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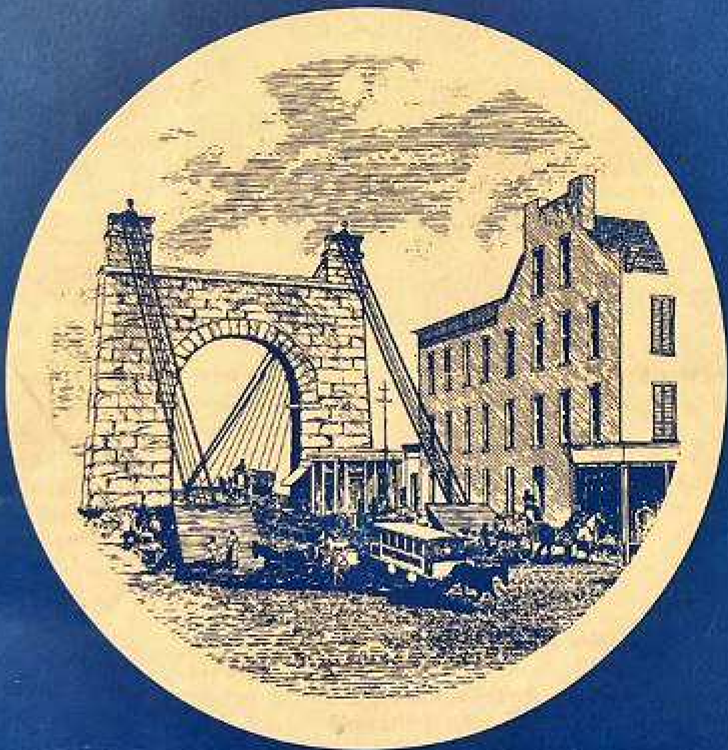


McLure House Hotel & Conference Center
Wheeling, West Virginia, USA

The Early Development of the Long Span Suspension Bridge In Britain, 1810-1840

By Roland A. Paxton

This paper commences with a description from little-known sources and comments on one of the earliest essays in the application of wire to a suspension bridge. This relates to the proposal of Thomas Telford [1757-1834] for a bridge at Runcorn, England in 1814 and the creation and load-testing of an iron wire model bridge 50 ft. [15.2 m.] in length. In addition to its intrinsic interest this model bridge precedes what are believed to be the earliest wire bridges elsewhere, namely, White & Hazard's short-lived wire footbridge at Philadelphia, U.S.A. [1816],[1,2] and footbridges at Galashiels, Scotland [1816],[3] and Annonay, France [1822].[4]

This paper then outlines the principal developments in suspension bridge practice relating to main chains, anchorage, towers, saddles, deck stiffening and theoretical calculations made by engineers to determine maximum chain strain from 1817 to 1830. It also includes some previously unpublished information on another key figure of this period, Capt. Samuel Brown [1774-1852], and details of an unusual tower feature at Gattonside Bridge, Scotland, erected by Redpath, Brown & Co.

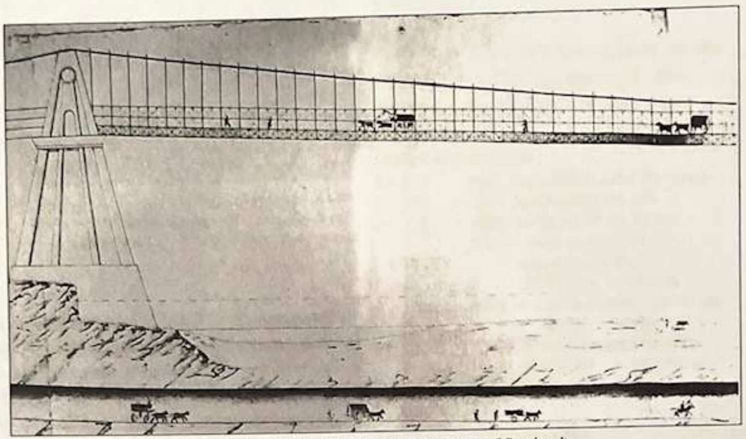


Fig. 1. Runcorn bridge or tunnel project 1814[18] (National Library of Scotland)

