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The Tay Rail Bridge disaster revisited

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A view is being expressed that the Tay Rail Bridge disaster of 1879 was not fundamentally due to the action of the wind but was a result of fatigue damage caused by dynamic loading. The paper discusses this proposition and shows that the evidence to support it is weak.

1. INTRODUCTION

In a previous paper[1] the results of an investigation into the collapse of the ill-fated Tay Rail Bridge (Fig. 1) based on a three-dimensional analysis of one of the pier structures are described. This indicated that the trigger to the collapse was uplift of the base of the windward column causing an increase in load in the under-strength diagonal ties resulting in their failure and leading to toppling collapse.

Lewis and Reynolds[2] have asserted that the failure of the bridge was not mainly due to wind effects but instead was due to dynamic loading and fatigue. This represents a major change of viewpoint in relation to the fundamental reason for the collapse and merits serious consideration.

In this paper the possible effect of dynamic motion and fatigue on the bridge are examined, demonstrating that there is little evidence to support the view that these were important issues. A new analysis of wind strength records taken near the time and near the location of the collapse provides further evidence to support the 'blown down by the wind' hypothesis.

2. SUMMARY OF FINDINGS FROM THE STUDY BY MARTIN AND MACLEOD[1]

For this study a three-dimensional skeletal frame element model of one of the support piers of the original navigation
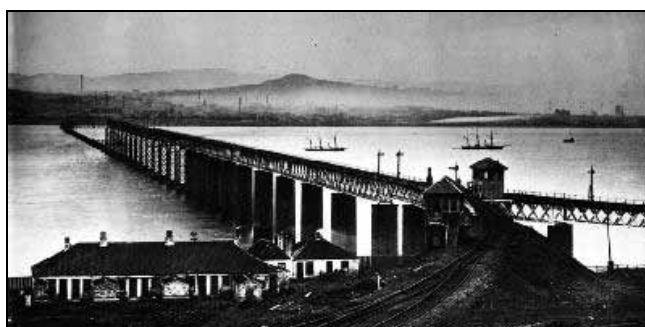

Fig. 1. Tay Rail Bridge after the collapse, looking from the south (courtesy of the University of St. Andrews)

spans was established. The following features of behaviour were included in the model

(a) vertical load of the train
(b) lateral load of the wind
(c) an allowance for uplift of the windward columns.

The analysis indicated the following.

(a) As the columns were not properly anchored to their supports (the wind forces used by Bouch indicated that there would be no uplift) there was a tendency for the columns to lift on the windward side taking two courses of masonry with them. That they did lift in this way is clear from the photographs taken after the event (Fig. 2).
(b) This lifting preceded failure of the bracing ties.
(c) The lifting of the windward columns threw more load into the bracing for the same lateral load.
(d) The maximum bracing load was at the second level of bracing above the base. In some of the collapsed piers the lowest level of bracing survived.
(e) The force of the wind on the train provided a greater overturning effect than the weight of the train in resisting overturning. Therefore the worst condition for overturning was with the train on the bridge.
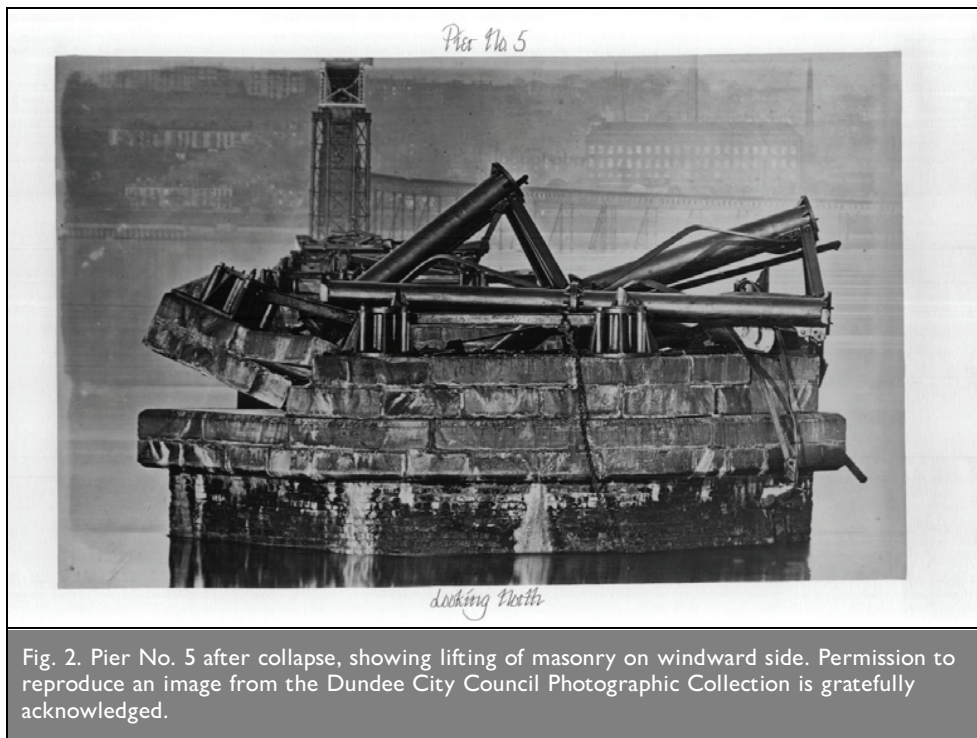
Thus the collapse scenario based on the analysis results was defined.

