IMAGE EVALUATION
TEST TARGET (MT-3)

Cana

Technical Notes / Notes techniques

The Institute has attempted to obtain the best original copy available for filming. Physical features of this copy which may alter any of the images in the reproduction are checked below.

L'Institut a microfilmé le meilleur exemplaire qu'il lui a été possible de se procurer. Certains défauts susceptibles de nuire à la qualité de la reproduction sont notés ci-dessous.

- [x] Coloured covers/ Couvertures de couleur
- [ ] Coloured maps/ Cartes géographiques en couleur
- [ ] Pages discoloured, stained or foxed/ Pages décolorées, tachetées ou piquées
- [ ] Tight binding (may cause shadows or distortion along interior margin)/ Reliure serré (peut causer de l'ombre ou de la distortion le long de la marge intérieure)
- [ ] Coloured pages/ Pages de couleur
- [ ] Coloured plates/ Planches en couleur
- [ ] Show through/ Transparence
- [x] Pages damaged/ Pages endommagées
- [x] Additional comments/ Commentaires supplémentaires — Fold-out maps, charts, etc., may be filmed at a different reduction ratio than the rest of the book.

Bibliographic Notes / Notes bibliographiques

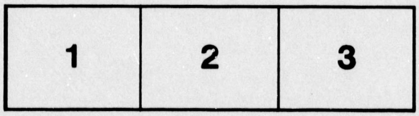
- [ ] Only edition available/ Seule édition disponible
- [ ] Bound with other material/ Relié avec d'autres documents
- [ ] Cover title missing/ Le titre de couverture manque
- [ ] Plates missing/ Des planches manquent
- [ ] Additional comments/ Commentaires supplémentaires
- [ ] Pagination incorrect/ Erreurs de pagination
- [ ] Pages missing/ Des pages manquent
- [ ] Maps missing/ Des cartes géographiques manquent

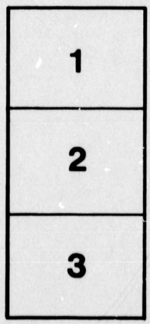
The
poss
of th
film

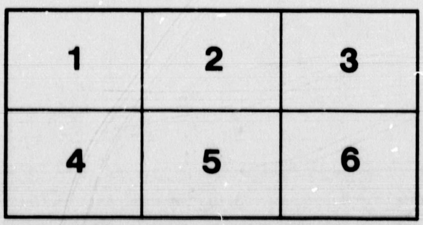
The
cont
or th
appl

The
film
insti

Map
in or
uppe
bott
follo


1
2
3


1
2
3


1
2
3
4
5
6

8.7

THE

FORTH BRIDGE

BY

B. BAKER

READ AT THE BRITISH ASSOCIATION
MONTREAL 1884

London:
PRINTED AT THE BEDFORD PRESS
20 & 21, BEDFORDBURY, W.C.
1884

1884
(46)

THE

FORTH BRIDGE

BY

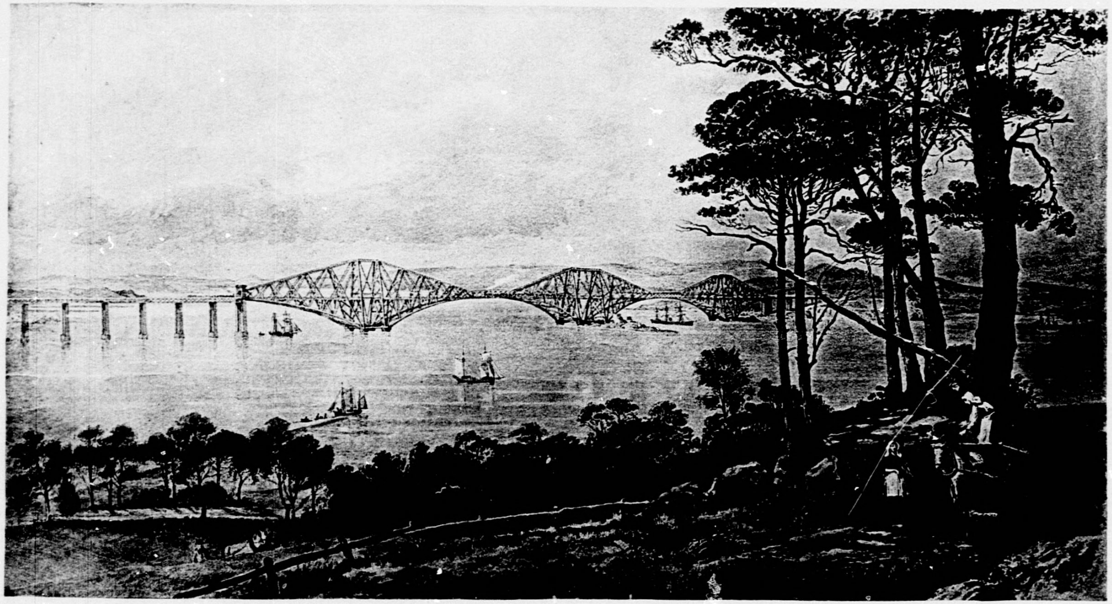
B. BAKER

READ AT THE BRITISH ASSOCIATION

MONTREAL 1884

London:
PRINTED AT THE BEDFORD PRESS
20 & 21, BEDFORDBURY, W.C
1884

1884
(4to)



"THE FORTH BRIDGE."

THE FORTH BRIDGE.

—::—

Two years ago I read a paper on the proposed Forth Bridge at the Southampton meeting of the British Association. Until the other day I had not since glanced at the paper, and a re-perusal was in many respects suggestive; for during the past two years the works have progressed, and some of the theories advanced in the first paper have been put to the test of actual practice. In one respect the re-perusal was a painful one, for the opening sentence contained a reference to Sir William Siemens, and I was reminded of the loss of a friend who took the greatest interest in the Forth Bridge, and whose vast experience and matured judgment could always be drawn upon in times of doubt or difficulty.

Taking up the narrative of the proceedings from the date of my last paper, I may state, in the first place, that five tenders were submitted for the construction and erection of the bridge, the amounts varying from £1,487,000 to £2,301,760, and that the contract was finally let to Messrs. Tancred, Arrol, and Co., on the 21st of December, 1882, for

£1,600,000, which was within £5000 of the estimated cost of the work as prepared by Mr. Fowler and myself for Parliamentary purposes.

The total length of viaduct included in this contract is about $1\frac{1}{2}$ miles, and there are:

2	spans of	1700	ft. each
2	,,	675	,,
15	,,	168	,,
5	,,	25	,,

Including piers there is thus almost exactly one mile of main spans, and half a mile of viaduct approach. The clear headway under the centre of the bridge is 150 ft. above high water, and the highest part of the bridge is 361 ft. above the same datum. Each of the three main piers consists of a group of four cylindrical masonry and concrete piers, 49 ft. in diameter at the top and from 60 ft. to 70 ft. in diameter at the bottom. The deepest pier is about 70 ft. below low water, and the rise of tide is 18 ft. at ordinary springs. In the piers there are about 120,000 cube yards of masonry, and in the superstructure about 45,000 tons of steel.

Operations were commenced in January, 1883, so the works have now been some twenty months in progress, and about 170,000*l.* have been expended in plant and temporary works and 200,000*l.* in the permanent works of the bridge. At South Queensferry an area of about 20 acres of ground has been laid out in shops and yards for the manufacture of the 1700 ft. span steel girders and for other purposes. These shops are in direct communication with the North British Railway, and are connected by an incline and winding engine with a temporary timber viaduct 2200 ft. in length and 50 ft. in width, extending from the South Queensferry shore to the first of the groups of four cylindrical iron caissons which constitute the lower portions of the main piers of the bridge. At Inch Garvie stores and offices have been built, and as this is an exposed island in the middle of the Forth the staging for the pierwork is of iron pinned to the rock. Similarly at North Queensferry, on the Fife side of the Forth, stores, offices, and iron staging have been erected.

The state of the works at the present time is as follows :

PIERS.

South Queensferry Main Piers.—One of the 70 ft. diameter caissons has been sunk to a depth of 16 ft. below low water, a second is in position, and the two others are advanced to the required extent to follow on.

Inch Garvie Main Piers.—One of the piers is practically complete, another is well advanced, and the pneumatic caissons for the other two are being constructed.

Fife Main Piers.—Three of the piers are built and the remaining one is in progress.

Cantilever End Piers.—One of these piers is carried to a height of 66 ft. above high water, and the other to a height of 6 ft. above Low water

Viaduct Piers.—Eleven out of the total number of thirteen piers are built up to the height at which it is proposed to erect the girders in the first instance before final raising into position by hydraulic jacks.

SUPERSTRUCTURE.

About tons of steel have been delivered for the 1700 ft. spans. The first portion to be erected will be that over the Fife Main Piers, and the bed-plates, skew-backs, and 12 ft. diameter tubes for this work are well advanced. A further quantity of tons of steel girder work for the approach viaduct is now in course of erection.

PLANT.

The plant includes 14 steam barges, launches, and other vessels; 22 steam, 12 hydraulic, and 38 hand-power cranes; 28 single and double engines for shop machines, hydraulic work, air-compressing, electric lighting, pumping, and other purposes; also gas furnaces for heating the steel plates, a 2000 ton hydraulic press for bending them, and planing machines, multiple drills, hydraulic rivetters, and other specially designed tools too numerous to mention. Having reference to the novelty and magnitude of the work and the amount of preliminary preparations required, it may be considered that fair progress has been made during the past twenty months.

