	6	1,103	1863
Gunboat	Gemse	4	280	,,
Ditto	Grille	4	280	,,
Paddle steamer . .	Achilles	4	157	,,
Caisson of the basin				
Frigate	Donau	31	1,840	,, Drawn on shore.
Ditto	Billona	35	1,206	1863
Paddle steamer . .	Fantasie	..	292	,,
Ditto	Triest	..	730	,,
Ditto	Greif	..	1,100	,,

APPENDIX A.—*continued.*

NAMES and TONNAGE of VESSELS, &c.

Class of the Ship.	Name.	Number of Guns.	Tonnage, B. M.	Date.
Iron-clad . . .	Drache	22	1,878	1864
Frigate	Novara	50	2,268	,,
Corvette	Friedrich . . .	21	1,267	,,
Iron-clad . . .	Don Juan de Austria	32	2,128	,,
Ditto	Kaiser Max . . .	32	2,128	,,
Ditto	Salamander . . .	32	1,878	,,
Paddle steamer . .	Fiume	..	..	,,
Iron-clad . . .	Prinz Eugen . .	32	2,128	,,
Corvette	Dandolo	21	1,280	,,
Frigate	Donau	31	1,840	,,
Brig	Hudzar	16	485	,,
Caisson of the basin				
Sloop	Kirka	6	524	,,
Frigate	Schwarzenberg . .	51	2,313	1865
Ditto	Radetzky	31	1,826	,,
Gunboat	Dalmat	4	..	,,
Ditto	Rika	4	597	,,
Paddle steamer . .	Fantasie	..	292	,,
Ditto	Curtatone . . .	6	560	,,
Gunboat	Streiter	4	597	,,
Paddle steamer . .	Elisabeth . . .	6	1,103	,,
Frigate	Adria	31	1,840	1866
Ditto	Schwarzenberg . .	51	2,313	,,
Paddle steamer . .	Fiume	..	450	,,
Frigate	Novara	50	2,268	,,
Iron-clad . . .	Kaiser Max . . .	32	2,128	,,
Ditto	Prinz Eugen . .	32	2,128	,,
Ditto	Don Juan de Austria	32	2,128	,,
Ditto	Drache	22	1,878	,,
Ditto	Salamander . . .	22	1,878	,,
Line-of-battle ship .	Kaiser	91	3,225	,,
Corvette	Friedrich . . .	21	1,267	,,
Gunboat	Dalmat	4	590	,,
Ditto	Velebrish . . .	4	590	,,
Paddle steamer . .	Greif	..	1,100	,,
Ditto	Stadium	..	800	,,
Gunboat	Hum	4	590	,,
Ditto	Wall	4	597	,,
Ditto	Streiter	4	597	,,
Ditto	Seahund	4	597	,,
Paddle steamer . .	Vulcan	6	483	,,
Ditto	Sta. Lucia . . .	8	931	,,
Iron-clad . . .	Ferdinand Max . .	16	3,066	,,
Paddle steamer . .	Fantasie	..	292	,,
Ditto	Elisabeth . . .	6	1,103	,,
Frigate	Donau	31	1,840	,,
Gunboat	Sansego	4	280	,,
Ditto	Streiter	4	597	,,
Paddle steamer . .	Curtatone . . .	6	560	,,
Ditto	Garguano	..	530	,, Drawn on shore.
Ditto	Greif	..	1,100	1866

APPENDIX A.—*continued.*

NAMES and TONNAGE of VESSELS, &c.

Class of the Ship.	Name.	Number of Guns.	Tonnage, B. M.	Date.
Gunboat	Hum	4	590	1866 Drawn on shore.
Sloop	Kerka	4	524	1866
Ditto	Narenta	4	524	1867
Frigate	Radetzky	31	1,826	,,
Corvette	Friedrich	21	1,267	,,
Gunboat	Hum	4	590	Drawn back into dock.
Sloop	Saida	6	344	
Iron-clad	Don Juan de Austria	32	2,128	

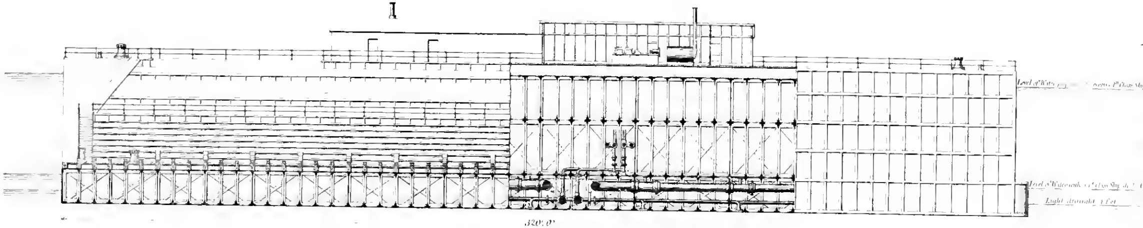
APPENDIX B.

VESSELS taken on and off the FLOATING DOCK at the ARSENAL of CARTAGENA, SPAIN.

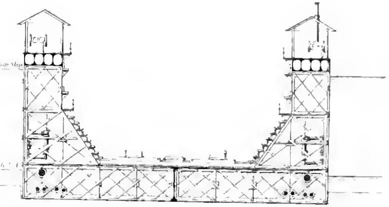
Class.	Name of Vessel.	Taken on.		Taken off.		Draught.	Approximate Weight in Tons.
		1866.		1866.		Feet.	
Dredger	Diligente	July	5	August	1	9	900
P.W. gunboat	Vigilanté	August	6	,,	18	12	
Ditto	Alerta	,,	18	,,	27	13	
Corvette, 32 guns	Alcedo	September	15	September	29	17	1,500
Corvette	Ferrolana	November	8	November	17	21	2,000
Transport	Laborde	,,	17	,,	20	8½	
Brig	Graviana	December	7	December	15	15	
		1867.		1867.			
Mail steamer	Santander	January	11	January	17	23	3,696
Transport	Pinta	,,	18	,,	22	17½	
Gunboat	Ceres	February	7	February	9	11	
Frigate	Resolucion	,,	11	June	9	21	2,000
Ironclad	Zaragoza	June	10	,,	27	20	4,400
Gunboat	Liniers	July	15	July	29	11	
Dredger	Diligente	,,	15	,,	29	9	900
Frigate	Princesa de Austria	August	7	August	13	21	3,810
Brig	Alcedo	,,	27	September	12	17	1,500
Tug	Veloz	,,	27	,,	12	12	
Tug	Relampago	September	23	October	12	5½	
7 pontoons	. .	,,	23	,,	12		
Frigate	Villa de Madrid	December	10	December	19	27½	
		1868.		1868.			
Ironclad	Zaragoza	March	16	April	6	25	4,972
Gunboat	Vigilanté	April	15	,,	25	12	
Ironclad	Numancia	,,	27	July	15	24	5,600

[Mr. G. B. RENNIE.

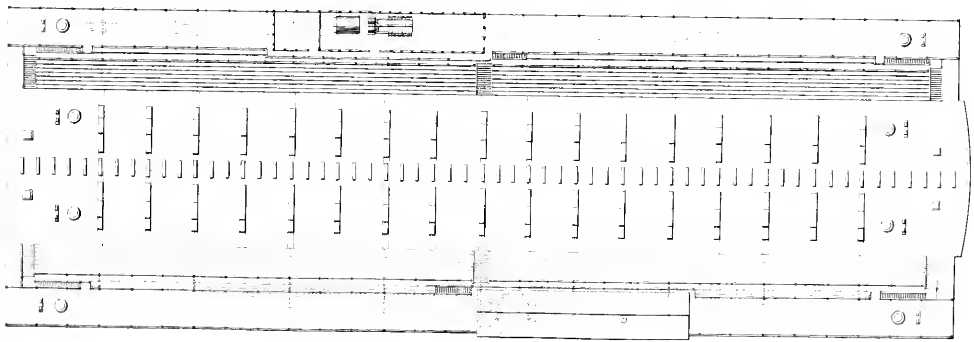
IRON FLOATING DOCK,
AT CARTAGENA.



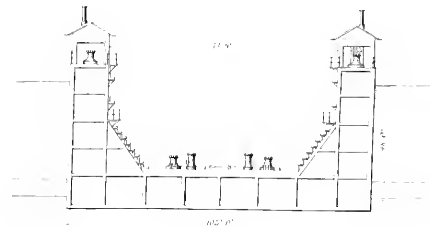
LONGITUDINAL SECTION ON CENTRE LINE. SECTION SHEWING ARRANGEMENT OF PIPES. SIDE ELEVATION.



TRANSVERSE SECTION.



PLAN.



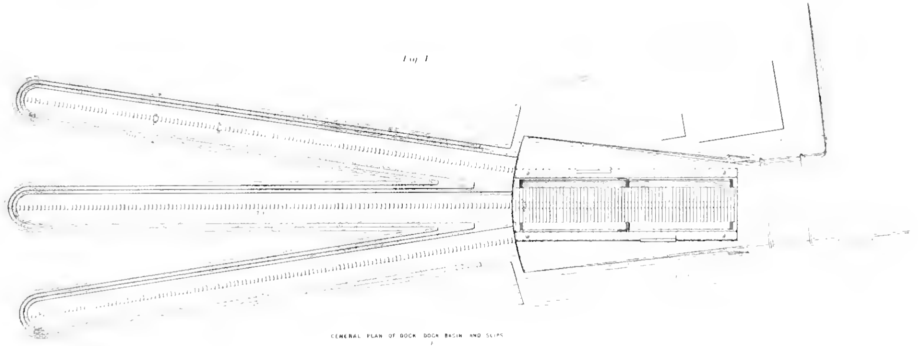
END ELEVATION.

Scale of Feet

IRON FLOATING DOCK,

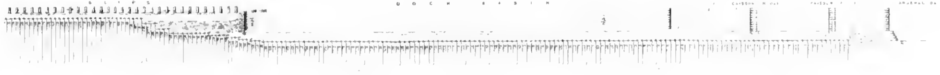
AT CARTAGENA

Fig. 1.



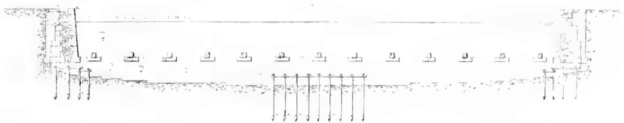
GENERAL PLAN OF DOCK, DOCK BASIN AND SLIPS

Fig. 2.

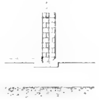


LONGITUDINAL SECTION THROUGH CENTRE OF SLIPS, DOCK BASIN AND ENTRANCE FROM ARSENAL BASIN

Fig. 4.

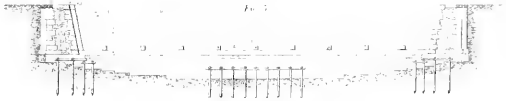


TRANSVERSE SECTION ON A LINE … FEET FROM ANGLE OF CURVE



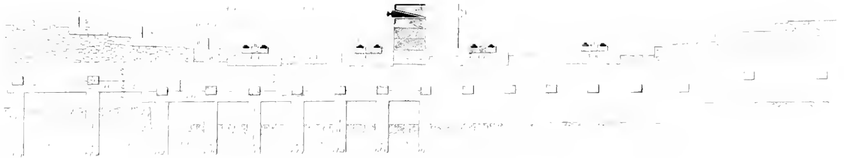
CROSS SECTION

Fig. 5.



TRANSVERSE SECTION THROUGH CAISSON GROOVE

Fig. 6.



TRANSVERSE SECTION THROUGH SLIP

Mr. G. B. Rennie said that Mr. Turner, the British Consul at Cartagena, had informed him that at the present time the working of the dock was perfect, and that it was in excellent order. The slipways for hauling vessels into the dock were constructed, but as yet no machinery had been ordered for the purpose. The Director-General of Engineers, General Nava, in a letter from Madrid, dated the 24th of November, said that the dock "functions" well, and "does good service," and that the docking of such vessels as the 'Victoria' and the 'Numancia,' with a displacement of 7,300 tons, fully equipped, and 1,000 tons less when lightened, was considered to be a very satisfactory test of the capabilities of the dock.

Mr. Hemans, Vice-President, said it would be difficult to discuss the subject in the absence of any statement as to the cost of this structure, which would enable a comparison to be made with the cost of dry docks constructed to fulfil similar purposes. He should be glad therefore if Mr. Rennie could supplement the other details, which were of the most complete character, with a statement of the cost per ton of the ironwork, including the transport to its destination.

Mr. G. B. Rennie replied, that roughly speaking, the cost of the dock was between £150,000 and £160,000. He had no account of the cost of the masonry work; the sum he stated was for the iron dock only. Many people had been employed at Cartagena who did not know much about their work, and who consequently were a long time about it; but he understood the Spanish government nevertheless considered the undertaking was cheaper than if a stone graving dock had been built there.

Mr. Barlow said this arrangement had been made with a view of taking ships upon a floating dock, and putting them on to a landing at a fixed level. He would ask whether that operation had ever been attempted, and if so, what had been done in respect to the difference of level which existed between the dock and the surface to which the ship had to be floated.

Mr. G. B. Rennie replied that the hydraulic machinery at Cartagena was not completed, but at Pola, where the docking arrangements were not on quite so large a scale, several vessels had been hauled ashore from the dock. The arrangement for keeping the levels of the slip and of the dock the same was a simple matter. The dock was floated into the small basin, the water was let into the dock, and it was grounded in the basin; the timber-ways on the slip were then placed in an exact line with the timber-ways on the floor of the dock, and the vessel was hauled along horizontally. The dock was moored where the water was about 50 feet deep. If it was wanted to haul the vessel ashore, the dock was transported into the smaller basin, then grounded, and the vessel hauled on shore.

Mr. Hamilton E. Towle, of New York, said that two kinds of floating docks had been adopted by the United States Navy Department, one called the 'sectional dock,' and the other the 'balance dock.' In the former, swinging gates were substituted for caissons, which latter were only used at the mouths of the basins. The floats at the ends of sectional docks were not held by pawls but by racks and pinions, and a worm-wheel gearing driven by a screw, which served the same purpose as pawls, and also permitted the floats to be moved up or down, an improvement devised by Messrs. Burgess and Dodge of New York.

The dock at Pola was 2 feet 8 inches wider than the Pensacola dock in the Gulf of Mexico, which was of the same dimensions as that at Portsmouth, New Hampshire; but the Pola dock was superior in construction, and had much of the character of the Cartagena dock. No iron docks had, up to the present time, been constructed in the United States. The shafting of the balance docks did not run the entire length of the dock, as stated in the Paper, but only along the centre over where the pumps were placed. In the sectional docks the shafting was coupled by universal joints, where the sections were united together, and it ran the whole length of the structure, on either side.

In hauling a ship out of the dock, when once raised and floated into the basin, the dock was first grounded upon the ways at the bottom of the basin. These ways were perfectly level, the dock rested upon them, and there were other inclined ways upon the dock itself, corresponding with similar ways on the land, and having the same inclination. The docks in the United States had ways of greater inclination than that at Pola, where the inclination was only about 4 feet in 500 feet. The hydraulic press at Pola had a piston 15 inches in diameter, and one was split under a test of 600 tons, though some doubt was at this date entertained as to the possibility of exactly measuring such high pressures.

The docking of the 'Kaiser,' weighing 4,223 tons, was made a crucial test of the dock at Pola. He calculated that the entire weight then raised was 5,066 tons, made up by the ship itself, with its guns, and other ballast. This test practically determined that the dock would lift a considerably greater weight than was required of it before construction.

The original design and plans of the trussed work had been modified, in consequence of the short length of the timber available in Austria. It was found, on advertising for timber, that speculators combined, and put such a high price upon it, that it was cheaper to bring timber of the larger scantling required from New York; and the very large and long timber which was used in the Pola dock grew in Ohio. The ways, upon which the ships were drawn out, were also made principally of Ohio oak; the sticks for the centre way being 18 inches square. The railway ways consisted of three parallel lines of timber, one in the centre under the keel of the vessel, and two others, one on either side. The centre way confined the hydraulic press upon the railway when pulling or pushing a ship upon the cradle.

At Pola there were two sets of railways constructed parallel to each other, each railway was over 700 feet long, and each was intended to provide length enough for two vessels, one behind the other.

The hydraulic press, or, rather, the hydraulic locomotive, consisted of the press, its pumps to force in the water, and two vertical steam engines with an ordinary locomotive boiler; and the whole slid upon the centre bed-way, and was confined to it by horizontal keys to secure the hydraulic press in position. To draw a vessel from the dock, the hydraulic locomotive went down to the cradle, on which the vessel rested, and was attached to it by a pair of iron hauling bars 8 inches square; the hydraulic cylinder was then confined by the keys, and the engines above forced the water into the cylinder, which, pushing the piston out the entire length of the stroke of 8 feet, gave the desired hauling motion. The motion was now reversed by means of a screw, which drew the cylinder along 8 feet, sliding the piston in again, when it was ready for a fresh stroke. Applying the force pumps once more, the ship was brought another 8 feet, and this process was repeated till she was hauled entirely out of the dock, and to the desired position on the railway. To put a ship back, the hydraulic locomotive was simply turned round, and the piston-head made to push against the cradle. This was the process adopted at Pola and at Pensacola, and at other naval yards in the United States.

At San Francisco and Philadelphia the docks were upon the sectional principle, but they all had the same kind of hydraulic apparatus for moving vessels.

The 'Franklin' was the first vessel, weighing 2,300 tons, hauled out upon the railway at Portsmouth, New Hampshire, and some vessels had been cut in half on the ways, and lengthened there. The Pola basin had about 40 feet depth of water at one corner, and 8 feet or 10 feet less at the opposite corner, and at the corner nearest the land the water was about 22 feet deep. A few years previous to its construction, several attempts, by Austrian and French engineers, to build an ordinary excavated dock had failed. They blasted and excavated the rock to a depth of about 16 feet or 18 feet; but at this depth subterranean passages and fissures were met with in the volcanic rock which communicated with the sea. The enterprise was consequently abandoned.

January 17, 1871.

THOMAS E. HARRISON, Vice-President, in the Chair.

No. 1,239.—"On the Strength of Lock Gates." By Walter Raleigh Browne, Assoc. Inst. C.E.

This subject has been treated of in the publications of the Institution, first by Mr. Peter Barlow,[1] and, secondly, by Mr. Kingsbury,[2] in the course of his Paper on the Victoria Docks; but neither of these investigations is altogether satisfactory.

Mr. Barlow points out the two strains to which a lock gate is subjected, viz., a transverse strain, arising from the water pressure, and acting at right angles to the length of the gate; and a longitudinal strain of the nature of a thrust, arising from the pressure of the other gate. He gives the ratio of these strains to each other, and proceeds to reduce the longitudinal to a transverse strain, in order to simplify the investigation. This he does by the help of some experiments of Girard, which are confined to oak timber, and which, as giving only the breaking strain, furnish no certain information as to practical cases, where the strain is only a fraction of the breaking strain. Hence the result, even if correct as far as it goes, is clearly not applicable in general.

So far Mr. Barlow speaks only of straight gates. He makes some further remarks on cambered gates, which appear to be based altogether on a false supposition, namely, that the pressure at the mitre end of a curved gate must be in the direction of the tangent to the curve at that point. This is only true in the case of a straight strut kept in equilibrium by a force at each end. It does not apply where a third force acts on the strut, as the water pressure does here. At the point of junction of two gates two surfaces are in contact; and, therefore, neglecting friction, the pressure between them must be normal to each, that is, at right angles to the axis of the lock. This original error vitiates all Mr. Barlow's reasoning on this part of the subject.

Mr. Kingsbury's chief object is to show the advantages of using

[1] *Vide* Trans. Inst. C.E., vol. i., p. 67.
[2] *Vide* Minutes of Proceedings Inst. C.E., vol. xviii., p. 445.

what he calls the cylindrical form of gate, *i.e.*, the form in which the backs of the two gates, when closed, make a single circular arc. The appearance in this case is that of an ordinary arch, and Mr. Kingsbury uses the common formula for arches under normal pressure, viz.:—

Thrust at any point = pressure per lineal unit × radius of curvature.

On the other hand, in straight gates, he assumes it to be necessary to give sectional area enough, first to withstand the transverse strain, and, secondly, to withstand the longitudinal or compressive strain. But it does not seem impossible that these two strains should partly counteract each other; as in a common arch the reaction of the abutments tends to counteract the vertical pressure of the load. Moreover, Mr. Kingsbury assumes that in cylindrical gates the unit strain on the two flanges is the same, which is not the case. The formula he uses gives the resultant compression on any section, but tells nothing as to how it is distributed. It might even happen that there was tension at one part, neutralized by an excess of compression at another.

In working out the mathematical conditions of the problem, no account is here taken of that element of strength in beams which Mr. W. H. Barlow calls the resistance to flexure. The character and laws of this resistance are not sufficiently ascertained for calculation; and it seems probable that it will exercise a greater effect proportionally under a breaking than under a working strain. In determining moments, the alteration of form by deflection has been neglected.

The investigation will then be as follows:—In the first place, all questions of form and material are left out of consideration; a pair of gates is taken as consisting simply of two rigid rods, 1 inch thick, supporting the water pressure, and meeting at the mitre.

In Fig. 1, let s = span of lock.
,, r = rise.
,, l = length of either gate.
,, w = water pressure in tons per inch run.[1]

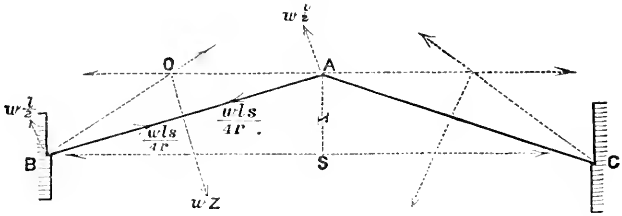
Then taking one gate as A B, the forces acting upon it are—

1. The water pressure, equivalent to $w\,l$, acting at the centre or middle point of the gate.

[1] The dimensions are taken in inches, because the stress is always measured in tons to the square inch.

2. The reaction at the mitre post. This by symmetry will act directly across the lock in the direction of A O.

Fig. 1.



3. The reaction at the heel post, which, by the triangle of forces, must pass through the intersection of the lines of action of the other two forces.

Resolving the two latter forces in directions parallel and perpendicular to the line A B, it follows, by the ordinary conditions of equilibrium, that the reaction at either end perpendicular to A B $= \frac{w\,l}{2}$, and the reaction parallel to the gate (or compressive

strain along the gate) $= \frac{w\,l}{2} \times \frac{\frac{s}{2}}{r} = \frac{w\,l\,s}{4\,r}$.

This longitudinal strain will be the same, whether the gate be straight or cambered.

Having established these general values for the forces, the most important and complicated case—that of a cambered iron-plate girder, may be considered. The investigation will differ from that giving the strength of such a girder in ordinary cases, because the longitudinal strain, which is there assumed to be zero, is here, in consequence of the end pressures, considerable.

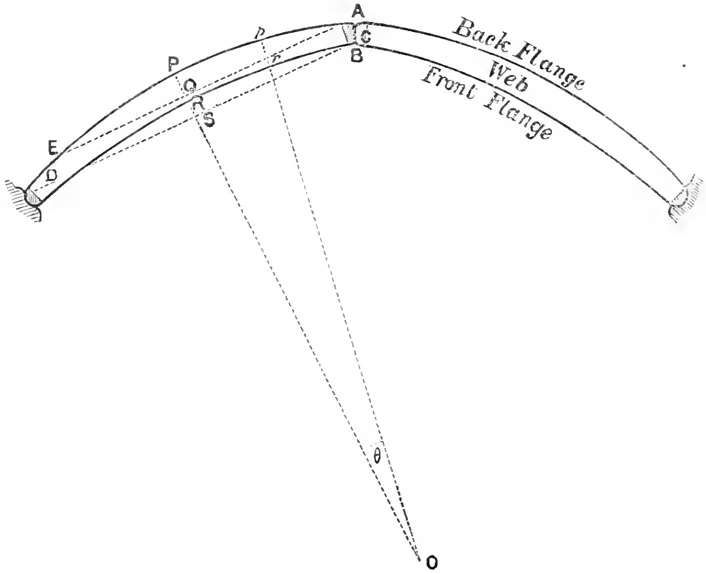
Figure 2 (p. 320) represents the plan of such a girder. It consists of a web and two flanges, of which that next the water (as A E) will be hereafter called the back flange, and the other the front flange. The resistance of each flange will be considered as acting at the centre of resistance of that flange; and the centre of resistance of the back flange will be taken to coincide with the back or upper edge of the web plate. This will in all practical cases be very near the truth.

The external forces acting on either gate are as before.

1. The water pressure. On any elementary arc $d\,s$ of the back

flange the water pressure will be $w\,d\,s$, acting along the normal to the arc.

Fig. 2.



2. The pressure on the mitre post from the other gate. This, as shown above, is equivalent to a pressure $\frac{w\,l}{2}$ at right angles to the axis of the gate, and a pressure $\frac{w\,l\,s}{4\,r}$ parallel to the axis.

3. The pressure on the heel post from the hollow quoin. This is equivalent to two pressures similar to those just described.

As to the exact centre of resistance, or point of application of either of those forces, it is to be remarked that the tendency of the water pressure will clearly be to flatten the curve of the gate by forcing it in at the crown. Now, supposing the joint of the mitre post at B to be perfectly hard, and the gate or rib to be flattened as described, the joint at A would evidently open, and the two ribs in their deflected state would meet one another only at the point B. In experiments made on small bars of wood, in 1868, this opening of the joint was very apparent when the strain approached the breaking point. In some cases it extended as much as three-fourths of the way down. Thus it appears that the pressure on the mitre joint A B will not be uniform, but will increase very considerably between A and B. The exact position of its centre of resistance is difficult to determine, theoretically or practically. Supposing the pressure to vary uniformly as the distance from A, the point would be two-thirds of the distance from A to B, and this will probably be a sufficient approximation to employ until the question can be accurately determined.

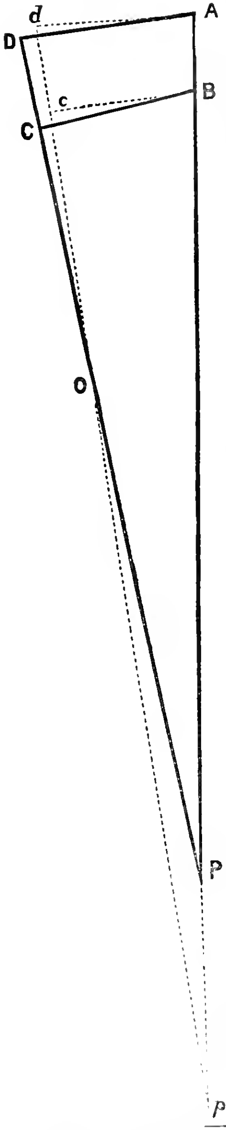
A line C D (Fig. 2) drawn from this point however fixed, to the centre of the heel post (through which all the normal pressures on the heel post must pass), will be the proper axis of the rib A E; a line drawn through A, parallel to C D, will be the chord of the curve of the back flange. Let P Q R S be a normal section to the rib drawn through the centre of its length. Then the following symbols will be used:

P Q the camber of the back flange	$= C$
P O the radius of curvature of back flange	$= \rho$
P R the depth of the web at centre	$= d$
Distance from P to centre of resistance, front flange	$= d'$
S Q distance from axis to chord of back flange	$= \beta$
Thickness of web	$= \tau$
Sectional area of back flange at centre	$= A$
Sectional area of front flange at centre	$= B$
Working strain per square inch	$= E$
Longitudinal compression $\left(\frac{w\,l\,s}{4\,r}\right)$	$= P$

The object is now to determine the relations which hold between these quantities and w, the unit pressure due to the water, on the supposition that the greatest strain on any part of the rib shall not exceed the given value E. For this purpose, suppose the rib divided by an ideal section at the centre, and consider the equilibrium of one half as A P R B. The forces acting on this half are the same in character as on the whole, except that for the pressures upon the heel post are substituted the resistances of the iron in the section at the centre. The nature of these resistances will, therefore, be the first point to discuss.