(a) With the train on the bridge the overturning action of the wind caused the windward columns to begin to lift at a wind speed of about 53 mph (1 mph = 0·45 m/s).
(b) This increased the load in the bracing assembly to failure which would occur at about 66 mph for the lowest measured tie strength or 68 mph for the average of the measured tie strengths. These wind speeds are at the middle and upper levels of force 10 on the Beaufort wind scale which was deemed to be realistic in relation to the reports about the wind made at the time of the collapse.

3. FATIGUE IN CAST IRON

3.1. The cast iron of the original bridge

The type of cast iron used in the columns of the Tay Bridge piers had low ultimate tensile strength (UTS)[3] (i.e. 9 tons/in^2; 1 ton/in^2 = 13·789 N/mm^2) and probably had high phosphorous

Fig. 2. Pier No. 5 after collapse, showing lifting of masonry on windward side. Permission to reproduce an image from the Dundee City Council Photographic Collection is gratefully acknowledged.

the failure load was reduced from between 10 to 15% depending on the rate of loading.

3.3. The contribution of fatigue to the collapse

This information about modern cast iron indicates that structures built in this material are resistant to fatigue failure for stresses up to 47% of ultimate. The main difference between the modern material and that used for the Tay Bridge is the higher phosphorous content of the latter. It is, however, reasonable to infer that the cast iron used on the Tay Bridge had a similar resistance to fatigue to that of the modern low-phosphorous cast iron.

content relative to modern cast irons. As there is no fatigue data for the cast iron produced at the time of the Tay Bridge disaster, information about the behaviour of a modern cast iron with a similar UTS is presented here.

3.2. The fatigue behaviour of a modern cast iron

A modern low-phosphorous cast iron of similar UTS (9 tons/in^2), has a fatigue endurance limit of 47% of the ultimate tensile load for approximately 1 million reversed cycles.[4] That is, if the stress is not higher than 47% of the ultimate stress then there is no likelihood of fatigue failure in the absence of stress concentration factors. Gilbert[5] has reported the following results for modern cast iron.

(a) The endurance limit of weak cast irons can be increased to a remarkable degree by previous under-stressing at or just below the endurance limit.
(b) Weak cast irons show a remarkable ability to absorb overstress in the notched and un-notched condition.
(c) Cast irons in general have low notch sensitivities in fatigue.
(d) In some cast irons, notching has no effect on the endurance limit.

Fallon (Casting Technology International, personal communication) reported that, in a career spanning 30 years, he had examined hundreds of grey cast iron failures due to cracking. With the exception of components rotating at high speed (e.g. crank shafts) he can only recall one where fatigue was the proven cause of failure. All of the others had failed catastrophically as a result of overstressing. The component (UTS 10 tons/in^2) that did fail in fatigue was in a printing press (1862) that was subject to major stress reversals throughout its 130 years of life. He also intimated that tensile tests on cast iron under fast loading conditions had been carried out by Casting Technology International. They found that

