Resource C

PAPER 1

THE LAW REPORT (TAY BRIDGE INQUIRY)

t839_resourceC_New.indd 3 1/3/2007 5:33:16 Shobha

TAY BRIDGE INQUIRY.

REPORT

TO THE

COMMISSIONERS FOR THE TAY BRIDGE CASUALTY,

BY

HENRY LAW, MEMB. INST. C.E.

Dated 9th April, 1880.

t839_resourceC_New.indd 4 1/3/2007 5:33:16 Shobha

5 & 46, Queen Anne's Gate, Westminster,
9th April, 1880.

To the Commissioners for the Tay Bridge Casualty.

Gentlemen,

In obedience to the instructions contained in your communication of the 22nd of January, 1880, I have now the honour to lay before you the following Report, embodying the information which I have been able to obtain upon those matters which have a bearing upon the casualty which occurred to the Tay Bridge on the night of the 28th of December, 1879.

In accordance with your subsequent instructions, in the present Report I have confined my attention exclusively to that portion of the bridge which has fallen; and for the sake of brevity and distinctness, I have omitted all reference to those details and particulars of the structure, which, although they may have an important bearing upon the question of reconstruction, have no connection with the cause of the catastrophe.

The bridge as constructed, consisted of 85 spans, namely, 28 still standing upon the southern side of the river, varying in span from 67 feet to 145 feet, 13 spans which have fallen, and 44 still standing on the northern side of the river, and varying in span from 162 feet 10 inches to 28 feet 11 inches.

It will not be necessary to refer to the construction of any other portion of the standing parts of the bridge beyond the two spans immediately contiguous to those which have fallen.

These consist of wrought iron lattice girders resting upon piers, each of which are composed of six cast iron columns, braced with wrought iron struts and ties, resting upon foundation piers of masonry, brickwork and concrete. The southern span is 145 feet, and the northern span is 162 feet 10 inches. Each girder is 16 feet 6 inches in height, and their distance apart from centre to centre, varies from 9 feet at their in-shore ends, to 14 feet 10 inches at the ends adjacent to the fallen spans.

These girders rest upon seven cast iron rollers, bearing upon raised surfaces on thick cast iron bearing plates, the rollers having beveled flanges to serve as girders, but there being no attachment between the girders and the piers. The ends of these girders are strengthened to enable them to

1 A 𝔍.

t839_resourceC_New.indd 5 1/3/2007 5:33:18 Shobha

carry the ends of the larger girders which have fallen, forming a table or shelf upon which the latter girders rested, three cast iron rollers being interposed to allow the girders to expand or contract. These rollers were provided with flanges similar to those below, but there was no attachment between the upper and lower girders. The upright ends of the lower girders were steadied by two transverse wrought iron girders, one at the top and the other at the bottom, with diagonal tee-iron stays.

In the portion of the bridge yet standing, the rails are carried upon transverse timber beams laid upon the upper surface of the girders, but in the portion which has fallen, the rails were carried upon transverse wrought iron beams, resting upon and secured to the lower booms of the girders.

The length of the portion of the bridge which has fallen is 3,149 feet, consisting of three separate girders, the southernmost one being 1,225 feet in length, divided into five equal spans, each of 245 feet, the middle girder being 944 feet in length, divided into four spans, of which the two outer ones are each 227 feet and the two inner ones each 245 feet, and the northernmost girder which is divided into four equal spans, each of 245 feet. It will thus be seen that the fallen portion of the bridge consisted of eleven spans, each of 245 feet, and two spans, each of 227 feet.

The gradient of the Railway over the southern standing portion of the bridge is a rising one of 1 in 35·368, and this gradient was continued over the first span. Over the second span the gradient changed to 1 in 490, still rising; the line then continued level for six spans, this being the most elevated portion of the bridge; the next span had a falling gradient of 1 in 130, and the remaining four spans had a falling gradient of 1 in 73·56, which continues over nearly the whole of the northern portion of the bridge.

The course of the Railway over the fallen portion of the bridge and for a considerable distance upon each side of it was a continuous straight line.

The fallen portion of the bridge consisted of wrought iron lattice girders 27 feet in height placed at a distance of 14 feet 10 inches apart from centre to centre. The upper and lower booms were trough-shaped, being each 2 feet in width, and between 15 and 16 inches in depth. The girder over each span was complete within itself, the vertical ends being of similar section to the booms, only 18 inches in width upon the face; the lattice bars, which had only a tensile strain to resist, consisted of flat bars in pairs, one being riveted to each side of the booms; those which were in compression consisted of I shaped struts, placed between the sides of the booms, and secured to them and to the tensile bars at their intersections.

t839_resourceC_New.indd 6 1/3/2007 5:33:18 Shobha

The upper booms were braced by transverse wrought iron beams with diagonal stays. The Railway was carried upon transverse wrought iron fish-bellied girders about 5 feet 5 inches apart, which rested upon the upper side of the lower booms, and being riveted thereto served as struts to the girders, the bracing being rendered complete by diagonal angle iron stays, crossing through the centre of each alternate transverse girder. In order to lessen the transverse strain upon the bottom boom, suspension bars of wrought iron were attached to the lattice bars at their intersections, and riveted at their lower extremities to the sides of the boom.