No special difficulties were encountered in founding the viaduct piers, notwithstanding their exposed position. Except in two cases the piers rest on the rock, and they were executed in half-tide or whole tide cofferdams, which call for no special remark. The cofferdam for the south cantilever end pier was necessarily a very substantial structure, being a quarter of a mile from the shore. It measured 126 ft. by 75 ft. over all, and had a double row of whole timber piles, with 4 ft. of puddle between, and internal struts, chain cable ties, and external raking struts and piles of great strength and solidity. A highly satisfactory bottom on boulder clay of rock-like hardness was found at a depth of 35 ft. below high water. The masonry of the viaduct piers and cantilever end piers consists of an Aberdeen granite facing, averaging a little over 2 ft. in thickness of rock-faced work, backed up with cement concrete or with rubble masonry set in cement, and bonded, about every 12 ft. in height, with courses of large stones carried across the entire area of the piers.

The main piers have on the whole, perhaps, given more trouble than was anticipated. On the Fife shore the whinstone rock bottom falls with a rapid slope of about 1½ to 1 to deep water, and it was necessary to step this slope for the masonry. Diamond drills worked from an iron stage were employed for the subaqueous blasting; but the removal of the rock proved a most tedious affair, and a substantial timber and clay cofferdam had, after all, to be constructed for one of the piers. With some trouble, the other pier was built within a makeshift half-tide dam, made up partly of the 60ft. diameter permanent iron caisson below low water, with a temporary iron caisson attached to it, the whole made tight to the rock as far as might be with concrete and clay filled in between the caisson and a few buckle plates. At Inch Garvie similar delay and trouble were experienced in carrying out the shallow piers. Some of the work could only proceed at low water of spring tides, and it generally happened to blow hard just at that long waited-for moment. Tidal work, and even half-tide work, are proverbially slow and worrying ; but we were all determined that, as the rock varied in quality, no foundation should be put in until the bottom had been laid dry. By perseverance and patience this has hitherto been accomplished, and we have the satisfaction of knowing that both the rock foundation and masonry are unexceptionable in strength and solidity. In our shallow rock foundations at the Forth we had much the same problem to deal with as Stephenson encountered, thirty years ago, when building the fine bridge across the St. Lawrence at this city, and our contractors dealt with it in much the same way. I am not concerned to defend the operations, as such details are usually left to those responsible, namely the contractors. Where speed is required, I am satisfied that in most cases pneumatic appliances offer incomparable advantages over cofferdam work on a rock bottom. French contractors generally resort to pneumatic caissons of ordinary type in depths exceeding 15 ft., but have employed, with great advantage, modifications known as the *caisson-batardeau*, the *bateau-plongeur*, &c., in depths as little as 6 ft. The six weeks required to build a pier with the aid of pneumatic appliances may often be taken up in stopping the leaks of a cofferdam on rock bottom. English contractors are not much accustomed to pneumatic appliances, other than an ordinary diving dress, and rarely resort to them. A diving-bell with shaft of access and air lock was provided and mounted on traveller complete at the Forth, and compressed air drills were fitted in the working chamber, but no use has hitherto been made of the apparatus.

The lower part of the South Queensferry main pier consists, as already stated, of a group of four pneumatic caissons 70 ft. in diameter. In the contract the option was allowed of sinking open-topped caissons by dredging inside, but, after experiencing the extreme hardness of the boulder clay, we were all agreed that it would be preferable to resort to the pneumatic process. Owing to the slope of the clay the four caissons will be sunk to varying depths, ranging from 68 ft. to 88 ft. below high water. The caissons, which were built on shore, launched and floated into position, are 70 ft. in diameter at the cutting edge, and taper 1 in 46 to facilitate sinking. At 1 ft. above low water, which is the top of the permanent caisson and commencement of the granite-faced masonry, the diameter is 60 ft. A working chamber 7 ft. high is provided at the bottom of the caisson, the roof of which is supported by four strong lattice girders 18 ft. deep, and cross girders 3 ft. deep spaced 4 ft. apart. An internal skin 7 ft. distant from the external skin, and vertical diaphragms, form pockets which can be filled with concrete at any point where, owing to the slope of the ground and the varying hardness of the silt and clay, a heavier pressure is desired to force down the caisson. Three shafts, 3 ft. 6 in. in diameter, with air locks at the top, pipes for admitting water and ejecting silt, and other of the usual appliances, are provided. The air locks for passing out the clay and boulders as designed by Mr. Arrol and myself have, instead of the usual hinged doors, two sliding doors like horizontal sluice valves, across the 3 ft. 6 in. shafts, which are worked by little hydraulic rams, or by hand, and are interlocked like railway points and signals, so that one slide cannot be opened until the other is closed. Mounted on the side of the air lock is a steam engine which, by means of a shaft passing through a stuffing-box in the side of the air lock and a drum inside, winds up the excavated material in skips containing one cubic yard. The operation of hoisting, opening slides, and discharging, is rapidly performed, so the two locks have a large working capacity. A third air lock, with side doors, ladder, and hoist, is also provided for the men.

The air-compressing plant consists of three engines with $16\frac{1}{2}$ in. diameter by 24 in. stroke steam and air cylinders, ample power being furnished by boilers of the locomotive type erected on the staging.

Reference has already been made to the two shallow piers at Inch Garvie, but there are also two deep piers which, being on a very irregular and sloping rock bottom, have required much consideration. It was finally decided to level a bed roughly with bags of sand, and to float out pneumatic caissons, and excavate the rock until a level bed was cut. Probably Mr. Fowler and I would not have adopted this precise plan if we had been contracting, although we might have resorted to the pneumatic process, but as M. Coisseau, a contractor of great experience in such work, offered to sub-contract for the sinking of the caissons at fair rates, we did not object. These caissons are 70 ft. in diameter at the bottom, and the rock slopes from 14 ft. to 19 ft. in that length, the lowest point being 75 ft. below high water.

All of the pneumatic caissons will be filled with concrete up to low-water mark, the mixture being 27 cube feet of broken whinstone, 7 cube feet of sand, and $5\frac{1}{2}$ cube feet of cement, which together make a full yard of concrete, having a crushing resistance of about 50 tons per square foot.

Above low water the cylindrical piers, which are 49 ft. in diameter at the top, 55 ft. at the bottom, and 36 ft. high, consists of the strongest masonry, the hearting being flat-bedded Arbroath stone with both horizontal and vertical bond, and the facing Aberdeen granite, the whole set in two to one cement mortar, and built in the dry within temporary wrought-iron caissons. In the shallow piers where the rock is stepped the masonry is carried down to the rock itself, and wrought-iron hoops 36 in. by $1\frac{1}{2}$ in. bind the bases of the piers. At the top of all the piers 18 in. by $1\frac{1}{2}$ in. hoops, and midway down 18 in. by $\frac{3}{4}$ in. hoops, are also built in, and it is believed that these cylindrical masses of masonry are as completely monolithic as can be attained or desired. In each cylindrical pier there are forty-eight steel bolts $2\frac{1}{2}$ in. in diameter and 24 ft. long to hold down the bed-plates and superstructure of the main spans.

A few words now as to the manufacture of the superstructure. About 42 miles of plates have to be bent for the tubular compression members, and the best method of doing this became a question of great practical importance. Bending cold did not answer, as the true curvatures could not be so attained. Theoretically, a 10,000 ton hydraulic press would be required to bend, truly, our 16 ft. by $1\frac{1}{4}$ in. thick steel plates, and practically a 2000 ton press was of no use. Heated in a gas furnace, the plates bent readily, but distorted considerably and irregularly in cooling. Covering with ashes, packing up, and a variety of expedients were tried before the proper method was hit upon, which was to bend the plates hot and to give them a straightening squeeze afterwards when cold. Uniform heating is secured by admitting the gas near the door and mid-way along the furnace, and an important incidental advantage of the use of tubular compression members thus is that every plate gets relieved from any internal strains which may have been set up by shearing or improper usage at the steel works, which is of the greater moment as steel having the comparatively high tensile strength of 34 to 37 tons per square inch is used for the compression members.

Some alarm was occasioned at the works by certain $1\frac{1}{8}$ in. thick plates breaking like cast iron on being bent cold to the flat radius of 6 ft. I felt certain, however, that the Landore steel was not at fault, as our inspectors test a shearing from every plate by bending it round a radius of $1\frac{1}{2}$ in. after being made red hot and cooled in water. On investigation I traced the cause of the fracture in the local damage the plates received from shearing. What the damage consists in is an unsolved riddle. It cannot extend more than $\frac{1}{16}$ in. from the edge, because planing to that extent relieves the plate and yet it affects the entire width, for the 4 ft. 6 in. plate snapped as readily as the 1 in. wide strip sheared from it. Neither can it arise from "nicking" by bad shearing, because making the plates red-hot cures the evil, though the "nicking," if previously existent, remains as visible as ever. Practically, the important point of interest to bridge builders is that with planed edges and drilled holes we have had no mysterious fractures, but the Forth Bridge plates have behaved as a material having as the higher limit a tensile strength of 37 tons per square inch and an elongation of 17 per cent. in 8 in. should behave. Our specification for steel in compression is 34 tons to 37 tons with an elongation of 17 per cent., and for steel in tension 30 tons to 33 tons with 20 per cent. elongation. The strength rarely varies as widely as the stated limits, and the elongation averages some 3 per cent. more. One of the plates which fractured from sheared edges when bent cold was tested by me in a variety of ways. A specimen made red-hot and cooled in water at 80 deg. stood 38.3 tons per square inch and elongated 21 per cent. Another specimen made hot and allowed to cool in air stood 36.6 tons and also elongated 21 per cent., whilst one planed from the plate direct without heating failed with 34.3 tons, but extended 25 per cent. For practical purposes, therefore, it mattered little how the plate was treated, provided the effect of the shearing was eliminated by planing or by heating.