4. TRAIN-INDUCED DYNAMIC EFFECT

4.1. The argument in favour of dynamic failure

Lewis and Reynolds[2] assert that lateral oscillations, induced by trains passing over a slight misalignment in the track, resulted in dynamic forces sufficient to fatigue the cast iron lugs attaching the bracing to the columns. The evidence for the dynamic effects is based on eye witness reports from painters and fitters that the high girders piers oscillated from side to side whenever a train crossed the bridge. Lewis and Reynolds, on the close inspection of digitally enhanced photographs used at the Court of Enquiry,[3] claim that

(a) crack arrest lines in the failed parts (pier 12) are evident;
(b) the failure of the cast iron lugs was due to fatigue rather than overstressing

4.2. Possible dynamic behaviour of the bridge

It is not possible to calculate the force in the cast iron lugs due to the dynamic action from the train but an estimate of the number of cycles the cast iron lugs sustained can be made. The estimate was based on the following data.

(a) Working life of the bridge: 2 years 4 months.
(b) Natural frequency of a pier (from the element model results): 0·2 Hz.
(c) Average length of a train: 248 ft (1 ft = 0·305 m).
(d) Number of trains per day: 40 (likely to be conservative).
(e) Speed of train: 25 mph (recommended speed for the bridge).

The result of this analysis is a total of 74 000 cycles of vibration. The loading on the ties would be uni-directional (tensile) in contrast to the reverse cyclic loading normally associated with fatigue testing and failure. This is because the ties buckle under low compressive load (i.e. Euler buckling load) and therefore the lugs will not sustain the same amount
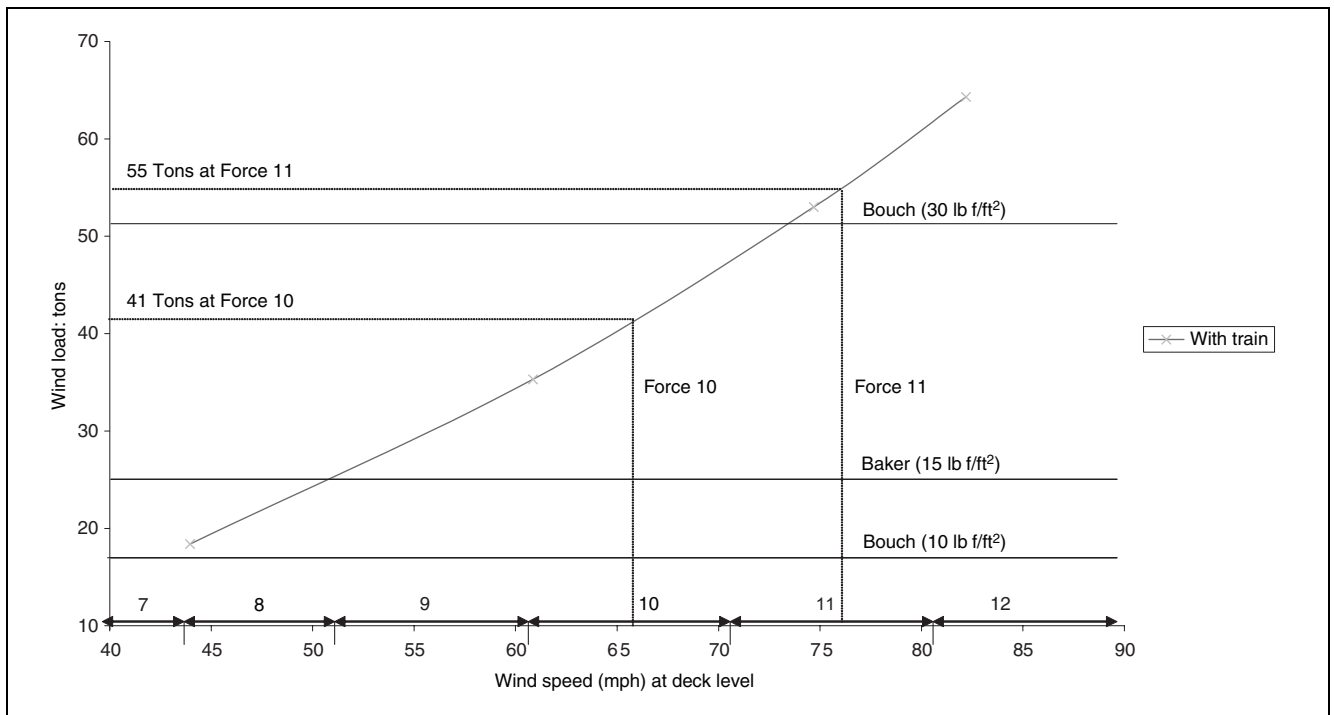
Fig. 3. Wind load in relation to wind speed (1 mph = 0·45 m/s; 1 ton force = 9·97 kN; 1 lb f/ft² = 47·880 N/m²)

of fatigue damage as would be the case with reverse cyclic loading.