The effect of the forces acting on the rib will be, as before observed, to shorten and flatten the curve to which it is formed. Hence an element ABCD (Fig. 3) of the rib, having centre of curvature P, will be altered to some such form as A B $c\,d$, having centre of curvature p, and the radius of curvature will be altered from ρ to $\rho + \delta\rho$. Thus there must be some point, as O, at which the length of the elementary arc will be the same as before. This point will be called the neutral point, and its distance O D from the back flange may be called k.

Fig. 3[1].



Now let ϕ be the angle subtended at the centre of curvature by any element of the rib, such as A B C D when unstrained: $\phi + \delta\phi$ the angle subtended by the same element when strained. Also let A P = ρ.

Then $\overline{\rho - k}\,\phi$ = length of element at O before straining.

= length of element at O after straining.

(by definition of neutral point)

$$= \overline{\rho + \delta\rho - k} \cdot \overline{\phi + \delta\phi}$$

Therefore $\delta\rho \times \phi + \overline{\rho - k} \times \delta\phi = 0$. (1)

Take any one of the elementary arcs, which may be considered to make up the element A B C D, and let y be its distance from the back flange, or A D. Then

Length before straining $= \overline{\rho - y} \times \phi$

,, after ,, $= \overline{\rho + \delta\rho - y} \times \overline{\phi + \delta\phi}.$

Hence the extension or compression is

$$\delta\rho \times \phi + \overline{\rho - y} \times \delta\phi$$

$$= \delta\rho \times \phi \times \left(1 - \frac{\rho - y}{\rho - k}\right) \;.\;.\;.\;. \text{ by (1)}$$

$$= \delta\rho \times \phi \times \frac{y - k}{\rho - k}.$$

Therefore by Hooke's law the strain per unit of area on this element is

λ[2] $\times \dfrac{y - k}{\overline{\rho - y} \times \overline{\rho - k}} \times \delta\rho.$

Suppose the value of this unit strain to be given as E, when y = some fixed quantity y' Then

$$\lambda\,\delta\rho = \mathrm{E}\,\frac{\overline{\rho - y'} \times \overline{\rho - k}}{y' - k}.$$

[1] In figure 3 the original and altered elements are superimposed on each other, by making the point A and line A P the same in both. Thus the figure does not show the *position* of the element before and after straining, but only its form.

[2] λ = Modulus of elasticity.

Hence at the former point, distance y from the back flange, the unit strain will be

$$E\,y - k \times \frac{\overline{\rho - y'}}{\overline{\rho - y} \times \overline{y' - k}}.$$

It is evident that the part of the rib which lies between D and O will be in compression, and the part, if any, which lies between O and P will be in tension. But in the case under consideration the whole compression must exceed the whole tension by the amount of the longitudinal thrust. Hence the point at which the unit strain is greatest will clearly be the back flange. Therefore the point at which the strain is to equal the maximum unit strain E is the point given by putting $y' = 0$.

Putting then $y' = 0$, the unit strain at distance y from back flange

$$= \frac{E}{k} \times \overline{k - y} \times \frac{\rho}{\rho - y}.$$

This is the value of the strain without reference to direction; the positive direction may therefore be assumed to be that in which this strain acts, viz., the direction of resistance to compression at the centre section of the rib.

If y becomes greater than k, the strain will become negative.

The flanges must now be considered separately from the web.

(α) Back flange. Putting $y = 0$. Strain $=$ A E.

(β) Front flange. Putting $y = d'$. Strain $= \mathrm{B\,E} \times \frac{k - d'}{k} \times \frac{\rho}{\rho - d'}$.

(γ) Web. Unit strain on element $\tau\,\delta y$ at distance y from back flange

$$= \frac{E\,\tau\,\rho}{k} \times \frac{k - y}{\rho - y} \times \delta y = \frac{E\,\tau\,\rho}{k}\left(1 + \frac{k - \rho}{\rho - y}\right)\delta y.$$

The total resistance to compression of the web will be found by integrating this from O to d. This integration is

$$\frac{E\,\tau\,\rho}{k}\left[d - \overline{k - \rho}\,(\log_\epsilon \overline{\rho - d} - \log_\epsilon \rho)\right] \quad . \quad . \quad . \quad (A)$$

Again, the moment about the back flange of the strain on the element $\tau\,\delta y$ is

$$\frac{E\,\tau\,\rho}{k} \times \frac{k - y}{\rho - y}\,\delta y \times y$$

$$= \rho \times \frac{E\,\tau\,\rho}{k} \times \frac{k - y}{\rho - y} \times \delta y - \frac{E\,\tau\,\rho}{k}\,(k - y)\,\delta y,$$

since $$y = \rho - \overline{\rho - y}.$$

The moment of the whole strain on the web about the back flange will be found by integrating this expression from O to d. The integration is

$$\rho \times \frac{\mathrm{E}\tau\rho}{k} \times \left[d - \overline{k - \rho}\,(\log \overline{\rho - d} - \log \rho) \right] - \mathrm{E}\tau\rho d + \mathrm{E}\tau\rho \frac{d^2}{2k} \quad (\mathrm{B})$$

This is a complete account of the resistances of the central section parallel to the axis of the rib. Besides these there is an unknown resistance to shearing perpendicular to this axis, which may be called F.

Next, to find the effect of the water pressure on the half rib under consideration:

Take any normal section pr (Fig. 2) making an angle ϕ with the radius of curvature at the centre P, the pressure on the element $\rho\, d\, \phi$ at this point is

$$w\, \rho\, d\, \phi.$$

And resolving this perpendicular and parallel to the axis of the gate, there result $w\, \delta l$ and $w\, \delta c$, δl and $\delta \phi$ being the projections of the element $\rho\, \delta \phi$ on A P and on P Q respectively.

Hence the whole resolved parts perpendicular and parallel to the axis will be

$$\frac{w\, l}{2} \text{ and } w\, c.$$

Again, the moment of pressure on the element $\rho\, \delta \phi$, taken about the back flange at the point P is

$$w\, \rho \times \delta \phi \times \rho \sin \phi.$$

And the extreme values of ϕ are $\phi = \mathrm{O}$, and $\phi = \cos^{-1} \frac{\rho - c}{\rho}$.

Hence integrating the above expressions between these limits, for the whole moment of the water pressure,

$$\int_0^{\cos^{-1} \frac{\rho - c}{\rho}} w\, \rho^2 \sin \phi\, \delta \phi = - w\, \rho \times \overline{\rho - c} + w\, \rho^2$$
$$= w\, c\, \rho\,;$$

or, since $\qquad 2\, c\, \rho = \frac{l^2}{4} + c^2,$

moment of water pressure $= w \left(\frac{l^2}{8} + \frac{c^2}{2} \right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\mathrm{C})$

It is now practicable to form the equations of equilibrium for the half of the gate under consideration. These equations are—

(1) Resolution of forces perpendicular to axis.
(2) ,, ,, parallel to axis.
(3) Moment of forces about the back flange at the centre section of the gate, or about the point P (Fig. 2).

The resolved parts of the water pressure, of the pressure on the mitre post A B, and of the resistances at the central section P Q, have been found.

The moment of the water pressure about the required point is given in equation (C).

The moment of the web's resistance about the same point is given in equation (B).

The moment of resistance of back flange is O.
,, ,, front flange is

$$-\mathrm{B\,E}\,\frac{k-d'}{k}\times\frac{\rho}{\rho-d'}\times d'.$$

The resolved parts of the pressure on the mitre post being $\frac{w\,l}{2}$ perpendicular, and P parallel to the axis, the moments of these parts will clearly be

$$-\frac{w\,l}{2}\times\frac{l}{2}\text{ and }\mathrm{P}\,\overline{\beta+c}\text{ respectively.}$$

Those moments are here considered positive which tend to turn the gate in the same direction as the hands of a watch.

Thus the three equations, as enumerated above, are

$$(1)\quad \frac{w\,l}{2}-\frac{w\,l}{2}+\mathrm{F}=\mathrm{O};\text{ whence }\mathrm{F}=\mathrm{O}.$$

$$(2)\quad \mathrm{A\,E}+\mathrm{B\,E}\frac{k-d'}{k}\times\frac{\rho}{\rho-d'}+\frac{\mathrm{E}\,\tau\,\rho}{k}\left[d-\overline{k-\rho}\left(\log\overline{\rho-d}-\log\rho\right)\right]$$
$$-\mathrm{P}-w\,c=\mathrm{O}.$$

$$(3)\quad \mathrm{P}\,\overline{\beta+c}+w\left(\frac{l^2}{8}+\frac{c^2}{2}\right)-w\,\frac{l^2}{4}-\mathrm{B\,E}\,\frac{k-d'}{k}\,\frac{\rho\,d'}{\rho-d'}$$
$$-\frac{\mathrm{E}\,\tau\,\rho^2}{k}\left[d-\overline{k-\rho}\left(\log\overline{\rho-d}-\log\rho\right)\right]+\mathrm{E}\,\tau\,\rho\,d-\mathrm{E}\,\tau\,\rho\,\frac{d^2}{2k}=\mathrm{O}.$$

Multiplying each of these latter equations by k,

$$(4)\quad \left[\mathrm{A\,E}+\mathrm{B\,E}\,\frac{\rho}{\rho-d'}+\mathrm{E}\,\tau\,\rho\left(\log\rho-\log\overline{\rho-d}\right)-\mathrm{P}-w\,c\right]\times k$$
$$=\mathrm{B\,E}\,d'\times\frac{\rho}{\rho-d'}+\mathrm{E}\,\tau\,\rho^2\left(\log\rho-\log\overline{\rho-d}\right)-\mathrm{E}\,\tau\,\rho\,d.$$

(5)

$$\left[\mathrm{P}\overline{\beta+c} - w\overline{\frac{l^2}{8} - \frac{c^2}{2}} - \mathrm{BE}\frac{\rho d'}{\rho - d'} - \mathrm{E}\,\tau\,\rho^2 (\log\rho - \log\overline{\rho - d}) + \mathrm{E}\,\tau\,\rho\, d \right] \times k$$

$$= -\,\mathrm{B\,E}\frac{\rho\, d'^2}{\rho - d'} + \mathrm{E}\,\tau\,\rho^2 \left[d - \rho(\log\rho - \log\overline{\rho - d}) \right] + \mathrm{E}\,\tau\,\rho\frac{d^2}{2}$$

If from these two equations (4) and (5) k is eliminated, a single equation is obtained between the variable quantities, which are A, B, d, τ, and c; d' being determined by d, and ρ being a function of c.

Any four of these five being given, the equation will enable the fifth to be determined.

The next step will be to find the section required at any other point in the rib.

For this purpose it will be sufficiently close to consider the water pressure as acting everywhere at right angles to the axis of the gate. Thus if z be the distance of the point considered from the mitre end, the water pressure will be merely $w\,z$.

Then the external strains on the part of the rib between this section and the mitre post will clearly be

1. A force P acting parallel to the axis.

2. A force $\frac{w\,l}{2}$ acting at right angles to the axis, and at the mitre post.

3. A distributed pressure $w\,z$, whose centre of pressure is mid-way between the section and the mitre post.

Now let $\beta + \gamma$ be the distance of P's line of action from the back flange at this section; so that γ takes the place of c in the investigation for the central section. Then the moment of the external forces about the back flange

$$= \mathrm{P}\;\overline{\beta + \gamma} + w\,z\frac{z}{2} - w\frac{l}{2}z.$$

But by the equation to a circle (Fig. 4)

$$\mathrm{C\,O \cdot O\,D} = \mathrm{A\,O \cdot O\,B} = \mathrm{A\,O} \cdot (\mathrm{A\,B} - \mathrm{A\,O})$$

$$= \mathrm{A\,O}\,(2\sqrt{\mathrm{E\,A}^2 - \mathrm{E\,F}^2} - \mathrm{A\,O}),$$

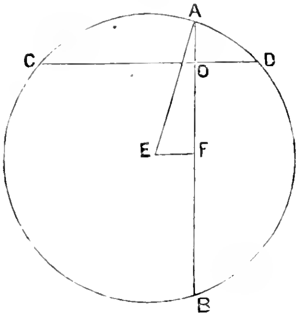
or in the case under consideration

$$z\,(l - z) = \gamma\left(2\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2} - \gamma\right).$$

But $\frac{\gamma}{\rho}$ may be neglected. Hence there may be put

$$\gamma = \frac{z(l-z)}{2\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}}.$$

Fig. 4.



And the expression for the moment becomes

$$P\beta + \frac{lz - z^2}{2}\left(\frac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}} - w\right).$$

As the amount of this moment determines the area of the front flange and web, it will be desirable to find its maxima or minima values. Taking the differential coefficient, and putting it equal to zero,

$$\left(\frac{l}{2} - z\right)\left(\frac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}} - w\right) + \frac{lz - z^2}{2} \times \frac{-P\left(\frac{l}{2} - z\right)}{\left[\rho^2 - \left(\frac{l}{2} - z\right)^2\right]^{\frac{3}{2}}} = 0,$$

or $$\left(\frac{l}{2} - z\right)\left[\frac{P \times \left(\rho^2 - \frac{l^2}{4} + lz - z^2 - \frac{lz - z^2}{2}\right)}{\left[\rho^2 - \left(\frac{l}{2} - z\right)^2\right]^{\frac{3}{2}}} - w\right] = 0.$$

It is evident that one maximum or minimum value is given by

$$\frac{l}{2} - z = 0.$$

To determine whether it is a maximum or a minimum, three cases have to be examined:

Case 1. $\left(\dfrac{P}{\sqrt{\rho^2 - \left(\frac{l}{2}\right)^2}} - w\right)$ negative.

In this case, since $\dfrac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}}$ diminishes as z increases,

it follows that

$$\frac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}} - w$$

is always negative, and increases (numerically) from $z = 0$ to $z = \frac{l}{2}$. Also $\left(\frac{lz - z^2}{2}\right)$ is always positive, and increases numerically from $z = 0$ to $z = \frac{l}{2}$. Hence the expression for the moment

$$P\beta + \frac{lz - z^2}{2}\left(\frac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}} - w\right)$$

diminishes from $z = 0$ to $z = \frac{l}{2}$; and hence when $z = \frac{l}{2}$, it is a minimum. In this case, then, the front flange and web should be strongest at the ends of the rib.

Case 2. $\left(\dfrac{P}{\rho} - w\right)$ positive.

In this case $\left(\dfrac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}}\right) - w$ is always positive, and hence the expression for the moment is a maximum where $z = \frac{l}{2}$; or the front flange and web should be strongest in the middle. This is the case of a pair of gates with very small rise, where P is consequently large; or of a gate with an undue amount of camber, where ρ is too small.

Case 3. $\left(\frac{P}{\rho} - w\right)$ negative, $\left(\frac{P}{\sqrt{\rho^2 - \left(\frac{l}{2}\right)^2}} - w\right)$ positive.

In this case the expression $\left(\frac{P}{\sqrt{\rho^2 - \left(\frac{l}{2} - z\right)^2}}\right) - w$ changes sign between $z = 0$ and $z = \frac{l}{2}$. As, however, it will always be small, it will be safe to take $P\beta$ as the constant expression for the moment.

Case (1) will be the general case, and then $P\beta$ will be the maximum value of the moment of external forces, and the equation determining the value necessary for the front flange and web at the ends of the rib will be

$$\left[S P - B E \frac{\rho d'}{\rho - d'} - E \tau \rho^2 (\log \rho - \log \overline{\rho - d}) + E \tau \rho d\right] k$$

$$= - B E \frac{\rho d'^2}{\rho - d'} + E \tau \rho^2 \left[d - \rho (\log \rho - \log \overline{\rho - d})\right] + E \tau \rho \frac{d^2}{2}.$$

And at the same time equation (4) becomes for the ends of the rib

$$\left[A E + B E \frac{\rho}{\rho - d'} + E \tau \rho (\log \rho - \log \overline{\rho - d}) - P\right] k$$

$$= B E d' \frac{\rho}{\rho - d'} + E \tau \rho^2 (\log \rho - \log \overline{\rho - d}) - E \tau \rho d.$$

The values to be given to d and d' are of course those at the ends of the rib.

Hitherto the rise of the gates and also the camber of the rib have been considered as fixed by considerations independent of the present investigation. An attempt will now be made to determine what the values of these quantities should be for a given span, in order that the quantity of iron in the rib may be as small as possible. For this purpose the central section will first be considered. Suppose the area of the front flange, and the depth and thickness of the web to be fixed by other considerations, such as the strength of the rib to resist the occasional blow of a vessel, this leaves A, the area of the back flange, as the quantity which is to be as small as possible. Recurring to equations (2) and (3), it will be seen that they may be written

$$(2') \qquad P + w\,c - A\,E = B\,E\,\frac{k - d'}{k}\,\frac{\rho}{\rho - d'} + \frac{E\,\tau\,\rho}{k}\left[d - \overline{k - \rho}\,(\log \overline{\rho - d} - \log \rho)\right].$$

$$(3') \qquad P\,\overline{\beta + c} - w\,\overline{\frac{l^2}{8} - \frac{c^2}{2}} = B\,E\,\frac{k - d'}{k}\,\frac{\rho\,d'}{\rho - d'} + \frac{E\,\tau\,\rho^2}{k}\left[d - \overline{k - \rho}\,(\log \overline{\rho - d} - \log \rho)\right] - E\,\tau\,\rho\,d + E\,\tau\,\rho\,\frac{d^2}{2\,k}.$$

In the first of these equations the right-hand side represents the whole resistance to compression of the front flange and web. In the second the right-hand side represents the moment of this compression about the back flange.

Now since the distance of any point in the section from the back flange is independent of the camber, it follows that this moment of compression cannot be increased except by an increase in the unit strain at each point. But this moment is increased if c is increased, because it is equal to $\left(P\,\overline{\beta \times c} - w\,\frac{l^2}{8} + w\,\frac{c^2}{2}\right)$.

Hence by increasing c the unit strain is increased at each point, and therefore the whole resistance to compression. Now equation (2′) shows that, by increasing this resistance, A is diminished, provided that $(P + w\,c)$ remains constant. This, however, is not the case, as $P + w\,c$ increases with c. Hence it has to be determined whether the decrease in the value of A from the increase of the resistance to compression is or is not counterbalanced by the increase in the value of A from the increase of $P + w\,c$; or, in other words, taking A as a function of c, whether its differential coefficient with respect to c is positive or negative.

To determine this question for the moment the web will be left out of calculation, and the rib considered as consisting of two flanges only, united by lattice bracing or otherwise. Any conclusion so obtained will apply *à fortiori* to the case where there is a web as well as flanges.

Omitting the web, or, in other words, making $\tau = 0$, the two equations become

$$\mathrm{P} + w\,c - \mathrm{A\,E} = \mathrm{B\,E}\,\frac{k-d'}{k}\,\frac{\rho}{\rho-d'}$$

$$\mathrm{P}\,\overline{\beta+c} - w\,\frac{l^2}{8} + w\,\frac{c^2}{2} = \mathrm{B\,E}\,\frac{k-d'}{k}\,\frac{\rho\,d'}{\rho-d'}.$$

Substituting from the second in the first,

$$\mathrm{P} + w\,c - \mathrm{A\,E} = \frac{\mathrm{P}\,\overline{\beta+c} - w\,\frac{l^2}{8} + w\,\frac{c^2}{2}}{d'}$$

Differentiating this with regard to c,

$$w - \frac{d\,\mathrm{A}}{d\,c} \times \mathrm{E} = \frac{\mathrm{P}}{d'} + \frac{w\,c}{d'},$$

or

$$\frac{d\,\mathrm{A}}{d\,c} \times \mathrm{E} = w - \frac{\mathrm{P}}{d'} - \frac{w\,c}{d'}.$$

But P, as has been shown, $= w\,\frac{l\,s}{4\,r}$;

hence

$$\frac{d\,\mathrm{A}}{d\,c} \times \mathrm{E} = w\left(1 - \frac{l\,s}{4\,r\,d'} - \frac{c}{d'}\right).$$

Now $\frac{l}{4}$ will always be much larger than d', and s will always be much larger than r; hence $\frac{d\,\mathrm{A}}{d\,c}$ will always be negative, and the larger the value of c can be made, the smaller will be the value of A. As a general conclusion, it may be assumed, that the value of c is to be as large as possible.

But it has been shown above that by increasing c the unit strain is increased on each point of the web and front flange. Hence this unit strain must be made as large as possible.

Now the unit strain at distance y from the back flange is

$$\frac{\mathrm{E}}{k} \times \overline{k-y} \times \frac{\rho}{\rho-y},$$

and the greatest value which this strain can have is E. This value will be attained by making $k = \rho$, for then the strain becomes

$$\frac{\mathrm{E}}{\rho} \times \overline{\rho-y} \times \frac{\rho}{\rho-y} = \mathrm{E}.$$

Put $k = \rho$. Then the two equations of equilibrium become

$$\mathrm{A\,E} + \mathrm{B\,E} + \mathrm{E}\,\tau\,d - \mathrm{P} - w\,c = 0$$

$$\mathrm{P}\,\overline{\beta+c} - w\,\frac{l^2}{8} - \frac{c^2}{2} - \mathrm{B\,E}\,d' - \frac{\mathrm{E}\,\tau\,d^2}{2} = 0.$$

The second equation assigns the best value to be given to the camber for any fixed value of P. Solving this equation, and substituting for P its value $w\frac{l\,s}{4\,r}$, then

$$c = -\frac{l\,s}{4\,r} + \sqrt{\frac{2}{w}\left(\mathrm{B\,E}\,d' + \frac{\mathrm{E}\,\tau\,d^2}{2}\right) + \frac{l^2\,s^2}{16\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{2\,r}}.$$

It has lastly to be determined what will be the best value to be given to P, or, since $\mathrm{P} = \frac{w\,l\,s}{4\,r}$, the best value for $\frac{s}{r}$. The first equation above shows that A will be least when $\mathrm{P} + w\,c$ is least, and hence that the best value of P will be that which makes $\mathrm{P} + w\,c$ a minimum.

Now the expression for c gives

$$\mathrm{P} + w\,c = w\frac{l\,s}{4\,r} + w\,c$$

$$= w\sqrt{\frac{2}{w}\left(\mathrm{B\,E}\,d' + \frac{\mathrm{E}\,\tau\,d^2}{2}\right) + \frac{l^2\,s^2}{16\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{2\,r}}.$$

The variable part of this expression is

$$\frac{l^2\,s^2}{16\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{2\,r},$$

which is accordingly to be made a minimum.

Expressing the quantities involved in terms of the angle which the axis of the rib makes with the span of the lock, and calling this angle α, then clearly

$$l = \frac{s}{2} \times \frac{1}{\cos\alpha} \qquad r = \frac{s}{2} \times \tan\alpha$$

$$\frac{l^2\,s^2}{16\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{2\,r}$$

$$= \frac{s^2}{4\cos^2\alpha} \times \frac{s^2}{16} \times \frac{4}{s^2\tan^2\alpha} + \frac{s^2}{16\cos^2\alpha} - \frac{s\,\beta}{2} \times \frac{s}{2\cos\alpha} \times \frac{2}{s\tan\alpha},$$

$$= \frac{s^2}{16} \times \left(\frac{1}{\sin^2\alpha} + \frac{1}{\cos^2\alpha}\right) - \frac{s\,\beta}{2} \times \frac{1}{\sin\alpha},$$

$$= \frac{s^2}{4} \times \frac{1}{\sin^2 2\,\alpha} - \frac{s\,\beta}{2} \times \frac{1}{\sin\,\alpha}.$$

The ordinary method for determining the maxima and minima of this expression leads to an equation of the fifth degree.

However, the object can be attained indirectly by assuming a value for β.

Now the depth of the rib will always be a small fraction, say one-twentieth of the span; and, since β has been taken to be two-thirds of the depth, $\beta = \frac{1}{30} \times s$. Then the expression becomes

$$\frac{s^2}{4} \times \left(\frac{1}{\sin^2 2\alpha} - \frac{1}{15 \sin \alpha}\right).$$

All the values which in practice are given to α may be taken as comprised between 10° and 40°. Comparing the values of the above expressions between these limits—

(1). $\alpha = 10^\circ$ $\sin\alpha = \cdot 174$ $\frac{1}{15 \sin \alpha} = \cdot 383$

$\sin 2\alpha = \cdot 342$ $\frac{1}{\sin^2 2\alpha} = 8 \cdot 547$

$\frac{1}{\sin^2 2\alpha} - \frac{1}{15 \sin \alpha} = 8 \cdot 164.$

(2). $\alpha = 20^\circ$ $\sin\alpha = \cdot 342$ $\frac{1}{15 \sin \alpha} = \cdot 195$

$\sin 2\alpha = \cdot 643$ $\frac{1}{\sin^2 2\alpha} = 2 \cdot 421$

$\frac{1}{\sin^2 2\alpha} - \frac{1}{15 \sin \alpha} = 2 \cdot 226.$

(3). $\alpha = 30^\circ$ $\sin\alpha = \cdot 500$ $\frac{1}{15 \sin \alpha} = \cdot 133$

$\sin 2\alpha = \cdot 866$ $\frac{1}{\sin^2 2\alpha} = 1 \cdot 333$

$\frac{1}{\sin^2 2\alpha} - \frac{1}{15 \sin \alpha} = 1 \cdot 200.$

(4). $\alpha = 40^\circ$ $\sin\alpha = \cdot 643$ $\frac{1}{15 \sin \alpha} = \cdot 104$

$\sin 2\alpha = \cdot 985$ $\frac{1}{\sin^2 2\alpha} = 1 \cdot 031$

$\frac{1}{\sin^2 2\alpha} - \frac{1}{15 \sin \alpha} = \cdot 927.$

These results show that the value of A always diminishes as that of α increases, and, therefore, that the rise ought to be as great as possible.

So far, however, only the area of the middle section of the rib has been considered. But it is obvious that the quantity which is really to be made a minimum is the whole quantity of metal in the rib, and that the increase of length caused by an increase in the rise might counterbalance the decrease so obtained in the value of A.

To obtain the accurate expression of this quantity, it would be necessary to find the value to be assigned to A at any point of the rib, and to integrate the expression so found for the whole length of the rib. It will, however, be practically sufficient to consider the area throughout as determined by the value of P. This will exactly represent the true case at the ends of the rib, and approximately everywhere else, because $w\,c$ is always small compared with P.

Assuming this, l P will clearly be an expression representing the whole quantity of metal in the rib, and only the minimum value of this quantity need, therefore, be found.

$$\text{Now } l\,\mathrm{P} = l \times \frac{w\,l\,s}{4\,r} = \frac{w\,s}{4} \times \frac{\dfrac{s^2}{4} + r^2}{r} = \frac{w\,s}{4}\left(\frac{s^2}{4\,r} + r\right).$$

Differentiating for maxima or minima,

$$-\frac{s^2}{4\,r^2} + 1 = 0 \qquad r = \frac{s}{2}.$$

It will be seen also that the second differential coefficient is positive; and hence that this value of r is a minimum, *i.e.*, the quantity of metal in the rib will be least when the rise is equal to half the span, or when the angle at the mitre is a right angle.

Thus everything appears to show that, theoretically, a pair of gates should meet at a right angle. There are, however, several practical reasons which prevent the rise from being made as great as is requisite for this.

1. The greater the rise the more oblique is the thrust on the hollow quoin with respect to the side wall of the lock, and, therefore, the more difficult to meet.

2. The longer the gate the weaker it is to withstand the strain of opening and shutting, and any blows, &c., which it may receive when home in the recess.

3. The less the value of P the greater will be the value of c necessary to bring the web and front flange into uniform compression. But a large camber entails a great depth of recess, which is inconvenient; and, moreover, in ordinary cases, long

before P attains its minimum value, the necessary camber will cause the two gates to assume the cylindrical form, beyond which they clearly must not be allowed to go.