When bent, the plates are planed at the edges in the usual way, and at the curved ends by a specially designed radial machine. They are then, with the internal stiffeners, temporarily built into a tube round a mandrel, and drilled through plates, covers, and bars at one operation. Four specially-designed annular drill frames, surrounding the tubes, and furnished each with ten traversing drills, capable of attacking every hole, travel along lines of railway in the building yards, so laid out that four lengths of tube, each of about 400 ft., can, if desired, be dealt with at once. In a 16 ft. length of 12 ft. diameter tube there are about 1600 holes to drill through from $2\frac{1}{4}$ in. to $3\frac{1}{4}$ in. thickness of steel, which operation takes about 52 hours' working of the drills. Continuous working is, of course, not possible, as the machine has to be advanced every 8 ft., which is the shift of the butts in the plating of the large tubes.

Over the piers the arched tubular lower member forms a connection with the upper bed-plates, the vertical and diagonal tubes, and the lateral and vertical cross bracing, so that considerable thought had to be given to the details at this point. A full-sized model was prepared, and different modes of arranging the junctions were set out and modelled. Finally it was decided to gradually change the tubular lower member into a box form with one rounded upper corner, where it meets the skewback or part over the pier, and by internal vertical and horizontal diaphragms, to make the latter a cellular structure of enormous strength and stiffness, offering facilities for attachments in any required direction. Several layers of plates form the bottom of this skewback, and constitute what may be termed the "upper bed-plate" of the bridge. The "lower bed-plate" consists of similar layers of plates rivetted together and bolted to the pier; and the two bed-plates are free to slide on each other within certain limits to be referred to more particularly hereafter. The layers of plates run longitudinally and transversely, to meet the different stresses; and, after the edges are planed, the plates are fitted together, clamped between girders, and drilled by special machines through their whole thickness. About 1000 lineal feet of $1\frac{1}{8}$ in. holes have to be drilled in each bed-plate, which in practice with the 8-drill machine, takes about eighteen days, including stoppages. In the upper bed-plates holes about 11 in. square, with corners rounded to a 3 in. radius, are required, in some instances, to clear the nuts of the holding-down bolts, and these are cut readily by a simple tool devised by Mr. Arrol. In other cases, 12 ft. diameter recesses, 2 in. deep, have to be bored for what may be termed a huge key or dowel, which will connect the upper and lower bed-plates, but allow a slight rotation; and this also requires a special tool.

The tension members and cross bracing generally consist of box lattice girders which are drilled by travelling machines of similar type to those already referred to in connection with the tubular members. All of the rivets are of steel, having a tensile strength of about 27 tons, an elongation of about 30 per cent., and a shearing resistance of from 22 tons to 24 tons per square inch. It is hardly necessary to state that hydraulic rivetting will be used throughout. The nuts and washers of the holding-down bolts and some other parts are of cast steel, having a tensile strength of 30 tons per square inch, and an elongation of 8 to 10 per cent. It may be interesting to mention that the contractors have used steel in preference to iron in some parts of the temporary works, and that at their request the 168 ft. span viaduct approach girders were changed from iron to steel with a view to save expense.

The two years' additional consideration given to this bridge since the date of my first paper, has led to no modifications of importance in the design, or in the weight of steel required for the construction, a satisfactory result which is largely due to the care and ability of my colleague Mr. Allan Stewart, who has had charge of the detailed calculations and designs from the inception of the undertaking. Originally the cantilevers had a varying batter towards each other from 1 in $7\frac{1}{2}$ at the piers to vertical at the ends, where they meet the central girder. We have now made the central girders slope inwards and maintained the batter of 1 in $7\frac{1}{2}$ throughout, thus getting rid of the previous "winding" which somewhat complicated the details of the cantilever, and at the same time preserving and emphasising the pyramidal form of cross-section characteristic of the design. In models of the bridge a feeling of great solidity results from this feature, as will be the case no doubt in the bridge itself, of which a geometrical elevation necessarily gives but a poor idea.

We have also modified the attachment of the superstructure to the piers. Formerly the intention was to put an initial stress upon the 12 ft tubes between the double piers as described in my first paper, and to bolt the superstructure rigidly to the masonry. Now we secure the superstructure to one only of the four cylindrical piers in each group by the great circular key already referred to, and permit a certain amount of sliding on the others. Owing to the enormous size of the structure elastic deformations which may be neglected in ordinary cases have to be provided for. A very great deal of consideration has been given to this important point, and the calculations have necessarily been complex and tedious, but we think we have now made the best disposition attainable to resist all possible and improbable hurricanes striking the bridge locally or throughout the whole span, and all variations of temperature likely to be met with at the Forth.

The question of clothing the tubes between the piers with some non-conducting material will be left for future settlement after the movements under changes of temperature have been registered by the tube itself. Fortunately we are not troubled with Canadian variations of temperature and the correspondingly great changes of form in metallic structures. At the new Clyde Viaduct in a length of 376 ft., the observed annual range is 2 in., or a fraction over half an inch in the 100 ft., and this is an open lattice construction, whilst the Forth Bridge horizontal members between the piers are closed tubes. Obviously during the early stages of erection, before much weight comes on the bed-plates, the tube will be practically free to expand and contract. Ultimately, when the whole weight of the completed structure rests on the piers, the friction between the two surfaces of the upper and lower bed-plates will probably be sufficient to prevent movement except under extremes of temperature and heavy wind pressure of rare occurrence. The attachment of the superstructure to the piers partakes thus of the character of a safety friction clutch. Movement will not occur under ordinary circumstances, and if an excessive shock from some unforeseen cause arise on the superstructure, it can only be transmitted to the masonry of the pier through the sliding surface of the upper and lower bed-plates. Should a wave of deflection from the impact of a tornado pass along the great cantilever, as some critics suggest, then it would be arrested by skidding as an express train is arrested, and not by running into a buffer stop.

Provision is made for lubricating the surfaces, and as the result of experiments made by myself during the past two years, probably some crude petroleum will be applied to the bed-plates every time paint is applied to the rest of the bridge. Calculations have been made of the extent of sliding and of the stresses on the piers under the twisting action of a hurricane blowing on one cantilever, whilst the balancing cantilever is in a dead calm, and various coefficients of friction have been assumed. During erection sliding can, if desired, be made practically free by carrying one cantilever further out than the balancing one, and so relieving two out of the four bed-plates of weight. In the completed bridge the position of the bed-plates could be adjusted by temporarily loading the end of a cantilever.

Experiments on friction vary considerably, but when such large surfaces as 2200 square feet, which is the joint area of the four bed-plates of each main pier, are concerned, there would no doubt be an equalising effect which would make the proper coefficient of friction for the bed-plates approximate to the mean of the results obtained with a number of experiments on small areas. The coefficients obtained by Morin for iron on iron greased ranged from .09 to .115, and with the grease wiped off, .16 to .19, the pressure being about 27 tons per square foot, or considerably greater than that on the Forth Bridge bed-plates. On a large scale, the mean values of coefficients for different surfaces are derivable from launching ways of ships, brake experiments, and other data. In launching ways the coefficient must be singularly small, for with declivities of $\frac{1}{2}$ in. to 1 in. in the foot, ships not only start, but acquire a velocity of ten miles an hour or more very quickly. In some of Mr. Denny's experiments it would appear that even with this flat slope the velocity acquired was fully half that which a body would attain falling freely, so the retarding friction during motion must be very small. Again, the coefficient of friction at starting could not exceed .06, but of course in ordinary launching ways the pressure per square foot and the character of the surfaces are different to those in the Forth Bridge, though the total weight of the moving mass may be the same, and the facts are worth mentioning on that account. Brunel's broadside launch of the Great Eastern in 1857 affords, however, valuable data directly applicable to our sliding bed-plates, for the weight of the ship was 12,000 tons, and the launching ways were iron on iron somewhat rough on the surface, and imperfectly lubricated or not lubricated at all. As a result of experiments with a small section of the launching ways, the inclination was made 1 in 12, as it was thought a small force would then start the ship, and a similar force restrain it from acquiring undue velocity, the observed coefficients of friction ranging from .125 at starting to .067 at moderate velocities. On commencing the launch an estimated force of about 500 tons was required to assist gravity on the 1 in 12 incline, hence the starting coefficient with the 12,000 ton load would be about .125 as in the model. Again, when started, the 1 in 12 was more than sufficient, for the vessel ran on some 3 or 4 ft., and, spinning round the handles of the winches, injured five men. Subsequently, however, owing to want of rigidity in the ways, rusting of the rails, or some other disturbing cause, considerable trouble was experienced, and successive additions had to be made to the hydraulic presses during the three months occupied in the launch. For the last 30 or 40 ft. Brunel estimated the power required, inclusive of gravity, at one quarter of the weight, or double that which started the ship at the top of the launching ways.

Railway trains are not as heavy as ships, but afford valuable data as to the coefficient of friction of steel on steel under severe pressures, such as must obtain at the point of contact of the tyre with the rail. Captain Galton's experiments show that the coefficient varies widely with the speed and other elements, being sometimes as little as .05. With dry rails the adhesion of the driving wheels indicates a coefficient of about .20 to .25, and with wet rails, .15 to .20. Probably with "greasy" rails it would not exceed the .10 arrived at by Morin fifty years ago as an average.