4.3. The likely contribution of dynamic behaviour to the collapse

Assuming that the cast iron used for the Tay Bridge has similar fatigue properties to that of a modern equivalent, the estimated number of non-reversed cycles due to the dynamic action of the trains, experienced by the bridge in its lifetime (74 000), is low compared with the 10^6 reverse cycles taken by the modern material. Fatigue failure from this source therefore requires the unlikely situation that the stresses in the lugs were close to ultimate when the trains passed over the bridge.

5. ESTIMATES OF WIND FORCE ON THE BRIDGE

Lewis and Reynolds[2] state that the maximum wind pressure was no greater than 15 lb f/ft² (1 lb f/ft² = 47·880 N/m²) on the night of the disaster (based on Benjamin Baker's evidence to the Court of Inquiry[3]); this not being sufficient to cause the collapse of the bridge.

A wind pressure of 15 lb f/ft² corresponds to a Beaufort force 8 whereas the estimated Beaufort value was force 10–11[2,3,6–9,12] (Fig. 3). The upper end of force 10 corresponds to a pressure on the bridge of 28 lb f/ft² and the upper end of force 11 to a pressure of 36 lb f/ft².

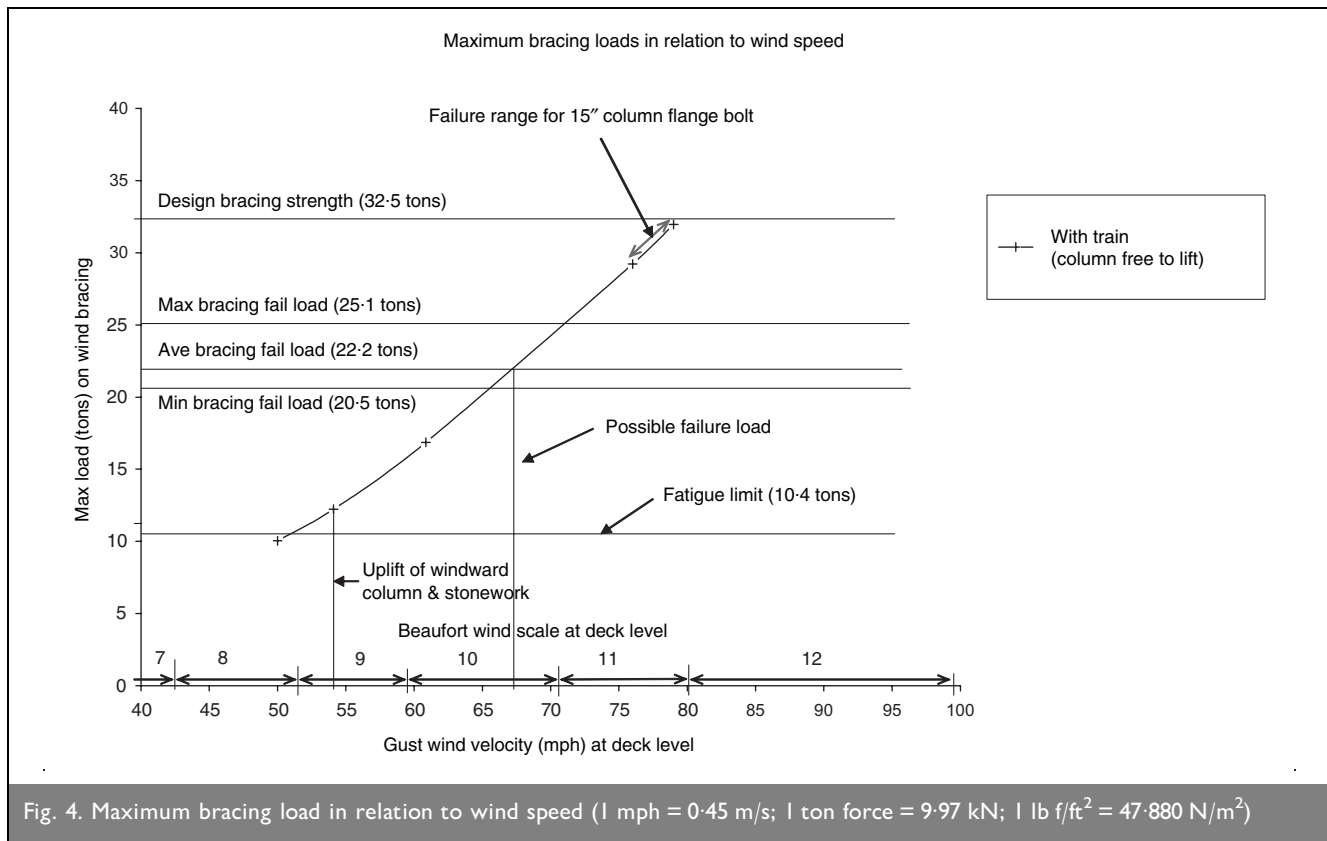In order to glean further evidence of the strength of the wind at the time of the disaster, the authors carried out a search of the records in the Meteorological Office in Edinburgh. During the lifetime of the bridge observations of wind strength were recorded at 0900 and 2100 h daily at an observation position near Dundee approximately 3 miles NE from the bridge and at an altitude of 164 ft. Table 1 gives the data for the lifetime of bridge. The Meteorological Office recordings,[10] based on a scale of 0–6, were converted to Beaufort wind force as shown in Table 1, which also shows the corresponding pressure in lb f/ft² based on the CP3 code.[6]