The point at which this limit is reached will depend upon the value given to the areas of the front flange and web, since the value of c depends on these quantities.

By calculation in the case under discussion, in which, however, the web and front flange were lighter than usual, the limit appears to be reached when the angle α was a little less than 30°. This points, therefore, to an angle of 25°, which is about the value given in the best modern construction as being not far from the truth.

Closing here the general investigation, and proceeding to consider results, there will be taken as an example one of the lower ribs of a gate actually designed for a lock now under construction, and it will be shown how to calculate its dimensions by the present method.

In this case the water pressure per lineal inch of the rib, or the value of w, was 0·15 ton; and the maximum strain allowed was 2·5 tons per square inch.

The length of the gate was 417 inches.

The angle α was about 23°, and was such that $\frac{l\,s}{4\,r}$ was equal to 516.

The value of the front flange, or B, was 8·6 square inches, being composed of a bar 4 inches by $\frac{3}{4}$ inch, and an angle iron $4\frac{1}{2}$ inches by $4\frac{1}{2}$ inches by $\frac{5}{8}$ inch.

The value of d was about 29·5 inches.

The value of d' was about 28·25 inches.

The value of β was assumed to be 13 inches.

The thickness of web, or the value of τ, was $\frac{1}{2}$ inch.

From these quantities the proper camber is to be found from the equation

$$c = -\frac{l\,s}{4\,r} + \sqrt{\frac{2}{w}\left(\mathrm{B\,E}\,d' + \frac{\mathrm{E}\,\tau\,d^2}{2}\right) + \frac{l^2\,s^2}{16\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{2\,r}}.$$

Substituting the above values, it will be found that

$$c = -\,516 + \sqrt{\frac{2}{\cdot 15}\left(8{\cdot}6 \times 2{\cdot}5 \times 28{\cdot}25 + \frac{2{\cdot}5 \times \frac{1}{2} \times (29{\cdot}5)^2}{2}\right) + (516)^2 + \frac{(417)^2}{4} - 2 \times 516 \times 13}$$

$$= 42{\cdot}2 \text{ inches}.$$

Taking this for the camber, the value of A, the back flange, is given from the equation

$$A\,E + B\,E + E\,\tau\,d - P - w\,c = 0\,;$$

or substituting the present values,

$$A = \frac{\cdot 15\,(516 + 42\cdot 2)}{2\cdot 5} - 8\cdot 6 - 29\cdot 5 \times \frac{1}{2}$$

$$= 10\cdot 14 \text{ square inches.}$$

This determines the section at the middle of the rib. Since, however, as has been shown, the external forces have a tendency to increase towards the ends of the rib, it will be desirable to calculate the areas necessary at each end.

For this purpose it will be assumed that the strain at the end section is uniform throughout, and is equal to 2·5 tons per square inch; and thence will be derived the areas corresponding to such a condition.

The equations for this case are

$$A\,E + B\,E + E\,\tau\,d - P = 0$$

$$P\,\beta - B\,E\,d' - \frac{E\,\tau\,d^2}{2} = 0.$$

The value of d, in the example chosen, was here 21 inches, and that of d' 20 inches. The remaining quantities the same. Then the second equation is (since $P = \cdot 15 \times 516 = 77\cdot 4$)

$$77\cdot 4 \times 13 - B \times 2\cdot 5 \times 20 - \frac{2\cdot 5 \times \frac{1}{2} \times (21)^2}{2} = 0,$$

or

$$1006\cdot 2 - B \times 50 - 275\cdot 6 = 0\,;$$

whence B = 14·6 square inches.

Substituting this value in the first equation, it becomes

$$A \times 2\,5 + 14\cdot 6 \times 2\cdot 5 + 2\cdot 5 \times \frac{1}{2} \times 21 - 77\cdot 4 = 0\,;$$

whence A = 5·86 square inches.

It appears, therefore, that to obtain the most uniform distribution of strain, it is necessary to add considerably to the strength of the front flange towards the ends of the rib, diminishing that of the back flange by nearly the same amount.

Having thus fixed the dimensions of the lowest and most important rib, those above will be discussed.

Now, recurring to the expression giving the best value of c, viz.,

$$c = -\frac{l\,s}{4\,r} + \sqrt{\frac{2}{w}\left(\mathrm{B\,E}\,d' + \frac{\mathrm{E}\,\tau\,d^2}{2}\right) + \frac{l^2\,s^2}{16\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{2\,r}},$$

it appears that c will vary with w, increasing as w decreases. It will, therefore, be especially advantageous to space the ribs so that the pressure on each may be as nearly as possible the same, in which case the dimensions of the rib will, of course, be the same also.

Hitherto the rib has throughout been considered as a plate girder. Supposing it to have lattice bracing, the same equations will apply, leaving out all connected with the web, or, in other words, making $\tau = 0$. The two equations will then be:

To determine c, $c = -\frac{l\,s}{4\,r} + \sqrt{\frac{2}{w}\mathrm{B\,E}\,d' + \frac{l^2\,s^2}{4\,r^2} + \frac{l^2}{4} - \frac{l\,s\,\beta}{r}}.$

To determine A, $\quad \mathrm{A\,E} + \mathrm{B\,E} = \mathrm{P} + w\,c.$

Again, supposing the section to consist only of an oblong, *i.e.* suppose the gate to be a wooden gate. Here there are two cases in practice:

1. Where the rib is made of whole timbers; 2. Where the rib is built up of shorter balks, with a post in the middle of its length.

In the first case it will be impossible to give the proper camber, unless the timber be bent. In the second case it will be best to take each half of the rib as a separate rib, and the two halves as forming in themselves a pair of gates with small rise. They will then be reduced to Case 1. And here it will be safe to consider the camber as nothing, except in estimating the value of $\overline{\beta + c}$.

But if $c = 0$, $\rho = \infty$.

$$\rho\,(\log \rho - \log \overline{\rho - d}) = \rho\left(\frac{d}{\rho} + \frac{1}{2}\left(\frac{d}{\rho}\right)^2 + \frac{1}{3}\left(\frac{d}{\rho}\right)^3 + \&\text{c.}\right)$$
$$= d + \frac{d^2}{2\,\rho} + \frac{d^3}{3\,\rho^2} + \&\text{c.}$$
$$= d.$$

Hence the two general equations (4) and (5) become, putting $\mathrm{A} = 0$, $\mathrm{B} = 0$, $c = 0$,

$$(4).\quad (\mathrm{E}\,\tau\,d - \mathrm{P})\,k = \mathrm{E}\,\tau\,\rho\left(d + \frac{d^2}{2\,\rho} + \&\text{c.}\right) - \mathrm{E}\,\tau\,\rho\,d = \mathrm{E}\,\tau\frac{d^2}{2}.$$

$$(5).\quad \left(\mathrm{P}\,\beta - \frac{w\,l^2}{8} - \frac{\mathrm{E}\,\tau\,d^2}{2}\right)k = -\,\mathrm{E}\,\tau\,\rho^2\left(\frac{d^2}{2\,\rho} + \frac{d^3}{3\,\rho^2} + \ \&\text{c.} + \mathrm{E}\tau\rho\frac{d^2}{2}\right.$$
$$= -\,\mathrm{E}\,\tau\,\frac{d}{3}.$$

Or eliminating k,

$$\mathrm{P}\,\beta - \frac{w\,l^2}{8} - \frac{\mathrm{E}\,\tau\,d^2}{2} = -\frac{2}{3}d\,(\mathrm{E}\,\tau\,d - \mathrm{P}),$$

or

$$\frac{\mathrm{E}\,\tau\,d^2}{6} = \frac{2\,\mathrm{P}\,d}{3} + \frac{w\,l^2}{8} - \mathrm{P}\,\beta.$$

This equation will give τ, the thickness of the rib required.

In conclusion, it may be remarked that the quantity of metal used in existing lock gates appears by the present investigation to be much beyond what is necessary. In the example given above, the pressure was that due to a head of 28 feet of water, and the distance between the ribs was more than 2 feet. Yet the area required for the back flange comes out to be only 10 square inches. A continuous skin, $\frac{1}{2}$ inch thick, would itself give an area of 12 square inches, and this is the least thickness that could possibly be admitted. This shows a large waste of material in the double skinned gates which are now so common; and this, combined with the acknowledged difficulty of keeping the gates permanently watertight, seems to form a strong objection against their use. On the other hand, wooden gates are liable to more rapid decay, and cannot be given the requisite amount of camber. Thus the form of gate, which so far would seem to have the preference, is that adopted, for example, in the Bute docks at Cardiff, viz., a wooden skin supported by plate girders at intervals, and strengthened by vertical and diagonal ribs. Here a leak is at once detected, and can easily be repaired, either by fresh caulking, or by cutting out and replacing a plank; whilst the dimensions and position of the girders can be exactly adjusted to the requirements of theory and practice.

[Mr. G. H. Phipps.

Mr. G. H. Phipps said, the chief object of the Paper appeared to be, to point out a manner of ascertaining the degree to which lock-gates, either straight or of various amounts of curvature, were subject to the effects of transverse strain. He thought that Mr. Barlow's Paper, to which the Author had referred, although apparently erroneous in the views expressed relative to the combined effects of transverse strain, and pressure applied endways, on balks of timber, had the merit of being one of the earliest, if not the very first, in its advocacy of gates of considerably greater curvature than had been previously constructed. Such an increase in the curvature, it was obvious, could not well be carried into execution while timber remained the only material at command; but this difficulty vanished so soon as iron came so largely into use.

Of the gates of the Victoria Docks, described by Mr. Kingsbury, which were of iron, and of very considerable curvature, the Author seemed to apprehend that such gates might, by flexure, throw the strain away from the true axes of the gates, where they met at the mitre-posts, and so give rise to an unequal pressure on the two boiler-plate skins of the gates. But the axes of such gates, when properly set out, formed a continuous circular arc, drawn through the centres of the heel-posts and the centre of the gate's thickness at the mitre-posts; and, consequently, were subject to no other strain than direct compression of the material. The quantity by which the compression shortened each gate would indeed tend to a slight 'nipping' of the gates where they met, but rather at the outer extremity than at the inner one, as supposed by the Author. However, the difficulty, if it existed, could be easily got over by slightly rounding the touching surfaces of the gates.

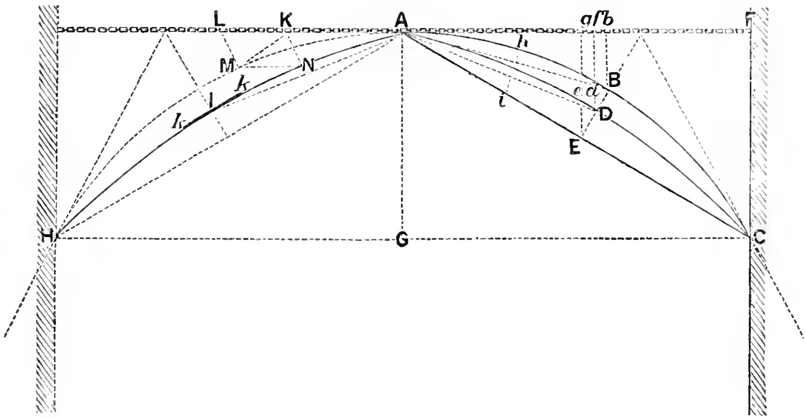
Several important advantages appeared to be possessed by curved gates over straight ones; amongst others, that they contained far less material,—that by absence of flexure they were more likely to fit closely to the cill,—and that the curved figure enabled the roller to be placed upon the straight line passing through the centre of the heel-post and the gate's centre of gravity, thereby avoiding any tendency of the gate to cant over.

Although not referred to in the Paper, he would observe that the floating power obtained by forming lock-gates of boiler-plate made watertight appeared so great an advantage, that the objections, as regarded the difficulty of maintaining them watertight, ought to be critically examined before so great an advantage (particularly for heavy gates) was given up.

He would now offer a few observations upon the various strains upon lock-gates, with a method of determining them, founded upon simple geometry and statics, which he thought fully competent to give all the useful information required; but at the same time he desired to give the Author of the Paper full credit for his able use of the higher mathematics.

Let H G C, Fig. 5, be the span of the lock, and H A B C a continuous circular curve: and let A B C, A D C, and A E C, represent

Fig. 5.



the axial lines of three different outlines of gates; the first being the truly cylindrical; the second, a flatter circular curve; and the third, a flat gate.

Let the gate H A be removed, and the other gate A B C kept from turning on the centre C by a chain A F, lying at a tangent to the curve at A.

Calling W the total pressure of the water at right angles to A C, acting at the middle of its length, and T the tension upon the chain A F, T would be $= \frac{W \times C E}{C F}$. Next, let the dotted line A B be the chord of half the arc A B C, and B D E the arc of a circle drawn with the radius A B, cutting the two inner gates at D and E. From these lines being all of equal length, each had the same water pressure to sustain; and hence they all had the same turning moment around the respective points B, D, and E; but the chord A B was kept from turning around B, by the force T into the lever B *b*, which, therefore, equalled the previous turning moment W × C E. On the two inner gates, therefore, the moments T, into *f* D and *g* E, respectively, were too great by the distances *d* D and *e* E.

which accordingly, *i.e.*, T × *d* D, and T × *e* E, represented the bending moments at the above points of the two gates. The same method would also evidently apply on any other arc, *h i*.

When the gate was straight, as A E C, the pressure, endways of the material, would be at all points equal, and would bear the same proportion to the uniform pressure upon the cylindrical gate (equal to the tension T) as the radius to the cosine of the angle F A C.

When the gate was of some curvature A I H, intermediate between the cylindrical and the straight, the pressure at right angles to any given section, at I, must be obtained by first finding the direction and magnitude of the resultant K M, due to the conjoint action K N of the water upon the chord A I, and the tension T = K L, and then reducing the force of the resultant in the ratio of radius to cosine of the angle between the resultant and a tangent *k k* to the curve at the section I in question.

Having now obtained the bending moment of any section, and also the direct pressure upon it endways, the total strain upon any fibre of the material was obtained by adding the compression due to transverse strain to the direct compression, at the one extremity of the section, and by adding the tension due to transverse strain, as a minus quantity, to the direct compression, for the total strain at the other extremity of the section; which would be either compression or tension, according to whether the plus or minus quantity preponderated.

Mr. Bramwell said he would endeavour to explain the conclusions to which he had arrived in considering the question, with the aid of the few simple rules that seemed to him sufficient for the purpose. The Author had alluded to the one or two previous communications on the same subject; especially to a Paper by Mr. P. W. Barlow, which he condemned as containing the erroneous statement, that the line of pressure at the mitre or shutting-post would always be a tangent to the curve of the separate gates, whatever that curve might be. No doubt that was an error, as the line of pressure must always be at right angles to the centre line of the lock, and thus could only be a tangent to the curves of the gates when they were such that the two gates, on being closed, formed a segment of a circle; or, at all events, that their junction at the shutting or mitre-post formed, at that point, part of some continuous curve.

Passing next to the Author's views, he seemed to urge that when gates were closed, they were deflected under the pressure, and thus the junction at the mitre or shutting-post was disturbed, so that the pressure was not uniformly distributed, but was thrown towards the front of the gate; that it was necessary to take this fact into account in calculating the strains upon the gates; and that when taken into account, it would be found there were differences of strength required between the back and front flanges of the ribs of the gates, and that it would be a better construction to give such a curvature to the gates, as that, when closed, they should assume the form of a Gothic arch, and not the form of a segmental arch.

Mr. Phipps had well remarked that in practice the mitre-posts were ordinarily bevelled at the edges; and that thus the pressure was, under any circumstances, preserved in the centre line of the gate. Mr. Bramwell, however, thought that when gates were of the proper form, there would be no deflection whatever; and that this proper form must be one which, when they were closed, made a segment of a circle; and he based this opinion on the following reasons. Suppose it were required to construct a self-sustaining, that was an unstrutted, dam, in a lake to support fluid pressure; it could not be doubted but what the plain circular form Fig. 6 would be the one adopted. No one would think of adopting a hexagonal structure with curved sides, such as Fig. 7. Again, suppose it

Fig. 6.

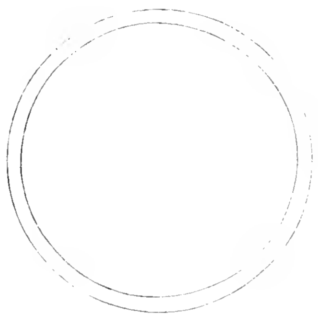
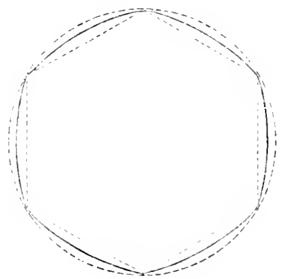
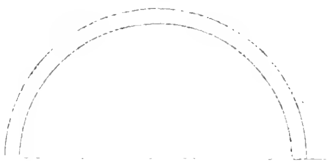


Fig. 7.



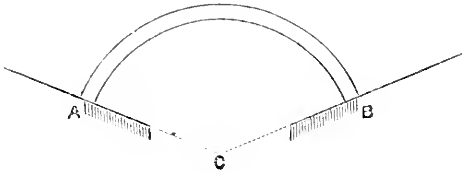
were required to make a self-sustaining dam against the face of a flat wall; it could not be doubted that the semicircular form of Fig. 8 would be the one adopted. And supposing it were desired

Fig. 8.



to make a self-sustaining dam against two flat walls, inclined to one another as A B, Fig. 9, the form selected would be that

Fig. 9.



of the segment of a circle, having for its centre the point at which the faces of these two walls, if produced towards each other, would meet in C. Now if this were true of a self-sustaining, *i.e.*, an unstrutted, dam, it should be equally true of a lock-gate made in one leaf, having its hinge-post at A, and its shutting-post at B, Fig. 9. For such a gate, when closed, was a portion of a self-sustaining dam. But if true of a lock-gate when made in a single leaf, it was equally true of lock-gates when made in a pair of leaves, with their hinge-posts at A and B, and with their mitre, or more properly, shutting-posts at the centre between the two. It was manifest that the whole of any cylindrical dam was subject simply to a uniform compressive strain, and except that this uniform strain was one of unstable equilibrium, it would be possible to make such a dam by the mere abutment of contiguous piles, like the staves of a cask. Now the junction of the two shutting-posts in a pair of gates was no more important than the junction of any other two staves round about the dam or gates, and was no more likely to be subjected to an alteration in the point of pressure than the junction of any other two staves in the dam or gate. He should have thought the foregoing propositions self-evident and indisputable; and that it was equally indisputable that gates, forming, when closed, a segment of a circle, could not be subjected to transverse strain, and therefore not to any liability to distortion which transverse strains alone could produce; and that it was clear that the whole of the material in the gates would be in compression. He had been at the pains of finding what was the curvature of the gates which the Author had brought forward as an instance in which the calculations had been employed. In the Paper these gates were stated to have such a rise that $\frac{l s}{4 r} = 516$, the length of each gate being 417 inches; and also to have a versed sine to each gate of 42·2 inches. He had fully expected to find that these gates, when closed, would form a Gothic arch; but, to his surprise, he found that they formed a segment of a circle. Thus it appeared that the Author, in the instance brought forward, had adopted the segmental curve, while apparently advocating in the Paper the view that that curve was not the best. But while the Author adopted the segmental curve, calculations were nevertheless given, based upon the suggestion that there would be flexure of the gates under pressure, and that the point of pressure at the mitre-post would be shifted; and thus the conclusion was arrived at that there must be varying proportions of metal in the ribs of the gates. Further it was suggested that this was done because of the variations of strain in different parts of the gate, although that gate, when closed, formed with its neighbour a true segment of a circle. According to Mr. Bramwell's already expressed views, such a proportioning of the metal must be erroneous and injurious; as in every pair of gates forming a segment of a circle when shut the pressure must be uniform throughout the segment.

As to what must be the radius of a pair of segmental gates in relation to the width between the centres of the heel-posts, in order to obtain the minimum of metal, he might state that the quantity of metal would clearly depend upon the girth of the gates, multiplied into the requisite sectional area; and the least metal required would be when this product was a minimum. Now the girth would vary with the radius multiplied into the angle, the pressure would vary directly as the radius, and the quantity of metal would therefore vary as the angle into the square of the radius; and this quantity would vary as the area of the sectors, and could be represented by them. In Fig. 10 he had assumed a pair of gates having half the width of the lock for their radius, and therefore a sector of 180°. Calling this radius unity, the area of such a sector would be 1·5708.

In Fig. 11 he had shown a pair of gates subtending an angle of 90°, and having therefore a radius of $\sqrt{2}$. It was manifest that a 90° sector of a circle of a radius of $\sqrt{2}$ would be equal in area to a sector of 180° of a circle of a radius of 1, and that therefore the amount of metal would be the same in the gates of Fig. 10 and in those of Fig. 11. The minimum amount must be somewhere between these two sectors of 180° and of 90°, and it would be found to be, as shown in Fig. 12, 133·56°, as with this angle the square of the radius in the degrees was a minimum. Such an angle gave a rise of gate of ·65916, and for the chords of the gates, angles at the heel-posts of 33°·39, and at the mitre-posts of 113°·22;

and these proportions and form of gate he believed to be the most

Fig. 10.

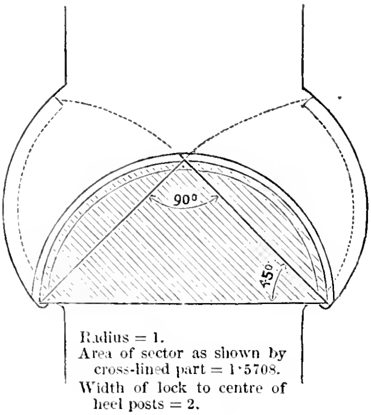
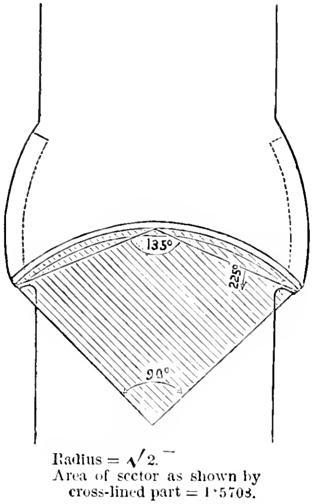


Fig. 11.



economical of any. Moreover, they did not require inconveniently deep recesses to draw into when open, nor did they possess, so far as he knew, any other practical objection. It would be found that while Figs. 10 and 11 required each a unit of metal, Fig. 12 would require ·8785 of metal. From this it appeared that there was not

Fig. 12.

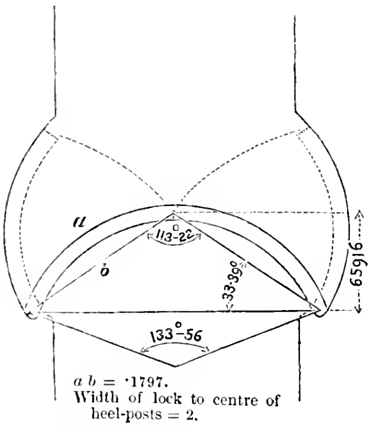
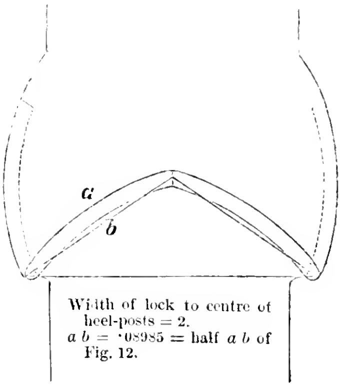


Fig. 13.



so very much difference between the metal of the most economic form of Fig. 12 and the metal of the very different forms of Figs. 10 and 11; and that thus if for any reason an engineer wished to have gates subtending either a greater or a less number of degrees than Fig. 12, he might do so without material increase in the amount of the metal. This was true so long as the gates when shut formed a segment of a circle, but directly this form was departed from, and curves were used for the individual gates which formed a Gothic arch when the gates were closed, the increase of metal necessary to resist the cross strain tending to produce flexure became considerable.

For example, take a pair of gates as in Fig. 13, with girders, say ·06 wide and having the same central rise as those of Fig. 12, but with each gate made with a flatter curve, so that the rise of each gate at *a b* was only ·08985 instead of ·1797 as in Fig. 12. There would then come, in lieu of a compressive strain of $\frac{1}{2}$ on the inner and $\frac{1}{2}$ on the outer flanges of the girders as in the girders of the gates Fig. 12, a compressive strain of 1·2809 on the back, and a tensile strain of ·283 on the front flange, so that ·5639 part of metal would be required in lieu of the one part of Fig. 12.

He had only one other remark to make, and that was upon the statement by the Author, that although gates which subtended an angle of 180° were clearly the most economic, a statement which he trusted he had disproved, they had various practical disadvantages, and among them that they threw an oblique strain on the masonry. Now in this latter view he could not coincide. Gates which formed a semicircle when shut could not throw an oblique strain on the masonry; on the contrary, the strain must be exactly parallel with the centre line of the lock. Not only was this true with gates which formed a semicircle when shut, but it was true of all gates which subtended an angle of 180°, or which, in other words, made an angle of 90° at the mitre-posts whatever the curvature of those gates might be, or even if they were quite flat. Let Fig. 14 represent a square dam in a lake, the sides being sufficiently stiff to bear the pressure. Then such a dam would clearly not have any tendency to alter its form, either into that of Fig. 15 or that of Fig. 16; but the sides would be in equilibrium and the square shape would remain unchanged. But this must be equally true if instead of a whole dam the half of it were taken, as in Fig. 17; that was to say, such a figure would have no more tendency for the apex A to lower, and for the feet B and C to spread, than it would have for the apex A to rise, and the feet B and C to come together. He thought this was evident from these diagrams; but if not, a simple calculation of the pressure on such a pair of gates would prove that their heels had neither any

Fig. 14.

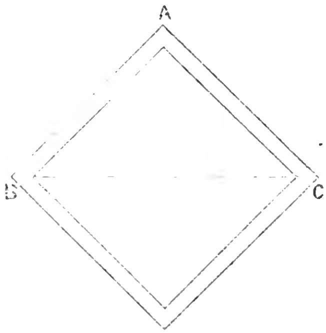


Fig. 15.

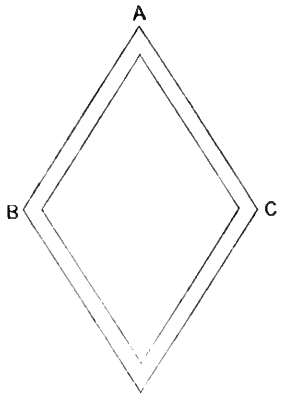


Fig. 16.

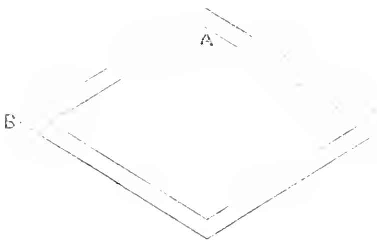
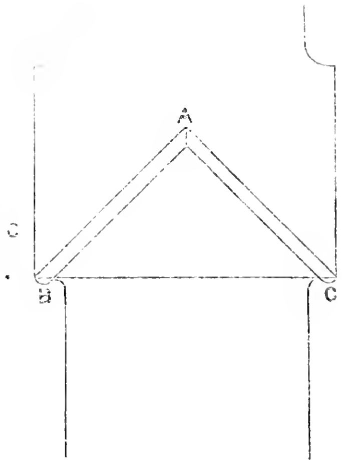


Fig. 17.



tendency to spread nor to come together, and that thus the strain they put on the masonry, so far from being oblique, must be parallel to the centre line of the lock.

