

The Tay rail bridge disaster — a reappraisal based on modern analysis methods

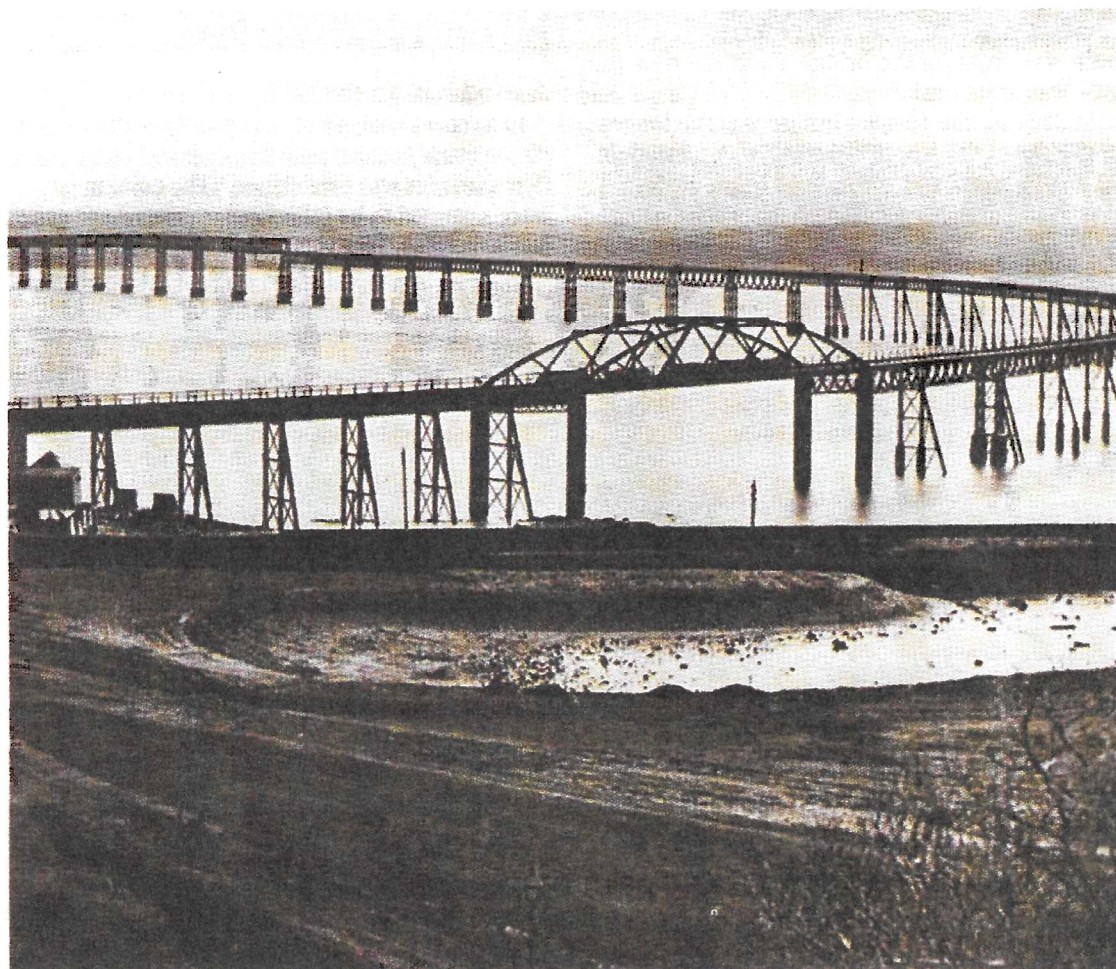
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The collapse of the world's longest bridge in 1879 with the loss of 75 lives exposed a major weakness in British civil engineering. While American and French engineers were using significant wind loadings, few of their British counterparts even considered it. But there is still speculation as to exactly why the first Tay rail bridge in Scotland failed and as to whether its designer, Sir Thomas Bouch, was to blame. This paper, based on a recent three-dimensional computer analysis using a modern approach to wind loading, throws new light on the collapse.