From Table 1 the following should be noted.

(a) The single observation of wind strength above Beaufort 9 occurred 1·75 h after the collapse of the bridge. This was the only measurement of wind strength greater than Beaufort 9 during the lifetime of the bridge. It is, of course, possible that there were force 10 or greater winds between the observation times.

(b) Over 88% of the wind loading on the bridge during its

Wind strength: Met. Office	Wind strength: Beaufort	Wind pressure[6]: lb f/ft²	No. of readings	Percentage of total	Percentage of total
0	0	0	1100	64·9	
1	1	0·06	322	19·0	
2	5	8	80	4·7	
					88·6
3	6	14	85	5·0	
4	7	22	56	3·3	
5	8–9	34–47	52	3·1	
6	10–12	47–84	1	0·1	
					11·4

Table 1. Wind force distribution for the lifetime of the bridge (1 lb f/ft² = 47·880 N/m²)

Fig. 4. Maximum bracing load in relation to wind speed (1 mph = 0·45 m/s; 1 ton force = 9·97 kN; 1 lb f/ft^2 = 47·880 N/m^2)

lifetime was no greater than force 5 on the Beaufort scale (i.e. fresh breeze). The bridge was not constantly buffeted by strong winds.

(c) Beaufort values and the Meteorological Office values for wind are based on sea states. They correlate more closely with velocity than wind force although the Beaufort values are normally referred to as 'force 8', for example. Since wind force is proportional to the square of the velocity, the increase from Beaufort 9 to 10 represents a significant increase in the wind force.

Although the data do not give direct evidence of the strength of the wind at the time and place of the disaster, they do show that the wind conditions near the bridge and near the time were exceptionally high thus supporting the hearsay evidence given at the Court of Enquiry that the wind strength at the time of the collapse was greater than Beaufort force 9.

There is therefore strong evidence to support the conclusion that the wind pressure on the bridge at the time of the collapse was significantly greater than 15 lb f/ft^2.