The various parts of these girders have been carefully proportioned to the several strains to which they had to be exposed, and as the catastrophe did not result from the failure of these girders, it is not necessary more particularly to describe them. It is, however, desirable to make an observation with reference to how far each division should be regarded as having formed a continuous girder. As already mentioned, each girder was complete in itself, and the booms of these separate girders were connected by cover plates with the intention of making them continuous; but in the face of the evidence given at Dundee, of the manner in which these connections between the girders were made, I do not think that these divisions can be considered to have been continuous in such a manner as to produce an increased pressure upon any of the piers. It was stated by William Oram (Question 6,494), that the connecting cover plates were temporarily secured by service-bolts, which were afterwards removed and replaced by rivets; the bridge, in the mean time being used for the passage of heavy ballast trains. (Questions 6,821 to 6,825). It is true that the ends of the girders had been originally raised before the cover plates were bolted on; but it must be evident that no strain such as would produce continuity in the girders in the sense now under consideration could have existed, for if it had it would have been quite impossible to have removed any of the bolts.

Judging from the portion of the bridge which is standing, the permanent way appears to have been very carefully constructed. The rails are laid upon longitudinal timbers or way-beams, 18 inches wide by 15 inches in depth, the rails themselves are of steel, 75 lbs. to the yard, with guard rails of the same weight and material, both rails being secured in the same chairs, which are placed 3 feet apart; a flat wrought iron tie bar is also introduced at distances of about 19 feet apart to preserve the line in gauge.

The platform of the bridge was formed of planks 4 inches in thickness, covered with asphalte and with a few inches of ballast as a preservative against fire.

I now proceed to describe the most important part of the structure in connection with the subject under consideration, namely, the piers upon which the fallen portion of the bridge was supported.

3

t839_resourceC_New.indd 7 1/3/2007 5:33:19 Shobha

These piers each consisted of an assemblage of six cast iron columns, braced by means of wrought iron studs and ties. Their foundations consisted of hexagonal-shaped piers of concrete, faced with brickwork, measuring 27 feet 6 inches in length from point to point of the cutwaters, and 15 feet 6 inches in width. These piers were carried to a height of 5 feet above the level of high water of spring tides, the upper four courses being faced with stone, and no movement or settlement appears to have taken place in them.

The height from the top of the upper course of the masonry to the under side of the lattice girders varies from 83 feet to 81 feet 3 inches; in the following description and in all the calculations the highest pier is referred to; as, however, the height of the pier affected the strength, it may be desirable to give in a tabular form the heights of the several piers above the masonry and the spans of the girders which they supported; the numbers in the first column are the numbers of the piers in the structure, counting from the southern side, and, to avoid confusion, will be adhered to throughout this Report.

No. of Pier.	Height of Pier.	No. of Span.	Width of Span.	Description of bearing on Pier.
28	67 „ 6	29	245	3 rollers on lower girders.
29	82 „ 6	30	245	8 rollers on pier.
30	83 „ 0	31	245	8 rollers on pier.
31	83 „ 0	32	245	Bolted to top of pier.
32	83 „ 0	33	245	8 rollers on pier.
33	83 „ 0	34	227	6 rollers and an expansion joint.
34	83 „ 0	35	245	8 rollers on pier.
35	83 „ 0	36	245	Bolted to top of pier.
36	83 „ 0	37	227	8 rollers on pier.
37	82 „ 8	38	245	6 rollers and an expansion joint.
38	82 „ 4	32	245	8 rollers on pier.
39	82 „ 0	40	245	Bolted to pier.
40	81 „ 8	41	245	8 rollers on pier.
41	66 „10			3 rollers on lower girders.

Cast iron base-pieces, 2 feet in height, for the reception of the columns, were secured to the piers, each piece having four holding-down bolts passing through the upper two courses of masonry, each of which was 15 inches in thickness.

The six colums were arranged in the form shown upon the plan, Drawing No. 1, so as to form two clusters, each triangular on plan, and having no other connection at their upper extremities beyond the struts and

t839_resourceC_New.indd 8 1/3/2007 5:33:21 Shobha

ties. The two extreme columns marked 1 and 4 on the plan were each 18 inches in diameter, and inclined inwards at the top 12 inches in their whole height; the other four columns, 2, 6 and 3, 5, were each 15 inches in diameter. They stood in vertical planes parallel to the direction of the bridge, but in those planes 2 and 6 and 3 and 5 were each inclined 12 inches towards each other in their whole height.

Each column was composed of six flanged pipes, connected at their joints with eight screwed bolts, each $1\frac{1}{8}$ inches in diameter. Each triangular cluster was surmounted by a wrought-iron box girder L shaped on plan, taking its bearing upon the three columns; and upon this box girder another wrought iron cellular girder was placed, running in the direction of the axis of the bridge, and vertically under the longitudinal lattice girder of the bridge itself. Upon the upper side of this cellular girder was bolted a massive cast iron plate, a similar plate being also bolted to the underside of the longitudinal lattice girders of the bridge, and between these two plates were placed the cast iron rollers, each 5 inches in diameter and 2 feet in length, upon which the weight of the bridge was carried. This description applies to all the piers, excepting Nos. 31, 35 and 39, in the case of which piers the rollers were omitted, and the longitudinal lattice girders were united to the cellular girders by screwed bolts.

Measuring across the bridge, the cellular girders were equally distant from the centres of the tops of columns 1, 2 and 6, and 3, 4 and 5, and consequently the pressure of the girders of the bridge was borne half by each outer 18 inch column, and one-fourth by each inner 15 inch column.