Mr. G. J. Morrison said he thought that the Author of the Paper, in speaking of circular gates, did not attempt to show that the pressure upon them, if they were very thin, would be anything but simple compression; but that when the thickness of the gates was considerable, the pressure throughout the whole of the section might not be equally distributed, and with that he agreed. A cylinder subjected to pressure, at right angles to the surface at every point, must be subjected to uniform compression if the thickness of the cylinder was small, but to a different amount on the outside and on the inside if the thickness was considerable. It was found that large guns (which were thick cylinders subjected to pressure from within) did not increase in strength when the metal passed a certain limit; which showed that the strain was not uniformly distributed through the metal. He thought that where lock-gates were composed of horizontal girders with back and front flanges, the conclusion of the Author was true, viz., that the strains on the front and back flanges were not the same, even when the gates were circular. True he made an assumption with regard to the pressures at the mitre-post which might not be strictly correct, but which if not correct was approximately true, and the error, if any existed, would only slightly alter the exact amount of strain he arrived at.

With regard to the example which the Author had given at the end of the Paper, although he arrived at the conclusion that the camber to be given to the gates was such as to make them circular, yet he thought it was only accidental. The Author had assumed a certain value for the front flange of the gates, and from that had calculated the proper camber for the gate, to get equal strain throughout, and the result was a circle; but this system of calculation would not invariably give a circle. Mr. Bramwell had spoken about the building of a cofferdam where a circular form would be adopted, and he considered a gate was simply part of a cofferdam; but the Author took for granted that a certain size must be adopted for the front flange of the gate, otherwise it would be injured by blows from vessels, &c., and if the size of the front flange was fixed, it did not then follow that a circle was the best shape of gate. If a considerable quantity of metal had to be put into a gate to resist blows from vessels, it might be more economical to adopt a straight gate, which would be shorter than a circular one, and so take full advantage of the metal which had to be put in for other reasons.

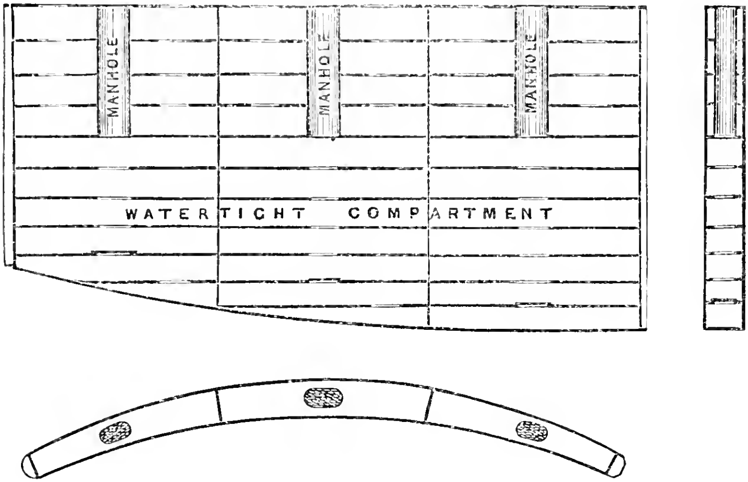
The thrust on the side walls in a semicircular gate would be in a direction parallel to the centre line of the lock; but that was inconvenient where the gates were required near the outside of the lock, for it might be undesirable to put a great amount of masonry in front of the gates; therefore it was often advisable to direct the thrust, if possible, across the wall where an abutment could be obtained.

In the example given by the Author, a certain size of gate had been assumed, with a flange of $8\frac{1}{2}$ inches for the front, and the back flange had been calculated at 10 inches, so that the pressure might be the same throughout. Now, if each of the flanges were $9\frac{1}{4}$ inches, there would be the same amount of metal, and there would be only 2·6 tons pressure per square inch in one case, and 2·3 tons pressure per square inch in the other. He believed the Author was the first to call attention to the different strains on the back and front flanges of lock gates, but the Paper showed that the difference, in ordinary cases, was so slight that it might practically be disregarded.

Mr. F. W. Sheilds said that a question had arisen as to the principle on which gates of this kind should be designed, upon which there seemed to be considerable difference of opinion. He thought it was erroneous to compare a structure of this kind to a circular cylinder; for in a lock gate, just as in the arch of a bridge resting on a solid abutment, the supports were upon two immovable points, viz., in the case of the arch on the abutments supporting the springings, and in the case of the lock gate on the hollow quoins supporting the heel-posts; whereas in a cylinder there were no such fixed points of support, but the whole circular structure was in equilibrium throughout. For that reason it might be assumed that there would be deflection in a lock gate, just as in an arch, and that the centre of a gate would bulge in or bulge out under pressure as an arch not in equilibrium would do. The question also arose as to the rounding or making square of the meeting points of the mitre-posts. He thought it better to make the mitre-posts meet at the outside skin, where the water-pressure came directly upon the gate, than at any other point, such as midway upon the gate, which would occur when the mitre-posts were rounded as Mr. Phipps described; because if the mitre-posts met at the outside skin, the arch of the gate would be continuous at the point of meeting, and the gate would be in a better position to resist the strain upon it. The rounding might be further justified for this reason—that the heel-post was round, and, if one was rounded, the other should be. But in that respect the heel-post had the advantage of acting in a rounded bearing, viz., the hollow quoin; whereas the other had nothing of the kind, and the indirect bearing he spoke of was in full force. He therefore thought the Author was theoretically right on that subject. But the Author had also said that the mathematical conditions of a structure of this kind were different from those of an arch or girder. He was not able during the reading of the Paper to follow the mathematical reasoning sufficiently to say whether that was really the case. It might be so, but in his own practice he had followed the common arch and girder rules; and, unless there was good reason to the contrary, he was an advocate for great simplicity in engineering mathematics; and his own impression was the ordinary arch and girder rules were sufficient for the practical designing of works of this nature.

Mr. HARRISON, Vice-President, remarked that no doubt, if dock gates were always subjected to such conditions as had been assumed, a constant and equable pressure might be obtained with a perfect portion of a circle; but in practice that was not always the case. He had met with a variety of causes which had produced a transverse strain upon dock gates. Sometimes a piece of chain was wedged in between the bottom of the gates and the sill, or a piece of wood got jammed in at half tide level so that it could not be moved; and there were other similar accidents to which the gates were liable, and in all these cases they were subjected not only to transverse strain in every direction, but they could not meet except by being twisted, and they would at once act on girders. Though advocating the principle of the circular form of gate for the outer skin, which he believed to be perfectly right so far as it went, yet he had adopted for practical reasons a form different to that of having the outer and inner skins parallel. One reason why he had adopted different forms was that the nearer the sill approximated to a straight line the better the gates would fit; at the same time he had found he could make a circular sill sufficiently water-tight. Then, again, he believed that in iron gates it was desirable to adopt the system of flotation. Fourteen years ago he had constructed a pair of iron gates for a lock 80 feet in width (Figs. 18); in this case the heel-post was raised considerably above the sill. When these gates were put in, they were made water-tight from top to bottom; consequently when there was a high tide it was found necessary to let in a certain amount of water to prevent them floating away, and there was a depth of 6 feet to 8 feet of water in the gates. A large valve let the water out, but practically it was not always attended to; and, as there was a heavy weight thrown upon the roller paths, they were worn away. To avoid these objections, he had adopted the following plan with perfect success:—He had limited the water-tight compartment to the part marked in Figs. 18, letting the water flow in and out of the other compartments. A water-tight tube was constructed sufficiently large to admit of a ladder being put in, by which a man could descend to any part of the gate at any time of tide for the purpose of examination.

Figs. 18.



This water-tight compartment was made so large that, when the water reached the upper part of it, it floated the gate within about 5 tons, so that the gate could never float away; and having, as shown on the plan, a larger buoyant power towards the outer end, the gate had, as nearly as possible, its weight divided between the roller and the heel-post. He was satisfied that this was one of the best principles that could be adopted for iron gates.

Gates were liable to vessels bumping against them, and they were exposed to a variety of accidents. To make a gate water-tight there must be a certain thickness of iron, which would bear caulking; and calculation based on the mere assumption that the only thing to be provided for was a uniform state of equal pressure would give a less thickness of skin than it was desirable in some cases to put in. The angle at which these gates shut was as nearly as possible 24°. When closed the outer skin made a complete circle.

Mr. J. Coode said, some remarks having been made in the earlier part of the discussion with regard to the advantages of continuity of skin in lock gates, he must observe that if continuity of skin were an advantage, he was unable to recognise it, and that he regarded it rather as a drawback. It was very properly stated that in the case of all circular gates there was a tendency, when the pressure came upon them, to 'nip' at the crown or extrados of the arch: that being so, if the skin were continuous, the greater part of the strain would fall upon the skin, which was the weakest part of the gate. To his mind the office and functions of the skin and of the frame of a lock gate were as distinct and as different from each other as the functions of the skin and of the frame of the human body. The rule in practice was to cut away a portion of the meeting posts at the salient angle which they formed when closed—in fact, to chamfer them off in some degree; and the effect of this was to prevent that very continuity which was contended for. Generally the amount of chamfer was from about one-fourth to one-third of the breadth of the mitre or meeting posts, and by that means instead of bringing the principal strain on the mitre-post at an acute angle, it was brought at an obtuse angle on the face of the mitre-post.

He would take this opportunity of referring to an arrangement which he never saw or heard of till a few months since: that was a means of resisting the very severe strain which gates were subjected to when struck by waves, even small waves, when the water level was nearly the same on both faces. The arrangement was devised by Mr. Ramsey, resident Engineer at Ramsgate harbour, and had proved most successful. In the case of one of the pairs of gates of the floating basin in that harbour, the evil was so great that the masonry became seriously dislocated, solely by the concussion of the gates caused by their being struck by the waves coming in at the harbour entrance, when the tide on the outside of the gates was nearly level with the water in the basin. The question arose as to what was to be done to prevent the recurrence of this mischief? Under these circumstances Mr. Ramsey devised the following arrangement:—One end of a stout beam or stay of greenheart timber was connected by means of a massive iron moveable joint, to the fore part of each gate, near the mitre-post, on the inner or basin side, and at about the level of high water of equinoctial spring tides; the opposite end of this beam passed into an opening in the masonry of the side walls of the entrance. The inner end of this beam was supported by a small bogie truck running upon a pair of rails laid in a channel under the ground, the object of this being to support the inner end of the beam, and admit of its ready motion in a horizontal plane. On the vertical face of the beam, nearest the gate, there was a strong toothed rack, working into a pinion at the end of a train of gearing, the first motion of which was a screw-and-worm wheel. By means of this gearing the power of the men was communicated to the gate through the beam or stay just described; the action was so perfect that the gate would remain stationary in any intermediate position if the gearing were let go, even when there was a considerable amount of disturbance in the harbour. This was a simple contrivance, and the men connected with the working of these gates, whose lives had frequently been in jeopardy under the old arrangement, had the utmost confidence in it, though at times the gates were opened and closed when there was a considerable amount of undulation in the harbour. The span of the gates was about 50 feet; the shock from the waves was occasionally so great, previous to the adoption of the arrangement described, that the masonry of the entrance had to be entirely taken down and rebuilt. The manipulation of the gates was so simple and easy by means of the gearing and stays, that the men preferred these on all occasions, and had abandoned the original ordinary gearing and chains, which still remained available if desired. The arrangement described would in his opinion entirely obviate the necessity of sea or storm gates.

Mr. W. R. Browne, in reply upon the discussion, said the chief point to which he would address himself was as to the cylindrical or circular gates, bearing the form of an arc of a circle. It had been argued that this form was the best that could be given to lock gates; but he could not assent to that doctrine. The principle which all those who advocated that form had rested upon was this—that in the case of a cylinder pressed by a uniform pressure outside, the pressure produced upon the cylinder, and tangential to the cylinder, was a certain simple pressure uniform everywhere: but there was a difference, which had not been remarked, between the case of a cylinder and the case of a pair of lock gates.

He would take the case of Fig. 6, exhibited and commented upon by Mr. Bramwell. When a cylinder like a cofferdam was pressed by external pressure, the form of the cylinder did not alter; but it was not true that the diameter of the cylinder did not alter, because a certain pressure acted throughout the circumference of it, and pressure always produced contraction. The whole cylinder would contract; it would remain a perfect circle, but it would be of smaller diameter than before, as was the case with an india-rubber ring placed upon an elastic tube, where the diameter was smaller though it remained a circle. But in the case of lock gates resting upon abutments, while the gates yielded to the pressure of the water, the abutments did not yield; and instead of their being in the condition of a cylinder, they were in the condition of an arch, Besides, in the case of a cylinder, the pressure would not be uniform throughout the whole of its section; the pressure on the outside of the cylinder would not be the same as on the inside; and it might happen that there was compression at the outside fibre, and tension at the inside fibre of the cylinder; and the strains would not be confined to the safe working limit if there was just enough iron to support the pressure upon the cylinder if equally distributed. For a proof of the fact that the pressure was not uniform all over the section, he referred to Rankine's "Applied Mechanics," art. 273; but it might be seen thus:—The pressure came first upon the outside fibre. It was only by the yielding of the outside that the strain was brought upon the next one, and that strain was less than that on the outside fibre by the amount of the resistance offered by that fibre in yielding. It was the same with the second and third fibres, and so on, until at last a point was reached at which there was no pressure whatever.

Mr. Bramwell said the examples given in the Paper proved his case, because he discovered that the gate which Mr. Browne thought the best form came out to be a cylindrical gate. That was true in the case of the particular gates for which calculations were given, but it was not to be argued from this that the cylindrical was always the best form of gate. It was the result of design. The fact was, when designing those gates the engineer wished to have the advantage—if advantage it was—of the cylindrical form; and had therefore requested him to make the quantities such as would cause the cylindrical form to come out the best in calculation, and a few of the quantities which could be altered had been manipulated, so that that was the case. When he wrote the Paper he did not take the trouble of new calculations, but copied what he had already done; and he had not imagined that the persistent energy of Mr. Bramwell would have seen through his calculation, and found that the gate happened to be of a cylindrical form.

With regard to Mr. Harrison's remarks as to floating gates, he would say, whatever the advantages might be, in some respects there were attendant disadvantages. Twice in his experience the Bristol caisson gates floated from off their bearings, though that might not have happened if the people whose duty it was had properly looked after them, in which case considerable inconvenience and expense would have been avoided. Mr. Harrison's remarks, however, pointed to what was perhaps the best form of gate, viz., one with a sort of box near the bottom for flotation, and a wooden skin with girders above.

Mr. Phipps had urged that there was too much in the Paper in the way of symbols, and that the question was unnecessarily complicated, and wished it had been settled by some geometrical method. He should have been happy to do it if he could; but unfortunately Nature was against it in this as in some other cases. It was not very easy to do it by analysis; it was much harder, if not impossible, to do it by geometry. He was aware of the liability to fall into mistakes in an investigation of this nature; but he had not only gone over these calculations several times himself, but they had been checked in a variety of ways.

He could not agree with Mr. Bramwell that the point C on Fig. 2 would be about the middle of the gate. From various experiments he had made with small models, it was clear that the strain would be much more nearly at the point B; but that would not make a serious difference in the calculation.

Mr. R. P. Brereton remarked, through the Secretary, that he did not consider there was any necessity for treating the calculations of the strains to which dock gates were subject differently from those of ordinary bridges, roofs, or girders spanning a given opening. The thrusts or pressures against the side walls or abutments would be of the same nature, and would be lateral, varying according to the flatness, or vertical, according as the arch or strut form of the girder was adopted. The only distinction in gates being the application of the loads, the water pressure uniformly diffused throughout acted always in the same directions, which, with an arch or strut, reduced the lateral thrust against the abutment or hollow quoin.

Local circumstances would generally determine the form of gates that it was most desirable to adopt; but with meeting gates, in the proportions most commonly in use, he could conceive no form in which theoretically the strains would be so favourably provided for as when the two leaves were formed as a continuous segment of a circular arc, that being the line of equilibrium naturally sought by water pressure; the metal being economically disposed in a continuous sheet abutting fairly against the hollow quoin, and not requiring fortification by ribs or girders, as must be the case with gates meeting at a point with separate curves or camber requiring to be restrained from distortion or change of form.

It was suggested that calculation showed that the best form for gates would be where the sally, or rise, was equal to the half span, or meeting at 90° in the centre. This would not appear to be the case: whether treated as segments of a circle or as struts, and with dock walls capable of resisting lateral thrust, a rise of about one-half of the half span, or an angle of 127° at the centre (which was very commonly adopted), would be more economical, besides having many practical advantages.

Taking this proportion of one-half, and contrasting it with a greater or a less rise, such as double or one-half, or equal to the span or one quarter of it, the calculations would be favourable; and whether considered as segments of the complete circle, or cambered as segments of separate circles, or merely as straight struts, the thrust would be about the same. In comparison with the case of 1 to 1, whilst the strains would be somewhat reduced, the length of the strained part would be increased in a greater ratio; and, on the other hand, in the case of $\frac{1}{4}$ to 1, whilst the strain would be increased about two-thirds, the length of the strained part would be reduced in a much less ratio, resulting in the loss of upwards of one-half. But amongst the practical advantages of increasing the angle of the gates were to be found the greatly increased effective bearing of the heel-post surface for abutment in the hollow quoin, as well as reduction in the extent and lateral direction of the thrusts. With the rise of 1 to 1 the bearing surface would be about one-half of the semicircle of the heel-post, without producing lateral thrust. With $\frac{1}{2}$ to 1 the surface would be reduced to about two-sevenths of the semicircle, and the thrust would be increased one-fourth, with a considerable lateral direction. And with $\frac{1}{4}$ to 1 the bearing surface would only be a sixth or a seventh of the semicircle, and the thrust would be increased in amount about two-thirds, and its lateral direction more than doubled.

In designing dock gates a large excess of strength beyond that calculated must always be provided, particularly high up in the gate, where the calculated strains were smallest, and where the gate was subject to blows from vessels and from the sea in exposed situations; nor would it be practically prudent to make use of boiler plate less than $\frac{1}{4}$ of an inch in thickness.

As regarded the question of relying entirely upon horizontal ribs at intervals to convey the thrust, considering the skin, in the case of curved gates, only in the light of planking to retain the water, it must be borne in mind that a single skin or plating outside the ribs, as usually applied, could not be available for resisting thrust. Assuming that the gates could always meet true at the centre, so that the skin at the meeting posts' end should be in line, the other end attached to one side only of the heel-post would not approach near to the abutting surface in the hollow quoin. A double skin was therefore indispensable, that the thrusts of both should be brought by the heel-post symmetrically to the bearing surface; or a single skin could only be applicable for thrust when passing through the centre of the heel-post.

With large spans many advantages accrued from double skins of boiler plates, particularly when near the bottom and middle of the gates. They were important for diffusing the pressure of the ribs uniformly along the heel-post. In some cases the thrusts amounted to 50 tons or 60 tons per foot in height with the gates at an angle of $\frac{1}{2}$ to 1, and the effective bearing surface obtainable with the heel-post 20 inches in diameter did not exceed 4 inches in width on either side of the centre line. Enormous pressures were produced even when distributed in the most careful manner. A double skin also admitted of sufficient buoyancy, from the adoption of air chambers within, for the easy working of the gate, and to relieve the rollers and heel-post pivots from excessive loads. His practice had been to restrict the capacity of the air chamber so that there should always be a sufficient preponderance of weight to prevent the gate from lifting.

The iron gates used in the muddy water at the Bristol Docks, and constructed more than twenty years ago by Mr. Brunel, had air chambers of a capacity to give sufficient buoyancy when mud had deposited in the bottom. When this was removed, water was admitted into the air chamber to retain the gates in position. There had, he believed, been one or two instances where this had been neglected, and the gates had floated on extraordinary tides, doing some mischief to the fastenings.

In well-constructed gates the use of double skins tended materially to preserve the form under racking forces, and to equalize the strains, diminishing the thrusts at the bottom where greatest, and distributing them higher up. He had known instances of gates constructed in this manner undergoing excessive strains without injury, when single-skin or plank-ribbed gates must have been destroyed. On one occasion a large single-leaf gate, 65 feet in length, broke away from its opening chains during a heavy sea, continuing for some time uncontrolled, opening by the runs of the sea as much as 15 feet or 20 feet, and closing violently against the meeting post; but no mischief ensued beyond the starting of a few rivets.

January 24, 1871.

CHARLES B. VIGNOLES, F.R.S., President, in the Chair.

No. 1,284.—"Train Resistance on Railways." By W. Bridges Adams.[1]

To analyze this question, it is necessary to determine the theoretical conditions under which the resistance might be reduced to a minimum on a level line. The first condition is, that the rails should be perfectly straight and level, *i. e.*, free from all irregularities, and of such section that they would not materially deflect, either vertically or horizontally, under the heaviest load borne on them by the pressure of the wheels. Secondly, that they be fixed in supports at sufficiently close intervals to prevent deflection, the supports being as firm and immovable as the piers of a bridge. Thirdly, that the rails be supported elastically on the rigid supports in such mode that no blow can take place, or any greater pressure at one point than another, the elastic action being equally distributed throughout. Fourthly, that the joints of the rails be so connected that they be equally strong, level, and even with the solid portions of the rails. Fifthly, that the two rails be perfectly parallel throughout to the required gauge when straight, and concentric when curved. Sixthly, that supposing the rails to be sufficiently hard to resist crushing, the bearing surface should be as narrow as possible, inasmuch as on curved lines the friction increases in proportion to the breadth of contact.

The structure of the vehicles requires that each vehicle must be supported on four wheels as a minimum. If the wheels be fixed on their axles, so that each axle becomes practically one wheel, analogous to a garden roller, the lines of rail being perfectly straight and level, with the axle arranged at right angles, and the wheels parallel with the rails, it follows that the only resistance will be the axle friction, and that tire friction will be absolutely nil, supposing the tires to be formed with coned peripheries, permitting exact movement in a straight course without forcing the flanges into contact with the rails. The amount of this axle friction, under the most favourable circumstances, of abundant oil, lubrication, and bearing surface, equivalent to 90 lbs. per square inch, is generally assumed to be 4 lbs. per ton of insistent load on the level. If, therefore, the practical resistance per ton

[1] The discussion upon this Paper extended over portions of three evenings, but an abstract of the whole is given consecutively.

is found to amount to 10 lbs., 20 lbs., or upwards, per ton of load, it follows that this surplus friction must be generated between the tires and the rails, and it is important to inquire whether this is a matter of necessity, or an evil that can be avoided. It may arise either on a straight line or on a curve. On the straight line, by the malformation of the vehicle, owing to the axles being out of parallel with each other; or by the axles, though parallel with each other, not being rectangular to the line of traction, and the wheels at intersecting angles, with constant grinding between flange and rail. On curves, the friction, both of the flanges and of the wheel treads, may be caused by the flanges being constantly at an intersecting angle with the rails, and by the differing lengths of the rails, producing a sledge or sliding movement; the wheels on the outer rail requiring to work on larger diameters than the inner, and for want of compensation involving great torsion of the axles, eventually leading to their breakage. In such vehicles the amount of the tire friction will be increased in proportion to the increase in the width of the gauge and in the curvature of the rails. The evil may be exaggerated by faulty structure, either original in the workshop, or, as a consequence of collision, making the frame of the vehicle—which should be a true oblong—a rhomboid, the wheels and axles taking the same relative position. And, practically, the rails, assumed to be straight and even, are, by faulty workmanship and wear, a succession of small curves on which the wheels, by the action of their coned peripheries, are seeking the path of least friction, and by reason of the rigid lateral fixtures of the wheels to the upper structure, that, with its whole load, is carried with them; as any one may verify by watching the sinuous course of a loose coupled goods train. Passenger carriages, close coupled, are prevented from making the same movement, but they become sledges or sliders, and grind away the flanges and treads of their wheel tires, and the collars of their bearing brasses, at a large cost of engine power, and with an extra development of noise and vibration not compensated for by the mere perfect vertical action of their springs.

Goods and coal trains are loose coupled, for the reason that otherwise their resistance would overpower the engine. But this involves another difficulty in snatches and concussions, breaking the chains and couplings, and inducing accidents more or less fatal and costly. Nor does this loose coupling contrivance get rid of the difficulty. Torsion of the axles goes on with tire rubbing and flange concussion, 'wringing the necks' of the revolving axles, and gradually, but assuredly, destroying them, unless so enormously heavy that the destructive action is confined to the wheels and rails instead. In coal trains, on a given line, the breakage of an axle per week is the average result.

As the loads on railways increased, the wheel-tires were crushed out, and rails were made heavier to resist them. Then tires were made of steel, and rails were crushed beneath them; and so rails in turn were made of steel, and it is assumed that, by its hardness, the steel will have great durability. But hardness has little to do with the question, which is one of homogeneity. Steel rails rolled from solid ingots do not split like iron rails rolled from imperfectly welded blocks and drawn out into a skein of fibres. But what are called steel rails are, in their most perfect condition, not steel but iron rails; the carbon which has served the purpose of enabling it to be melted into an ingot having disappeared in the subsequent processes. Steel proper is a very treacherous metal, and must either be equally hardened and tempered throughout, or perfectly annealed throughout. In the former case or in the latter it will not be subject to breakage. But if unequally hardened, or if hardened throughout and not tempered, it will break under concussion. But even when so hard as to break like glass, it is not secure against the wear of attrition. The engineer of a London line laid down some exceedingly hard and brittle steel rails, carefully guarding against risk of breakage; and twelve months' wear produced the removal of one-sixteenth of an inch of metal from the surface, as perfectly smooth as though planed off in a machine. The reason was, the amount of sledging or sliding movement, in which the sand embedded in the softer wheel tires cut the harder metal.

The cause of wear between tires and rails resolves itself into the fact, that the wheels do not roll, or only partially roll, and so become, more or less, sledges. It is a process of rubbing friction analogous to that of the axles without a lubricant, the noise and vibration experienced by the passenger indicating the amount, which is greatest in dry weather. In heavy rain the water acts as a lubricant between wheel and rail, and much of the noise and vibration disappear if the rails be clean. There is one common case demonstrating the amount of the sledging action. When a rail is turned, after being hammered and notched in the chairs, the square notches gradually become small curves, then larger curves, then the prominences begin to disappear, and after a given time, proportioned to the weight of the engine and the mileage, the rail becomes a nearly true and level surface, unless crushed out in the process. It is quite clear that mere rolling could not produce this result, and that it is practically a coarse planing process at the expense of engine power.

It is obvious, that if, by a process of better structure, tire friction can be got rid of, trains may run with only the normal amount of resistance belonging to the axle friction. The source of the tire friction is in the rigidity of the structure, and the only mode of avoiding it is by flexibility and compensation yielding. The first condition is, that instead of keying the wheels fast on to the axles, and so involving axle torsion and breakage, every wheel should revolve freely, so as to compensate for the varying lengths of the rails on curves or irregularities. Secondly, that the wheels should be enabled to slip, or slightly rock, within the tires on elastic cushions, enabling the tire to tread equably on uneven surfaces, intercepting noise and vibration and pressing always on the rails with equal pressure without jumping. In this mode all risk of bursting the tire will be avoided. Thirdly, that instead of fixing the wheels between horn plates, always involving mischievous flange contact on curves, the axles should be 'caster centred,' so that the wheels may move freely from side to side radially, constantly maintaining their parallelism with the rails, and their axles in true radii to the curves, under all conditions. Under such circumstances imperfect structure of the frame, whether original or distorted by collision, will not affect the true running of the wheels. Fourthly, that the buffer contact should be in true radial lines, so that close coupling may be used without involving lateral friction of the wheels. Fifthly, that the structure of the bearing springs should be such as to enable them to carry varying loads with equal ease and with minimum risk of breakage.

Wheels were originally made of wood, then of cast iron, then of cast-iron bosses, with wrought-iron spokes and tires, and then of solid forged wrought iron, either spokes or discs, and then of wood discs. The Americans still make them of cast iron in the solid with their tires chilled. The chilled tire is not desirable, for it increases the liability to breakage, and is apt to grind into flats on the tread, which spoils it. In all these wheels, excess of strength is needed to resist axle torsion and flange pressure against the rails.