6. WIND-INDUCED CUMULATIVE FATIGUE DAMAGE PREDICTION

In an attempt to allow for reduced fatigue life, due to casting defects, conical bolt holes and the consequent stress concentration in the lugs, the fatigue limit has been conservatively taken as 47% of the average fail load (22·2 tons; the test result from Court of Inquiry) of the bracing assembly tested (which included the lugs) and not the design strength of the lugs (59 tons). The design strength was calculated by the authors as the load that can be taken by the two lugs (with parallel rather than conical bolt holes) across the critical cross-section for lug failure (see Fig. 6) using a failure stress for cast iron of

9 tons/in^2. Fig. 4 shows that the wind strength required to reach the fatigue limit load in the tie is about Beaufort force 8/9. Therefore it appears that the wind caused loading in the ties slightly above the fatigue limit on approximately 10 out of 1696 12-h periods (i.e. 0·6% of the total time) and only significantly exceeded the fatigue limit load on the night of the disaster.

An estimate of the cumulative damage due to turbulence of the wind was carried out based on the following assumptions.

(a) The peak wind velocity above the mean is due to a 15 s gust.
(b) The distribution of differences from the mean wind velocity is normal (i.e. Gaussian).
(c) Goodman's Rule[11] was used to compensate for the mean wind load by reducing the stress range for the number of cycles to failure at zero mean load.
(d) Miner's Rule[11] was used to identify the cumulative fatigue damage.

This analysis gave a prediction of only 7% fatigue damage during the lifetime of the bridge indicating a low level of probability that failure of the lugs was due to fatigue caused by the turbulence of the wind.

7. THE STRENGTH OF THE TIE BAR ASSEMBLY

7.1. The failure of the cast iron lugs

Observations from the wreckage[3] indicated that failure of the tie bar assembly occurred at the cast iron lugs in all cases. Fig. 5 shows the lug details.[3] An estimate of the strength of the lugs is 59 tons (1 ton = 4·44 kN) based on an ultimate stress of 9·0 ton/in^2 in the cast iron. This compares with the design strength of
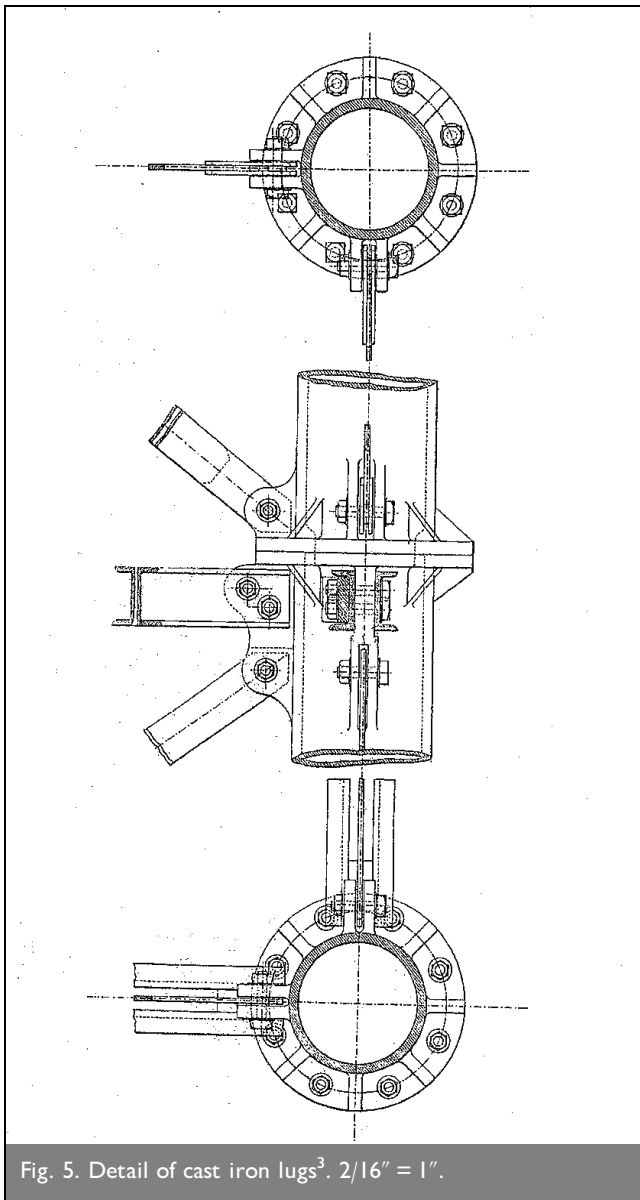
Fig. 5. Detail of cast iron lugs[3]. 2/16″ = 1″.

executed. The holes in the lugs were as cast and caused the bearing surface to be conical. This would cause the load on the lug to be eccentric resulting in a magnification of the axial stress in the lug due to bending. Fig. 6 shows the bearing of a pin on a lug with a conical bearing support resulting in an eccentricity (*e*) relative to the central plane of the lug plate.