The first Tay rail bridge (Figs 1 and 2) was completed in February 1878 to the design of Thomas Bouch for the North British Railway Company. Bouch was responsible for the design, construction and maintenance of the bridge. He had made his reputation as a railway engineer

by building bridges quickly and on a tight budget. Because his structures were built economically the railway companies were keen to engage his services. Most of his bridges were lattice girders supported on slender cast iron columns braced with wrought iron struts and

Fig. 1. First Tay rail bridge before collapse (viewed from north)

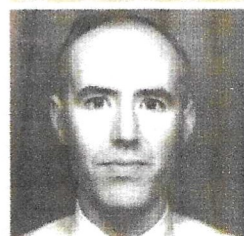


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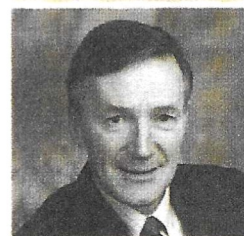
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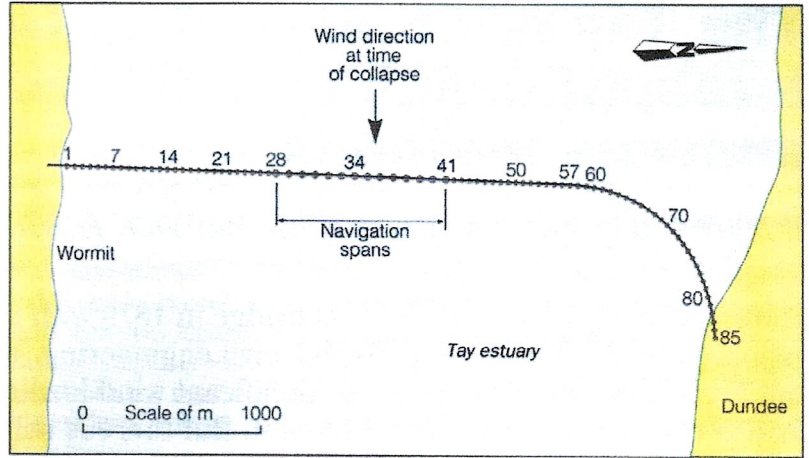


Fig. 2. Schematic plan of first Tay rail bridge showing pier numbers

ties (Fig. 3). The contractor who built his biggest bridges described them as 'flesh without muscle'.¹

The disaster occurred on the stormy night of 28 December 1879. A train of one engine and six carriages crossing the bridge from the south to north was lost when the structure of the navigation spans section collapsed (Fig. 4). Detailed accounts of the disaster are given by Thomas¹ and Prebble². There were no survivors from the 75 people aboard the train. The gale blowing along the Tay estuary at right angles to the bridge was estimated at force 10 to 11 on the Beaufort Scale by local naval officers. Local amateur meteorologists also reported that the train was crossing the bridge about the time the gale was at its peak.³

At 3264 m, the 85-span bridge was the longest in the world. Its single rail track was carried on 72 deck girders and through 13 'high girders' across the main navigation channel. The navigation span girders were 8.2 m high with a 26.8 m

clearance above the high water mark. It was these spans which fell. Most of the deck girders, all of which remained standing, were transferred to the present Tay rail bridge which opened eight years later (Fig. 5).

The piers of the first bridge were originally designed to be of brickwork. However, after the fourteenth caisson was sunk, the solid rock foundation identified in the borer's report proved to be a thin layer of very hard rock overlying mud and clay. The pressure on the piers thus had to be reduced. It was achieved by substituting brick with hastily designed and potentially more expensive cast-iron columns braced with wrought iron struts and ties.