The cast-iron wheel, properly made, is the simplest and cheapest, and not necessarily the heaviest, and by improved construction it may be quite free from all risk of breakage. In constructing a railway wheel to revolve on the axle, the first consideration is length of wheel boss. This should be equal to one-half the diameter of the wheel, to insure truth of structure and freedom from wear. In experiments with loose wheels, years back, a soft cast-iron boss 9 inches through was used to a wheel of 4 feet in diameter, and the result was that, in technical phrase, it 'got drunk' in three weeks, and the principle was dogmatically pronounced a failure upon the imperfect practice. With a boss half the wheel diameter in length this defect would not exist. Fig. 1 (Plate 8) shows this arrangement. The wheel boss is bored out, and the axle turned to a diameter one quarter of an inch less than the bore. The intermediate space is fitted with a sheet of perforated brass, with an open joint, so that the axle can revolve within the brass; or the wheel can revolve round the brass or both, the lubricant being contained in the perforations; or the boss may be cast without boring, and the space run in with white metal.

In considering the question of axles, there are several points. First, the normal strength of the axle, and next, the bearing area of the journals. As respects vehicles, a 10-ton wagon gives the practice of the heaviest axles, viz., $5\frac{1}{4}$ inches in diameter at the back of the wheel, 5 inches through the wheel, and $3\frac{3}{4}$ inches by 8 inches for the journal. Assuming that this size, if answering at all with the existing disadvantages of structure, would be ample were the defects removed, the question narrows itself into bearing surface of the journal and lubrication. In the experiments of Nicholas Wood and George Stephenson, to ascertain the rubbing friction of axles and wheels mounted on fixed bearings removed from the vehicles, and consequently without tire friction, they came to the conclusion, that a bearing area of one square inch to every 90 lbs. of insistent weight, and perfectly lubricated with oil, was the most favourable condition. Now, the area of the journal of the 10-ton wagon is only 40 square inches, equal on the four bearings to a load of 6 tons 8 cwt., and no doubt it was the small size of bearings, calculated from fixed machinery, that originally led to the use of viscid soap instead of fluent oil for lubrication. The small size of the journals was induced probably by the object of obtaining a back and front collar on the axle, to retain the bearing brass without materially increasing the diameter through the wheel. In Fig. 1 it will be seen that a proposed improved axle is 5 inches in external diameter throughout. It is not solid, but a tube with a bore of $2\frac{1}{2}$ inches, leaving walls of $1\frac{1}{4}$ inch, thus reducing the weight by one-fourth. If the external diameter were increased to $5\frac{1}{2}$ inches, the weight would be about the same as that of the solid 5 inches, having a bore of $2\frac{1}{2}$ inches and walls of $1\frac{1}{2}$ inch; and the strength would be increased one-fifth, by the increased external diameter. But the increase would be considerably more than that, inasmuch as metal $1\frac{1}{2}$ inch in thickness would be much better worked and manipulated than metal 5 inches thick. Taking the 5-inch axle, with a bearing 5 inches in diameter and 10 inches in length, it would give 70 square inches of area, equal to a load on four wheels of 11 tons 5 cwt., and without any sinking or reduction of diameter to cause breaking down. To keep the free revolving wheels to the exact gauge, collars are shrunk on to the axle at the back, and screw studded. Against these collars the wheels abut, retained by the bearing brasses in front of them facing against the wheel-front, turned true, and with ample surface. The front collar of the axle is $6\frac{1}{2}$ inches in diameter, and is screwed into the hollow of the axle and secured by a key; the bearing tapers slightly to the front, tending to carry up the oil towards the wheel. The axle box is fitted below with a metal oil well in a 'keep' of hard wood, cross keyed into position, and fitting close round the bearing and brass. A slip of hard wood between the box and end collar prevents end wear on the brass, which is so serious an evil in ordinary arrangements. The hollow of the axle within may serve for an oil magazine, and there is also a feed on the box top. Waste of oil is prevented by glands in hollows round the external joints. An angle band, carried in a recess of the axle collar, lifts the oil continuously from the well as it revolves. The wheel is turned on the periphery to a cross curve bearing on the centre of its breadth. The tire is $1\frac{1}{2}$ inch thick in the central width, and 1 inch at the edges, where the retaining rings rebate. The space between the bolts is flushed with wood packing, on which is laid a thick band of vulcanised rubber, faced with a thin hoop of tempered steel. On this hoop the wheel rests, and the bolts are screwed up. Black-lead powder is placed between the wheel and steel hoop, and the wheel can thus slip round, and the tire can rock laterally to suit the tread on the rails. These axles and wheels, with the tire free from blows, would carry oil, under the most favourable conditions, to enable the vehicle to run 100 miles per day for twelve months. The wheel-tire is shown in section in Fig. 1.

There remains the question of axle radiation true to curves, and the parallelism of the wheels to the rails under all conditions. Vehicles have been constructed on four wheels, on six wheels, and on eight wheels. On the whole, for several reasons, four-wheel vehicles are preferred, though eight-wheel vehicles are well adapted for large saloons or family carriages. Bogie carriages on eight wheels have hitherto been imperfect for several reasons; frictional central bearings carrying the load, and uncertain automatic guidance of the wheels by the flanges, the outer wheels recoiling when they ought to advance, an evil which it has been sought to remedy by lengthening the bogie, and thus increasing the flange friction by irregular action. On no curve can the fixed parallel axles be perfect even on the bogie; and it is only by keeping them as close together as possible that this evil can be minimised, using the same 'caster' guidance as that proposed for the four-wheel vehicle.[1]

Figs. 2 and 3 show a half carriage on eight wheels, with long vertical shackle-rods pendent from the side bearing springs, which are firmly fixed to the axle-boxes, in such mode that the wheels and axles may move freely in every direction, either laterally or fore and aft, or diagonally. Such freedom of movement as this would, of course, result in irregularities if uncontrolled. To insure true movement a pivot is fixed to the upper frame, round which the wheels and axles revolve. If this pivot were placed directly over the centre between the four wheels, the pressure of the wheel flanges against the curves of the rail would probably place the axles in positions abnormal to the curves; but, passing through a plate on the cross bar of the wheel frame centred near the inner axle, it gives a 'caster' movement, insuring that the wheels are always parallel to the rails by flange guidance, whether on the straight line or on the curve; on the curve by flange pressure, and on the straight line by gravitation action, or the vertical action of the spring shackles. The total wheel base is 35 feet, and it can run round S curves of two chains radius without grinding the flanges.

It will be seen that there is no frictional resistance to the radial movement, as no load is borne upon the central pivot; nor is any special structure of the vehicle required to carry the load, which is borne by side springs from the axle-boxes as usual, but with the advantage that, as the horn plates are dispensed with, the frame may be made to overhang the axle-boxes to any extent desired, and the width of the spring plates may be greatly increased, giving equal strength, with less thickness and greater elasticity. The width of the frame of the vehicle may in all cases

[1] By 'caster' movement is to be understood the principle of applying a vertical axis at a short distance behind or before a horizontal axis, so that it 'casts' or turns the horizontal axis out of a straight line. In house furniture the vertical axis is in front of the horizontal axis, the tendency being, to follow the guidance of the mover. On the rails the rail is the guide, and so the vertical axis is placed behind the horizontal, in preference.

be twice the width of the gauge; but if the gauge be broad, it may even exceed that, the height of the vehicles remaining in all cases the same, and the centre of gravity being lower in proportion to the width. There is therefore—supposing the traffic abundant—an economical advantage in the broad gauge, for the length of the train may be shortened, and the number of vehicles be reduced, in proportion to their width, with manifest advantage in haulage. And with free wheels and radial movement, the resisting friction on the broad gauge may be as free as that on the narrow gauge. The apparent anomaly of lessened friction on the narrow gauge is simply owing to the fact that the wheels so called, on both, are not true wheels, but garden rollers, and the narrower the roller the less is the friction. With wheels proper, the friction is alike in both cases, relatively to the load, and perfect or imperfect structure. The 7-feet gauge might have vehicles 15 feet in width running with no more friction per ton of load than any narrow gauge; and it is only a question of population and traffic, whether it would be commercially desirable or not. With proper construction, the proportion of paying to non-paying load should be less on a broad gauge than on a narrow gauge. The chief question to consider, if it be a fluctuating traffic, is as to the average maximum, and transit convenience.

There is one article of commerce in which the quantities are more unvarying and regular than any other—coals. The object in this case is to get as large a load on each wagon as possible, short of damaging the wheels and rails. It is customary to run these trains with a foot or more of loose chain-coupling between each wagon. The motive for this is, that if close-coupled the friction of the wheel flanges against the rails would become so enormous as to overpower the engine, and the amount of the friction may be estimated from the sinuous courses the wagons take when left to their own guidance. If the wheels were free to revolve on their own axles, and the axles freely radial to allow the wheels to be parallel to the rails in all cases, flange friction would cease, and in such case the wagons might be close-coupled, provided their ends were curvilinear to keep contact without binding at the corners. In such case axle torsion and axle breakage might cease to be a large and unknown quantity, and the chances of breaking couplings, with all the contingent accidents, would be minimised, with a lessened consumption of fuel, and a removal of the chief sources of wear and tear.

There is another important advantage in the radial system. Ordinary wagons or vehicles as they leave the workshop may be accurately constructed, with their axles parallel to each other and rectangular to the line of traction—their supposed normal condition of perfection—or they may not; a question involving much supervision. But supposing them originally true, they may chance to meet with a collision on a sharp curve. In that case, if deficient in buffer yield, they cease to be square frames, and become rhomboidal in form. The two sole bars remain parallel to each other and so do the axles, but one end advances beyond the other, and the axles become askew to the line of traction, and the wheels form constant intersecting angles to the rails. The vehicle is thus converted into a constant sledge under all circumstances, with great risk of running off the rails. This will account for the fact that, in some cases, trains of wagons are unable to run down inclines of 1 in 72 without engine power to help them, the resistance being equal to about 31 lbs. per ton.

In the radial wagon, Figs. 2 and 3, the guiding action is from two central pivots; in the non-radial, from four horn plates. The horn plates in a damaging collision change their relative position to each other. The central pivots in a similar collision retain their relative position, and the wheels being free, the wagon will continue to run true on its wheels even after the frame is distorted. The radial wagon has a compensating action; the non-radial wagon has none. The non-radial wagon if not repaired goes on 'wringing the necks' of its axles, by flange pressure against the rails, till it breaks them—an incipient cut round the exterior gradually deepening, and the centre lessening, till the axle inside or outside of the wheel becomes severed. This radial wagon is shown without swinging spring shackles, the springs being fixed to the sole bars, with the radial axle guards and axle-boxes sliding beneath the buckles, but with common axle-boxes. The radial axle guards can be formed with arms clipping the tender rod, and made to move laterally, to right or left, round the curves, and ensuring truth of movement with rails out of gauge, as indicated by dotted lines in the diagram.

When the wheels, instead of rolling, slide along the surfaces of the rails, especially with sand or dust between, contact is incessantly made and broken, a vibratory sound is induced, and also a vibratory movement by no means pleasant to the ear or to the nerves generally. If the rails were quite clean and oiled on the surface there would be smooth sliding without breaking contact, and the unpleasant vibration would cease, while the lubrication of the axle bearings would be perfect and free from concussion. There is no reason, other than faulty construction, why vehicles moving on rails should be appreciably more noisy and concussive than vehicles at rest.

The same conditions of frictional resistance apply to short vehicles as well as to long ones, but in long vehicles the defects of rigidity are exaggerated. With length there is less oscillation but more vibration. Of course on very long carriages more wheels must be used to carry the load, but there is no reason why eight-wheel vehicles should not have their friction reduced nearly to the same amount as four-wheeled. The system of radial vehicles permits the application of self-acting breaks throughout the whole train, worked either by the guard or driver, or by both.

Engines with coupled driving wheels must of course have the axles of those wheels parallel with each other, and on curves those tires must grind and induce impedimental friction on the rails, and for this reason what are called single drivers are commonly used for high speeds. But if the tires be applied as friction clutches by the agency of springs, they will yield a compensating movement on curves, preventing the torsion and liability to breakage of the axles. Six coupled wheels close together may thus be used without surplus friction, radial wheels being applied to the trailing end, so as to adapt the engine to run without flange friction round curves as sharp as 3 chains radius, either one, two, or three pairs of radial wheels being used, making the engine twelve-wheeled, and free from flange friction. Of course, providing the wheels between tire and wheels with elastic springs will serve greatly to moderate concussion. But the structure of the rails is worth consideration—such a structure as will diminish to the utmost the chances of loose movement among the several parts, and make those parts as few as possible in number. Rails are practically a portion of the engine. A rigid permanent way with no loose parts or any chance of working loose might be hard, but would not in itself be concussive. There is an important principle which should not be overlooked: the strength of a rail is as the square of its depth and width to prevent flexure. But if the rail be supported on its lower edge, it will require a much larger amount of metal in its central web than if it be suspended below its upper edge or table; and therefore the metal of a rail 5 inches in depth may, without increase of weight, by changing the prop below into a suspension above, produce a rail 8 inches in depth. As the strength is as the square of the depth, the increase will be as 64 to 25. The 5-inch double-headed rail placed in chairs is supported on its base, and needs a thick web to prevent it from buckling under the load. To keep the rail in position wooden keys are driven on the outside between the rail and chair. The outward pressure of the wheels crushes the keys, and then the rails are loose, and jump up and down, breaking the chairs and crushing the rails, the load being multiplied in its mischievous effect by the looseness. To avoid this the wooden keys are in some cases applied inside the rails. The rail, 5 inches in depth, passes through an arc of that radius if yielding laterally. The 8-inch rail, elevated only 2 inches above its bearing, is free from the rocking action. The total depth from the ordinary rail top to the bottom of the cross sleeper is 12 inches, and if the sleeper sinks or gets loose, 12 inches more or less of ballast must be dug out and removed in order to get at the sleeper to repack it; and when packed and the space refilled, a mass of loose ballast is filled in for the reception of rain. By the forward motion of the engine, the sleepers continually rock fore and aft, and by flange pressure the sleepers are driven endways transversely to the line, which gets out of gauge and into irregular curves. It is a great defect in railway structure that almost all rails are applied in this imperfect manner, whether used with or without chairs.

It is generally assumed that the hardness of a railway is lessened by the use of wooden sleepers as compared with iron. But the iron rail in an iron chair is practically an iron permanent way, and is subject to jolting. One objection is the ringing sound, and the more perfect the iron way is, the freer from all loose movement, the greater is the ringing. But the ringing is induced by a ringing wheel and a ringing rail combined. If the ringing be taken out of the wheel, by an elastic tire, the rail also will cease to ring, as is the case with a wood wheel compared with an iron one.

Time was when wooden sleepers were objected to on account of their tendency to decay, and so creosoting was invented. But by the increase of the engine weight the sleepers were destroyed by mechanical action before rotting could begin. Cast-iron sleepers have been substituted for wood in warm climates, but have not made way in England, possibly on account of the cost, for if heavy enough to prevent brittleness at high speeds, the weight would require to be increased from 84 lbs. to 112 lbs., or more.

The Author has come to the conclusion, that wrought iron is the best material for sleepers, and that surface bearing with surface packing is the most suitable mechanical arrangement, the head of the rail being kept as low as possible on the sleeper; that a deep keel under the line of the rail is necessary to maintain the gauge; that screw bolts as fastenings are a disadvantage; that fishes as hitherto constructed are very inefficient and troublesome:

moreover, that the rails should be so applied as to maintain their position without fastenings, and that even in case of breakage they should not get out of the sleepers; that the sleepers should key into the ballast in such mode that if the sleeper were to lift it should lift the ballast with it. And all this with a long and wide bearing area of the rail on the sleeper, and of the sleeper on the ballast, while the line should be well adapted to lay into sharp curves as well as maintain its straightness on straight lines; and all this with the minimum number of parts, and the minimum number of types of parts.

With ordinary rails supported on the lower table, whether with a broad foot direct on the sleeper, or with a duplicate table in a cast-iron chair, it is obvious that a high upper table requires a web proportionately broad to prevent lateral overhang and crushing, and so it becomes very heavy. The foot rail if wide enough on the foot to keep it steady and of great height takes more metal than a double headed rail, for the foot must be thick where it joins the vertical web, to prevent buckling upwards at the edges. But if the rail be suspended beneath the upper table these difficulties may be avoided. To prevent crushing a sufficient mass of metal must be applied on the running table.

Fig. 6 shows a single-headed rail 7 inches in depth, with a lower table 1 inch by 1 inch, a vertical web $\frac{1}{2}$ inch thick, and an upper table $2\frac{3}{4}$ inches wide by $1\frac{3}{4}$ inch in depth. The weight is 75 lbs. per yard. The sleepers, formed of rolled plate iron, are 2 feet 3 inches in length and 12 inches in width, giving each a bearing area in the ballast of $2\frac{1}{4}$ square feet, and weighing 65 lbs. each. They are spaced 3 feet apart from centre to centre. From the bearing surface the plates pass downwards, forming a deep channel groove spread outwards below, like an inverted wedge which keys into the ballast and holds the sleeper firmly down. The extreme edges are ribbed to thicken them and retain the ballast. To keep the rail in the sleeper a cross key of parallel round iron is driven through both the sleeper and the rail; the end being tapered for easy entrance, the drift being parallel, and curving the key downwards. This leaves no discretion to the platelayer, such as would be needed with a key taper throughout. The elastic action of the key prevents any working loose, but at any time it may be driven another inch and retightened. The holes through the sleeper and the rail web are $\frac{7}{8}$ inch in diameter, and the round key is $\frac{5}{8}$ inch in diameter. The bearing surface of the rail on each sleeper is 60 square inches, or about six times the rail area of an ordinary chair. One cross key is used to each intermediate sleeper, and four keys to each joint sleeper. The sleepers, which serve the place of fishes, are 2 feet 3 inches in length as compared with 1 foot 6 inches, and 6 inches in depth as compared with $2\frac{1}{2}$ inches. To keep the gauge of way, cranked tie bars are applied, the cranks lipping in between the sleeper and the rail, and sufficiently low in the ballast to prevent disturbance. Practically the chief use of the tie bars is to determine the exact gauge when laying down. When once in the ballast the deep keel gives an amount of lateral stability which, combined with the small elevation of the rail above the surface, renders it almost immovable. The amount of ballast required with this rail under ordinary circumstances is only one half the usual depth, and it may be laid in two trenches instead of over the whole surface. Cross sleepers, deep down, only get partially packed and require the removal of 12 inches of ballast, called 'opening out,' to get at them, leaving the upper ballast afterwards open to the penetration of rain. These iron sleepers, on the contrary, are packed from the surface without any 'opening out,' and they shield the ballast below them from access of rain, the water passing at an angle below the bottom of the keel. The sleeper has an elastic form both where it rests on the surface and also at the bottom of the keel, and the support of the rail is the reverse of that of an anvil, as when resting in a heavy cast-iron chair. There is a general impression that an iron way must necessarily be a rigid way, and that timber sleepers lessen the rigidity; but this can scarcely have any effect with heavy iron chairs. The wrought-iron sleepers will be less rigid than rails jumping up and down in the chairs by the crushing of the keys. No doubt cast-iron sleepers being heavier, and more resembling an anvil, have a harder and harsher effect than rails borne directly on timber sleepers without chairs, but the real annoyance is to the ear rather than the body, by the ringing sound which the timber helps to moderate.

Many years back the Author produced a system of permanent way known as the Girder Rail. It was something analogous to the present, inasmuch as it was supported on the surface, or rather suspended. But the sleepers were not short channels, but long angle irons bolted to the rails and through them, and breaking joint throughout like a long T beam one solid mass for miles. But the vertical web of the angle irons was not so deep as in the present system, nor the rail so strong at the joints, and the number of screw bolts all beneath the horizontal bearing were inconvenient to get at. Moreover, it was difficult to lay into curves, or to make it take any form but that of its manufacture, which was not very perfect, so that it was impossible to get rid of small curves or kinks by any amount of packing. If forcibly restrained by the ballast, the engines that passed over restored it to its normal condition. And so large a mass of iron without breaks—a continuous beam—was much affected by changes of temperature. The ringing sound under the trains was very remarkable. For the sake of keeping down the weight the rails, which were double-headed and 84 lbs. to the yard and 7 inches in depth, were only 2 inches in width, and the vertical webs of the angle irons were only $3\frac{1}{2}$ inches in depth.

It is generally assumed that surface bearing prevents any effectual holding in the ballast, but that system demonstrated this idea to be a fallacy. When laid in loose-blowing sand, the sand below the horizontal webs became consolidated into a kind of sandstone, which when lifted clung to the bolts and rails as if it were a mass with them. The objections may be thus summed up: insufficient bearing-table, insufficient depth of the angle-irons, expansion and contraction as an endless bar, difficulty of dealing with curves, and inconvenicnee of the multiplicity of screw-bolts in a buried position; to which may be added the difficulty of replacing a rail owing to the long length requiring to be taken apart by reason of the continuous break-joint. In the present structure these difficulties are got rid of by the shallow angle-irons being changed into deep channels, and short lengths substituted for break-joint, permitting the laying in to curves by the substitution of one large solid table for two small ones, and by getting rid of screw-bolts altogether, and substituting simple keys easily driven in and driven out, with the advantage that the rails will be safe to run over even after the keys are removed, a most important consideration for quick repairs, and especially on lines of constant traffic, with small intervals, such as the London suburban lines. If there be any advantage in the application of wood to prevent jarring, there is no difficulty in laying the sleepers on inch planking, as a mere surfacing to the ballast below the sleeper wings. In Fig. 6 the types are four, the posts per mile 10·064, and the total weight per mile 228 tons. It will be observed that timber fishes and screw-bolts are dispensed with.

On a rough estimate, the surplus resistance of trains is fully one-third more than it ought to be, in consequence of imperfect structure, and this necessitates the employment of one-third more engine power than would be needed were this evil corrected. If the present sledging or sliding movement of the so-called wheels over the rails were reduced to simple rolling, and the loads per wheel regulated to prevent crushing, and mechanical destruction of the sleepers avoided, there is no apparent reason, other than collisions, why wheels and rails should not last twenty years. In the prevention of collisions breaks should play an important part. A given power applied will stop a single carriage on a given incline with a specific rate of speed. If every carriage in the train were supplied with a similar break, every length and weight of train could be stopped in the same space and time. To do this effectually the breaks in their normal condition must be pressing on the wheels, and be taken off by the guard or driver, or both, when requiring to proceed. They should also be rapidly self-acting in case of impending collision, or breakage of a coupling. There is no difficulty in this arrangement.

On the question of engine traction the necessity for larger and larger trains has gone beyond the adhesive power of a single pair of driving wheels with a crushing load on them, and perforce the number must be multiplied by coupling four or six or more. The objection to coupling is the increase of friction between tires and rails, increasing with the speed. The cause of it is rigidity and inflexibility, and the remedy is to be found in flexibility. If the tires, instead of being shrunk on to the wheels with enormous tension, involving their bursting, were simply applied as friction clutches, the compensating movement needed would be provided for, and axle torsion and tire rubbing would disappear. Fig. 1 shows spring tires both with steel and india-rubber springs. Coupled driving wheels of course can be used with straight axles. Inside cylinders require cranked axles—a source of breakage. Outside cylinders tend to produce more unsteadiness than inside, but they can be used with straight axles, and the unsteadiness can be obviated by length of wheel base. But length of wheel base involves friction of the flanges, and more especially on curves. The remedy for this is to keep the driving wheel-base as short as possible, and to make the trailing wheel-base radial. The numerous engines with radial axle-boxes which have been working for the last four years on the suburban lines of the London, Chatham and Dover and the Great Northern railways show how practicable this is. They are adapted for 4 chains radius, and are steady at all speeds, the length of wheel-base being 20 feet. The water-tank being at one end and away from the driving-wheels, the load on the driving-wheels is as constant as that of a tender engine, independently of the consumption of water. Tenders were originally contrived to carry a large quantity of water, and the tank-engine has been rarely used for long distances on account of the limited quantity of water. The radial engines on the suburban lines are six-wheeled, but there is no difficulty in making them eight-wheeled, to carry two thousand gallons of water with an increase of wheel-base, and yet fitted to roll freely round curves of 3 chains radius. Such engines are adapted for the highest speeds and longest journeys. An engine is shown in Figs. 7 and 8, Plate 8, with four driving-wheels 6 feet in diameter, and four trailing-wheels 4 feet in diameter. But it is not merely for long journeys on main lines that an abundant water supply is needed. The journeys on suburban lines are practically a continuous journey, and it is more convenient and economical to carry a large supply of water than to pay frequent visits to the water-crane, deranging the traffic meanwhile. There are three classes of engines for curvilinear lines—the radial engine, the double bogie engine, and the twin engine, and they may all alike be constructed with two cylinders or four. Their steadiness must in all cases depend on the length of wheel-base. The movement of the radial engine is by curvilinear slides laterally; that of the double bogie engine is by a four or six-wheel frame moving on a central pivot at each end of a long frame. The twin engine is radially coupled at the centre of the frame, at the foot-plate between the fire-boxes.

Practically, the double bogie engine is two separate engines as much as the ordinary engines, each formed on a separate frame, with a difference in the connection. The two bogie engines are connected by a central pivot each to a long boiler, much in the same mode that two railway timber trucks are connected by pivoted saddles to a long tree. The boiler is formed with a double fire-box at the centre of its length, and each connected by fire tubes to a chimney at either end, the steam space being in common. To obtain space for footplates, the fire-boxes are cramped and narrowed in the centre, and the driver and stoker are separated—one having the fires in charge, and the other the steam; with the disadvantage that neither of them can perform the duties of the other in case of accident to one or other. To connect the engines to the boiler separate frames are fixed to it, having no connection with one another, by a pivoted centre plate, and a circular sway bar next the fire-boxes, the chimney ends taking no bearing on the engine frames. The load of the boiler therefore is not borne on the extreme wheels, but overhangs them. It will be seen that the two engine frames are so arranged, that they can swivel round each independently of the other, so that they can work without friction on irregular curves, whereas if they were obliged to move simultaneously they could only work to advantage on regular curves. But the traction and buffing strains are carried through the two engine frames, and consequently through the central pivots or circular plates which are attached directly or indirectly to the boiler. If therefore, in case of collision, the pivot withstands the force of the blow, it must be by the strength of the boiler at the risk of loosening the rivets, and, if giving way, the engine frames must drive into the fire-boxes at each end as the ultimate result. Other things being equal, fewness of parts in any machine is desirable, with equal efficiency. The double bogie engine is disadvantageous in four cylinders being substituted for two, but in balance for this, greater power is claimed, which might be a fair equivalent; but the cylinders are placed in awkward positions, overhanging weights at the ends of the loose bogies tending to sway them down and induce steam leakage. It is therefore worth considering whether the four cylinders may not be so disposed as to attain many greater advantages while remedying their defects.

There can be no doubt that the demand for a concentration of greater engine power is increasing with the increase of traffic. To obtain this concentration the number of driving-wheels must be multiplied, so as to distribute the necessarily increased load, without crushing the rails and road. This must, of course, extend the wheel-base, and necessitates provision against increased flange friction on curves. It is notorious that engines with six wheels coupled are frequently less efficient than engines with four wheels coupled and the same steam power, and this arises either from extra flange friction, or from defects in the adjustment of the coupling rods, and their multiplication. It is not a favourable condition for cylinders placed at one end of an engine to be working a train of coupled wheels, say eight in number.

The true mechanical position for the cylinders is at the mid-length of the engine frame, out of the way of collision, and with their weight supported by wheels both fore and aft. To accomplish this, one cylinder on a side will not suffice, and there must be two, one above the other, each piston being made to work a pair of coupled driving-wheels, the pistons working in opposite directions. As the wheels are all drivers, it is obvious that no radial movement can be permitted or obtained, but as an equivalent for getting rid of flange friction some of the flange tires may be replaced by broad rolling tires without flanges, leaving the flange base sufficiently long for lateral steadiness to the gauge of way.