Calculations of the stresses on the piers have been made upon the hypothesis that coefficients of .10 and .25 obtain on different bed-plates at the same moment in the manner most unfavourable to the structure. A variety of other assumptions and test calculations have been made. As a final result we are of opinion that the maximum stress on the masonry of the main piers will be something between 9 tons and 12 tons per square foot. To attempt a closer approximation would only serve to advertise our incapacity to appreciate the complex character of the problem and the uncertainty of some of the data. So far as compression is concerned, our concrete, which has a crushing resistance of 50 tons per square foot, would thus give a factor of safety of at least four. The solid Arbroath stone piers are, of course, of far greater strength both as regards compression and the shearing and possibly tensile stresses to which the piers may be subject under the extreme hypotheses made as to force and distribution of wind.

Very valuable data as to the ability of a massive rubble pier in cement to resist a heavy lateral force were afforded by the experimental arch of 124 ft. span and 7 ft. rise built in Paris some fifteen years ago. The thrust of the arch was about 1400 tons, and, treating the abutment as an elastic solid, the stress upon the masonry would range from 14.7 tons compression to 8.7 tons tension per square foot. To ascertain the ability of cement concrete to resist heavy shearing and tensile forces I tested a number of concrete beams having different proportions of cement. Such concrete as that used at the Forth developed a tensile strength under transverse stress of about 10 to 12 tons per square foot, so that it was from no inherent weakness in the concrete that masonry was substituted for it in the 36 ft. upper length of the main piers. Our reason for its adoption was that we believed by using natural flat-bedded Arbroath stone set in two to one cement mortar, with both horizontal and vertical bond, we made certain of obtaining practically a monolith, whilst with concrete, however careful the inspection, there might be cleavage planes of perhaps dangerous extent in places. The special stresses on the piers arising from the cantilever system of construction have received, as I have already said, our most close consideration, and we doubt not that the desired factor of safety of four will be obtained as regards all shearing, tensile, and compressive stresses to which the masonry may be conceived to be liable under any reasonable hypothesis which can be framed.

Happily, we are relieved from all anxiety as to the foundations, since the piers rest either on rock or on a boulder clay, which for all practical purposes is as hard as rock. It may be mentioned, however, that the heaviest load at the base of any of the 70 ft. diameter caissons, including the tilting action of a 56 lb. per square foot wind, is about 24,000 tons, or at the average rate of a little over six tons per square foot, deducting nothing for the water displaced by the pier.

As it is impossible to produce the working drawings of the bridge on the present occasion, a short tabular statement of some of the leading features may be desirable.

Steel in main cantilevers	... ...	40,000 tons.
„ centre girders ...	... ...	1,600 „
„ viaduct approach	... ...	2,800 „

Cantilevers—680 ft. projection; 343 ft. and 40 ft. deep.

Bottom Member of Cantilever, a pair of tapering tubes:

Distance apart centres at piers	...	120	feet.
„ „ „ ends	...	31.5	„

Tube at piers, 12 ft. diameter, $1\frac{1}{4}$ in. thick, 830 sq. in. area.
„ ends, 5 ft. „ $\frac{3}{8}$ in. thick, 120 sq. in. „

Top Member of Cantilever, a pair of tapering box lattice girders:

Distance apart centres at piers	...	33	feet.
„ „ „ ends	...	22.25	„
Girder at piers, 12 ft. × 10 ft.	...	506	sq. in. net area.
„ end, 5 ft. × 3 ft.	...	60	„

Columns over Piers, 12 ft. diameter, 368 to 468 sq. in. area.

Diagonal Struts (tubes), 8 ft. to 3 ft. diameter, 198 to 73 sq. in. area, and 337 ft. to 74 ft. long.

Diagonal Ties (box lattice), 8 ft. to 3 ft. deep, 163 to 67 sq. in. net area, and 327 ft. to 82 ft. long.

Horizontal Wind Bracing (box lattice), 11 ft. to 3.5 ft. deep, 88 to 20 sq. in. area, 142 ft. to 40 ft. long.

Vertical Wind Bracing (box lattice), 4.75 ft. to 2.5 ft. deep, 84 to 28 sq. in. area, 160 ft. to 60 ft. long.

Central Girder, 350 ft. span, 51 ft. and 41 ft. deep; bottom members, 32 feet apart centres, 142 sq. in. net area; top members, 22.25 ft. and 19 ft. centres, 139 sq. in. gross area.

Internal Viaduct, lattice girders with spans of 39 ft. to 145 ft

Floor, buckle plates and trough girders.

Wind Fence, close lattice work 4 ft. 6 in. high.

Viaduct Approach, lattice girders, under rails continuous over two 168 ft. openings, 22.5 ft. deep, 16 ft. apart; floor and wind screen as for internal viaduct. Masonry piers 25 ft. × 8 ft. at top, and 50 ft. × 20 ft. at base.

Rolling Load: (1) trains of unlimited length on each line of rails, weighing one ton per foot run; (2) trains on each line made up of two engines and tenders, weighing in all 142 tons, at the head of a train of 60 short coal trucks of 15 tons each.

Wind. A pressure of 56 lb. per square foot, striking the whole or any part of the bridge at any angle with the horizon, and acting square or obliquely upon an area equivalent to twice the plane surface of the front girders, with a deduction of 50 per cent. in the case of tubes. The total wind pressure so derived amounts to 2000 tons on the 1700 ft. span, and 7900 tons on the whole superstructure included between the two cantilever end piers. The following table shows the magnitude and intensity of the heaviest resultant stresses in tons on some of the principal members from dead load, live load, and wind, distributed as already described :

——	Dead.		Live.		Wind.		Total.	
	Gross Stress.	Stress per sq. in.	Gross Stress.	Stress per sq. in.	Gross Stress.	Stress per sq. in.	Gross Stress.	Stress per sq. in.
Bottom member ...	2282	2.8	1022	1.2	2920	3.5	6224	7.5
Top „	2253	4.4	997	2.0	544	1.1	3794	7.5
Vertical „	1550	3.3	705	1.5	1024	2.2	3279	7.0
Diagonal struts	802	4.1	167	0.8	414	2.1	1383	7.0
„ ties	754	4.6	186	1.2	194	1.2	1134	7.0
Hor. wind bracing...	80	0.9	5	.1	265	3.0	350	4.0
Ver. „ „ ...	42	0.5	169	2.0	108	1.3	319	3.8
Central girder top...	337	2.4	303	2.2	182	1.4	822	6.0
„ „ bottom	330	2.3	301	2.1	247	1.8	878	6.2

Owing to the batter of 1 in $7\frac{1}{2}$ of the main girders and the adoption of double piers, not merely at Inch Garvie, but for the main piers on either side of it, the calculation of stresses involves some interesting and complicated problems. It is fortunate that from the nature of the ground no unequal settlement can occur in the foundations or some of the stresses would be indeterminate by reason of the double piers which, I may add, were not a feature of the original design.

At the centre cantilevers any unequal loading is supported by the double bracing between the piers at Inch Garvie, but at the north and south cantilevers, the support is twofold, namely, the resistance of the bracing, and the pull on the holding-down bolts at the cantilever end pier. The first problem that arose, therefore, was to ascertain how much of the load would be supported by each of these reactions.

But the weight of each part was not known, and in a structure of this magnitude, it required to be ascertained with accuracy. This could only be done by the method of trial and error, and as the stresses produced by the weight of the structure itself are very considerable, and as the secondary bracing is an important item, detailed drawings had to be made and carefully measured several times before the stresses could be determined. The inward slope of the cantilever gave rise to forces of sensible amount from the component of the vertical forces square to the plane of the cantilevers. A much more difficult problem, however, was presented by the stresses produced by wind. Thus, for example, a force acting horizontally on the cantilever near the 350 ft. girder, and at right angles to it, produces at the main pier a downward pressure on the leeward side, and an upward action on the windward side, together with a twisting action, and vertical and lateral pressures on the cantilever end pier. All these forces had to be traced through the different members and bracings, and the same had to be done for a wind force acting as a moving load at every point of the cantilever. It was also essential to consider the wind as blowing not only at right angles to the line of the bridge, but also at such other angle as would impose on each member the greatest amount of stress. By a reference to the diagram it will be seen that the stresses produced by wind are very severe, and, therefore, exactness was required in order that no part might be unnecessarily heavy, nor on the other hand be strained beyond the allowed amount. After all the stresses as a framework structure arising from the dead load, a travelling train, and from wind had been ascertained, a new set of problems arose, chiefly from the magnitude of the work, and the weights of the different members themselves which cause local stresses of considerable importance in some instances.

An all-important point was, of course, the stress per square inch admissible upon the several tension and compression members. The only limit practically imposed upon us by the Board of Trade was that the stress should not exceed one-fourth of the ultimate strength of the steel without reference to the question of the relative proportions of live and dead load, or the character of the stress. In settling the sectional areas we did not bind ourselves to any formula derived from Wöhler's or other experiments on the fatigue of metals, but considered each member separately and had reference to the whole of the circumstances, including the character of the rivetting and other details of construction. As many competent engineers are of opinion that the rational way of proportioning structures is to assume varying ultimate resistances of the metal for different proportions of dead and live load, and adopt an uniform factor of safety of 3, we tested the actual areas of the Forth Bridge members by the following rules, and found in all cases an excess on the requirements :

a. For a constant load assume the ultimate tensile strength to be 30 tons per square inch.

b. For a load varying from *nil* to a maximum assume the strength to be 20 tons per square inch if the alternation of stress is frequent, and 22.5 tons if it is seldom, as in the case of a hurricane.

c. For alternate tension and compression assume the ultimate strength to be 10 tons if frequent and 15 tons if seldom.