The three columns forming each triangular group were braced to each other at every joint by wrought iron struts and ties; the struts were horizontal and consisted of two channel irons placed back to back and bolted at each end by two $1\frac{1}{8}$ inch bolts to lugs cast upon the columns. Each of the rectangular openings formed by the columns and struts was stayed diagonally by flat wrought-iron bars $4\frac{1}{2}$ inches broad and half an inch in thickness, the upper ends being connected with the columns by $1\frac{1}{8}$ inch bolts passing through lugs cast upon them, and the lower ends being secured to two sling plates, each $4\frac{1}{2}$ inches by $\frac{3}{8}$ths of an inch thick by gibs and cotters, and the sling plates being connected with the columns by $1\frac{1}{8}$ inch bolts passing through lugs cast on to them.

The two triangular clusters of columns were braced to each other in a similar manner by struts and ties between the 15 inch columns; that is to say, between columns 2 and 3, and columns 5 and 6. Furthermore at each joint a wrought iron rod $1\frac{1}{2}$ inches in diameter was introduced horizontally to tie together columns 2 and 5, and columns 3 and 6.

5 B g

t839_resourceC_New.indd 9 1/3/2007 5:33:21 Shobha

Having thus given a general description of the portion of the bridge which fell, I proceed to consider the strains to which the several parts were exposed under varying circumstanses, and how far the structure was capable of resisting those strains. In order, however, to render this Report as brief as possible, and to avoid as far as can be done the introduction of technicalities, I shall here confine myself to a statement of results, but for your information the mode of arriving at those results is annexed in the form of an Appendix.

The four forces to which the structure was liable to be exposed were those resulting from changes of temperature, from the weight of the structure itself, from the weight of a passing train, and from the lateral pressure of the wind.

For our present inquiry the strains produced by changes of temperature may be disregarded, and those resulting from the weight of the structure itself, or when loaded with a train, are very easily ascertained. Assuming for the reasons already stated that no additional strain is produced upon any of the piers in consequence of the continuity of the girders, and assuming a train with the weight and conditions of that which fell with the bridge, namely having a weight, including the passengers, of 120 tons, and supposing it to be placed over one of the piers in the position which would produce the heaviest pressure, I find that the structure alone would produce a compressive strain upon the 18 inch columns of 1·47 tons, and upon the 15 inch columns of 1·06 tons to the square inch; and that with the train over the pier these strains would be increased to 1·84 tons on the 18 inch columns, and to 1·30 tons on the 15 inch columns.

There are so many doubtful elements, the values of which have to be *assumed* in attempting to determine the amount of the strains to which the several parts of the piers would be exposed by the action of a powerful wind pressure, that it is impossible to arrive at any positively definite result.

As regards the actual pressure of the wind upon the structure, I have adopted the same views as those taken by Dr. Pole and Mr. Stewart, namely, as regards the lattice girders, for the windward girder I have taken the entire area of the outer face, including the way-beams and rails; for the leeward girder I have taken only the surface above the level of the rails, and I have supposed that the wind would only exercise half of its force against this surface, in consequence of the shelter afforded by the windward girder. As regards the train, I have wholly deducted the surface of the leeward girder which it would shelter, and for the train itself I have only taken half the round surfaces, and have reduced the pressure of the winds by a sixth, that being the extent to which the train would be sheltered by the windward girder.

6

t839_resourceC_New.indd 10 1/3/2007 5:33:22 Shobha

In the case of the pier I have again adopted the views of Dr. Pole and Mr. Stewart, namely, in supposing that there would be one 18 inch column and three 15 inch columns exposed to the wind, and that the tie-bars and struts would be equivalent to one-fourth of the space (when seen in end elevation) between the columns.

Now, it is a matter of the first importance to determine what wind pressure would suffice to overturn any portion of the train; it is at once evident that the second class carriage, being the last but one in the train, was the one which had the least stability; and Dr. Pole and Mr. Stewart state that a wind pressure equal to 28½ lbs. upon the square foot would suffice to overturn this carriage. They have, however, assumed that the carriage was empty, whereas the evidence of those who collected the tickets shows that there were eight second class passengers.

In my own calculation I have assumed the average weight of the passengers at 140 lbs. each, and I have taken into account the vertical pressure resulting from the action of the wind upon the curved surface of the roof, and the conclusion at which I arrive is, that the second class carriage could not have been overturned with a less wind pressure than 35·68 lbs. upon each square foot; and as there is no position in which this carriage could have been placed where it would have been sheltered to a greater extent than between one-seventh and one-eighth of its entire surface, it results that the actual pressure of the wind must have exceeded 40 lbs. on the square foot to have overturned this particular carriage, in the condition in which it was upon the night of the catastrophe, and without regarding any assistance which the couplings might afford in retaining the carriage upon the rails.

The next subject that I have investigated is the effect which the wind would have in lessening the weight of the superstructure upon the windward rollers, and in increasing the same upon the leeward ones, and the results are shown in the following table:—

	Without any Wind.	With Pressure of Wind equal to			
		10 lbs.	20 lbs.	30 lbs.	40 lbs.
	lbs.	lbs.	lbs.	lbs.	lbs.
Without any Train:—					
Pressure on west rollers	322,450	300,190	277,930	255,670	233,410
„ east rollers	322,450	344,710	366,970	389,230	411,490
	644,900	644,900	644,900	644,900	644,900
With a Train:—					
Pressure on west rollers	427,615	399,205	370,795	342,385	313,975
„ east rollers	427,615	456,695	485,775	514,855	543,935
	955,230	855,900	856,570	857,240	857,910

7

t839_resourceC_New.indd 11 1/3/2007 5:33:23 Shobha

The slight increase which will be observed in the total pressure upon both rollers with an increased wind is owing to the vertical pressure resulting from the action of the wind on the curved roofs of the carriages.