Figs. 9 and 10 illustrate this plan of twin-cylinder engine. The driving-wheels are eight in number, and 4 feet 6 inches in diameter. The four cylinders, cast in pairs and fixed at the mid-length of the frame in strong horn plates, to work crank pins on the wheels, are 15 inches in diameter, with 24-inch stroke. The extreme wheel base is 18 feet. The central wheel-base, with flange guidance, is 8 feet; the same as the 'White Raven,' which was perfectly steady with radial movement of each pair of end wheels, and a total wheel base of 22 feet. The end wheels have flat tires 8 inches in breadth, without flanges. Each pair of end wheels has a coupling-rod to a pair of central wheels. The steam is taken from a central dome near the fire-box, and enters the two cylinders on each side simultaneously, by the action of a single slide valve worked by the ordinary number of eccentrics, which open and close two ports at opposite ends, working the two pistons in opposite directions. An obvious advantage is gained by this arrangement, as the steam, working in opposite directions on two pistons at the same time, neutralizes all oscillation and may dispense with the balancing of the wheels. A possible objection to this simple arrangement of single slide valves with double ports may arise from unequal slip of the front and hind groups of wheels, which are unconnected, interfering with the due steam entrance and exit. If so, two sets of valves can be used, the forward set within, and the hinder set without the wheels, without interfering with the fire-box. By a simple arrangement, the steam could be shut off from either front or hind wheels, by the driver on the foot-plate, where only small power or speed were needed. The buffers are formed on radial curves, with broad steel spring plates bearing on sheet rubber on curved blocks of timber, adapted to meet any class of buffers on the same level. The couplings are on radial traction bars between the radial buffers. The engine will run with the minimum of flange friction round curves of 3 chains radius. All the wheels are spring tired, preventing blows and destruction between wheels and rails. Wedge break blocks, actuated by steam, press upwards and arrest all the eight driving wheels. The tractive force at the rails is equal to 18,500 lbs. Two side springs spanning from axle-box to axle-box carry the front end of the engine, and two cross springs, one to each axle, carry the hind end, the whole load resting on four points of support, with a total wheel-base of 18 feet, and a total length of

28 feet. If used as a tank engine, the wing tanks will carry 1,600 gallons of water, and the coal bunk has 72 cubic feet of space. The boiler is of large size, and the ordinary coal fire would suffice to produce steam enough; but there is a better fuel, which must ultimately displace coal for locomotive purposes—petroleum. It is lambent flame in contact with heating surfaces, that is the chief agent in giving out heat to water, and not mere incandescent fuel; and lambent flame can be most effectually induced by burning gas. Petroleum, flowing from an upper cistern in thin streams into heated perforated pipes in the furnace, will be instantly flashed into gas, which, mingling with the requisite amount of atmospheric air, will fill the whole of the fire-box and tubes with lambent flame in greater or less amount at the will of the driver, who by the presence or absence of smoke will know exactly how to adjust his oil supply to the supply of air, and stop or urge the steam-making at pleasure, without needing a stoker to dirty his hands or his footplate, keeping him to his legitimate business of looking out. It is quite clear that the question of cost may have a considerable margin. If an engine boiler of a given size can be made to double its power, it may be worth while to pay double price for the fuel, and the minimum price at which fuel manufacturers may supply the material of olefiant gas has not yet been arrived at. There is another important consideration: there is no sulphur in petroleum to destroy bars, tubes, or copper fire-boxes.

There is another principle not yet applied to locomotives, though used to a considerable extent in steam-vessels—using the steam twice over before dismissing it to the chimney in blast, by passing it through two or more cylinders, working it expansively. But this application would need some complication in the eccentrics, to keep the cranks at right angles.

A passenger engine could be made on the same principle, and of the same size and length of wheel-base, but with only four driving-wheels and no coupling-rods. The driving-wheels, 7 feet in diameter, might be at the ends, with an 18 feet base. There would be four central carrying wheels, 3 feet 6 inches in diameter, sliding laterally in the horn plates, so that the engine would work round curves of 4 chains radius with a tractive force at the rails of 12,000 lbs.

This Paper has dwelt on many of the most important details making up railway transit—some in actual use, and some awaiting use. A practical people are too prone to ask, "Has it been done? Is it in actual use?" And many are they who try to

Fig. 1.

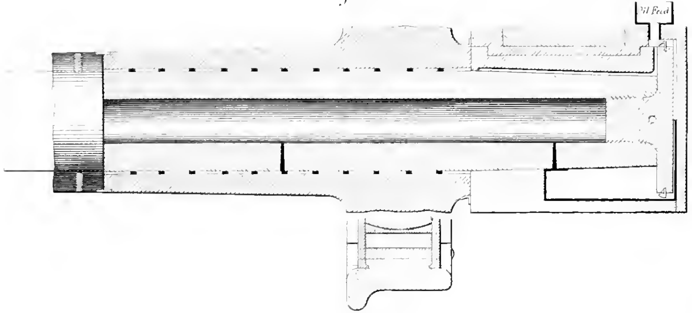


Fig. 3

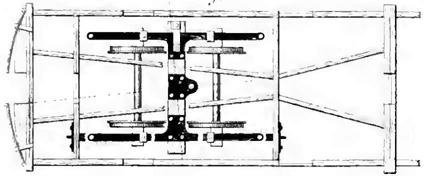


Fig. 8

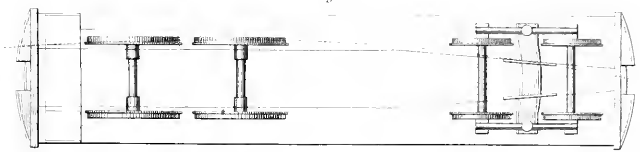


Fig. 2

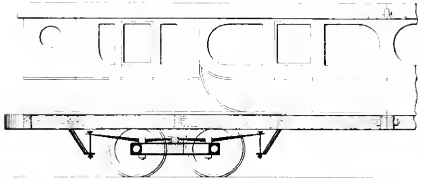


Fig. 7

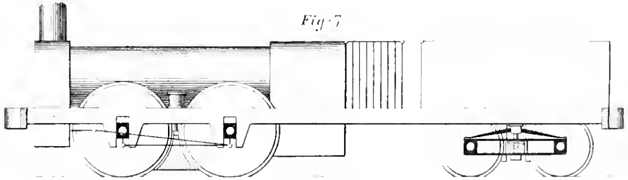


Fig. 6.

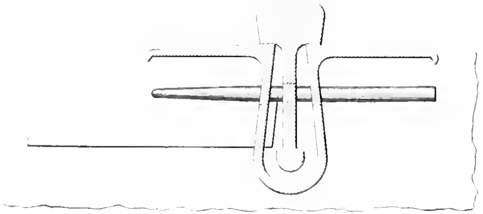


Fig. 5

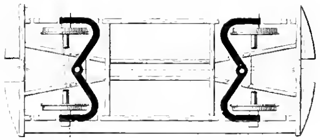


Fig. 4

Fig. 10.

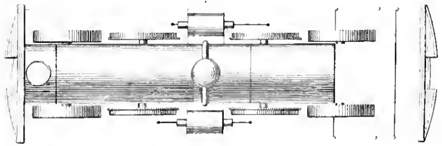
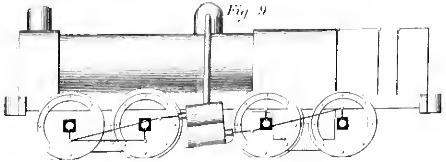


Fig. 9



avoid all possible mistakes by never doing anything that has not been done before. But had there been no previous looking into the unknown there would not have been any railways. The true philosophy is to examine carefully all useful suggestions that may be forthcoming; and when satisfied that the balance of probability is in their favour, to try them fairly, for the result is a very important one. To reduce train resistance by one-third, and take 300 tons of coal at the cost of 200, and at the same time to multiply the haulage power of engines in a similar proportion, reducing the cost by possibly one-half, is an important consideration for those railway companies dealing largely in coal, and still more for the vast mass of their customers, to whom it is a necessary of their existence.

The communication is accompanied by a series of diagrams, from which Plate 8 has been compiled.

[Mr. C. Douglas Fox

Mr. C. Douglas Fox said, it appeared to him important in the interests of the profession that a more comprehensive series of experiments should be made as to the effect of sharp curves upon tractive force for drawing a train. On the Queensland railway a series of experiments were made in 1869 on a comparatively small scale, but on very severe curves down to 5 chains radius, combined with gradients up to 1 in 50.[1]

These experiments showed that the effect of a 5-chain curve, on the 3 feet 6 inches gauge, was to increase the tractive force necessary in the ratio of 6 to 5, as compared with a straight line; and that, if the curve exceeded 10 chains in length, this ratio was further increased towards the end of the curve.

As regarded the reduction of cost of maintenance and repairs of permanent way and rolling stock, there would seem to be two primary things necessary to be reconciled in order to accomplish this object. In the first place the construction of railways must be economical, and so also must be their maintenance. As far as the latter point was concerned, what was the present condition of railway rolling stock over the greater part of Europe? It consisted of a very heavy dead weight compared with the live load carried. A great deal, nay almost the whole of it, was

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[1] Southern and Western Railway of Queensland.

Main Range (Section 5).

Results of Experiments made with the dynamometer to ascertain the relative resistance due to curves and gradients.

May 11, 1869.

The load consisted of six empty wagons and a break van. Gross load 32 tons 13 cwt.

Description and weight of wagons.

Description.	Weight. Tons.	cwt.		Number of Wheels.
No. 8. Break van	5	4·1 / 10·0	ballast.	4
,, 16. 14 feet covered goods	3	18·1		4
,, 25. 14 feet covered truck	2	15·0		4
,, 31. 20 feet covered goods	5	8·2		6
,, 68. ,, low-sided	4	19·0		6
,, 75. ,, ,,	4	19·0		6
,, 78. ,, ,,	4	19·0		6
Total	32	13·0		36

A speed of 10 miles per hour was maintained throughout from Murphy's Creek to Toowoomba.

constructed with a rigid wheel base of considerable length. As a rule, very little of the radial action which the Author spoke of had been introduced in Europe; the tires were still coned; the centre of gravity was rather high; the stock was screwed up tightly with screw couplings; and it was provided with double buffers—four to each carriage or wagon, besides a central draw-bar and two safety chains. But there was a constant grinding on sharp curves between tires and rails; a considerable and undetermined amount of gauge concussion due to oscillation and sinuous motion, and severe vertical shocks, owing to the heavy

TABULATION OF RESULTS.

Gradient.	Radius of Curve in Chains.	Tractive Force in lbs. per Ton.
Level	Straight	17·15
Level	5	20·55
1 in 625	Straight	18·86
1 ,, 625	5	24·00
1 ,, 300	5	27·44
1 ,, 226	7	34·30
1 ,, 120	Straight	34·30
1 ,, 120	57½	37·13
1 ,, 112	12	41·10
1 ,, 110	7	45·51
1 ,, 100	10	46·30
1 ,, 100	5	48·02
1 ,, 99	Straight	41·10
1 ,, 87	Straight	45·59
1 ,, 80	Straight	46·30
1 ,, 80	6	48·00
1 ,, 80	5	48·88
1 ,, 71	5·5	54·88
1 ,, 64	5·5	56·60
1 ,, 60	Straight	51·11
1 ,, 60	20	54·88
1 ,, 60	12	58·31
1 ,, 60	5·5	60·03
1 ,, 55	Straight	56·29
1 ,, 50	15	62·49
1 ,, 50	10	65·11
1 ,, 50	8	66·91
1 ,, 50	7	72·03
1 ,, 50	6·5	75·43
1 ,, 50	6	77·18

These experiments were made with a dynamometer registering cwts. only.

N.B. These results only hold good for about 10 chains on sharp curves. If the curve exceeded 10 chains in length, the tractive force necessary at the end of the curve was much increased. At the end of a 5-chain curve, whose length was 18 chains, and gradient 1 in 60, it was 64 lbs. instead of 60 lbs. as given above. This accounts for certain seeming discrepancies in the total.

rolling loads, which took place both on curves and straight lines. He thought one reason why more radical remedies had not been applied, in England at least, was the extreme difficulty of introducing fundamental alterations in the existing rolling stock —a difficulty which must more or less exist on all old systems of railways, and which could not be lightly ignored. Thus, as these difficulties had grown with the use of steeper gradients, sharper curves, longer trains, and higher speeds, heavier rails had been laid down and more powerful locomotives had been introduced to struggle through the difficulty rather than remove it. He understood the object of this Paper was to show one way in which the tractive force necessary, or in other words, the train resistance to be overcome, might be reduced. He was not going to speak to the exact question of a particular system raised by the Author, he rather desired to dwell on the question of train resistance generally. One remedy was that of radiation of the axles as they passed round curves, and for this purpose there were three systems which might be easily compared. There was, first, the old-established American bogie truck, and next, Mr. Adams' radial system, and Mr. Clark's radial system. The bogie used so universally in America accommodated itself very beautifully to rough roads. Any one who had travelled in America must be convinced of this; and these bogie-trucks ran over roads,

Table showing the tractive force required on different gradients irrespective of the curves, *i. e.*, taking an average of all the different results arrived at on the total lengths of the same gradients, though these gradients occurred both on straight portions of the line and on curves of different radii.

Gradient.	Tractive Force in Cwts.
Level	5
1 in 625	$5\frac{1}{2}$
1 ,, 300	8
1 ,, 226	10
1 ,, 110	13
1 ,, 100	14
1 ,, 90	15
1 ,, 80	16
1 ,, 70	18
1 ,, 60	19
1 ,, 50	20

The curves on the Main Range are so short for the most part, that it is difficult to obtain very accurate results. The foregoing were compiled from a great number of experiments.

especially in winter which European rigid-rolling stock would have difficulty in contending with. At the same time the bogie did not support the centre of the carriage, and it left a great length to be trussed or supported in some other way. It required an extra pair of wheels, and slightly raised the centre of gravity of the rolling stock. He believed that an immense advantage, as regarded the question of train resistance, was derived from a well-arranged system of radiation.

With reference to loose wheels, he had found it necessary to provide some good plan of getting round curves of 5-chain radius, and for that purpose he had tried several systems. Amongst others he made some experiments with loose wheels, one on each axle of a four-wheeled wagon. These wheels were 2 feet in diameter, carefully fitted with a hard gun-metal bearing on the axle 12 inches long, and were provided with arrangements for lubrication. The axles were $4\frac{1}{4}$ inches in diameter through the wheel-boss, and were not fixed, but free to turn as usual. It was thought that an ample length of bearing had been given, but the result was not satisfactory, and these wheels had soon to be keyed to the axle. He was, however, far from saying that a loose wheel could not be devised that would answer the purpose without 'wobbling.' Mr. Brunel had tried many experiments on this subject, but the results were not satisfactory.

He had for some years tried experiments with conical tires, and he believed that though it was true, on curves of large radii and at high speed, that the flange of the hinder wheel of the carriage did not touch the inner rail, but that both flanges were thrown on to the outer rail, yet that on sharp curves, and at moderate speeds, this did not obtain, and that the hinder wheel did begin to drag on the inner rail. The result was that the advantage of the cone was neutralised on such curves. Again, the cone on a straight line was not so satisfactory in its action as the cylindrical tire, as unless the centre line of the coned tire exactly coincided with the bearing on the rail, the wheels became of different diameters; and he thought a great deal of the oscillation on a straight line was caused by the irregular action of the cone.

Another important matter that deserved consideration, where engineers were free to act, was that of the central buffer. The introduction of this was almost an impossibility in Europe, where the use of the double buffer was firmly fixed; but he had watched the action of the central buffers in America and in Norway, and he thought that there lay in them the means of introducing a great improvement. The side buffers caused serious racking strains to the framing and did not act properly on curves, and on sharp curves had a tendency to throw trains off the rails; and, though the balance-lever had been introduced to enable the outside buffers to touch one another upon curves, this was only a partial remedy of the evil. The American central buffer was a rough and ready contrivance. It had answered the purpose well, but the mode of coupling was not first-rate. He had lately been left behind, in the last carriage of a train, in the middle of the night, in consequence of the coupling pin jumping out; but the Norwegian buffer, designed by Mr. Carl Pihl, the Government engineer, met this objection, and as far as he could ascertain answered admirably, and he was in consequence adopting it in a modified form in Canada for the whole of the rolling stock on the lines under his charge there. It caused no racking strains to the framing; the whole of the buffing and drawing strains passed through the centre, and could be carried through by a balk of timber, or by a proper arrangement of angle-irons. Consequently, the frames could be lightened, and the combined buffing and drawing apparatus was hardly more than one-third of the weight of that in ordinary use in England.

Another question deserving of discussion was that of loose couplings. Two totally opposite systems obtained in this respect in England and America. The practice in England was to screw up the carriages very tightly together with screw couplings, and he believed that was essential with such short carriages as those used; but in America, where the carriages were long and heavy, loose couplings were used; and those who had ridden on American railways could bear testimony that, for some reason or other, the American cars ran with a much pleasanter motion than English carriages. There was much less of the disagreeable tremor, which rendered reading difficult here; the movement of the American car was more of a roll than a shake; each car ran freely without in any great degree communicating its motion to the adjacent cars; and he was inclined to believe that this reduced the train resistance. A little more care in starting the train was all that was required.

Another important point was to have the centre of gravity low down. It was always an object to reduce its height as far as possible, inasmuch as by that means the gauge concussion was reduced, and the steadiness of the rolling stock was improved. He had ridden at considerable speed on a line of 3 feet 6 inches gauge, with carriages 8 feet 6 inches wide, which was the extreme width he now adopted for that gauge, and those carriages ran with great steadiness; no doubt in a great measure owing to their low centre of gravity.

There was a wide field open for the exercise of the ingenuity of the profession in reducing the dead weight of rolling stock. Why should it exceed from one-fourth to a maximum of one-third of the gross weight? The improvements in springs had reduced the shocks caused by shunting, which had until lately been the bugbear of engineers; and if the German system of a double set of bearing springs were introduced, that evil could be almost got rid of.

He found that, on the sharp curves on the Queensland railways of which he had spoken, one point tending to reduce train resistance was to lay the rails comparatively wide to gauge. Thus on 5-chain curves on the 3 feet 6 inches gauge, by laying the rails fully $\frac{3}{8}$ths of an inch wide to gauge, the friction and the wear of the tires were much reduced.

When he was in America a short time since, a pneumatic break was in use on the Pennsylvania Central railway, which appeared to answer all essential conditions. It was applied by the engine-man to every car by the simple turning of a small handle, and the great point was, that it acted, though almost instantaneously throughout, yet on the hinder end of the train first, thus keeping it drawn out tight. It was a simple arrangement of an air cylinder on the engine, in which the air was kept constantly compressed by means of a small steam cylinder. Thence the air was forced throughout the train through pipes, which could be quickly coupled or uncoupled, and in case of accident to the train, had self-acting valves, which closed the supply of air. The breaks were applied to the wheels of each carriage by means of pistons working in small air cylinders placed under the centre of the frames.

Mr. J. A. Longridge said he thought the question was one of reducing the friction of the train, whether of the axle bearings of the carriages themselves, or of the rolling resistance upon the rails; and whatever might prove best in that respect would be equally satisfactory if the dead weight could be reduced, although he thought this could not be done to any great extent.

He had for many years been of opinion that loose wheels were suitable for railway carriages. He believed, too, that radial axles were exceedingly important, and on this matter he could speak partly from experience, and partly from the mathematical nature of the problem. On the Mont Cenis railway, which had curves of 40 metres radius, being the sharpest curves of any railway in the world, loose wheels and radial axles had been adopted with perfect success. He was aware that great objections were made to loose wheels, and that it was said they got a 'wobbling' motion, which rendered them dangerous. His experience on the Mont Cenis railway disproved this. The carriages which had these loose wheels had a wheel base of 14 feet, and they had never given the slightest trouble; in fact, there was a considerable diminution in the wear of the tires. The carriages were of two classes, viz., four-wheel carriages of the ordinary kind, with a wheel base of 6 feet, and carriages on Mr. Clark's radial system, with six wheels, having a wheel base of 14 feet; and he had no hesitation in saying the friction of the six-wheeled carriages was less than that of the four-wheeled in going round the curves.

It was the general opinion that on a curve the outside rail must be elevated to counteract the effect of centrifugal force, and that the centrifugal force would drive the carriage against the rails on the outer side of the curve. When he first went to Mont Cenis, in company with Mr. Crampton, they looked into that question, and they found that when a four-wheel carriage passed round a curve one of the leading wheels was always running hard against the outside rails, whilst the trailing wheel on the opposite side was running equally hard against the inside rails. Where the centre rail existed the guide wheels showed the same action, viz., that the front of the carriage was passing outwards, and the hinder end inwards, as regarded the curve, and this action took place at as high a speed as 35 miles to 40 miles per hour, thus showing that the centrifugal force never brought the carriage up against the outer rails. It could not do so, because for that the whole carriage must have a sliding action in a horizontal direction upon the rails; and at the small angle at which that was found, the coefficient of friction was so great that the sliding action was never overcome.

In the case of a carriage passing along a curve there was, besides the forward motion, which, of course, would be rectilineal, another motion causing the carriage to rotate horizontally round its centre; so that when the curve was an entire circle, the carriage had made an entire revolution in passing round it; and, as there was no fixed pivot, this could only be caused by two forces acting as a couple, one at the front wheel, another equal, and in an opposite direction, at the back; and, as a matter of fact, it was found that, in going round sharp curves, the flange of the hinder wheel never touched the outside rail at all. Mr. Fox's remark, that the cone of the wheel in such a case acted in the wrong direction to what it ought was perfectly correct, and therefore it was better to abandon cone-wheels altogether.

Mr. Crampton and he had tried an experiment in the mode of coupling. The carriages on the Mont Cenis line were coupled by a short link joining two pins in the centre of the carriages. When so coupled, this action of the wheels was, as before described, in each carriage of the train. But on doing away with the link altogether, and joining the carriage by one pivot without an intermediate link, it was found that the tendency of the hinder wheels of the first carriage to run against the inner rail was counteracted by the tendency of the second carriage to run against the outer rail. Under these circumstances, all the carriages excepting the last ran perfectly well, the last carriage having the same defect as in the ordinary case. He did not think that was an advisable way of getting over the difficulty he had pointed out, because it brought a greater side strain upon the couplings and frames of the carriages. The system of radial action in the six-wheel carriages on the Mont Cenis answered exceedingly well, and he thought that for six-wheel carriages nothing could be better; but it was not applicable to four-wheel carriages. He was now adopting radial axles to some rolling stock; but there was one defect in that plan, which was this—unless the radial bars were connected, and they were not, that the hind wheels had just the reverse action to what they ought to have. Therefore in the carriages he was now having constructed, he had adopted a provision by which the axles must move simultaneously, but in opposite directions, so that both would always be pointing to the centre of the curve. As the front axle took the curve first, he had no doubt it would bring the hind axle into the proper position, and the two axles on a curve would be in their proper position. There had been no difficulty with loose wheels, as he had already said, on the Mont Cenis railway; the axles did not revolve, but the wheels revolved on the axles. In the system he was now adopting the axles would revolve in ordinary axle-boxes, whilst the wheels would revolve independently only in passing round curves. He believed it was better to have cylindrical wheels than conical wheels; and he was putting on some of his carriages wheels with conical tires, and on the hind wheels of the same carriage cylindrical tires by way of experiment; but he had no doubt the loose cylindrical wheels would work well, for he knew from experience they were doing exceedingly well on Mont Cenis. The wheel bearing was about 9 inches.

Mr. W. Naylor remarked that the St. Helen's and Runcorn Gap Company once undertook to haul the wagons from a convenient site at the collieries, and to deliver them at a convenient siding at Runcorn Gap. The colliery people found their own wagons, and collected them in the siding, and for that purpose horse power was employed. The curves were very sharp indeed, and two of the colliery owners adopted the plan of loose wheels, or one loose wheel on each side, so that each wheel could revolve freely and independently of the other. This was considered at first a great advantage, as two horses could take as heavy loads round the sharp curves with loose wheels as three horses could do with the wheels fast on the axles. Notwithstanding this great advantage, and notwithstanding that forty years ago the trains travelled on that line at comparatively slow speed, the oscillation was considerable, and there was a sinuous motion from side to side; and that was so much the case when descending the steep inclines at Widnes and Sutton, that the owners of the wagons became alarmed. The engineers of the permanent way complained that those wagons damaged their line, and the result was that, rather than incur the risk of danger, they keyed the wheels fast and decided upon using more horses for hauling the wagons to the railway siding.

In a four-wheel carriage designed by Mr. Bergin for the Dublin and Kingstown railway in 1836, each wheel had a separate axle. The carriage came out of the workshop on to the siding with the greatest ease; but when it was attached to a train travelling at a speed of 30 miles an hour the sinuous motion from side to side—he might almost say jumping from side to side—was so great that it was never again used.

He next met with a six-wheeled engine by Mr. Joseph Woods, in 1839, which was placed on the then London and Southampton railway. The leading axle had one loose wheel, and the others fast wheels. The engine was sent out and worked well, but Mr. Woods, wishing to go a little further, put on one end of the axle a wheel $\frac{5}{8}$ths of an inch larger in diameter than the other; one wheel being still loose on the axle. That engine did its work well; but after a time it was determined that the wheels should be made fast on the axle; accordingly the wheel was keyed fast to the axle, and the engine was sent out to work. When the train arrived at Southampton the driver called his attention to the engine, as he could not imagine what was the matter with it, and when he was informed that one wheel was $\frac{5}{8}$ths of an inch smaller in diameter than the other, he at first refused (and was at last with difficulty persuaded) to take the train back with that engine, stating that he would sooner be discharged than risk his life on the road.

On another occasion, in the year 1839, when he went into the Southampton district to take charge of that section of the South

Western railway, he found an engine that had been used by the contractors as a ballasting engine. The wheels were in a very strange condition, with the flange on one side cut down as square as if it had been done in the lathe. On examining the engine, it was found that on one side the wheels were 5 feet 9¼ inches apart, while on the other side they were 5 feet 10 inches apart. The engine in that condition had the axles as if they had been radially constructed for a curve of 560 feet radius; while the previous engine he had referred to, with the difference in the diameter of the wheels of ⅝ths of an inch, was adapted for a curve of 350 feet radius. Notwithstanding, these engines did not run off the rails.

The line from London to Southampton was opened on the 11th of May, 1840, and on the 17th of the same month an engine by Messrs. Sharp and Roberts, while going over a piece of straight line, in good order and well ballasted, ran off the rails in a cutting, turned over, killed the engine-man on the spot and injured a number of other people seriously. The engine when examined appeared to be perfect, and the question was raised, how could the accident have happened? He never heard it solved, but he would attempt to do so. At that time there was a strong bar of iron between the engine and the tender, keeping them together. If the engine ran from side to side with a sinuous motion, the tender ran from side to side also, but in opposite directions; consequently there would occasionally be a tremendous thump between the engine and the tender quite sufficient to snap an iron bar 1¾ inch in diameter; and he thought that was the reason why the engine was thrown off the line, the momentum of the train acting upon the tender as a rudder on a ship.

Again, on the line between Salisbury and Bishopstoke, a train was travelling at a speed of about 40 miles an hour over a piece of bad road; the road had been newly laid, and the ballast was not good. While travelling round a curve of a radius of ¾ths of a mile, the engine ran off the line on the inside of the curve, continued along the embankment for some distance, and finally went down the slope into a field.

With regard to axle friction, in 1849 he investigated the question of what was the cause of oscillation in railway trains, and for that purpose he made experiments with a pair of wheels 3 feet 6 inches in diameter keyed on an axle. These were run down an incline from the Dorchester station; then over a level; and then down another incline a distance of 9,960 feet, the average gradient being 1 in 178. The wheels then travelled up an incline of 1 in 200 for a distance of 2,050 feet, coming to a state of rest on an average gradient of 1 in 262, equal to a gravity of $8\frac{1}{2}$ lbs. per ton. This he attributed to there being no axle friction; there was nothing upon the wheels; it was simply a pair of wheels turned loose, and he could only attribute the result to tire friction and to atmospheric resistance. The centre of the axle was the moving point, and the upper part of the wheels would be moving at twice the velocity of the axle, and consequently the resistance of the atmosphere would be quadrupled. The object of this experiment was to notice the effect of the cone in causing the wheels to run from side to side; to observe this he followed upon an engine close after them, and saw that the lateral movement was considerable.