A modern analysis

In a recent analysis of the collapse, a three-dimensional linear elastic frame model of a navigation span pier was established.⁴ The column bases were assumed to be fully fixed except where uplift was simulated. Wind loading was applied using

Fig. 3. Belah Viaduct, 1858





Fig. 4. Tay rail bridge after 1878 collapse (viewed from south)

the British Standard for wind loading, CP3.⁵

Fig. 6 shows the predictions of behaviour. The main variables considered in the analysis were as follows.

- *Wind velocity* — no records of velocity were taken at the time of the disaster but a velocity of 34 m/s corresponding to the middle of the force 11 range is a likely value based on observer reports.³
- *Whether or not the train was on the bridge* — the train appears to have had a significant effect on the overturning action.
- *Tensile failure of the column baseplate bolts* — these bolts were not expected to be in tension but Fig. 6 shows that failure could have occurred under force 10 conditions with no uplift.
- *Uplift of the windward column base* — this factor was not considered at the time of the original inquiry⁶ but it does have a significant effect on the bracing load. It is likely that this was the first step in the collapse. The 5 mm uplift used in Fig. 6 is based on an estimated weight of foundation resisting the applied load at a wind speed of 34 m/s (with no vertical restraint at the column base in the analytical model). Such a lift moves the likely column



baseplate bolt failure to the next row of columns. The windward columns failed in bending just above the baseplates.

- *Bracing strength* — the design strength of a bracing member in tension was 32.5 tons force (1 ton force = 9.97 kN) but measured strengths were less than this as shown in Fig. 6. The uplift of the windward columns significantly increases the bracing load. An important observation from the analytical results is that the

Fig. 5. The existing Tay rail bridge designed by Barlow



Fig. 6. Maximum bracing loads in relation to wind speed
(1 mile/h = 0.45 m/s,
1 ton force = 9.97 kN)