The bending stress in the lug at the critical section would be

$$\boldsymbol{\sigma} = P/A + M/Z$$
$$= P/bc + Pe/(bc^2/6)$$
$$= \frac{P}{A}(1 + 6e/c)$$

where *A* is the cross-sectional area, *Z* is the elastic bending modulus, *b* is the width and *c* is the depth of the lug at the critical section. The factor $(1 + 6e/c)$ is a magnification on the uniform axial stress due to the eccentricity.

The actual magnification is $59/22 \cdot 2 = 2 \cdot 67 = 1 + 6e/c$. This corresponds to an *e*/*c* ratio of 0·28. The diameters of the holes in the lugs as cast varied from 1·125 in at the outside face of the lug to 1·25 in at the inside face of the lug. The taper was therefore significant and it is not difficult to imagine that an *e*/*c* ratio of 0·28 would have been achieved. The reduction in the measured strength of the lugs against the design strength may therefore be explained by the non-parallel bearing surfaces for the pins. In both the tests and in the examination of the wreckage there was no evidence of failure of the bolts which attached the sling plates to the cast-iron lugs although the bolts recovered from the wreckage were distorted. The conical seating is likely to have been a major cause of this distortion.

7.2. The strength of the tie bars
The Kirkaldy tests included a set where the lugs were not present and where the failure was at the bolt connection in the

32·5 tons based on the strength of the ties themselves[3] (Fig. 4) and the average measured strength of five tie bar assemblies (which included the lugs) quoted at the Court of Enquiry[3] of 22·2 tons. These test specimens all failed at the lugs. The testing was carried out by David Kirkaldy at his laboratory in Southwark, London. The loads were measured using a 300-ton capacity hydraulic tensometer with the loads applied slowly. Kirkaldy's testing service was considered to be one of the best available at the time and the testing work for the Tay Bridge tie assemblies appears to have been competently
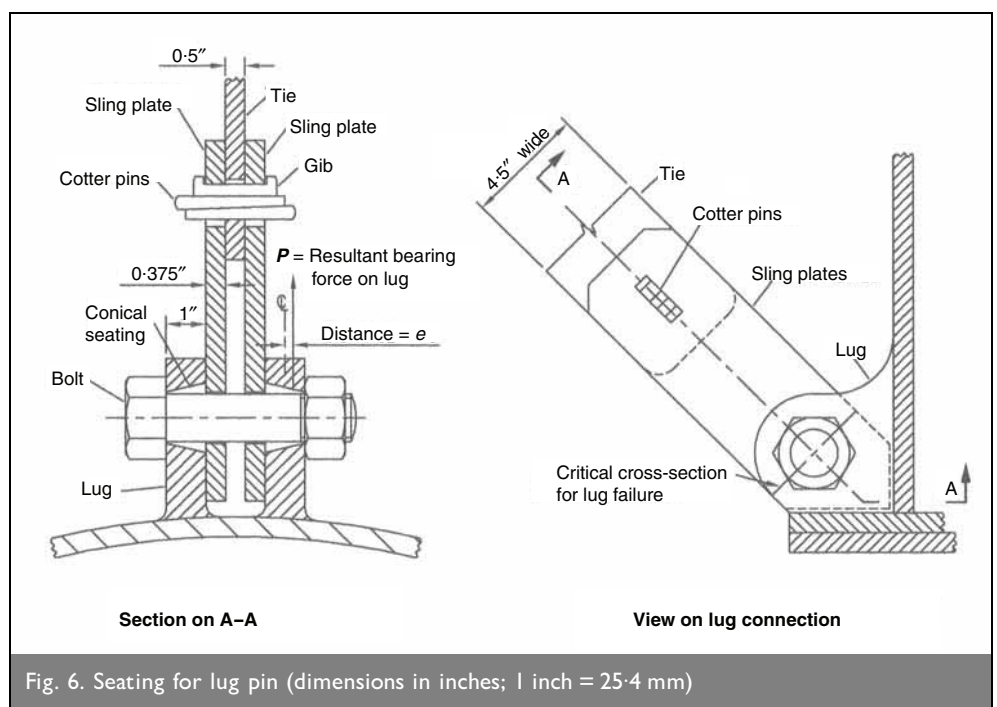