These pressures upon each set of rollers are, as I have already explained, equally divided between one 18 inch and two 15 inch columns; these pressures are, however, still further modified by the horizontal pressure of the wind acting against the exposed surfaces of the superstructure, pier and train, but to what extent it is very difficult to determine.

If for a moment it is assumed that the pier may, by virtue of the system of bracing, be considered as a rigid structure, and the effect of the bolts in holding down the columns be disregarded, then the wind pressure required to overturn the structure, about the east 18 inch column as a centre, would be 36·38 lbs. without any train, and 32·69 lbs. on the square foot, with a train over the pier.

But, unfortunately, the piers must have been very far from being rigid structures, in consequence of the imperfect manner in which the struts and ties were connected with the columns. The struts consisted of channel irons, placed back to back with the lug of the column between, and connected therewith by two $1\frac{1}{8}$ inch bolts at each end; the holes for the bolts were cast $1\frac{1}{4}$ inches in diameter, and being rough and larger than the bolts, and the ends of the struts having no bearing surface to abut against, the struts themselves were only retained in their positions by the pinching action of the bolts. But the security thus afforded must have been very slight, because owing to inequalities in the surfaces of the lugs themselves, and to the fact that in some cases the holes in the struts had been roughly enlarged with a blunt tool so as to leave a burr, the actual bearing surface of many of the struts against the lugs was very small.

As regards the flat ties, when the structure was first erected they were tightened up by means of gibs and cotters, but owing to the slots in the bars against which the gibs and cotters bore being rough, and the gibs and cotters also roughly forged, and further, owing to the holes cast in the lugs not being cylindrical, and to a screwed bolt being used to secure the ends of the ties instead of a pin, the real bearing surface was exceedingly small, and a comparatively slight strain would suffice to crush the edge of the hole in the lug into the thread of the screw.

In reference to the tie bars it should also be observed that the bearing surface of the gib against the slot in the bar was quite inadequate, for while the area of the section of the bar exposed to a tensile strain was 1·625 square inches, the bearing surface of the gib being in compression should have had an area of 1·86 square inches, whereas it had only a surface of 0·375 square inches, or about $\frac{1}{5}$th of the strength of the bar.

8

t839_resourceC_New.indd 12 1/3/2007 5:33:24 Shobha

From these circumstances it would result that a lateral pressure against the columns would produce movement in the struts and ties, resulting in the latter becoming slack. And this movement actually did take place, in some of the tie bars still standing I found packing pieces of iron a quarter of an inch in thickness had been introduced between the gibs and cotters, and on enquiry I learned that these had been introduced from time to time since the opening of the bridge.

From the accounts which have been furnished to me it appears that about 150 of these packing pieces were inserted in the ties between the middle of October of 1878 and the time of the bridge falling, and that the necessity for them arose before the bridge had been opened five months. This circumstance clearly shows that there must have been a considerable racking movement in the piers under the united action of passing trains and wind, and I cannot but consider points to the primary cause of the disaster.

For the slackening of these ties and struts means the removal of that condition upon which alone the power of the structure to resist being overthrown by a lateral pressure depends. And it is easy to conceive that a storm of the violent character of that of the 28th of last December would produce such movements, in the connections of these struts and ties with the columns, as would render the columns unable to sustain the additional weight of the train and the lateral pressure of the wind.

An examination of the ruins of pier No. 32, being that over which the train was situated when the structure fell, indicates that the columns doubled up about their joints as the lower lengths of the westward 15 inch columns were pushed over to the west, or in the reverse direction to that in which the rest of the structure fell. A similar action in pushing back the westward columns is seen in piers No. 36, 39 and 40.

The present state of piers Nos. 29 and 31 affords conclusive proof of a weak point existing in the structure at the time of the overthrow in each of those piers, namely, in pier No. 29 at the level of the top of the second tier of columns, and in pier No. 31 at the top of the lower tier; for the strain at the point of fracture was, in the former case, only $\frac{5}{7}$ths, and in the latter case only $\frac{6}{7}$ths, of the strain at the base of the pier, while theoretically the strengths of the pier at the base and the points of fracture were the same. It is clear, therefore, that the power of resistance of these two piers had been reduced at the points of fracture in the case of pier No. 29 to the extent of $\frac{2}{7}$ths., and in the case of pier No. 31 to the extent of $\frac{1}{7}$th, of their normal power of resistance.

Considering that the columns are 76 feet in height, that with a wind pressure of only 20 lbs. on the square foot, a pressure of 337 tons will be

9 c g

t839_resourceC_New.indd 13 1/3/2007 5:33:25 Shobha

thrown upon the eastward 18 inch column at the time of the passage of the train, and that a horizontal pressure of $37\frac{1}{2}$ tons is acting against the top of the column, it is easy to conceive what must have been the inevitable consequence of any slackness on the part of the ties.

It is also necessary to point out that owing to the double angle which the ties by which the 18 inch columns are braced make, with the direction of the force tending to overthrow the structure, the efficiency of these ties is reduced in the proportion of 1 to 2·73, or to little more than one-third of their full strength, and that any elongation or movement of the ties would allow of nearly three times that movement in a horizontal direction in the point of the column to which they were attached.

There are also other circumstances in connection with the construction and workmanship of the bridge which undoubtedly contributed to the catastrophe.