The above apply to tension members and are to be divided by 3 for the working stress. For struts the working stress equivalent to the above, from the results of my own experiments and from other considerations, I take to be 40 per cent. of the stress causing first flexure, as given by the following empirical formulæ :

$$f = (.44 - .002\, r)\,(t + 18) \text{ for tubes}$$
$$f = (.40 - .004\, r)\,(t + 18) \text{ for lattice}$$

where r = ratio of length to diameter and t = tons per square inch, as set forth in paragraphs *a*, *b*, and *c*, but increased in all cases in the ratio of 34 to 30, which are the specified minimum strengths of the steel used for compression and tension members respectively.

I have no doubt that any structure proportioned by the above rules would have an ample margin of safety, but as already stated the stresses in the case of the Forth Bridge are lower than indicated. The diagram of stresses shows that the lower tubular member is the most affected by wind, and, in fact, under the conditions assumed the leeward tube does the work and the windward is almost relieved of stress. Looking at the huge 12-ft. tubes as they now lie at the Forth Bridge works, with their ten longitudinal **T** bars 12 in. × 7 in. × $\frac{7}{8}$ in., having double angles rivetted to the web of the **T**, and with annular stiffeners every 8 ft., certainly nothing could appear better adapted to resist stress and fatigue, and I should not feel the least anxiety if they were subject to double the stress which will ever be imposed upon them. I may add that the preceding formula for struts is based upon my experiments with steel ranging from 26 to 56 tons in tensile strength, and fairly represents the average results, though in this instance, as in all others where columns are concerned, individual experiments differ rather widely, owing to initial stress, unequal bearing, or other cause. In proportioning the rivetted joints of the tubes and other members, the shearing area is generally made one and a half times the net sectional area of the plates connected if in tension, and half that for planed and butted joints in compression only.

In my first paper I said that the maximum wind pressure upon the 1700 ft. span had been assumed to be equivalent to a pressure of 56 lb. per square foot upon the double surface, and I regretted that such assumption necessarily involved many matters of pure conjecture, which rendered it impossible to state with precision what factor of safety would belong to the Forth Bridge. The same remark of course applies even now with equal force to every other bridge, because there exists a lamentable lack of data respecting the actual pressure of the wind on large structures. Mr. Fowler and I have spared no pains during the past two years to contribute something to the general fund of information; and other engineers, doubtless, are experimenting,—for experiments, and not speculations, are wanted. We have had now for two years, on the island in the middle of the Forth near the centre pier of the bridge, three wind gauges or pressure boards; the large one, 300 square ft. in area, is fixed square to the east and west winds, and of the two small ones of $1\frac{1}{2}$ square ft. area, one is fixed as above, and the other is free to swivel square to the wind in any direction. When speaking at the

Institution of Civil Engineers on the subject of wind pressure, previous to the erection of these gauges, I ventured to prophesy that, contrary to the opinion of many, the large board would show a smaller average pressure per square foot than the small ones. I have summarised the readings of the gauges for the past two years, and find them to fairly bear out my anticipations. In preparing the following table, the mean of all the readings of the revolving gauge between 0 and 5 lb., 5 lb. and 10 lb., &c., have been taken, and the mean of the corresponding readings at the same time of the small fixed gauge and of the large fixed gauge for easterly and westerly winds have been set forth opposite.

Revolving Gauge.		Small Fixed Gauge.		Large Fixed Gauge.	
Mean Pressure.		Easterly.	Westerly.	Easterly.	Westerly.
lb.	lb.	lb.	lb.	lb.	lb.
0 to 5	3.09	3.47	2.92	2.04	1.9
5 to 10	7.58	4.8	7.7	3.54	4.75
10 to 15	12.4	6.27	13.2	4.55	8.26
15 to 20	17.06	7.4	17.9	5.5	12.66
20 to 25	21.0	12.25	22.75	8.6	19
25 to 30	27.0		28.5		18.25
30 to 35	32.5		38.5		21.5
Above (One observation only above 32.5.)	65		41.0		35.25

I do not myself, nor does Mr. Fowler, place implicit faith in the registrations of our own or anybody else's anemometers, although we test the working of the gauges in the most careful manner, but at the same time I think it is pretty well established by our two years' experiments, that the effective pressure per square foot on a large and comparatively heavy board averages only about two-thirds of that indicated by an ordinary light anemometer. It will also be noticed that the heaviest gales have been from the west, and that the revolving gauge then indicated much the same as the fixed gauge. Some critics were of opinion that our 300 ft. gauge would be of little use, as it could not swivel square to the wind, but remembering the experiments made with a fan blast on oblique plates which showed that the resultant pressure was normal to the surface, I felt sure that having reference to the prevailing winds swivelling was of no practical importance at the Forth, and the results justified my anticipations.

The two heaviest gales occurred in the early morning of December 12th, 1883, and January 26th, 1884, respectively. On the latter occasion much damage was done throughout the country, and there was conclusive evidence from the extent as well as the intensity of the storm, that it was a very exceptional one in character. At Inch Garvie the small fixed gauge was reported to us as registering 65 lb. per square foot, but on inspection I found the index pointer could not traverse further or it might, perhaps, have indicated much higher. At Valencia very strong squalls covering short periods were stated to have attained a rate of upwards of 150 miles per hour. At Holyhead lengthened squalls of 120 miles and short squalls of higher rates were reported. At Alnwick we were told that several instances of 10 miles in five minutes, or 120 miles an hour, and squalls of 150 miles occurred. Now if we assume, as is common, the pressure of wind to be equal to .005 V^2, and accept the velocity of 150 miles as correct, we shall have to believe that pressures of 112 lb. per square foot were reached at Valencia on the west coast of Ireland, and at Alnwick on the east coast of England, on the 26th of January last. I confess I find it much easier to believe that the records of anemometers as at present obtained are utterly misleading and valueless for all practical purposes. I entirely mistrusted our own 65 lb. record, even before I knew that the index was at the end of its travel. On finding out the latter fact, however, I experimented with the gauge, and finally in the presence of the inspecting officers of the Board of Trade, made it register 65 lb. by the sudden application of a pressure not exceeding 20 lb. The momentum of the light index needle, and not that of the pressure plate which was bridled back, sufficed to cause the error.

I look upon the record of 65 lb., therefore, as valueless so far as regards the specific maximum pressure attained during the great storm, but of considerable value as evidence that the highest pressure, whatever it might have been, partook of the character of a smart jerk of too instantaneous duration to affect a structure of any size or weight. From the records generally and from my own watching of the movements of the three gauges, I have come to the conclusion that uniform velocity and pressure in a wind, whether it may prevail or not at cloud heights, can never obtain near the surface of the earth or in the neighbourhood of any bridge or other structure capable of causing eddies. Unsteady motion must be the rule in air as in water, and the threads of the currents moving at the highest velocity will strike an obstruction successively rather than simultaneously, so that the mean pressure per square foot on a large area must be less than that on a small surface from that cause alone, irrespective of possible differences in the partial vacuum at the back of the planes.

In the spring of this year, when running into Dublin Harbour during a heavy broadside gale, I took occasion, when in still water but in the full blast of the wind, to measure the heel of the vessel and from her elements to calculate subsequently the mean pressure required. My pressure board in this case was about 6000 square feet in area, and the deduced mean pressure was 12 lb. per square foot. From other data I estimated the corresponding anemometer pressure at fully double the preceding amount, and this was perfectly rational because the vessel kept steady at the constant heel, whilst heavy local gusts of very small area struck different parts of her in a distinctly recognisable manner.

In short, the large area and heavy mass of the hull equalised the jerky action of the numerous small blasts of high intensity, and a similar action doubtless takes effect in ordinary railway structures, and will to a still greater extent in such a large and heavy structure as the Forth Bridge.

Mr. Fowler and I are of opinion, therefore, as a result of our two years' further consideration, that the assumed pressure of 56 lb. per square foot over the whole of the bridge is considerably in excess of anything likely to be realised. It is another question whether the method of estimating the effective area exposed by the bridge, namely, double the plane surface with a deduction of 50 per cent. in the case of tubes, is right or wrong. We think it is a sufficiently near approximation to the truth, for reasons which I will briefly set forth.

As all engineers well know, one of the results of the panic caused by the fall of the Tay Bridge was the appointment by the Board of Trade of a Committee to consider the question of wind pressure on railway structures, which Committee advised the adoption of certain rules. Shortly stated these were : (1) That a maximum wind pressure of 56 lb. per square foot should be provided for. (2) That the effective surface upon which the wind takes effect should be assumed at from once to twice the front surface according to the extent of the openings in the lattice girders. (3) That a factor of safety of 4 for the ironwork, and of 2 for the whole bridge overturning as a mass when gravity alone comes in, should be adopted. In the case of the Forth bridge we took, with the approval of the Board of Trade, the highest ratio for the surface, namely, twice; but I must admit that I had not at the time the slightest idea whether the twice ought not to be thrice and even more, and the recommendations of the Committee did not assist me, as they were founded on no special experiments, and did not accord with my own experience so far as it then extended. Under these circumstances the necessity of further experiments was clearly indicated, and we have made them.