Introduction

The most important developments in the evolution of the long span suspension bridge following the pioneer work of James Finley in North America,[4,5,6] as exemplified in Merrimack Bridge [1810], occurred in Britain during the next two decades. The principal projects dictating these developments were: the 1814-18 Runcorn Bridge proposals of Telford and Capt. Brown; Union Bridge, near Berwick-upon-Tweed [Capt. Brown 1819-20]; Menai Bridge [Telford 1819-26]; Hammersmith Bridge [Tierney Clark 1824-27]; Montrose Bridge [1823-40]; and, to a lesser extent, the Clifton Bridge project [1829-64]. All these projects involved bridges with spans of 420 ft. [128 m.] or more, designed to accommodate carriage and pedestrian traffic. For a detailed description and assessment of the work of Brown and Telford. More generally, the reader is referred to papers by Drs. Day,[7,8,9,10] Kemp,[11] Paxton,[12,13,14] Smith,[15] Peters,[4] and Kranakis.[5]

Telford's Wire Bridge Proposals of 1814

From c. 1810 Telford, as Parliament's engineer, was actively progressing the improvement of travel times on arterial roads in Britain by means of road re-alignment, improved construction and gradients and the creation of structures of unprecedented scale. On the London to Holyhead Road he proposed to make the hazardous crossing of the Menai Strait by means of what would then have been the world's largest cast-iron arch with a span of 500 ft. [152 m.]. In order to support the ribs while they were being joined, he made the ingenious proposal of laying them on timber centering supported from above by iron stays radiating from temporary side towers. This concept, basically a stayed bridge, was publicised in 1811[16] and 1813.[17] Although not progressed at the time it was soon developed by others, for example Redpath & Brown in a wire bridge at Peebles, Scotland,[5] and by the turn of the 19[th] century for temporarily supporting the ribs of large steel bridges. In 1814, however, for his proposed suspension bridges at Latchford [200 ft., or 61 m. span] and Runcorn [500-1000-500 ft., or 152-305-152 m. spans] (fig. 1),[18] on the Liverpool-Newcastle-under-Lyme-London road, Telford prepared what were probably the first multi-wire cable suspension bridge designs.[19]

A wrought-iron suspension bridge of the magnitude proposed at Runcorn was then quite unprecedented in terms of design, construction and technology, and it was necessary for the projectors to demonstrate its practicability. In referring to its main cables Telford stated that the "metal should be kept as far as practicable in straight lines and also have few joinings".[20] These conditions ruled out the use of conventional chain. Telford, with his young assistant, William Provis [1792-1870], who 14 years later was to publish the definitive account of Menai Bridge[21] and later extended the life of its deck following storm damage, embarked on a series of over 200 experiments into the strength of iron wire and bars.[20] These experiments included many with the iron deployed to the curvatures with a dip of 1/20[th] and 1/50[th] span then envisaged for the bridge, from which the asterisked factors in the *'power of suspension'* calculation below were derived. Telford's findings exercised a major influence on bridge designers for several decades through their widespread dissemination in the publications of Barlow,[22] Navier,[23] Provis[21] and others from 1817.

Checking the results of one of the experiments on 1/10[th] in. [2.5 mm.] diameter iron wire (fig. 2).[24] by modern calculation, the tension at the point of maximum deflection, assuming a parabolic curve and a deflection of 19 ft. 8 in. in 900 ft. [6 m. in 274 m.], gives a theoretical

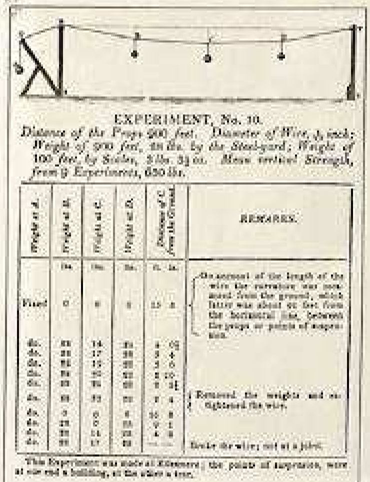


Fig. 2. Runcorn Bridge Project, 1814-strength testing with loaded wire[24] (P. Barlow)

strain of 609 lb.[21] This figure correlates quite closely with Telford's experimentally determined breaking strain of 630 lb. [286 kg.],[25] considering that the curvature of each was not identical because of differences in the load distribution. The determination of design details by experimental rather than theoretical procedures may seem time consuming and unnecessary now, but when considered against the inadequate state of "strength of materials" practice in 1814, this procedure undoubtedly represented the most reliable way of achieving the desired objective.