The mode of securing the holding-down bolts was not satisfactory as they had no anchor-plate or bearing at their lower extremities, but were merely inserted in a hole drilled through the two 15 inch courses of stone, and were then run round with cement, and as the angle of taper of the conical head was only $6\frac{1}{4}$ degrees it is evident that a very slight compression of the cement would allow of a considerable movement in the bolt; some of these bolts have evidently yielded as much as eight inches in screwing down the base-piece at the erection of the bridge, and in one or two cases the stones been burst by the wedge action of the conical head. It would have been better also if they had been carried to a greater depth, so as to have had a greater weight of masonry to be lifted, instead of trusting to the adhesion of the cement which appears to have been very slight, partly in consequence of the smoothness of the sawn face of the stone, and partly, I imagine, from the stone having been dry when set. In many cases the cement has parted from both stones forming a thin detached sheet of large dimensions. In many cases also the nuts at the upper ends of the bolts have a very imperfect bearing upon the base-piece.

Passing on to the columns, it is apparent that many of them have blow holes of considerable size, which have been filled in with a composition of resin and filings; sufficient care does not appear to have been taken to keep the cores from shifting, or in properly adjusting the upper flask, and as a consequence there are many instances of a considerable difference in the thickness of metal on opposite sides of the column; in some cases the metal on one side being only $\frac{5}{8}$ths of an inch, and on the opposite side, $1\frac{3}{8}$ths, or a difference of $\frac{3}{4}$ of an inch; and as is usually the case when the upper side of a casting is thin, the metal becomes chilled, and has accumulations of scum and air which very much deteriorate from the strength of the metal.

t839_resourceC_New.indd 14 1/3/2007 5:33:26 Shobha

The mode of attaching the ties to the columns by means of lugs was evidently insufficient, as in almost every instance the lugs have been torn away. it is difficult to believe that the burning on of defective lugs in the manner described by the witnesses examined at Dundee could have been sanctioned by any person who had the intelligence to understand that the whole security of the structure depended upon the strength of these lugs.

I consider that the mode of connecting the columns at the flange joints was also in some respects defective, the bolts being an-eighth of an inch less in diameter than the hole, and the flanges being separated in some cases as much as $\frac{3}{4}$ of an inch, the bolts could not act as steady-pins, and as in several cases there was no spigot on either of the pipes, there was nothing but the pinching of the bolts to prevent the columns from shifting, and there are evidences that some of them did so shift at the time of the catastrophe.

I have not regarded the concrete as having added in any way to the security of the structure, otherwise than in its increasing the weight of the columns and so increasing the moment of stability of the pier; and my reason for taking this view is that the concrete was so unequal in its quality that no dependence could be placed upon its being of proper strength in the place where strength was required.

Before leaving the columns, I should observe that some of the flanges were so imperfectly faced that the only portions of the metal in contact was a strip of about five-eighths of an inch round the margin of the flange.

In conclusion, I would sum up by the statement that, in my opinion, the base of the pier was too narrow, occasioning a very great strain upon the struts and ties, that the angles at which the latter were disposed, and the mode of connecting them to the columns, were such as to render them of little or no use, and that the other imperfections which have been pointed out lessened the power of the columns to resist a crushing strain; I consider that the yielding of the struts and ties was the immediate cause of the disaster, but that the other circumstances stated contributed to it.

It is only due to Sir Thomas Bouch, to his Assistant, Mr. Thomas Peddie, to Mr. Noble and the Officials of the North British Railway, to say that they have afforded me every facility for making the most thorough and searching investigation.

I have the honor to remain,
Gentlemen,
Your obedient servant,
HENRY LAW.