The tension members and the bracing of the Forth bridge, as already explained, are lattice box girders and the main compression members are tubes. Thus, in the case of the top tension members near the piers, we have the front surface of the girder with channel bars and projecting flanges, making it essentially different to the flat anemometer plate, and three corresponding surfaces, situated respectively about 7 ft., 33 ft., and 40 ft. to the rear of the front surface. In the case of the tubes we have the tube itself, then a couple of box lattice cross braces, with channel bar members, and finally another tube. No theory exists which could enable us to estimate even approximately the equivalent flat surface of such a network; and I felt until my scheme of experiment by models was realised with satisfactory results, that our calculation of stresses from wind pressure rested on anything but a logical basis. The problem to be solved was how far the eddies caused by the front surface affected the surfaces to the rear. In the recommendation of the "Wind Committee" a front plate girder was considered to give complete shelter to any girders to the rear of it, but I think anyone who has walked "Indian file" in a gale of wind will have noticed that unless he locked up pretty closely to the front man he felt practically the full force of the gale, and similarly unless the rear plate girders of a bridge be relatively close to the front girder the latter will not afford anything like complete shelter. It is obvious, therefore, that the depth of the girders, and the distance apart enter into the problem, as well as the question of their being plate or lattice; and I may add further that the position and character of the floor between the girders also materially affect the wind stresses.

My original idea was to prepare models and test them in actual wind at Inch Garvie, but the irregularity of the results, even with the flat boards, precluded the possibility of any useful data being so obtained. I determined, therefore, to abandon the attempt to measure actual resistances, but to arrive at the same end by getting the equivalent area in flat surface of the different bridge members and cross bracing, and for this purpose devised a very simple pendulum arrangement, consisting in effect of a cross bar with a model at one end and an adjustable flat surface at the other of exactly equal weight, which bar was suspended at the centre, so that the only resistance to turning was the torsion of the suspending string. On oscillating this pendulum, if the flat surface were not the exact equivalent in resistance of the model, one or the other would advance, and the sensitiveness was such that different observers would rarely vary more than 3 or 4 per cent. in their results.

To test the sufficiency of this simple apparatus I contrasted the resistances of thin flat surfaces and cubes, and my results agreed within 2 or 3 per cent. of those obtained in the most elaborate manner by Dubuat many years ago. Similarly, the results obtained with cylindrical surfaces and inclined planes were in strict accord with those obtained by previous observers and other apparatus. When experimenting with sheltered surfaces, however, my results differed considerably from previous experimental ones, which I must say are singularly few in number, having reference to the vast importance of the subject to engineers. Thus, according to Thibault, the resistance of the rear plate of a pair set at a distance apart equal to the diameter is .7 of that of the front plate, whilst in my experiments I found no such excess until the distance apart was $3\frac{1}{2}$ diameters. I experimented with discs placed at from 1 diameter to 4 diameters apart, and the resistance of the two discs in terms of that of the single one was in round numbers 1.0 for 1 diameter; 1.25 for $1\frac{1}{2}$ diameters; 1.4 for 2 diameters; 1.6 for

3 diameters ; and 1.8 for 4 diameters. An increased number of discs placed intermediately between the front and rear discs little affected the resistance. For example, by reducing the 4 diameters to 3.6 diameters, an extra disc could be introduced without increasing the resistance of 1.8, and by still further reducing the distance to 3.5 diameters 4 discs could be employed. This result is of great importance in its bearing on railway bridges where a succession of lattice bars may occur one behind the other, which would offer a very large surface to the wind if the proper way of estimating that surface were to take a slightly angular view of the bridge and measure up all that was visible.

It has been already mentioned that in the " Wind Committee's" report no addition is made for sheltered surfaces in the case of plate girders, whilst it might appear from the foregoing experiments that as much as 80 per cent. allowance should be made where the girders are four depths apart. This would, however, be a very fallacious deduction, for it omits all consideration of the floor of the bridge. Reasoning from the observed resistance of cubes it may be inferred that the resistance of a tubular girder, such as the Britannia Bridge, would be only 80 per cent. of that of a single flat girder, and clearly the floor of a girder bridge, if close plated, makes the conditions approximate to that of the tube. As a matter of fact I found that two plates connected by a floor plate at the bottom offered no more than 90 per cent. of the resistance of the single plate. Summarising my conclusions, for it is impossible to give details here, I should say that the effective surface of a plate girder bridge would range from 90 per cent. to 180 per cent. of that of the front surface according to the distance apart of the girders, the degree of openness of the floor, and its position relative to the main girders.

In many respects the preceding remarks apply to lattice girders, but the varying extent of the open spaces between the bars introduces an additional complication. When the openings were one-fourth of the whole area, I found for a distance of one diameter apart, an increased resistance of 8 per cent. from the second disc, whilst with openings of double the size, the increase was 30 per cent. At two diameters the respective amounts were 40 per cent. and 66 per cent., whilst at four diameters the more open lattice reached 94 per cent. In other experiments, sometimes with a small flat plate in front of a lattice, and sometimes in the rear, I obtained at four diameters distance resistance exactly equal to the sum of the two specimens tested separately.

The top member of the Forth Bridge consists as I have said of a pair of box lattice girders, or as may be said equally truly, of four single-web lattice girders. Models of these single-web girders, tested in pairs, gave 20 per cent. increase from the rear girder when the distance apart was equal to the depth; 50 per cent. for two depths; 70 per cent. for three depths, and 80 per cent. for four depths. When three girders were placed one behind the other, the middle girder gave rise to a further increase of about 4 per per cent. for three depths and for four depths; in short it mattered practically little whether two, three, or four girders were used. Two models of a complete bay of the top member were made, one as light as possible and the other somewhat heavy. The results were in accord, the resistance averaging 1.75 times that of the plane surface, whilst that of each of the lattice box girders tested separately was 1.15. As a factor of 2 instead of 1.75 was used in the wind calculations, the pressure on the lattice members has been somewhat over-estimated, but on the other hand that on some of the other members of the bridge, judging from the results of the experiments, has been somewhat underrated.

The bottom member and the main struts of the bridge consist of a pair of tubes braced together by box lattice girders. I tested a complete bay of the bottom member, and found the resistance of the two tubes, placed seven diameters apart, together with the two box lattice braces, of a depth equal to the diameter of the tubes, to be 1.1 times that of the plane surface. Substituting plate girders for the lattice braces the ratio was still only 1.24, so the tube evidently acted as a sort of a cut water, and by clearing a path for the flat surfaces lessened their resistance. This was further proved by removing one of the tubes and testing the single tube and cross lattice bracing. Tube in front the resistance was but 80 per cent. of that obtained when the lattice was to the fore. The lattice bracing tested alone had a resistance equal to 60 per cent., whilst when in position between the two tubes, it only increased the resistance about 5 per cent. This perhaps was to me the most re-assuring of all the experiments because, looking at a complete model of the bridge, it appeared as if the intricate mass of cross bracing must offer an enormous resistance to the wind. As a matter of fact, so far as my experiments extend, it would seem that the eddies caused by the front surface extend to a great distance in all directions, and in a complex structure the innumerable and conflicting eddies would almost appear to neutralise each other as regards some of the sheltered surfaces. On the other hand, in simple isolated structures, such as a pair of bars or tubes, the shelter is practically *nil* at distances equal to about six diameters, and the members might as well be abreast. This was well demonstrated in the experiments by arranging the models on the skew so as to imitate the effect of a wind blowing at an angle to the horizon, when constant results were obtained with widely different angles.

In the approach viaduct at the Forth the lattice girders are under the rails, and there is a wind fence on each side. Testing a model of this class of construction, I found that the resistance of the parapet and of the railway carriages was only two-thirds of the corresponding plane surface, a result due no doubt to the eddies thrown up by the girders. It would appear therefore that current estimates of the wind pressure required to overturn railway carriages on exposed viaducts should be further considered, for although an average carriage might overturn with a uniform pressure of 40 lb. per square foot, a 60-lb. wind may be necessary to produce the equivalent of that pressure. In our model of a pair of lattice girders with floor, wind fence, and railway carriage on the top, the total resistance was but 93 per cent. of that due to the plane surface. As by the present rules engineers would in such a case estimate the equivalent at about 150 per cent., it follows that in many recent and presumably future bridges the actual wind stresses may be considerably less than estimated.

The leading constituent parts of the Forth Bridge were tested, as described, by models of single members and of complete bays, but we proceeded a step further and tested both in air and in water a complete metallic model of two pairs of cantilevers with cross bracing, internal viaduct, and wind fence, together with the intermediate part over the Inch Garvie piers. The total resistance so ascertained was 9 per cent. greater than that obtained by calculation on the basis of taking double the plane surface with a deduction of 50 per cent. in respect of tubes. With the models of different parts tested separately the excess was 4 per cent. This excess would not apply to the moment of the wind pressure, because the highest parts of the bridge are lattice structures, the resistance of which was over-estimated. If a 56-lb. wind ever occurred as a mean over such an area as that we are dealing with, it would be something greater at the high level of the lattice top member and something less at the level of the bottom tubes.

Personally, therefore, I am satisfied that the assumption originally made by ourselves and the Board of Trade officers was a sufficiently close approximation to the truth for all practical purposes. I do not attach undue importance to the results obtained by the models, nor to the records of our large and small pressure boards at the Forth, but at the same time to me they have thrown a little daylight on many obscure questions respecting the actual wind pressure on railway bridges and other structures. Mr. Stewart and I would sometimes attempt to calculate the resistance of a model upon hypotheses of our own, and differ most widely in our results, as others who have attempted the same thing have generally done. A single swing of the long pendulum would solve all our doubts and difficulties. In arranging the experiments, I had regard to Froude's principles as to velocity relative to the scale of the models, and believe the eddies and interferences to be similar in kind in the models and bridge. Of course what is wanted is the measured resistance of actual bridges in actual storms, but this I have not yet been able to undertake.

Such experiments as I have been able to make have at least served to show how little is known about wind stresses, and how necessary it is that every engineer should seize such opportunities as may offer for contributing something to the general store of information.