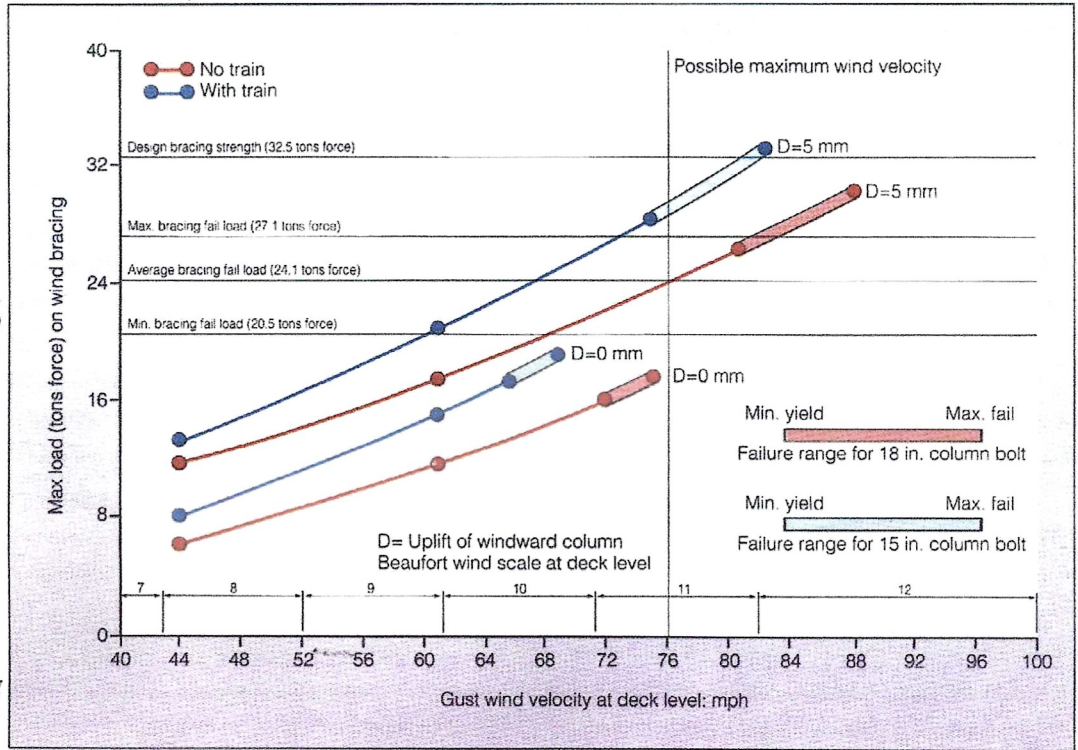
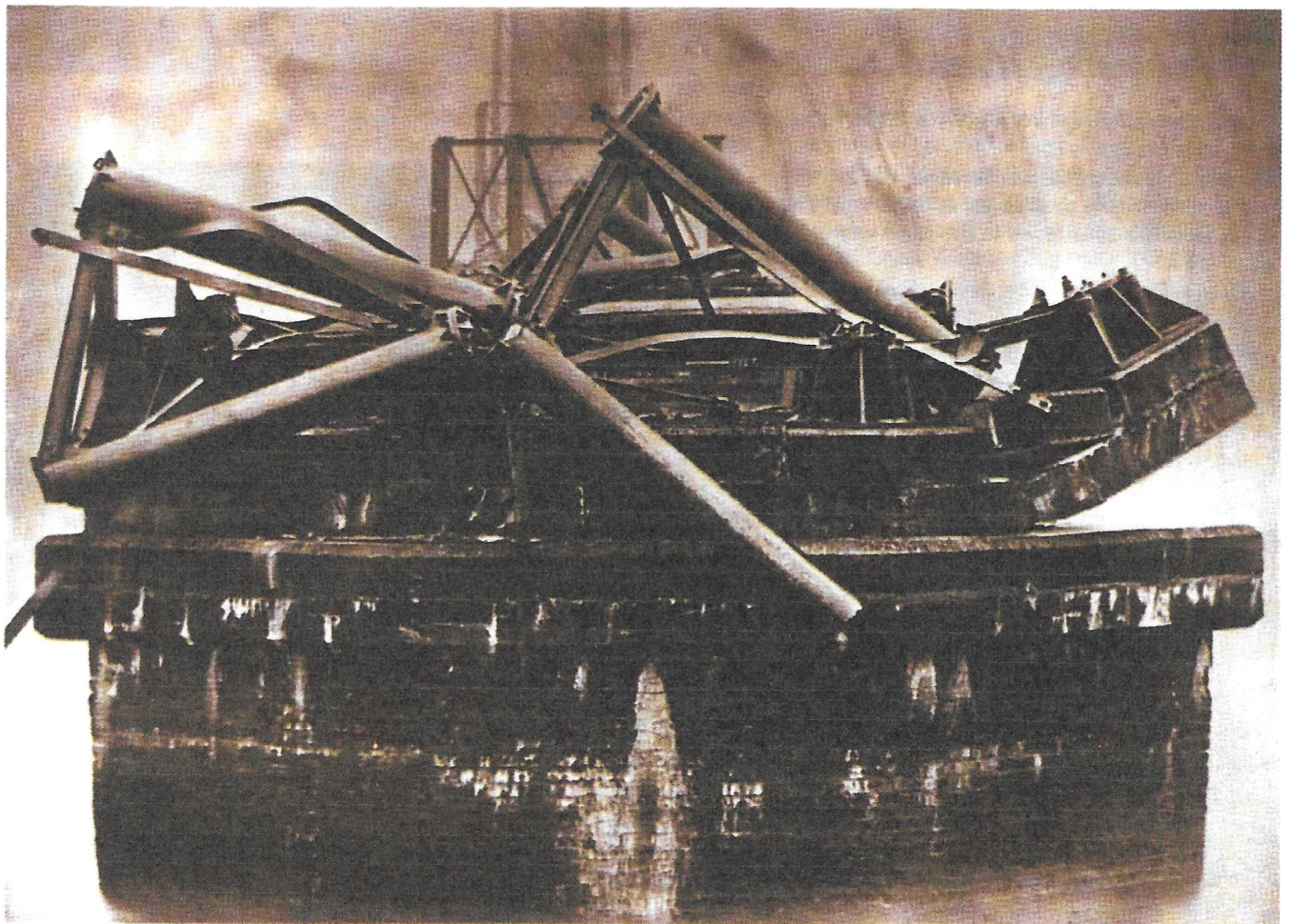