11

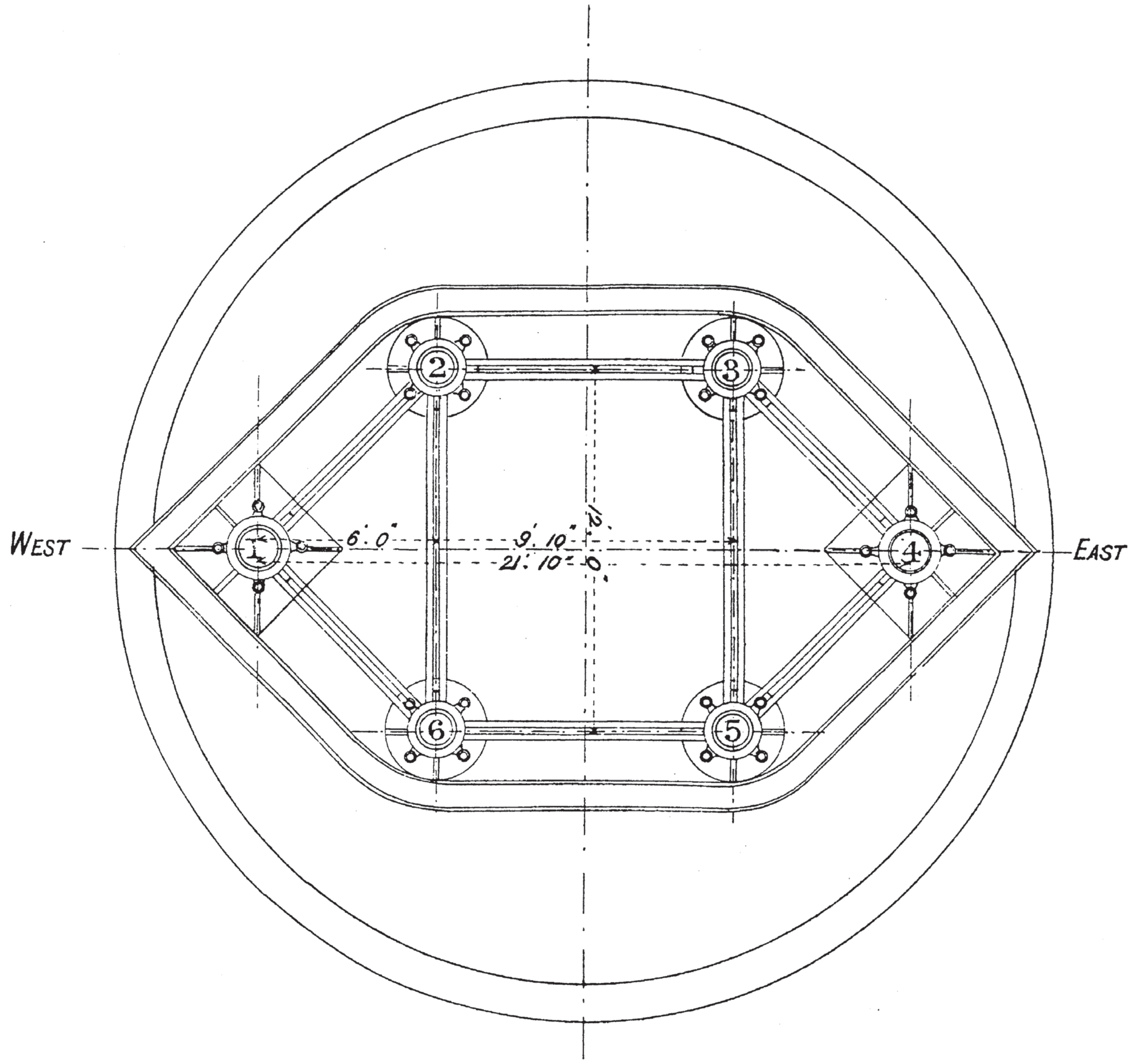
Vacher & Sons, Printers, 29, Parliament Street, and 62, Millbank Street, S.W.

t839_resourceC_New.indd 15 1/3/2007 5:33:27 Shobha

Drawing No. 1.

TAY BRIDGE.

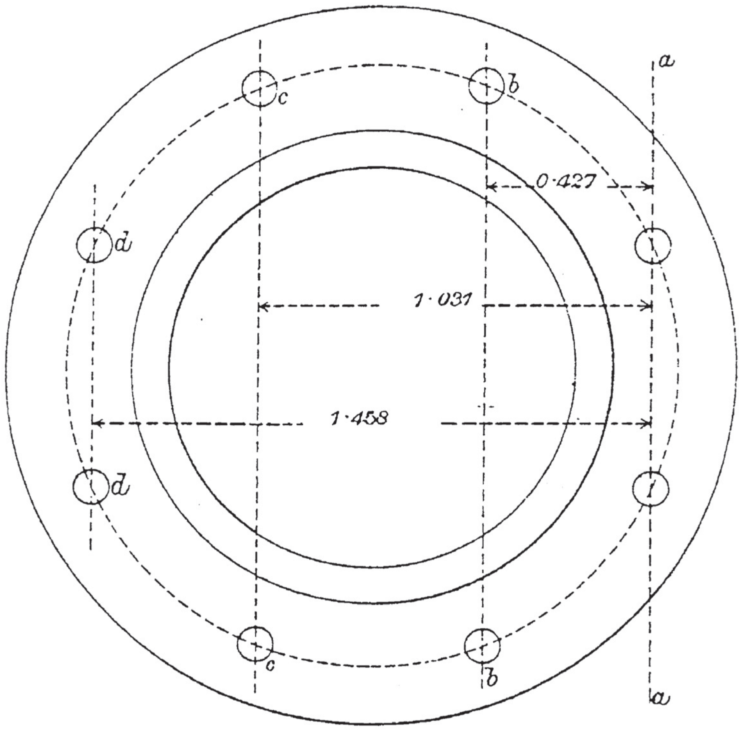
Sectional Plan of the base of one of the fallen piers.



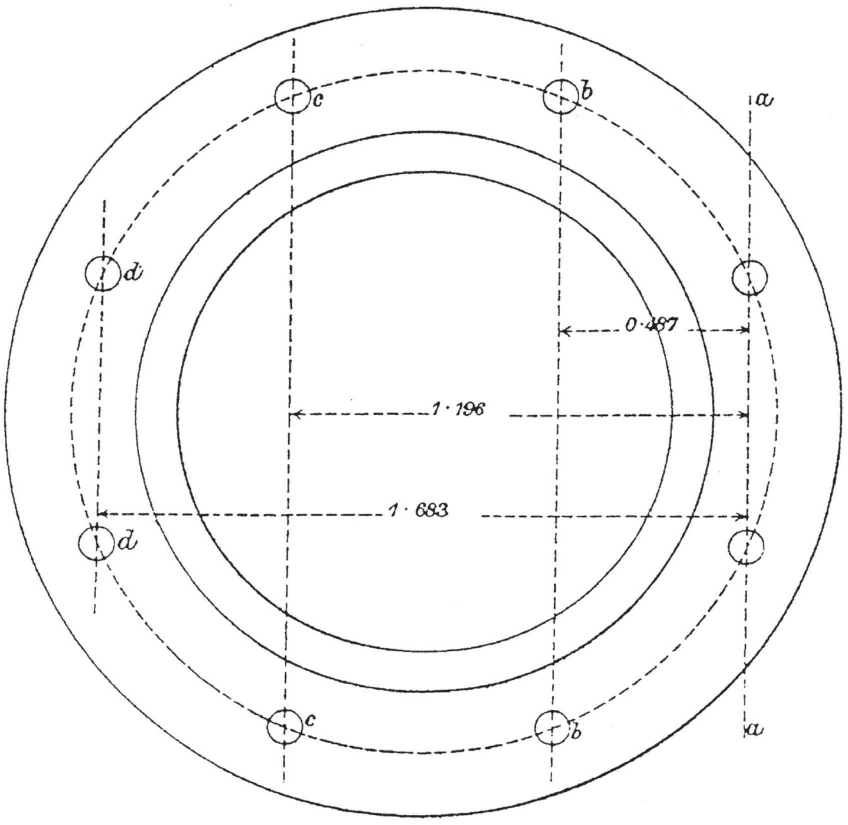
Scale 3/16ths of an inch to a foot.