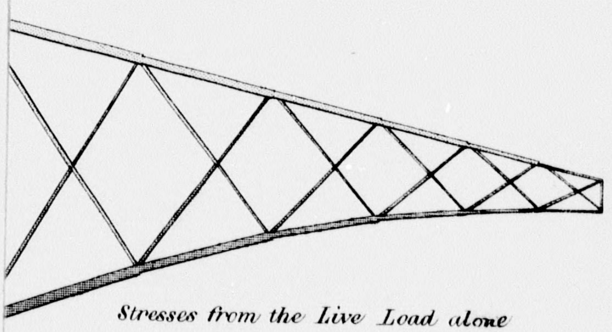
Two years ago I said I should have preferred to have postponed any communication on the subject of the Forth Bridge to the British Association "until the many points of interest and difficulties inseparable from so gigantic an undertaking had manifested themselves." I am in much the same position now, for it will be gathered from the present paper that no real strain has yet been put upon the resources of the contractors, or the capacities of the executive officers. Two years hence I may, perhaps, have a more thrilling tale to tell. Much interest in the work has been evidenced by Continental and American engineers, and the criticism on the whole has not been unfavourable but appreciative. Occasionally it has been suggested that the appearance will not be as elegant as could be desired, but I retort, mentally, in Lord Bacon's words, "Houses are built to live in, and not to look on; therefore let use be preferred before uniformity, except where both may be had." We aim at getting both, and our granite faced piers, with their simple but bold mouldings, certainly look better than cluster-columned metallic piers, however scientific. Thus far we have succeeded in satisfying our masters, and very keen critics, the directors of the North Eastern, the Midland, the Great Northern, and North British Railways, and the officers of the Board of Trade both as regards the quality and appearance of the executed work.

If I were to pretend that the designing and building of the Forth Bridge was not a source of present and future anxiety to all concerned, no engineer of experience would believe me. Necessarily, where no precedent exists, the successful engineer is he who makes the fewest mistakes. We cannot wait for precedents, and therefore as successive points of doubt or difficulty arise, we reason them out on the best data attainable, and then in the land of Burn's we act up to Burn's favourite motto,

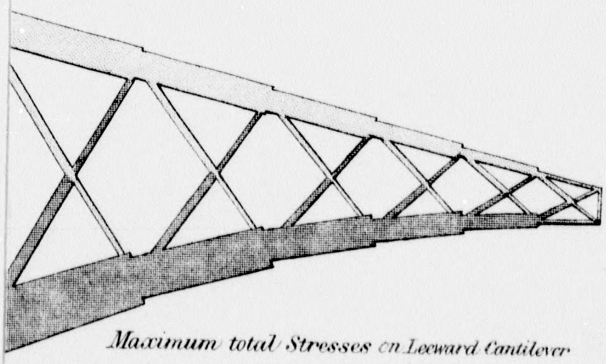
> "On reason build resolve—
> That column of true majesty in man!"

B. BAKER.

Montreal, September, 1884.



Stresses from the Live Load alone



Maximum total Stresses on Leeward Cantilever

FORTH

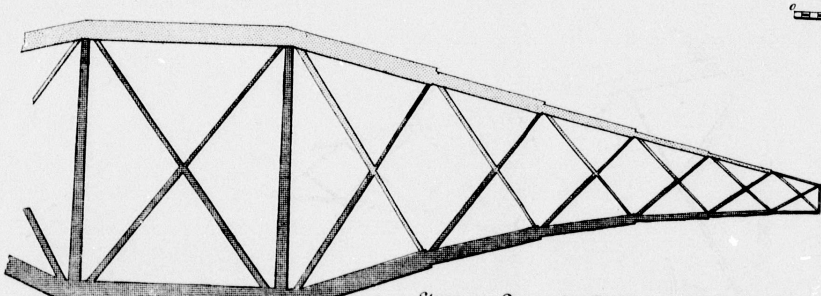
DIAGRAMS OF GR

Scale 200

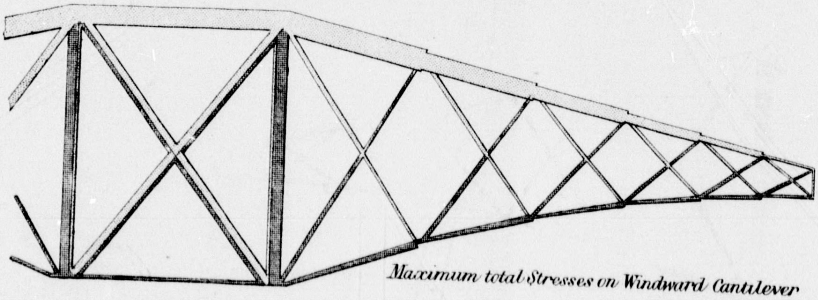
0 100 200

Scale 20,000

0



Stresses from the Dead Load alone



Maximum total Stresses on Windward Cantilever

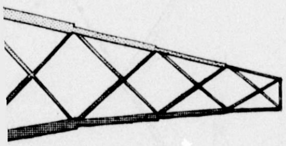
FORTH BRIDGE
DIAGRAMS OF GREATEST STRESSES.