Fig. 7. Pier no. 33 after collapse showing lifting of masonry on windward side (right)



highest bracing load is not at the level above the base of the columns but at the next level up. This is due to the fact that the moment connection at the base of the column induces the column to take some of the base shear thus relieving the bracing load at the base.

On the basis of these observations a scenario for the collapse is as follows. When the train reached the high girders there was a particularly strong gust. The presence of the train increased the overturning effect marginally and the bolts in the windward columns of the piers came into tension. Because they were intended for location purposes rather than to take tension they were not fully anchored into the masonry of the pier. The column bases started to lift off taking two courses of masonry with them (Fig. 7). This had the beneficial effect of reducing the loads in the baseplate bolts but caused a significant increase in the diagonal bracing members causing them to fail. These members first failed not at the bottom level but at a level above that. This triggered a toppling collapse with rotation tending to be at level 1 (Fig. 8). During the process of toppling there was a kickback to windward on the remains of the piers below level 1.

The scenario is in accord with pictorial records and with evidence given at the inquiry. Fig. 7 (where the windward side is to the right of the photograph) shows two courses of masonry having been pulled up by the column bolts. For two of the piers (29 and 31, the first and third navigation piers on the south side respectively) the base level bracing remained in place. The court of inquiry report,⁶ paragraph 12, says

The distance at which the girders were found from the piers, is such as would result from a fracture and separation taking place in the piers somewhere above the base of the columns.

The appendix of the report also adds

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 Nos. 36, 39 and 40.

Without the luxury of an action replay one cannot be certain about the sequence of events. The recent analysis has however provided satisfactory explanations for a number of observations and has added significantly to understanding of the collapse.

The court of inquiry suggested that poor workmanship may have been a cause of collapse. While the piers showed clear evidence of

poor construction practices which would have contributed to the collapse, the results of the analysis indicate that the pier structure could have failed due to the effect of the wind alone. Since the underestimate of wind loading could have been the sole reason for the accident, it is important to inquire how culpable was Bouch in this regard.

Was Bouch to blame?

Most designers of iron railway bridges of moderate height and span before the Tay Bridge did not take account of wind forces. It was generally accepted that providing for dead and live loads imparted sufficient lateral stability so that no special provision for wind forces was necessary. Bouch followed this philosophy for his original design.

At the time Bouch redesigned the piers for the Tay Bridge he was also working on the design of the proposed Forth rail bridge (Fig. 9). Because of the magnitude of the spans (488 m) he consulted some eminent engineers of his day (Hawkshaw, Bidder, Pole, Harrison and Barlow) on the question of wind pressure. These engineers in turn, not being satisfied with their judgement, consulted the Astronomer Royal, Sir George Airy. He reported that despite gusts of up to 40 lbf/ft² (1 lbf/ft² = 0.048 kN/m²) acting over 'very limited areas', an average pressure of 10 lbf/ft² acting over the whole length of the bridge was sufficient. This opinion was endorsed by the engineers, who considered the report highly authoritative.