APPENDIX TO COURT OF INQUIRY REPORT Z.C.E. Drawing No. 2.

TAY BRIDGE.



PLAN SHOWING POSITIONS OF BOLTS THROUGH FLANGES OF 15 INCH COLUMNS.



PLAN SHOWING POSITIONS OF BOLTS THROUGH FLANGES OF

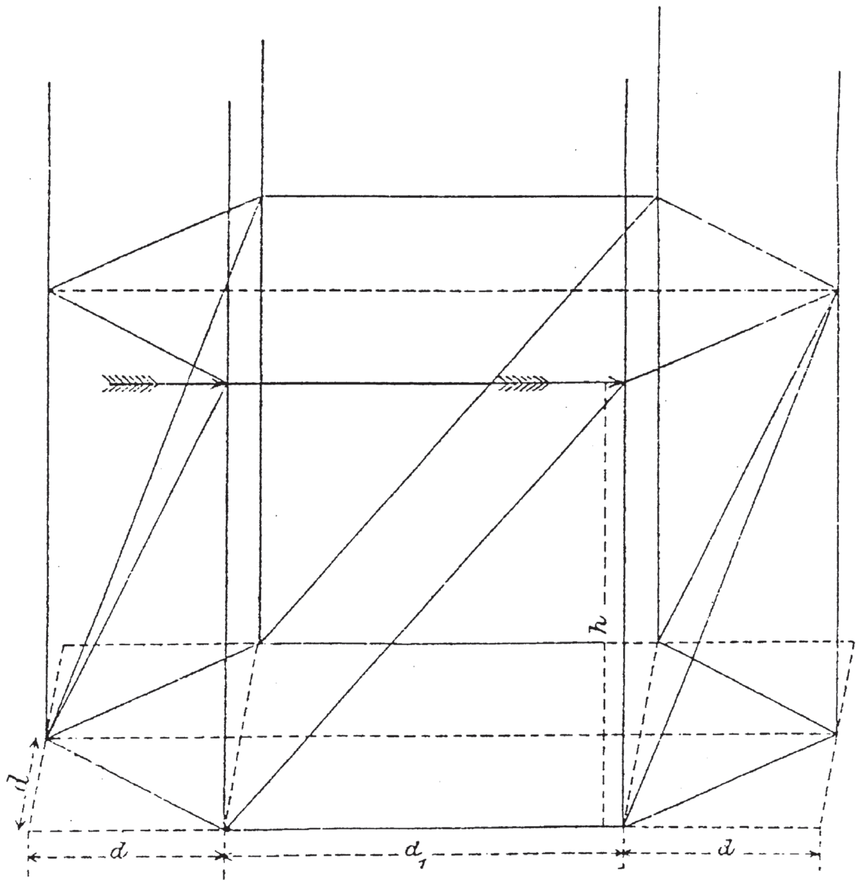
t839_resourceC_New.indd 17 1/3/2007 5:33:29 Shobha

Drawing Nº 3.

APPENDIX TO COURT OF INQUIRY REPORT I.C.E.

TAY BRIDGE

SKELETON DIAGRAM SHOWING THE STRAINS ON THE TIE BARS.