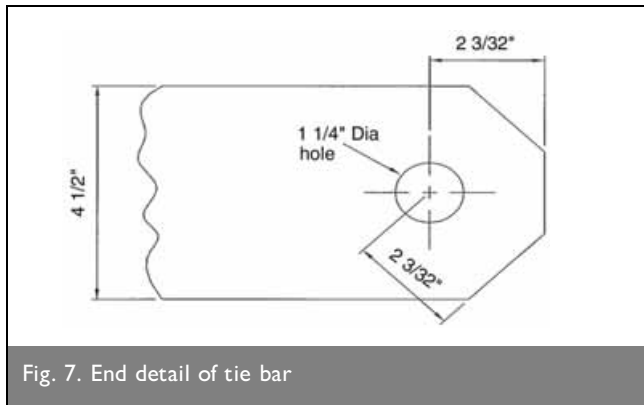Fig. 6. Seating for lug pin (dimensions in inches; 1 inch = 25·4 mm)

Fig. 7. End detail of tie bar

Dimensions shown on figure: 2 3/32", 1 1/4" Dia hole, 4 1/2", 2 3/32"

tie bar itself. Fig. 7 shows tie bar dimensions at the bolted end. The edge distances are within current code limits but the stress concentration due to the presence of the hole resulted in an average tie strength of 25·6 tons as compared with the design strength of 32·5 tons and the average lug strength of 22·2 tons.

The tie assembly was therefore doubly vulnerable to failure.

8. CONCLUSION

The reasons for the collapse of the Tay Bridge in 1879 have again come under scrutiny. A new suggestion[2] that the collapse was due to fatigue failure as a result of dynamic effects is shown in this current paper to be weakly supported by evidence as compared with the evidence for failure due to wind loading.

The design faults made the bridge unsafe from two directions.

(a) The applied wind load was very much greater than the design wind load.
(b) The actual strength of the ties was significantly less than the design strength of the ties, both with respect to lug failure and to the failure of the tie itself.

It seems to the authors that the evidence for the prime cause of the disaster being due to under-design for wind loading is overwhelming.

REFERENCES

1. MARTIN T. and MacLEOD I. A. The Tay Bridge Disaster—A reappraisal based on modern analysis methods. *Proceedings of the Institution of Civil Engineers*, 1995, **108**, No. 2, 77–83.
2. LEWIS P. R. and REYNOLDS K. (2002) Forensic engineering: a reappraisal of the Tay Bridge disaster. *Interdisciplinary Science Reviews* 2002, **27**, No. 4, 287–298.
3. COURT OF INQUIRY. *Report upon the Circumstances attending the Fall of a Portion of the Tay Bridge.* HMSO, London, 1880.
4. JONES G. N. J. *Engineering Materials.* British Cast Iron Association, 1953, Alvechurch, Research Report No 370.
5. GILBERT G. N. J. *Fatigue Properties of Cast Iron.* British Cast Iron Association, Alvechurch, 1953, Research Report No 370.
6. BRITISH STANDARDS INSTITUTION. *CP3, Code of Basic Data for the Design of Buildings.* BSI, London, 1972, Chapter 5, Part 2, Wind Loading.
7. MARTIN T. J. and MacLEOD I. A. *The Tay Bridge Disaster—a Study in Structural Pathology, Developments in Structural Engineering,* Forth Rail Bridge Centenary Conference. Spon, London, 1990.
8. PREBBLE J. *The High Girders.* Penguin, London, 1979.
9. THOMAS J. *The Tay Bridge Disaster.* David and Charles, Newton Abbot, 1970.
10. METEOROLOGICAL OFFICE. *Weather Records for Dundee (Eastern Necropolis), (1877–1879).* Meteorological Office, Saughton House, Edinburgh.
11. ASHBY M. F. and JONES D. R. H. *Engineering Materials 1.* Butterworth-Heinemann, Oxford, 1980.
12. SWINFEN D. *The Fall of the Tay Bridge.* Mercat Press, Edinburgh, 1979.

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