Mr. Bramwell remarked that Mr. Adams had brought forward a variety of very useful suggestions. He thought, however, if Mr. Adams hoped to get the great saving he had spoken of by reducing axle friction he would be disappointed: because it was unlikely that the resistance due to axle friction was so great as to admit of any large reduction. Last year he had the opportunity, on the South Western railway, of trying some experiments with a steam break; and to ascertain what was the value of the break it was necessary to find out what was the resistance of a train of a known weight, and running at a known speed. The weight of the train, including the engine and tender, on which he tried the experiment, was 160 tons, the speed was 40 miles an hour on a level, the rails in good order, and the weather propitious; the steam was shut off and the momentum of the train was left to die out. The distance the train ran was 5,396 feet; which gave an average resistance of 22·4 lbs. per ton weight of train from 40 miles an hour down to nothing. That included the train friction and the resistance of the air. There was also another source of resistance which he believed had never yet been allowed for, viz., that when an engine was running with the regulator shut there was a vacuum produced in the cylinder, and the indicator diagrams when worked out showed it was equal to $3\frac{1}{2}$ lbs. per ton of the weight of train, leaving for the other sources of resistance $18\frac{1}{2}$ lbs. Looking at these facts, he thought Mr. Adams could scarcely carry out his expectation of diminishing materially the power necessary to draw a train by diminishing the axle friction.

With regard to the question of loose wheels, he agreed with Mr. Longridge, and would not himself hesitate to run them at 60 miles an hour. He had been concerned, whilst an apprentice, in making the wheel designed by Mr. Brunel, and spoken of by Mr. Adams, and a more ill-advised piece of construction was never turned out. The wheel was 4 feet in diameter; the boss short—only 10 inches: it was made of soft iron and worked on a soft-iron axle. Let this workmanship be compared with that of ordinary carriage axles—Collinge's axles—these would run for years, because they were well made, a case-hardened box working upon a case-hardened axle: if such workmanship was applied to railway wheels he saw no difficulty in making wheels loose upon axles that would do their work. He did not propose, any more than Mr. Adams did, to fix the railway axles; these should still be allowed to revolve, and thus the amount of motion between the wheel and the axle, and the amount of wear, would merely be that due to the required differential revolution of the wheels as they, from time to time, came respectively on the outside and on the inside of the curves.

If anything were wanted to show that such wheels would effect the object for which they were intended it was contained in Mr. Naylor's statement with reference to Mr. Woods' engine on the South Western railway. In that engine the loose wheels were made of diameters differing by $\frac{5}{8}$ of an inch, and yet they ran successfully, till the men in authority keyed the wheels tight on the axle, and then they went so badly that the driver refused to take the train back with such an engine. If confirmation was wanted that the use of fixed wheels of equal diameter over curves was wrong, it was contained in that narrative of Mr. Naylor's, who had alluded to the danger of the corresponding defect, viz., the use of fixed wheels of unequal diameters on straight lines.

Further, on the question of loose wheels and radial axles, when the discussion upon the Mont Cenis railway took place,[1] he had exhibited a model of a railway curve having, instead of a pair of wheels, a long cylinder, which could be placed either in the ordinary way parallel with a radial line, or could be shifted into a radial line. With this model he showed that, when the two axles of the carriage were parallel instead of radiating, there was a strong endway thrust, and that if the axle of the long cylinder were left at liberty in its bearings the cylinder would move endways in the same way as in an ordinary printing press, where the rollers in passing over the inking table made an end-way as well as a rotary movement.

Now in a railway carriage on a curve, this continuous tendency of the wheels to move in the direction of the length of the axle

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxviii., p. 258.

could only be satisfied by a corresponding slipping motion between the wheels and the rails. To show the value of this in producing friction, and how much more important it was than the other friction, which arose from the wheels not being loose on the axles, he would make a comparison in a particular instance. He would commence by stating the friction arising from the wheels not being loose on the axle. Assume a curve of 621 feet radius—a little under 10 chains; a gauge of 4 feet 8½ inches; and a four-wheel carriage with a space of 12 feet between the wheels. The loss by the slipping caused by the wheels not being loose on the axle would be equal to the whole weight of the train through 20 feet in 1 mile; whereas the loss by slipping through non-radiating of the axles would be the whole weight of the train through 53 feet. He believed there was no difficulty in overcoming this loss arising from non-radiation of the axles, and that it might be done either by the mode suggested by the Author, or by other mechanical contrivances. He also concurred with Mr. Adams in the view that it was most desirable in locomotives to introduce the elastic medium at the earliest possible moment, and not to leave the 2-tons weight of the wheels and axle of an engine running without an elastic relief upon the rails. If the introduction of springs between the tire and the frame of the wheel were possible in practice, he believed a great benefit would result.

He quite concurred with Mr. Adams that it was undesirable to shrink tires on to railway wheels; and he also thought the use of spoked wheels in railway carriages was a mistake. He believed that both these errors had arisen simply from repeating in an iron wheel the old-fashioned structure of a wooden wheel. In a common road wheel with its nave, spokes and felloes, the only reliable bond for the whole structure was the tire; under these circumstances the tire was properly enough shrunk on; it thus drew the parts together; there was no fear of bursting the tire in the operation, because the wood would yield to its pressure, and, moreover, the spokes were made 'dished,' and thus allowed of any decrease in diameter of the wheel which the contraction of the tire demanded. But in an iron wheel with spokes not dished there was no such yielding; and therefore the shrinkage of tires on to such wheels was always most unsatisfactory, as there was constantly present the doubt whether the exact best point of shrinkage had been reached: on the one hand there was danger of bursting, on the other there was danger of becoming loose. He believed the most appropriate railway wheel was one without spokes, and where the tire was secured without shrinkage. Some of the earliest railway wheels were made of the disc construction; and looking at the uniform support such wheels gave to the tire, and to the less effect they had in disturbing the air, he believed they would eventually supersede spoke wheels for all railway plant.

There was one point on which he thought Mr. Adams was in error, and in which he was at variance with the views expressed in other parts of the Paper. He agreed that it was desirable to have a smooth road so as to get the least resistance; but Mr. Adams proposed to break the train by acting upon the wheels. If it were desirable to keep both wheels and rails in order that kind of breaking must be got rid of; for where it was used the wearing of flats in the wheels was almost inevitable, and wheels thus worn became revolving hammers. He thought the apparatus adopted by Mr. Brunlees for the São Paulo railway was an admirable one for curing this defect, and that it was applicable in all cases, though specially designed for extremely heavy inclines.[1]

The plan of breaking by 'contre-vapeur' of M. Le Chatelier, so largely used on the Continent, and now beginning to be employed in England (he believed Mr. Ramsbottom had it in operation in the Liverpool tunnel), was also an admirable one for preventing the rubbing of flat places on the wheels. In this system provisions were made which enabled the driver, when he wished to bring the train to rest, or to retard it down a steep incline, to safely reverse his engine while the steam was on. This put the pressure of the steam against the wheels; but it would be seen, on reflection, that this pressure could never be sufficient to stop the wheels from revolving; and thus, though a powerful retarding effect was obtained, the wheels could never be worn out of shape. He had experimented with this system of break and had found that with an ordinary train the driver had entire control, and could pull up in the usual distance and time on approaching a station, without making use of any other break, even a tender break. He believed, if this system were adopted, it would be found that although trains would still be fitted with tender and break-van breaks, these would never be used in the ordinary service of the trains, but would be brought into operation only when it was desired to pull up very sharply to avoid collision.

Mr. G. H. PHIPPS said he thought Mr. Longridge undervalued the usual practice of the super-elevation of the outer rail upon railway curves, arguing from experiments upon the Mont Cenis

[1] *Vide* Minutes of Proceedings, Inst. C.E., vol. xxx., p. 40.

railway, that the bite upon the rails was upon the foremost part of the parallelogram formed by the four wheels of the carriage, on the exterior rail, and, at the after part of the same, on the interior rail of the curve. Now, if the action referred to took place upon a line similar to ordinary lines of railway, without the central rail, the cross-cornered bite above referred to would, in his opinion, by no means prove that the lateral pressure against the forward wheel flange was not considerably greater than that against the after wheel flange, and thus still leave the necessity for super-elevation of the outer rail, as usually considered; but if to this was added the action of the horizontal wheels upon the central rails in the case cited, he feared that the experiment could not properly lead to the abandonment of the usual modes practised in this country for security against the effects of centrifugal force, whether consisting of super-elevation of the outer rail, or, on very sharp curves, of the check rail usually adopted.

Mr. J. A. Longridge explained that he did not dispute that centrifugal force had no influence on the movement of a railway carriage round a curve, but that it was not sufficient to throw the carriage wheel flanges against the outer rail. The proof of that was that the hind end of the carriage never bore against the outer edge.

Mr. J. B. Fell said he took considerable interest in the working of sharp curves, and could confirm to the fullest extent all that had been said by Mr. Longridge in regard to the satisfactory manner in which the system of loose wheels had worked on the Mont Cenis line. On the Paris and Scéaux railway, which had sharp curves of $2\frac{1}{2}$ chains radius, the loose wheel system had been in operation for more than fifteen years. The axle-boxes of the Mont Cenis carriages were made after the model of those used on that line, which he believed had run over 100,000 kilomètres with but trifling repairs. With regard to the plan Mr. Longridge proposed, in place of Mr. Clark's method, to use the diagonal rods for obtaining the radial movement, he had been told by M. Arnaud, of Paris, the inventor, that it answered exceedingly well: it was a more simple mode of imparting the movement to the axle. The result in carriages in which it had been tried was perfectly satisfactory. Mr. Naylor had told what happened to an engine with one leading wheel $\frac{5}{8}$ths of an inch larger in diameter than the other. A curious instance of the kind occurred some years ago on the Maria Antonia railway, a single line, where a movement was observed to take place in the rails on one side in the direction of Florence and on the other side towards

Pistoja. No one could make out for many months what was the cause of this; but at length one of the engines came into the shop for repairs, when it was found that one of the driving-wheels was $\frac{1}{4}$th or $\frac{3}{8}$ths of an inch larger in diameter than the others. That engine ran probably twelve months before the defect was discovered, which had no other than the above result; and he was not aware that the engine had at any time left the rails. After the wheels had been turned up and made even the movement of the rails ceased.

Mr. J. Clark observed that train resistance on curves might be resolved into flange friction, and unequal tread. Taking the 4 feet $8\frac{1}{2}$ inches gauge, when a train passed round a circle of any radius of the same gauge half the weight of the train had a slip backwards upon the inner rail to the extent of 31 feet. To overcome this slip one wheel should be loose upon the axle, and by slipping upon the boss, instead of on the tread, the friction would be reduced to about $\frac{1}{10}$th of what it was when the wheels were fixed; and again this might be much more reduced by lubricating the boss. Nothing was gained by coning the wheels, and fixing the axles parallel in the framing. But if the axles were made to radiate, the conical tread had a circumferential bearing $1\frac{1}{4}$ inch larger towards the flange. On a curve of 7-chains radius, it was equal to the 31 feet difference of rail; for one of 5 chains radius it was rather in excess the wrong way; and for one of 10-chains radius, there was a falling off. On a straight road the conical tread acted as a drawback, because the wheel worked to the high side, and the opposite wheel running on the small part of the tread had a scraping action on the rail; and this had the effect of throwing forward the wheel with the larger bearing or tread on the opposite side, which was the cause of the oscillation complained of.

In the case cited by Mr. Longridge, the leading wheels were stated to grind against the rail on the outside, and the trailing wheels on the inside; but if the trailing wheels were examined from behind they would also be found to grind against the outside. He thought the centrifugal force, at a speed of 60 miles an hour on a large curve, was so small that it had no effect when the ordinary cant was given. Loose wheels were the only practical solution of the difficulty; but he thought the bearing ought to be one-third outside the wheel and two-thirds inside. If the bearing was all inside, he thought the wheel would soon get too loose.

The bogie arrangements proposed did not seem to him to afford sufficient pressure to deflect the leading wheels the right way;

nor did he think that putting rods in any position would effect the purpose. The bearing followed the track of the wheels that carried it, and there was nothing to prevent them grinding upon the outer flange.

Mr. G. J. Morrison remarked, that centrifugal force upon carriages going round a curve was a horizontal force. Taking W to represent the weight of the carriage, V to represent the velocity in feet per second, and R to represent the radius in feet, the centrifugal force was given by the formula $\frac{W V^2}{32 \cdot 2 \times R}$.

In the case of a carriage passing round a curve, its weight acted in a vertical direction, and the centrifugal force in a horizontal direction. It was stated that centrifugal force in ordinary cases was insufficient to make both the leading and the trailing wheels press against the outer rail. Perhaps in many cases that might be so. He would suppose an extreme case, but one within the limits of possibility—a curve of 2-chains radius, with a speed of 45 miles an hour. At that speed centrifugal force would be exactly equal to gravity; so that the carriage would be in this position: it would be acted on by its own weight in a vertical direction, and by a force equal to its own weight in a horizontal direction; and it would be just as reasonable to build a vertical wall on a curve of 2-chains radius, and put the rails on the wall and set the carriage to run round that, as to lay the rails of a 2-chain curve without any cant and send a carriage round at that speed.

He understood Mr. Longridge to say that the centrifugal force was not sufficient to make the outer wheels bite the rails, and therefore it was unnecessary to give any cant, because the cant was put to make the friction somewhat less. In the case which he had supposed, which was an extreme one, the danger would be extreme; but in ordinary cases, on a curve of 2 chains radius, at a speed of 20 miles an hour, a speed which had been worked for years on the Mont Cenis line, if there were no cant, the carriage would be in a dangerous position. If a vertical line were drawn to represent the weight of the carriage, and a horizontal line to represent centrifugal force, and the parallelogram was completed in the ordinary way, then the resultant of these two forces would be the diagonal of the parallelogram, and the rails must be laid at right angles to that, to put the carriages in a proper condition for running round curves. Whether the outer wheels pressed against the rails or not had nothing to do with the question.

With regard to the friction of a four-wheel carriage passing round a curve, in the first place, the outer rail was longer than the inner rail: and taking G to represent the gauge, measured from centre to centre of rails, this excess of length was $2 \times G \times 3 \cdot 142$ in a complete circle, no matter what the radius might be. With the ordinary gauge that was about 31 feet; but it was more convenient to consider the excess of length in 1 foot, and this excess was $\frac{G}{R}$. Either the outer wheel would drag forward, or the inner wheel would slip back: but in any case, half the weight must be moved over that distance.

Let W = weight of carriage,
R = radius of curve,
G = gauge (centre to centre of rails),
L = length of wheel base,
F = coefficient of friction;

then the total friction arising from this cause would be—

$$\frac{G}{R} \times \frac{W}{2} \times F = \frac{W\,F}{2\,R} \times G \quad . \quad . \quad . \quad (a).$$

The next friction to be considered was that arising from the fact that the axles were parallel.

The wheels constantly tried to move in a direction at right angles to their axles; but each wheel was compelled to move along a rail which was inclined to that direction at a certain angle a, and the friction arising from this would therefore be expressed by the formula $W \times \sin a \times F$; but as $\sin a$ was equal to $\frac{L}{2\,R}$, this might be written $W \times \frac{L}{2\,R} \times F$; or, to compare with the last formula,

$$\frac{W\,F}{2\,R} \times L \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (b).$$

There was another cause of resistance; the force to overcome the last-named friction was not applied directly, but indirectly, by the grinding of the flanges against the rails. The friction arising from this cause was somewhat difficult of calculation, but he believed it might be expressed very nearly thus:

$$\frac{W\,F\,L}{2\,R} \times \frac{F}{5}, \text{ or } \frac{W\,F^2\,L}{10\,R}.$$

That was approximately right, but it varied with the projection of the flange and the diameter of the wheel. It however involved F^2, so that it must be a comparatively small amount. But the formulæ (*a*) and (*b*), which expressed the amounts of the principal resistances, might be written thus:

$$\frac{W F}{2 R} \times (G + L) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (c);$$

and he believed that expressed very nearly the total resistance to be overcome.

If the wheels were made loose upon the axles, friction (*a*) was altogether got rid of. If the axles were made radial, then friction (*b*) was got rid of; but L was always greater than G. Therefore it was more important to make the axles radial than to make the wheels loose. He had no doubt loose wheels could be made to work well. Those he knew most about were on the Mont Cenis railway, where the speed was not great, so the evidence was not conclusive for high speeds; still, as far as they were tried, they worked admirably. He thought it might be taken for granted that resistance (*a*) might be got rid of altogether, by making the wheels loose; but as regarded resistance (*b*), it was rather a difficult affair.

Mr. Clark's system of radial axles was, no doubt, nearly perfect, and almost mathematically correct; but it involved three axles in a carriage. That was frequently considered to be a disadvantage. The plan adopted in America did not get rid of friction (*b*) altogether; but by the adoption of bogies the length of the wheel base (L) was reduced, and that being the case the amount of friction (*b*) was also reduced, although it was not removed. A great deal had been said about bogies running very smoothly on American railways. Bogies had the advantage of adapting themselves to the particular portion of the road on which they were running. Now, in a system like Mr. Clark's, the position of the axles depended upon the position of the three points at which the wheels touched each rail; and if the road was very bad, all these axles might be in a wrong position. Therefore he thought the bogie was suitable for a bad road; but he considered radial axles superior to the bogie on a well-laid road, although they could only be adopted for carriages at present. For locomotives the radial axle, invented by the Author of the Paper, could only be applied to the leading and trailing wheels, and not to the driving wheels. He thought Mr. Clark's system of radial driving wheels would prove successful, and if so, the friction (*b*) could be got rid of; but the only method at present in use for reducing that friction (*b*), when all the wheels had to be driving wheels, was the adoption of a very short wheel base; and where the curves were severe, this object was best attained by the engine known as the double-bogie engine. The two driving wheels must always be keyed on to the same axle, and therefore it was apparently impossible to lessen the friction *(a)*, in the case of engines.

Mr. W. Atkinson remarked that on the one hand the super-elevation of the outer rail had been regarded as a theoretical question, only one carriage being dealt with at a time, and that carriage being supposed to receive the impetus within itself; while, on the other hand, it had also been treated as a practical question, in which the impetus of the carriage was not within itself but was given by the engine drawing it. Directly a train was brought on to a curve a new condition of things was instituted. The resistance of the latter part of the train tended to pull the whole train into the position of a chord; and as Mr. Longridge had said, the central portion of the train ran against the inside rail of the curve and not against the outside. From this it followed that the utility of the super-elevation was very much diminished, because the latter part of the train dragged the trailing wheels of the engine against the inner side and forced the leading wheels more strongly against the outer rail. In practice it was often found that when the theoretical super-elevation had been given it had to be reduced considerably, because the engines ran off the line. By giving super-elevation the outer wheel pressed less heavily on the rail, and the train dragged the trailing wheels in such a way as to drive the leading wheel of the locomotive over the rail; therefore, though he felt some diffidence in saying that the super-elevation should be done away with, yet in practice it had to be much modified.

There was, no doubt, some truth in Mr. Fox's theory of accounting for the easy travelling in the American cars, but the principal cause of the easy motion was the fact, that the carriages were supported upon two distinct systems of springs. First, they had two or three wheels on either side, which prevented the inequalities of the road from being felt; then there was the bogie frame itself on springs; again, the carriage was placed in the centre on a bar resting on other springs, and consequently almost the whole of the tremulous motion which existed in European carriages, and which was conveyed directly from the road to the passengers, was removed. To effect this, attention must be directed not to slack coupling, but to some intermediate system of springs. He thought cylindrical were preferable to conical wheels, because in the case of curves little advantage was derived from the conical form, on account of the greater part of the train running against the inner rail. The want of reliable information on the resistances of the train on curves was a disgrace to the profession. There was no information in a tabulated form in any engineering book. He had reduced to the single form a rule laid down by Mr. Latrobe—that on a curve of 100 feet radius the resistance was 30 lbs. per ton, and inversely as the radius. That was given as the result of American experience of American rolling stock, but its insufficiency was admitted by American engineers. Then in the experiments on the Erie railway[1] it was estimated that the resistance was equal to $\frac{1}{2}$ lb. per ton per degree at the centre of the circle subtended by 100 feet at the circumference. By reducing this rule to the terms in which Mr. Latrobe's had been expressed, it was found that 28·75 lbs. was given as against 30 lbs., so that it was evident that both rules had a common origin. In both cases the length of the train and the speed were ignored, although it was manifest that the resistance would depend upon the length of the train and its speed.

The only other formula he was acquainted with was that given by Mr. Molesworth, in which the resistance was stated to be equal to 1 per cent. of the ordinary resistance of the train on a straight and level line for each degree of the curve occupied by the train. If that was reduced to the same form as the American rules, the three rules agreed in the condition that the train in this last case was 435 feet in length, and that the ordinary train resistance was 12 lbs. per ton at a speed of 20 miles an hour. In this formula the resistance of the curve was partly dependent upon the speed and partly upon the length of the train, so that in a train of 100 feet the resistance would have been only one-fourth. Thus this rule must be entirely wrong, and as far as he knew these were the only data extant. It was therefore most desirable that further experiments should be made and tabulated, and at the same time the length and speed of the train, and the state of the rails, should be recorded.

The only experiments he had made were on the speed of trains round curves. He took a train up an incline; drove it first along a straight line, and noticed the speed on entering the curve, and then when he had passed it; but what was most remarkable was that, with a train of 120 tons on a gradient of 1 in 66, going at a low speed, the loss on the curve was 25 per cent. of the speed at which the train entered upon it; while on another occasion, at

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxviii., p. 369.

a speed of about $11\frac{1}{2}$ miles an hour, the loss was only a very small percentage of speed. This showed the difficulty of arriving at positive conclusions, as a great deal depended upon the state of the rails at the time of making the experiment.

Mr. STEPHENSON said that whatever experiments had been tried on the Mont Cenis railway, and whatever might have been the results obtained from those experiments, they would not induce him to recommend the great railway companies of this country to adopt loose wheels for the rolling stock upon their main lines, running at a speed of 60 miles an hour. The question of the merits of fixed and loose wheels was not a new one. Many first-class mechanics had formerly devoted their time in endeavours to bring out an axle that would enable them to surmount the difficulties referred to: the object being to make each wheel travel the distance that was delineated upon the length of railway which that wheel had to pass over on the curves.

His father in 1826 produced a double axle, which was shown in Fig. 1 (p. 400). He was aware of the fact that any pulley running loose on its axle 'got drunk.' A practical man going through a manufacturer's shop would notice that a loose pulley which had no work to do always wore out sooner than the pulley which had all the work to do. It was evident, therefore, that if the wheel was to turn round upon the axle it must have a broad base. The wheels for these axles were employed upon a colliery rolling stock, and were used both below and above ground at a speed of 4 miles or 5 miles an hour, and a considerable sum was saved by preventing wear and tear of the wheels. The men used to skid the wheels upon the inclined planes, and those wheels, when they needed repairing, were sent to him, and he had sixpence each for turning them up. He might have got a shilling if they had not been on loose axles.

There was an axle, Fig. 2, the details of which he could not quite recollect. He had drawn it to the best of his ability, but he thought it was invented by his father after 1826. In this case there was a hollow axle having a wheel keyed on it at one end, while a solid internal axle ran through it, to the end of which the other wheel was keyed, the wheels being kept in gauge by collars. When these axles were first used he was engaged, as an engine-man, in winding the coals up the inclined plane where they were in use. For twelve months he worked an engine with a boy as fireman, and found that at low speeds the loose wheel answered its purpose.

The axle, Fig. 3, had not been invented when he was at the lathe, so that he could not remember or describe it so well as those previously mentioned. He believed it was introduced about the year 1831.

Fig. 1.

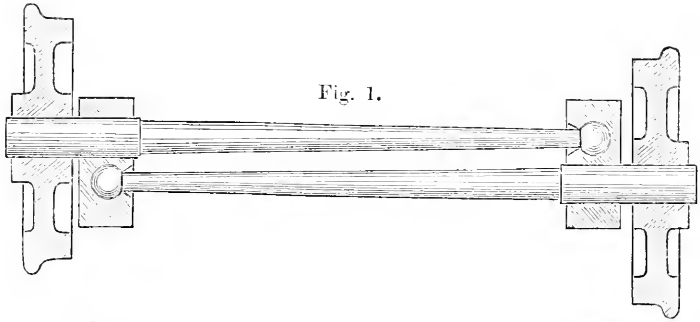


Fig. 2.

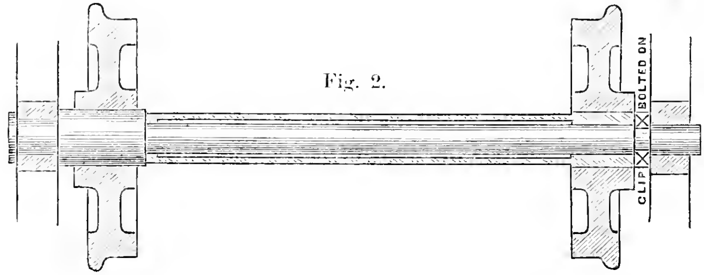
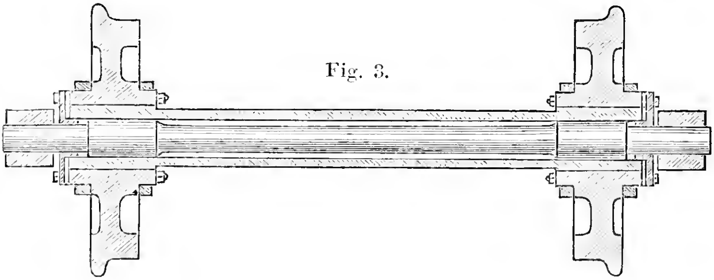


Fig. 3.



After the time to which he had referred, experiments were made which eventually led to the condemnation of loose wheels for high speeds, though they had been tried in every possible shape. The ball and socket joint, which appeared on the double axles, was changed into an ordinary bearing, but whenever it was worked at high speed it invariably got knocked to pieces.

He might incidentally state his opinion, that the mode of propelling a carriage was based on the fact that it was not always pulled upon the centre. The draw-bar in the middle of the carriage was not always pulled upon; occasionally the whole of the strain was thrown upon the safety chains. In the case of breakage of the draw-bar it was clear the strain would come first on one side and then on the other, according to the oscillation of the carriage. This principle had been entirely lost sight of in the arguments hitherto adduced. Mr. Phipps had made the original drawings of an axle which was projected by Mr. Robert Stephenson, and this question was discussed over and over again, not only by Mr. Stephenson but by Mr. Brunel, and they all came to the same conclusion, that there was an undeniably fatal defect in making one wheel detached from the other. In order to indicate this defect, he would ask any one who had ever passed through a village and seen the village blacksmith, after having hooped a wheel, place it on a cart-axle in order to take it home? He began to push, but when he pushed one wheel, instead of the other going forwards it had a tendency to go backwards.

If any two wheels, loose on the axles, were taken from a railway carriage, the power necessary to propel them forward might start the wheel, nearest to the point where the power was applied, 2 or 3 inches in advance, while the other wheel might not have started. Again, in a locomotive yard, supposing there was a pair of wheels keyed on the axle, and a man wanted to move them forward, what did he do? He placed his foot upon the spoke and his hand upon the periphery, and pulled, and both wheels went on simultaneously, but if he placed his foot upon the spokes of a wheel loose on the axles, the two wheels performed a pirouette. He submitted that the defect indicated by these facts was fatal to any application of the loose wheel to a railway carriage. Upon the question of coned wheels, he to a great extent concurred in what Mr. Fox had said. Between the years 1838 and 1842, during the construction of Charlestown tunnel, he happened on one occasion to be on a ballast train when an opportunity was afforded him for observing the peculiar effect of the wheels upon the curve. After careful observation, he discovered that the cones were acting in a diametrically opposite way to that generally believed, and there was not a shadow of doubt that at a speed of 10 miles an hour, with a train going round a curve of 10 chains radius, every cone, except those of the engine and as far as about the third carriage, was acting in a wrong direction. This was brought about by the engine having a tendency to pull the train into the line of the bow-string, thus bringing most of the cones into the wrong position.