Scale 200 Feet to One Inch

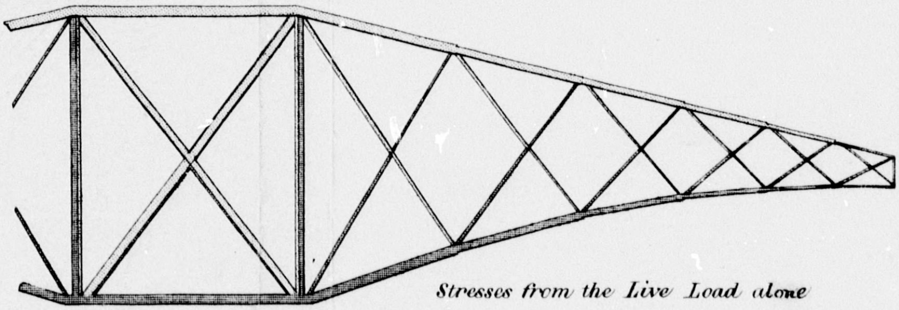
0 100 200 300 400 500 feet

Scale 20,000 Tons to One Inch

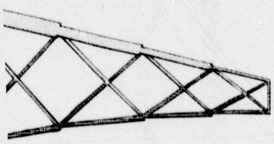
0 10,000 20,000 tons



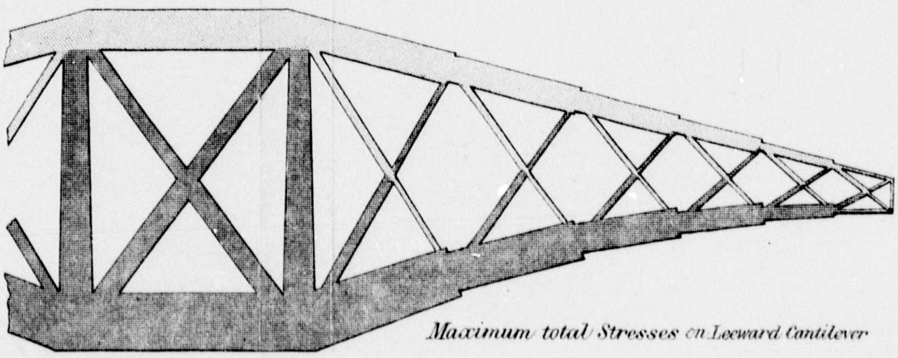
e Dead Load alone



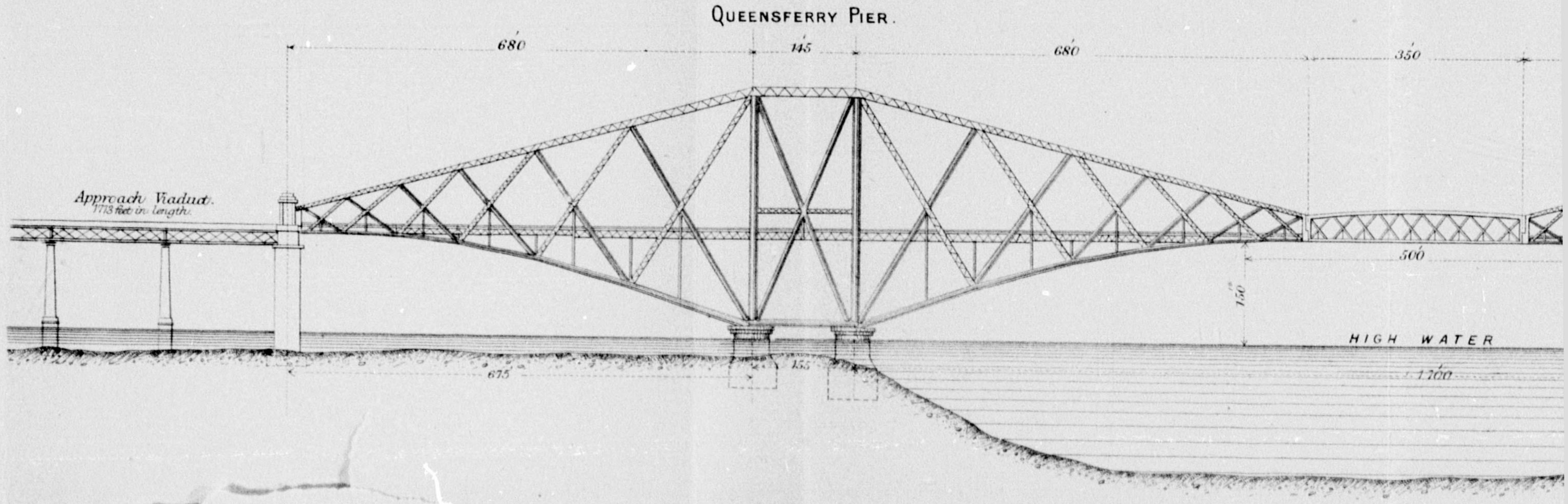
Stresses from the Live Load alone

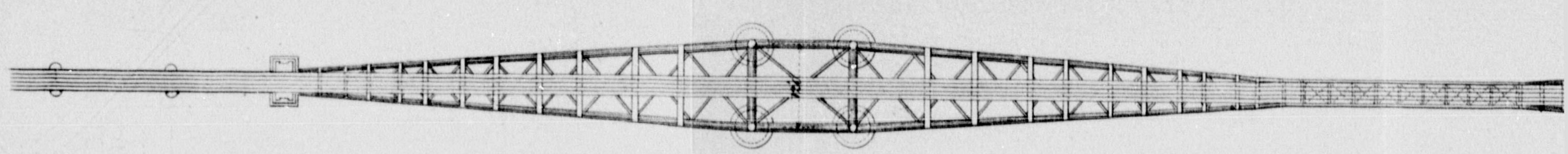


es on Windward Cantilever



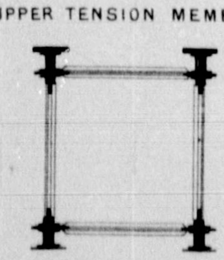
Maximum total Stresses on Leeward Cantilever



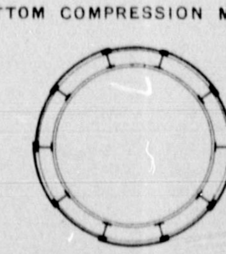


ENLARGED CROSS SECTIONS.

UPPER TENSION MEMBER.



BOTTOM COMPRESSION MEMBER.

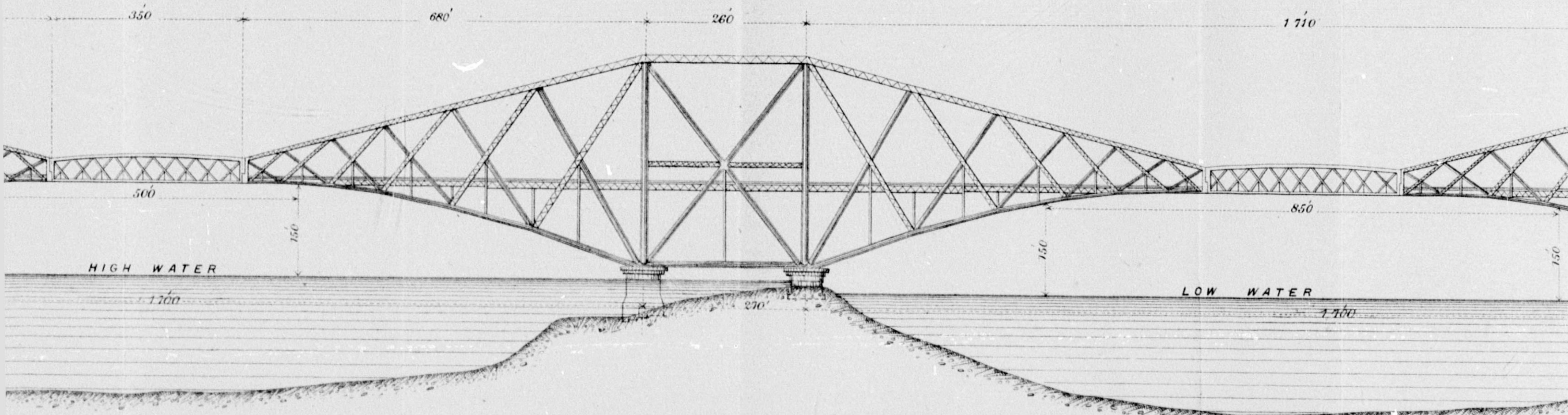


FORTH BRIDGE,

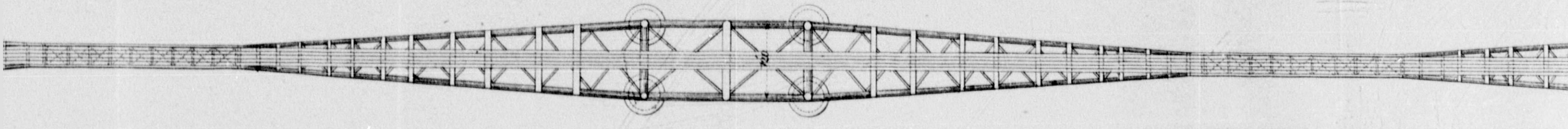
SESSION 1882.

Engineers { JOHN FOWLER, Engineer in Chief.
B. BAKER.

INCHGARVIE PIER.



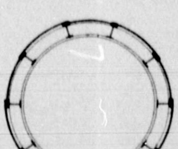
ELEVATION.



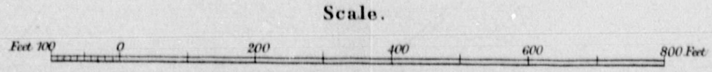
SECTIONAL PLAN AT RAIL LEVEL.

S.

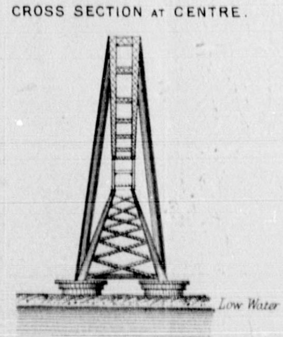
TTOM COMPRESSION MEMBER.

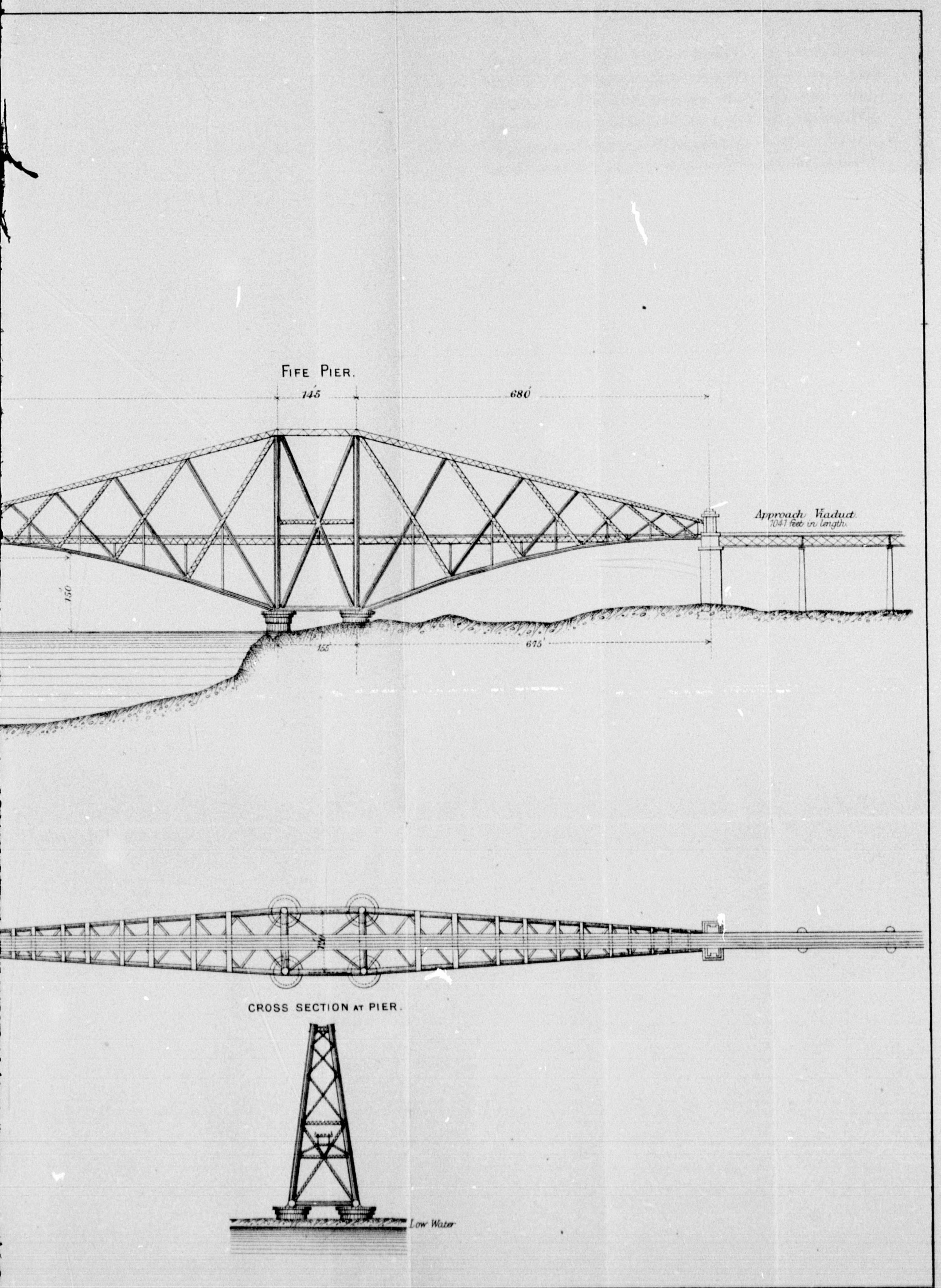


Scale.



CROSS SECTION AT CENTRE.





THOs KELL & SON, LITH. 40, KING STREET, COVENT GARDEN