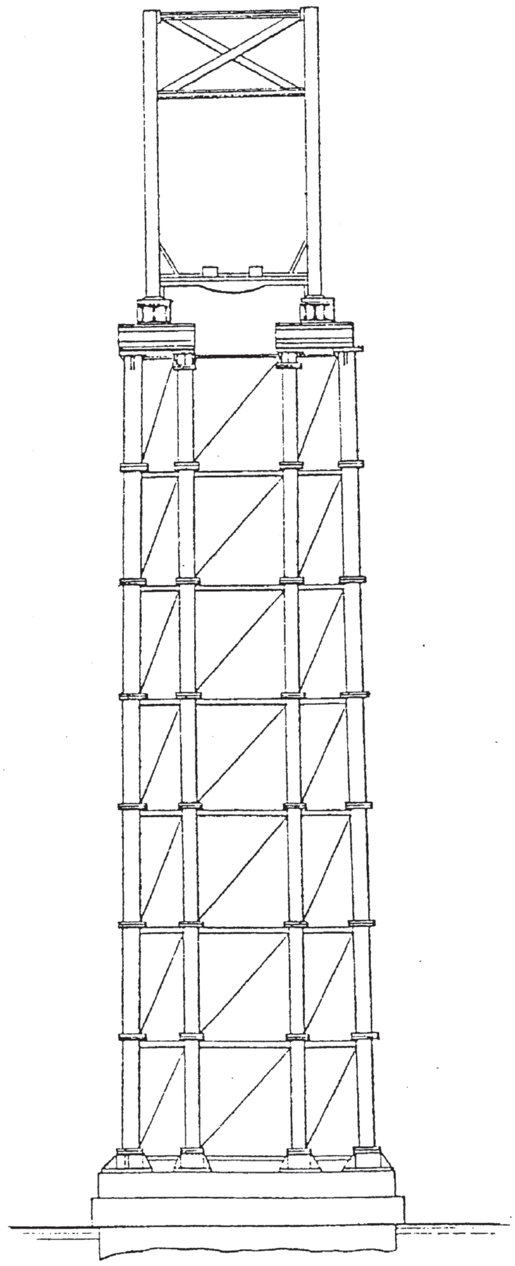


t839_resourceC_New.indd 18 1/3/2007 5:33:29 Shobha

Drawing Nº. 5

TAY BRIDGE.

SIDE ELEVATION OF ONE OF THE PIERS SHOWING ONLY THE BARS IN TENSION



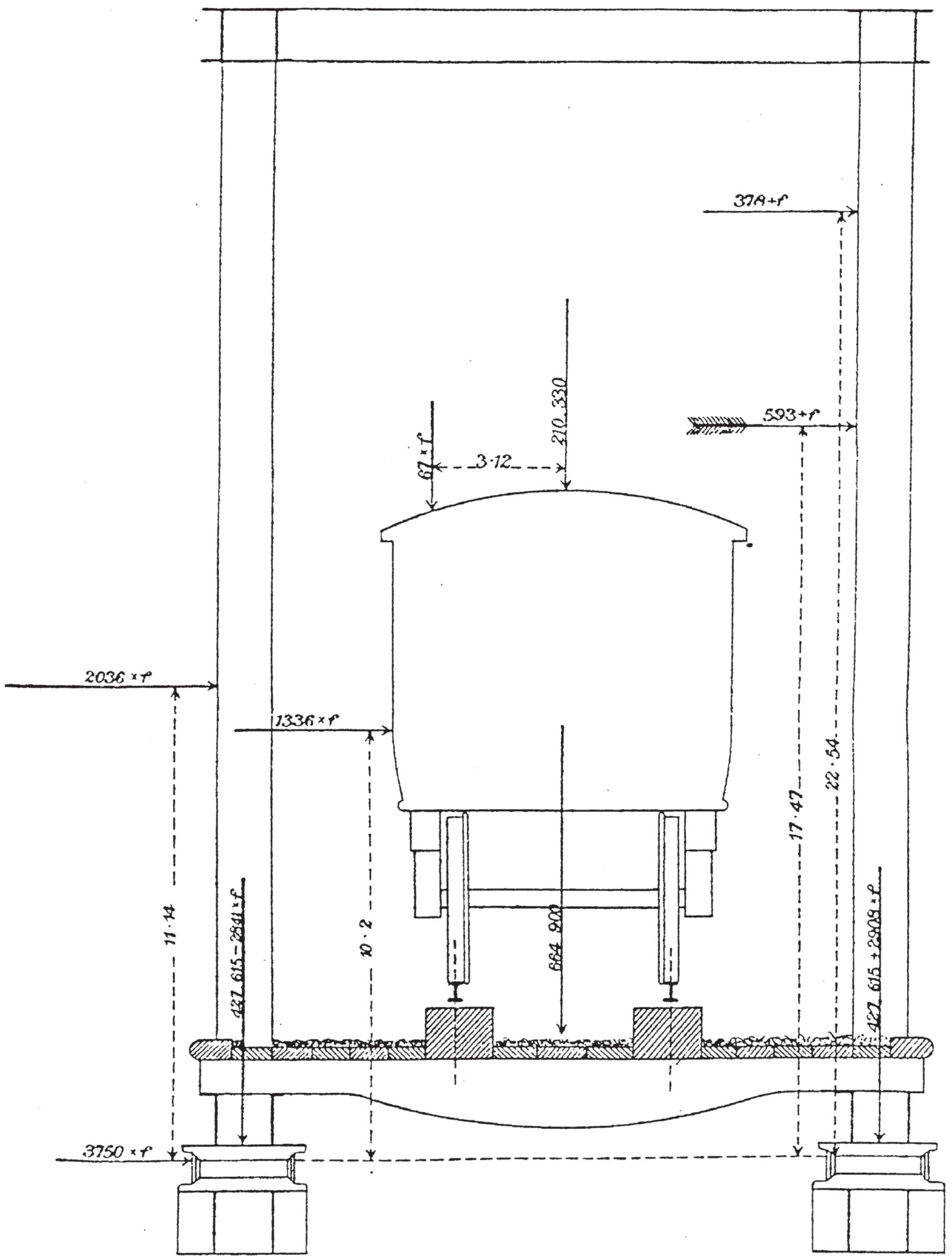
t839_resourceC_New.indd 19 1/3/2007 5:33:31 Shobha

APPENDIX TO COURT OF INQUIRY
Z.C.E.

DRAWING No. 5.

TAY BRIDGE.

TRANSVERSE SECTION OF THE SUPERSTRUCTURE SHOWING THE PRESSURES PRODUCED BY THE ACTION OF THE WIND COMBINED WITH THE WEIGHT OF THE STRUCTURE AND TRAIN.



SCALE ¼ OF AN INCH TO A FOOT.

NOTE. The feathered arrow shows the pressure of the wind when there is no train on the bridge.

t839_resourceC_New.indd 20 1/3/2007 5:33:32 Shobha