Mr. Brunlees said he could confirm all that had been stated with regard to the working of loose wheels and radial axles on the Mont Cenis railway. Nothing could possibly act better than the two combined in reducing train resistance. He thought that loose wheels could be applied to rolling stock with the same perfection of workmanship as fixed ones; and, as they possessed the great advantage of taking the strain off the axles, he considered their use would render railway travelling much freer from accident, and he would have no hesitation in travelling in a train fitted with properly made loose wheels at a speed of 60 miles an hour.

In adopting Mr. Clark's radial system on the Mont Cenis railway, he had to take into consideration the fact that there were on that line numerous curves of 2 chains radius, some of which were so long as to describe three-fourths of a circle, thus giving the outer wheels an additional distance to travel of 18 feet as compared with the inner ones; and he was convinced that the line could not, under these circumstances, have been successfully worked with fixed wheels and rigid axles. He was now applying the system to the stock for a much longer line, the Honduras Interoceanic railway, and was glad to find from the reports he had received, that it was acting equally as well there as upon the Mont Cenis railway. The Mont Cenis stock had not been quite correctly described. The goods wagons and a number of the passenger carriages were four-wheel, fitted with ordinary axles, and with fixed wheels on the one side and loose wheels on the other, but the remainder of the passenger carriages were made with six loose wheels, and radial axles which did not revolve.

There was no doubt but that much of what Mr. Stephenson had said, in condemnation of the use of loose wheels, was due to the fact that his experience of them belonged to an early period in the construction of railway rolling stock, when the defects to which he referred would be attributable to imperfect manufacture and fitting. Perhaps, also, Mr. Stephenson was a little prejudiced in favour of the old system. He might mention that the contractor for the Mont Cenis rolling stock was so much prejudiced against radial axles and loose wheels, probably from their being novel, that he declined to construct it with them unless he were previously allowed to make a trial of the system by fitting them upon the underframe of a carriage, and running it over the line. He had been so certain that the system was bad that nothing but an actual trial convinced him of its merits. It had been found in working that the carriages fitted with radial axles and loose wheels were much superior in comfort to the others with the fixed and loose wheels only, and were invariably preferred by persons who had opportunities of travelling by the two kinds of carriage, the motion being much steadier in the former, both upon the straight road and upon curves. He had no doubt that equally satisfactory results would be arrived at with regard to many of the improvements which Mr. Adams had suggested, and he hoped that ere long many of them would be carried into practical operation.

Mr. A. H. Macnair exhibited a diagram of a railway wagon upon a curve, the relative size of the wagon and the sharpness of the curve being exaggerated for the sake of illustration. A point marked A was the centre of the wagon, and the centre of gravity as well. The rails would cause the wagon to follow the curve, and therefore centrifugal force would come into action; but while the momentum of the wagon bore on the point A in the direction of the tangent, there would be an independent action in every other part of the wagon. If it were free from every force, except that which was inherent in itself, it would have a rotary motion of its own round the point A as a centre, which would affect it when upon a curve, and would tend to keep the inside of the wagon towards the centre of the curve, and the transverse axis of the wagon always radial. If the wagon had only one pair of wheels on an axle passing through its centre of gravity, this axle would be always radial, and the resistance would be a simple pressure against the outer rail. In the case of four-wheel wagons this resistance was not at the centre, but at the two axles. In this case, if the axles were radial the resistance would still be simply against the outer rail; but when the axles were parallel to each other the parallelism of the axles was antagonistic to the rotary motion of the wagon, and would destroy it. But the rotary motion was necessary; and therefore the outer rail must move the leading axle inwards, while the inner rail must move the trailing axle outwards, and this brought a transverse friction upon the rails which would not occur if the axles were made radial. However, the rotary motion of the wagon was not equally shared between the two axles. In illustration of this, he would take the case of a coach on a common road, in which the leading axle of the coach had radial motion, and the trailing axle followed approximately the path of the leading axle without radial motion, being fixed at right angles to the centre line of the coach. In the same way the trailing wheels of a railway wagon would keep upon the rails, even though they had no flanges, if they were only broad enough.

Therefore, if the curve was very great, as there was always a play between the wheels and the rails, that might be sufficient to prevent flange friction of the trailing wheel, and the whole flange friction would then come on the leading wheel. This was true of all parallel axles, even if they were provided with means for the revolution of one wheel independent of the other. When the wheels were fixed, the leading axle had still more friction to contend with than the trailing axle, for the longitudinal sliding action thus brought into play gave the outer leading wheel and the outer trailing wheel a tendency to hang back, which would aggravate the non-radiation of the former, but diminish that of the latter. Where the wagon formed part of a train some of these circumstances would be altered; the coupling of the wagons would tend to relieve the flange friction, as the trailing wheels of the wagon in front would tend to relieve the flange friction of the outer leading wheel, and the leading wheels of the wagon behind would tend to relieve that of the inner trailing wheel. At the same time the resistance of the train itself would not be diminished, but rather increased as it ran round the curve. He had pointed out these elementary matters in an abstract point of view, and apart from circumstances of imperfection in wheels and rails, or of oscillation, which might be called accidental. He thought radiation was the great question to be considered in the present day; one reason for which was that radiation would allow the full benefit from the coning of the wheels, if coning were of any benefit at all; but it was plain that no advantage could be obtained from coning without radiation.

There was another expedient that might tend to diminish friction in passing round a curve if the axles were radiated; that was the extra elevation of the outer rail, in order that it might be relieved of a certain amount of friction incidental to longitudinal slipping. He did not wish to underrate the importance of the independent motion of one wheel upon the axle, but considered it of less importance than radiation, though that was a question on which much difference of opinion prevailed.

Mr. Woods remarked that one observation struck him with some surprise at the outset of the Paper, an observation which he thought constituted the fundamental principle on which the conclusions of the Author were based. Mr. Adams said, "If the true principles of construction were accurately followed, resistance—other than that of gravity—should be reduced to the single element of axle friction." Mr. Adams stated that the axle friction might be taken at 4 lbs. per ton of weight, and then went on to intimate that, whereas in practice the resistance was found to be much greater than this, the excess was occasioned by disturbing elements due to faulty construction of the railway and the rolling stock. The problems referred to were by no means novel, but had engaged the attention of engineers for the last thirty or forty years; in fact, ever since the Liverpool and Manchester railway was completed. As far back as thirty-two years ago the resources of the Grand Junction and the Liverpool and Manchester railways were placed at the disposal of a committee, appointed by the British Association, whose duty it became to ascertain, what were called then, the 'constants' of railway resistance. The railways, the rolling stock, and the servants of those companies were placed at the disposal of that committee; and the conclusions arrived at were embodied in the volumes of the Transactions for 1838 and 1841.[1] Those conclusions he believed had not only never been disproved, but were now universally admitted. They showed that, beyond mere axle friction, a large amount of the resistance on railways was occasioned by the motion of trains through the atmosphere. It was, for instance, proved,—taking as an example a train of eight carriages,—that, whereas the friction of such a train at a low velocity scarcely exceeded 5 lbs. per ton, when the velocity was increased to 10 miles an hour it became 7 lbs. per ton; at 25 miles it was 12 lbs. per ton; and at 30 miles it was 15 lbs. per ton: so that the resistance was doubled, and in some cases trebled, simply by the action of the atmosphere alone. Having taken an active part in the conduct and analysis of those experiments, he could speak with the greater confidence on the subject; and he felt convinced that on railways of easy gradients and curves, like the London and North Western, Great Northern, and others, but little was to be hoped for in the way of reducing the resistance of trains by such appliances as had been proposed, whether by loose wheels, by radial axles, or by the other contrivances that had been described. In other and special cases some of these appliances were, he thought, of importance. With regard to Mr. Adams' radial axles, he had experienced the benefit of that system as applied to engines on lines where the curves were very sharp, thus enabling them to pass round curves much more easily than they would have done without it. He had no experience with Mr. Adams' radial system as applied to carriages and wagons, and much doubted whether it could be safely and advantageously

[1] *Vide* "Reports of British Association," vol. vii. (1838), p. 197, *et seq.*, and vol. x. (1841), p. 205, *et seq.*

applied to four-wheel vehicles. He had used Mr. Clark's system with success, and indeed was the first to introduce that system, on a railway in Chili, with curves of 500 feet radius, and it was there still working in a satisfactory manner. About a year ago he spent two or three days on the Mont Cenis railway, and he was pleased to find Mr. Clark's system of radial axles in satisfactory operation there. Although the carriages were fitted with loose wheels, the motion was perfectly steady, and the radial action complete, and all that could be desired, enabling the trains to travel smoothly and with ease round curves of 40 metres radius; but the speed, the gradient being 1 in 13 only, was necessarily limited, not more than, say, 7 miles or 8 miles an hour on the ascent, whilst in descending it got to 20 miles or 25 miles an hour; a speed as great as it could be desired to travel at down inclines of 1 in 13. He doubted whether it would be expedient or safe to apply loose wheels to trains travelling at express speed.

The reduction of resistance leading to economy of tractive power, became most important in those cases in which the engines had to put forth their utmost power, as, for instance, on steep gradients, or in travelling at high speeds; but in both those cases it would be observed, from what he had said, that the reduction of friction proper would amount to only a fraction of the whole resistance; for in the first case the force of gravity, and in the second case the resistance of the atmosphere, came largely into play.

No special reference had been made in the Paper to the methods used on American railways, but these, on the whole, had been found very effective; and in particular the use of the bogie truck, whether for engines or for carriages and wagons, was almost universal, in combination with the central buffing arrangement. With such combination trains readily went round curves of 500 feet radius. In working curves of that radius on the railways in Chili he had found great advantage, especially in the case of engines which had no bogie, in lubricating the wheels and rails by a jet of water, and the engines were arranged for this to be systematically done by means of pipes, which threw the water on to the rails in front of the driving-wheels.

The expediency of using elastic and loose tires on the wheels of coupled engines appeared to be very questionable; and he conceived that it would be almost impossible in practice so to adjust the tires as to allow of their slipping on the wheel centres, without at the same time being so loose as to be unsafe.

The suggestion of the application of four cylinders to an engine was not new. He believed the late Mr. Bodmer was the first to propose it many years since; but, in his opinion, the advantage that might be derived from the four cylinders would be more than counterbalanced by the increased complexity of the engine, a greater number of parts having to be attended to and kept in order.

Mr. Vignoles, President, said that before the appointment of the committee of the British Association on the subject of railway friction alluded to by Mr. Woods, the stock of the Grand Junction railway had been placed at his disposal, and he had made experiments during a period of several weeks. The result he came to was, that atmospheric resistance, both directly and laterally, formed a very large proportion of the total resistance; so much so, that in descending an inclined plane of 1 in 100 or in 120, it was about equal to the effect of gravity due to such a plane, and these resistances were increased in windy weather. He concurred to the fullest extent in the value of Mr. Adams' radial contrivance, particularly in respect of reducing friction, especially on curves; but it must not be assumed that with any contrivance of that kind, such a proportion of refinement could be obtained as would counterbalance the resistances on railways at high velocities.

Mr. Bridges Adams, in reply upon the discussion, proceeded to sum up the general objects of his Paper.

First, as to axles, his object was to obtain an unbreakable axle with the smallest quantity of material, and he therefore resorted to the tube principle as the strongest form, and as that in which the metal could be most perfectly manufactured by the reduced thickness of the hollow as compared with the solid. Hollow axles were not new, but they were new as he proposed to construct them. They had failed owing to the process of swaying down the bearings to produce collars for the bearing brasses, and in that process they were reduced in thickness where strength was most needed, and so failed. His axle was a true hollow cylinder from end to end, without any decrease of thickness at the bearings, and the collars were produced simply by the faced wheel boss at the inner side, and by a broad disc nut screwed into the axle hollow at the outer end. In this mode, collars of 2 inches or more in breadth could be obtained, instead of the insufficient ½-inch of ordinary practice, which collars were defective, and involved rapid wear and cost for replacement; moreover the increased diameter of the bearing could thus permit a larger surface for oil lubrication.

With regard to loose wheels, *i. e.*, wheels revolving on the axles, there seemed to be some want of clearness as to the object to be attained. The wheel was not wanted to revolve as in an ordinary road vehicle, for the revolution of the axle was more advantageous in many respects; but it was absolutely essential that each wheel should be enabled to slip easily round on the axle, in order to avoid the wasting of the tires and the destruction of the rails and torsion of the axles, together with the blows and vibration which were the conjoined causes of axle breakage. The worst mode of applying a wheel was by forcing it on to the axle, making the two as it were solid, with a very short boss; a boss so short, that the wheels could not be true without intense pressure, and the very solidity served to intensify the vibration. In the experiments with loose wheels they had failed from imperfect structure. In one case the bosses were too short for steadiness, 10 inches to a wheel 4 feet in diameter, and so became conical and unsteady by flange pressure. In another case, wheels only 2 feet in diameter, and with a boss 1 foot in length, failed because their heavy lining brasses were in parts adjusted by screws, which getting unadjusted caused failure. His own experience had satisfied him that it was practicable to use loose wheels with a boss 12 inches in length to a wheel of 3 feet in diameter. Many years back Captain Moorsom had a first class carriage built for the Birmingham and Gloucester railway to his design. It had four wheels, 3 feet in diameter and 12-inch bosses, simply bored out and without any lining brasses. Experiments were made with this vehicle at Euston Station, running it upon curves and straight lines by hand. On the straight line it could be observed that the wheels varied in their movement, sometimes advancing and sometimes receding slightly, as could be noticed by the positions of the spokes, but making an average movement that left them finally in their original position to each other. On the curves, a given distance increased the revolutions of the outer wheels, and the run back over the same distance restored them to their original position, as denoted by chalk marks placed on the spokes. This vehicle was in use for about twelve months, and the wheels did not get out of order, but they had a favourable condition of great importance: the axles were made to radiate, and consequently there was no unfair strain on them, or on the wheels, and there was no doubt that was the reason of their durability. Subsequently, the superintendent altered the carriage, and applied wheels and axles in the ordinary rigid mode; the only reason, so far as could be ascertained, was "to make it like the rest." But it would be preferable to make the boss half the diameter of the wheel in length, and lining it with thin sheet brass, which with black-lead powder dry, for a lubricant, would give double movement without any perceptible wear.

With regard to the application of tires, the most barbarous method was that of shrinking them on hot, putting all possible strain on them, so that they might burst under a slight blow. A loose tire was considered to be a source of danger, but in truth all tires were apt to stretch in the process of running over the rails, and the only way to prevent them from stretching was to support them by an elastic cushion round the wheel, which would absorb the blows and prevent vibration. The cushion should act as a friction clutch, holding the tire, but permitting it to slip to prevent undue strain. Experience had shown that such tires were applicable both to driving-wheels and ordinary wheels, and by the use of side rings, the tire might be made so deep and strong as always to maintain its true circular form under all circumstances. The tire so fitted became a channel in which either a steel or rubber cushion could be placed and safely held, and with rubber all vibration and ringing noise would disappear. With the wheels loose on the axles, the tires loose on the wheels, and the axles radial, the friction between tire and rail would be reduced to a minimum.

With regard to atmospheric resistance, that must be continually varying with the speed, and a still atmosphere might, under certain conditions, be more disadvantageous than a wind. Without wind the atmospheric resistance would be in proportion to the speed of the train, but with a stern wind travelling at the same speed as the train the atmosphere would give no resistance whatever. Again, a still atmosphere would give less resistance than a side wind, causing the flanges to come in contact with the leeward rail. A moderate side wind might thus produce a greater evil than a strong head wind, especially if the side wind drove the train and wheel flanges to the inner rail on a curve. In the Paper he had not alluded to the atmosphere, dealing only with the question of mechanical frictional resistance.

It had been said that certain experiments with passenger trains on the South Western railway gave very favourable results, but this depended on the circumstances. With a favourable wind and on a straight line of railway the best conditions might be attained. But the real question at issue was how to compensate by mechanical form for varying conditions. The American bogie vehicles did this imperfectly. Each bogie was centred to the upper frame, and in running the wheels could adjust themselves against the rails, the two pivot centres maintaining their relative position to each other under all circumstances, whatever might be the position of the upper frame. But he considered the position of the pivots in these vehicles to be injudicious, as it required a considerable length of wheel base to keep them steady. But by the application of the 'caster' principle to the pivots, placing them nearer to the inner than to the outer axle, true movement would be obtained even with a short wheel base. In this arrangement, the pivot carried no load, as in the American bogie, but simply served as a guide to regulate the radial movement of the wheels and axles on the free moving spring shackles. In this case, as in that of the four-wheel radial frame, and the longer frame, this same plan of 'caster' pivoting was used, and he insisted that on this plan the compensation of the free moving wheels would insure their true running, whether with badly constructed vehicles, or with vehicles distorted by accident, or with unequal diameters of the wheels on the same axle. The radial system of axles was even more important than loose wheels, for with radiation the conical tires of ordinary wheels could come into play, and so equalize the differing lengths of the pathways on curves, without disturbing the loads or upper structure by lateral movement.

A question had arisen as to the movement of the radial wagons on rails. It had been argued that the centrifugal force always kept the flanges of both the fore and aft tires in contact with the outer rail. But it was quite clear that this must be a question of speed. On the other hand, it had been contended that, whatever the speed, there was constantly going on what might be called a leeward movement, the foremost wheel flange hugging the outer rail and the hinder wheel flange hugging the inner rail, and this irrespective of radial movement or rigid fixture in the horn plates; and that therefore in the four-wheel 'caster' wagon, the front wheels might run normal to the curves of the inner and outer rails, and the hinder wheels abnormal to the curve, dragging against the inner rail. How much of this might be due to the super-elevation of the outer rail it would be difficult to say, but some engineers were beginning to doubt whether there was any advantage in the super-elevation of the outer rail, and in a line of sharp curves, where the rails had been levelled, no difficulty had occurred. So long as the wheels could be kept parallel to the rails, with the axles pointing to the centres of the curves, the flanges could not mount the rails, unless on the supposition of a projecting rail and at a bad joint, and the fishes must be very bad that permitted this. It was almost impossible that this should occur with the permanent way he had described, inasmuch as the rail was enclosed in a deep groove. With regard to the radial wagon under discussion it would be difficult for the hind wheels to drag abnormally to the curve, unless in the case of the gauge being laid abnormally wide.

A wheel 3 feet in diameter had two bases—the vertical base, a point or line vertical to the axle, and a horizontal base formed by the flange laterally against the rail. The flange base was about 14 inches in length, the 'caster' pivots being placed 7 inches within each axle; the distance was 14 inches less between them there, and the distance between the axle centres was 14 inches greater at the extremes of the flange bases. Supposing the leeward action to exist in the hind wheels hugging the inner rail, there would be a limited movement when the flange touched the rail, and the point where it touched it would be in the line of the centre pivot, without any tendency to sway the wheels either in one direction or another. Inasmuch, however, as the extreme flange base on the outer hind wheel was 14 inches behind the 'caster' pivot, it would at that point press against the outer rail and sway the axle round the eccentric centre to the normal position required by the curve, unless the gauge was abnormally wide. In such case there would be an advantage in a diagonal bar connecting the two radial axle guards and providing simultaneous movement, which he had foreseen. But in all ordinary cases, free movement of each pair of wheels, independently of the other, was desirable to minimize friction. A very long wheel base, as 18 feet or 20 feet, would, with the wheels coupled to produce simultaneous radial action, be a very unfavourable condition upon irregular curves or irregular rails, and tend to increase tire friction.

For the sake of keeping the parts few in number, the wagon frame was shown with the bearing springs fixed to it, and the axle boxes sliding beneath them with the radial movement; but, when high speed was required, he preferred to fix the bearing springs to the axle-boxes, and to give free sensitive movement by suspending the body from scroll irons or brackets on long vertical shackles, which easily yielded to the flange pressure of the rails on curves, and returned to their vertical position by gravitation of the vertical shackles on the straight line.

There was one other consideration; that of a permanent way of abnormally wide gauge giving no guidance whatever to the wheel flanges. In each case it was desirable to connect the 'caster' radial frames to the jointed traction rod at either end of the wagon, free lateral movement being given to the rod in slots of the head-stock, in such mode that the rod following the tractive force would keep the centre between the two rails, and enforce true radial movement of the wheels and axles.

In the application of breaks to radial vehicles, a departure from the common practice, of hanging the break blocks to the frame or body, was absolutely essential. He therefore suspended them from the radial frames in such a manner as to travel with the wheels, with provision for making them self-acting. If required, there could be a block to each wheel through the whole train. The action of the breaks was by long gravitation lines acting as steel-yards, on which the weight could be adjusted to the maximum, without stopping the revolution of the wheels and damaging them by sliding on the rails. The normal condition therefore was for the blocks always to be pressing on the rails, unless lifted, and this was accomplished by a system of sheeves and a running cord; one on the end of each lever moving with it, another running between the lever and the top of the vehicle over a fixed sheeve, and thence to a winch fixed on the tender or break-van; preferably the former. Thus, supposing the weight at the lever end to be 84 lbs., it would be reduced at the winch to one-fourth, and still more by the winch power. A cord passing from the winch ran over a fixed pulley at the top of the train, and then descended in a bight to the suspended pulley, rising upwards to the next vehicle, and so on throughout the train. Thus every gravitation lever was attached to a suspended pulley when in action, and each vehicle not required to be in action could have the lever suspended to a hook, out of use, the lifting cord passing onward to the next vehicle. In starting the train the driver must perforce lift all the breaks by the winch, and in stopping he would set the winch free, and then every lever in rapid succession would press the blocks on the wheels, with either the whole or a portion of the vehicles, as provided. In case of the train separating into two parts while ascending an incline, by the severance of a coupling, the lifting cord would also break, and all the breaks would act on the wheels before the train could acquire a reverse movement.

On the subject of hauling power, he directed attention to an engine with eight wheels, four coupled drivers in front and four carrying wheels behind, the carrying wheels arranged for common axle-boxes, and with a central radial bar midway between the carrying-wheels to produce radial movement on curves. In this mode the carrying-wheels sustained a tank with 2,000 gallons of water. In this arrangement, the load on the driving-wheels would be almost constant, the consumption of water at the hinder end scarcely affecting it. He had designed an engine for working curves of 2-chains radius, distributing the load over a number of wheels, all drivers, with a view to attain great power without damage to the rails. The cylinders were four in number, two on each side, placed above the other at a pitch corresponding to the centres of the driving-axles. The driving-wheels were eight in number, coupled in two groups, the extreme wheel base being 19 feet and the central base 8 feet 6 inches, the end wheels being the drivers, and without flanges. But, of course, flanges could be used to any or all of the wheels, broad or narrow, according to the curves. Each piston governed two wheels, and as the pistons acted in opposite directions on each side, the oscillating movement induced by single action was neutralized. The steam was taken from a dome on the centre of the fire-box, and the exhaust as usual, with the exception that longer exhaust pipes were needed to carry it to the chimney; but this was simply a question of proportion. As in Mr. Crampton's and other engines, and in the tank locomotive of Mr. Robert Sinclair, the cylinders could be easily covered in to prevent condensation. The eccentrics were outside between the wheels and the cranks. The load of the cylinders and piston was thus carried at the mid-length of the frame between the wheels, without end overhang. The engine could be used either as a tank engine, carrying 1,600 gallons of water in wing tanks, or as a tender engine. The buffer beams were radial. The wheels were all spring tired, which gave from 14 per cent. to 15 per cent. better adhesion, together with freedom from blows and vibration. Wedge breaks were applied to all the eight wheels by steam action, worked by the driver, in such mode as to retard, but not to arrest the wheels; and he considered that it would be as effective and as simple as the Chatelier break, both being alike a driver's operation.

With regard to rail breaks, he considered them to be useful under limited conditions. As far back as 1840 he had designed a rail break to clip the two sides of the rails, on the plan of a parallel ruler, the breaks being made to lift up and lower down, with a central lever to put on pressure. Captain Laws was about to apply this break to the Leeds and Manchester railway, but it was not thought advisable, on account of the difficulty of points and crossings. At a later period, he had devised a break of a similar kind, acting vertically on the rails, and it was ordered for some stock for a Northern railway, and actually constructed; but the superintendent of the line having expressed some dissatisfaction or doubt, he had it removed, not desiring to send an improvement to a distance to be worked under an adverse feeling. On the whole the pressure of blocks on the wheels he considered to be the most eligible arrangement, provided they were distributed over a sufficient number of vehicles, and guarded against setting the wheels fast.

The last consideration was the structure of the permanent way, which he considered to be an integral part of the rolling stock, without which rolling stock could not be used. He thought if all brittle and rotting material were excluded from permanent way, and the rail rightly constructed with regard to the work, what was called maintenance of way might almost disappear, and its annual cost go into dividend. If, for the heaviest work, it would be desirable to use a rail 8 inches deep with a single head, 3 inches in width, supported beneath the head by a channel sleeper of rolled iron, with a pair of upper wings lying on the ballast surface, and packed from the surface without need of 'opening out,' and a deep keel preserving the gauge; all the advantages of the longitudinal system, with a broad and almost continuous bearing, could thus be attained, but without the disadvantage of a continuity preventing their being laid in curves. The types of joints were only four in number, and the joints per mile a few over ten thousand—about half the usual quantity—and without any screw-bolts or brittle or rotting material. It was, moreover, a safety rail, first, by its great depth, adding to its strength; and next, by the greater part of that depth and length being buried in a wrought-iron channel and held down by cross keys, so that in case of a rail breaking it was scarcely possible for it to get out of position in the sleepers, or for the sleepers to get out of position in the ballast.

January 31, 1871.

G. W. HEMANS, Vice-President, in the Chair.

The discussion upon the Paper No. 1,284, "Train Resistance on Railways," was continued throughout the meeting.

February 7, 1871.

CHARLES B. VIGNOLES, F.R.S., President, in the Chair.

The following Candidates were balloted for and duly elected:—William Crouch, John James Montgomery, and Charles George Napier, as Members; Guybon Damant Atherstone, Thomas Aveling, Charles Colson, Henry Crabtree, Alexander Milne Dunlop, John Eunson, Martin John Farrell, George Fowler, John Russell Freeman, William George Freeman, Frank Alexander Brown Geneste, James Metcalf Hawkins, Peter Lindsay Henderson, James Archibald Hamilton Holmes, Joseph John Maclean, Samuel Lack Mason, Francis Ingram Palmer, Navigating Lieutenant, R.N., Daniel Pidgeon, Robert Carstairs Reid, Thomas Miller Rickman, Berkeley Craven St. John, Joseph Tomlinson, Douglas d'Arcy Wilberforce Veitch, Stud. Inst., C.E., John Waugh, and Francis George Wynne, Stud. Inst., C E., as Associates.

It was announced that the following Candidates, having been duly recommended, had been admitted by the Council under the provisions of Sect. IV. of the Bye-Laws, as Students of the Institution:—Rodolfo de Arteaga, William Dugald Campbell, Edward Alexander Dunn, Walter Faithful Garland, Harry Robert Kempe, Henry de Quincy Sewell, and John Slate.

The discussion upon the Paper No. 1,284, "Train Resistance on Railways," was continued and concluded.

[Index.

INDEX

TO THE

MINUTES OF PROCEEDINGS,

1870-71.—Part I.

END OF VOL. XXXI.

LONDON:
PRINTED BY WILLIAM CLOWES AND SONS, STAMFORD STREET
AND CHARING CROSS.