Bouch also consulted Yolland, the chief inspector of railways, who recommended that there was

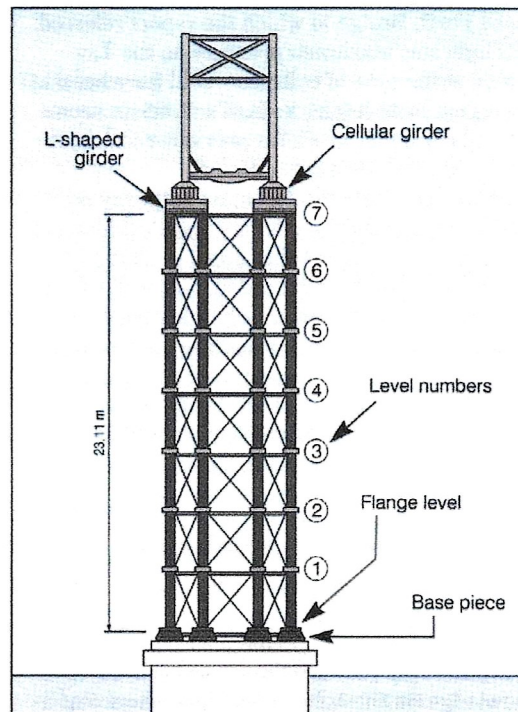
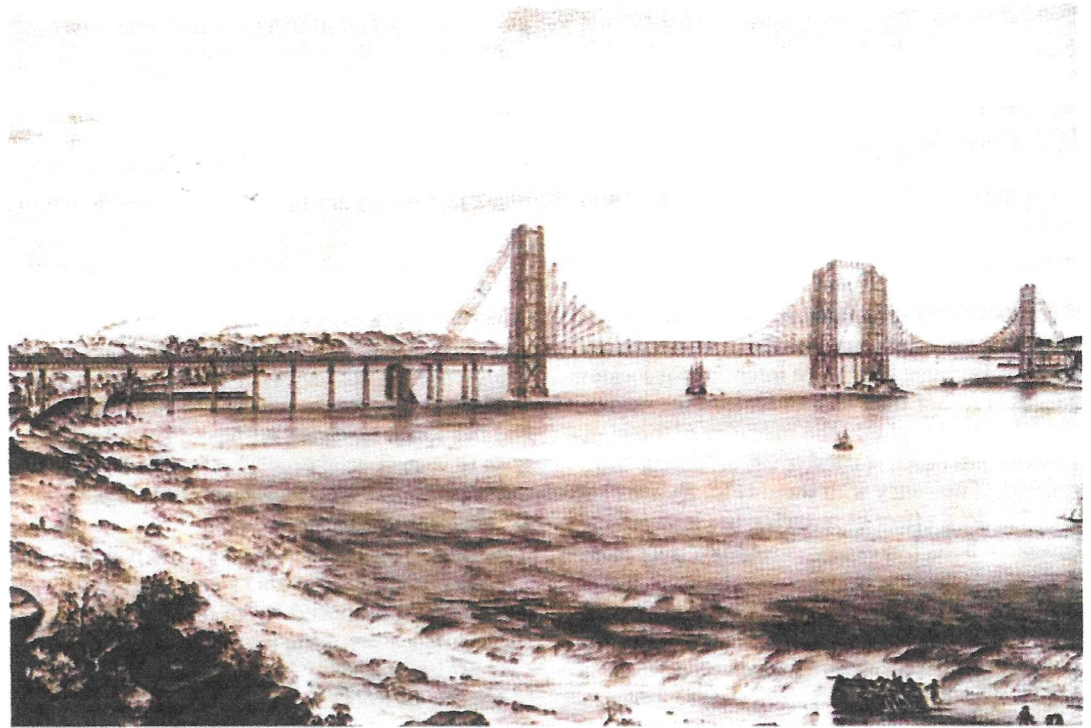


Fig. 8. Elevation of a typical navigation span pier.

Fig. 9. Bouch's design for the Forth rail bridge prior to the Tay collapse



no need to make specific allowance for wind pressure. Although realizing his new pier design would not impart as much lateral stability as brick piers, Bouch accepted this opinion and used Airy's average figure to check the design of the Tay Bridge piers. For this he was criticized by the technical experts — Barlow and Yolland — appointed by the court. They indicated that he should have used the gust pressure (40 lbf/ft^2) as this could have acted over the whole length of one of the Tay Bridge spans (74.7 m), which were significantly shorter than the spans for the proposed Forth Bridge to which the report referred. CP3 indicates maximum pressure on the Tay Bridge at the time of collapse would have been in the region of 46 lbf/ft^2 . Yolland's criticism seems particularly unfair given his previous recommendation on wind pressure to Bouch.