The main suspending element of Telford's Runcorn Bridge proposal of 1814 consisted of 42 cables comprising 35,128 continuous near parallel strands of 1/10[th] in. [2.5 mm.] diameter iron wire 2010 ft. [613 m.] long weighing 65 lb. [29.5 kg.]. Each of 16 main cables [there were to have been four one above the other in four lines] was to have been of about 4 in. [102 mm.] in diameter and to have consisted of 1256 wires.[19,5] The dip of the main or upper cables was to have been 50 ft. [15.2 m.]. The remaining 26 cables under and adjoining the roadway, including four diagonal cables at an angle of about 4 degrees to the line of the roadway to inhibit deck movement, were to have had a dip of 20 ft. [6.1 m.]. Telford rightly believed that these diagonal cables and the shallow cable curvature and weight would add stiffness and stability to the structure longitudinally, but neither he nor anyone else seems then to have had any real idea of the magnitude of oscillation-induced forces. With hindsight, it is unlikely that these measures by themselves would have been sufficiently effective in storm conditions.

Telford and Provis's calculations for the 1814 Runcorn Bridge proposal, estimated to cost nearly £150,000, were:

'Power of Suspension
8 x 754 = 6032 large cables under Roadway
6 x 500 = 3000 intermediate do. do.
8 x 500 = 4000 at the top of the 15 feet
13032 wires each capable of suspending 60lbs* = 781920 lbs or 349 ton
4 x 500 = 2000 in diagonal braces do. do. do. = 120000 53 tons
16 x 1256 = 20096 in main suspension cables at 228 lbs* = 4571888 or 2041 tons
Total power will suspend = 2443 tons

Weight to be Suspended
Roadway 1970 x 24 x 74 = 3496880 in two driving ways
1970 x 6 x 35 = 413700 [central path]
3890580 [sic] = *736 tons*
1970 x 30 = 59100 supl. feet of tin, 100 sheets of which weigh 189 lbs & cover 141 supl. Feet,
therefore 59100/141 x 189 = 9191lbs tin for covering the framing of the roadway = 35 do.
Iron in and above the roadway &c. 4543
2925
5200
12668 lbs of wire as per the other side
708814 wrot iron as per do.
108600 cast iron as per do.
830082 = 370 do.
2141 tons
which divided by two for the large arch = 1070 tons
Waggons &c. Supposing 10 waggons upon the large archway at the same time each equal to 10 tons = 100 do.
Coal ashes or slag on top of roadway, say = 30 do.
In all *= 1200 tons'*[19]

The above calculations appear to be in Provis's hand. Underneath Telford has written:

> In all the above calculations all the suspending wires are taken as suspending 600 lbs perpendicularly [34.1 tons in^2 - 527 Nmm^2] whereas they bore when fairly tried 736 & 770 Average 750 lbs [42.6 tons in^2- 658 Nmm^2] therefore say safely 700 lbs but this affects only the cables carried 1/50th [span].[19]

This design was too highly stressed to provide what Telford himself considered an adequate margin within a decade, that was, a maximum working design stress of about 1/5th of the ultimate. Nevertheless, the Runcorn Bridge project with its experiments and model test can be considered as part of a continually evolving design process that culminated in the successful completion of Menai Bridge eleven years later.

Fig. 3. Telford's wire bridge model 1814, as stored c. 1906[27] (British Waterways)

Telford's Wire Bridge Model of 1814

Telford's 1/20th scale wire model of the central span for Runcorn Bridge was 50 ft. [15.2 m.] long. It was erected and tested with a load of 3,035 lb. [1.377 tons] in June/July 1814 [probably at Weston Point near Runcorn, or at Ellesmere].[26] Fortunately for students of this subject a photograph of the model as it had been preserved in a Shropshire Union Canal building at Ellesmere at the beginning of this century has survived (fig. 3),[27] with a drawing which is probably contemporary with the photograph (fig. 4).[28] From the photograph and plan, even though the cables and deck of the model are not to their correct profile, it is possible to add to the account previously published by Dr. Smith.[15]

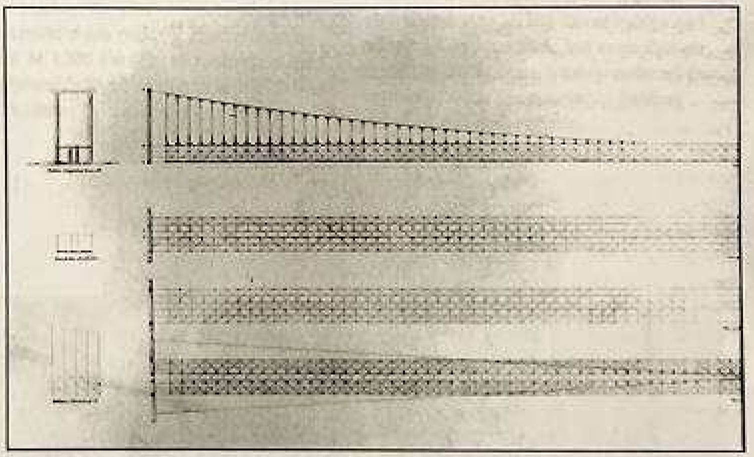


Fig. 4. Drawing of Telford's wire bridge model 1814 c. 1906[28] (British Waterways)