Stokes, probably the foremost authority on fluid mechanics in the country, gave evidence to the court. He reported that gusts of 50 lbf/ft^2 and greater had been recorded. Generally speaking, however, there was a lack of knowledge on the subject within the engineering community. Even an engineer as eminent as Benjamin Baker (appointed co-designer of the Forth rail bridge with Sir John Fowler after Bouch was discredited), told the court of inquiry that 27 lbf/ft^2 was the maximum wind pressure that could be realized over one span of the high girders. There must have been many engineers like Baker who were wise after the event.

After publication of the accident report, the Board of Trade recommended a figure of 56 lbf/ft^2 . The main reason for the lack of knowledge on the subject was that science of

wind effects on structures was in its embryonic stage of development — anemometers at the time were crude and could not measure gust of a short duration such as 3 s — and there was no way of estimating what is now called the drag coefficient.⁷ It is worth noting that the accurate wind load estimation on structures, especially reticulated structures, only became possible with the introduction of wind tunnels at the end of the nineteenth century.⁸

What about other sources? Was there any information available which would have guided Bouch in making a proper estimate of the wind loading? In France and America, engineers used 50 lbf/ft^2 for wind pressure.^{6,8} American engineers had arrived at these figures after bridges had collapsed in gale force winds. Unfortunately the maintenance of such information seems to have been neglected.⁸ Had such information been readily available it is likely the Tay Bridge disaster would not have occurred.

A key insight into the human cause behind the technical cause for the collapse of the bridge can be gained by comparing the design of the Belah Viaduct (Fig. 3) with the Tay Bridge. It is difficult to comprehend that both structures were designed by the same engineer. The court of inquiry in comparing the two structures reported

And the only conclusion to which we can come is, either that the former was extravagantly strong, or the latter inordinately weak.

Bouch was questioned by Barlow (designer of the present Tay rail bridge) why he deviated from the plan he had adopted in the Belah Viaduct for the horizontal ties. His answer that





They were so much more expensive; this was a saving of money

suggests that Bouch may have compromised the strength and stability of the redesigned piers because the project was well over budget and behind schedule. This view is corroborated by the fact that Bouch had intended to use eight columns in rows of two (similar to the Belah Viaduct) instead of six columns in the shape of a hexagon.¹ Most engineers would feel empathy for his predicament. If the funding of the project had been more favourable it is likely that a stronger, more stable pier structure would have been built.

It is interesting to note that Sir John Fowler was always pessimistic about the first Tay rail bridge's stability for he had refused to let his family cross it. Considering the height of the bridge deck, the narrowness of the piers and the exposed site on which the bridge was built, he must have felt that the stability of the piers had been dangerously compromised.

Many lessons were learnt from the disaster: research into wind effects on structures was stimulated; quality control during construction was highlighted as being of primary importance; and the use of steel as a structural material was sanctioned by the Board of Trade. All these lessons, bought at the cost of 75 lives, came to fruition when the Forth rail bridge was opened, quite appropriately in a gale, on 4 March 1890 (Fig. 10). British structural engineering had entered a new era of success.

Conclusion

The findings reported in this paper do not of course provide an unequivocal explanation of the

Tay Bridge collapse. They do however remind us of the potential consequences of design errors.

Bouch, as is normal nowadays, was under pressure to redesign quickly keeping costs to a minimum. He probably listened to the advice he wanted to hear and put aside doubts which he may have had about the need to design for wind loading. He took a risk. If the standards of today were transferred to his time, he would probably not be found to be criminally negligent but it is likely he would be held liable.

Fig. 10. Forth rail bridge as built to the design of Benjamin and Fowler.

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