The cross-section of the model was similar to that of the full-scale proposal, except that in the model the 16 main cables were represented by 6 wires and the 8 cables under the roadway by 10 wires (fig.5). The model drawing shows the wires adjoining the carriageway as horizontal and not curved to a dip of 1/50th span as intended in the full-scale bridge.

This feature seems to have been overlooked when fig. 4 was prepared. Telford's notebook indicates that the test load was applied incrementally at quarter points of the span, more or less uniformly. Great attention was paid to achieving satisfactory joints in the model. An entry in the notebook reads:

Wire Joints

No.1. Welded did break at welded part, but where the healed part joined the old part.
2. Joined with soft solder just bore 742 [lb] and broke.
i. Laid about 4 inch overlap and tinned at lower part
ii. Wrapped with brass wire and tinned what was broke.
iii. Covered with soft solder.
3. Welded and Brazed and wrapt with wire broke soon as No. 1.
4. Overlapped wrapped with iron wire and brazed-bore a considerablepull, broke between the heal and old as before-more at the joint.[26]

As a result of their experiments Telford and Provis considered the model to be 1/200th part of the strength of the full-scale structure.[26] The author has tried to investigate this factor, but it is not possible to do this accurately without knowing the cross-sectional area of the wires in the model. One possibility is that Telford and Provis discounted the relatively small suspending power of the lower cables [about 16% in addition to supporting their own weight] as an additional safety factor, considering the bridge to be supported solely by the 20,096 wires in the main cables. If so, dividing 20,096 by 1200 gives for the model say 16 wires of 1/10th in. [2.5 mm.] diameter. But, in the model, for the six main wires to have had the same strength as 16 wires of 1/10th in. [2.5mm.] diameter, they would have had to be about 0.24 in. [6 mm.] diameter. In the photograph the (upper) main wires do appear more than twice as thick as those by the roadway (fig. 6). On the assumption that Telford's strength test results and scaling down factor are of the correct order, the wires in the model would have supported conservatively about 16

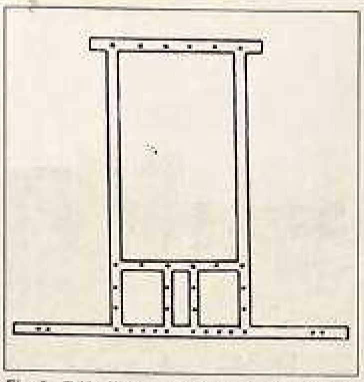


Fig. 5. Telford's wire bridge model 1814–sketch of cross-section at tower (Roland Paxton)

x 228 lb. = 3648 lb. [1655 kg.] plus their own weight. This figure is compatible with the model test weight of 3,035 lb. [1377 kg.] and Provis's statement that the model "would have carried considerably more without being injured".[29] The test load on the model was believed to represent more than the total estimated load of 1200 tons [1219 tonnes] on the proposed bridge, an outcome which was considered by the projectors to demonstrate the basic practicability of the project, but the necessary subscriptions for its execution were not forthcoming.

The Bar Link Main Chain

By March 1817 after "further consideration and discussion,"[20] Telford had abandoned the 1/10th in. [2.5 mm.] diameter wire cables in favor of cables made up of wrought-iron square section butt-welded bars and segmental facing pieces. He does not give a specific reason for this change, but referred generally the need for "effectually protecting the iron from the action of the atmosphere."[20] A major factor would undoubtedly have been what he believed to be the greater expense of the wire cables, that is £58 per ton as against £31.3 per ton for the composite bar cables,[20] for a strength advantage in the proportion of about 37 to 27. Telford also abandoned all the lower cables and adopted a lighter deck, but even though these measures considerably reduced the estimated cost, the necessary funding was still not forthcoming.

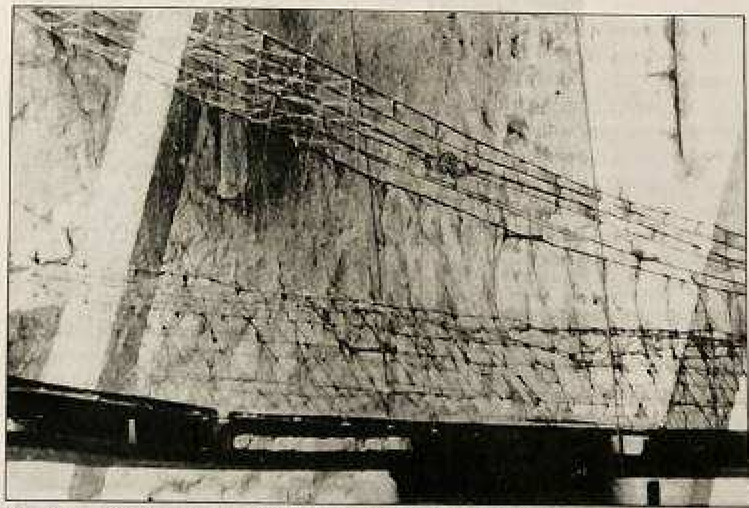


Fig. 6. Telford's wire bridge model 1814–close-up of part of Fig. 3 (British Waterways)

About four years later the bar cables, which Telford at first proposed to use for Menai Bridge, were abandoned in favour of the more practicable and economical long bar chains. These were essentially the bar chains patented by Brown in 1817 with Runcorn Bridge in mind and proved in use at Union Bridge in 1820,[9] but Telford and Provis improved on his practice of deploying chains in separate lines (fig. 7), by forming chains consisting of five rectangular cross-section bars cross-bolted in parallel. Also, the eyes of the bars were bored out rather than being forged which facilitated assembly. The manufacture, quality control and testing of more than 35,000 links for Menai Bridge, all to the highest standards and at the frontiers of technology, exercised a fundamental influence on the art of suspension bridge building, particularly in Britain. The parallel bar arrangement was readily adaptable to longer and larger cross-sectional area links and was almost invariably adopted afterwards by British engineers for large spans, for example, Clark, I.K. Brunel and Rendel. Brown himself adopted it at Montrose Bridge [1828], Wellington Bridge, Aberdeen [1830] (fig. 8), and for the Clifton bridge competition project [1829-30]. The larger cross-sections enabled the number of chains to be reduced and the appearance and strength of bridges was improved by carrying their chain curvature down to the level of the roadway at mid-span.

Brown's Clifton Bridge proposal, had it been built, would arguably have been his finest achievement (fig. 9).[30] It was drawn up by Charles Drewry who four years later produced the first text book devoted to suspension bridges.[31] Brown's experience of erecting suspension bridges was much greater than any of the other finalists in 1830. His design had a strength efficient dip of about 1/11th span and was estimated to cost "not exceeding £30,000" [rounded down from £30,600 [32]]. It was similar in elevation to Brunel's accepted design but was less strong, with a maximum working stress of 8.33 tons in^2 [128 Nmm^2], considerably more than the 5½ tons in^2 [85 Nmm^2] set by Telford and adopted by the adjudicating commit-

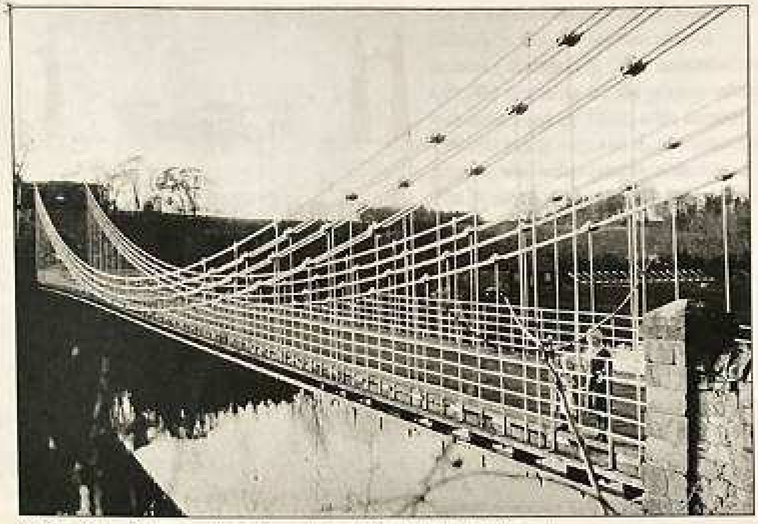


Fig. 7. Union Bridge near Berwick-upon-Tweed, Scotland (Roland Paxton)

Fig. 8. Wellington Bridge, Aberdeen, during refurbishment, 1987 (Dr.Tom Day)

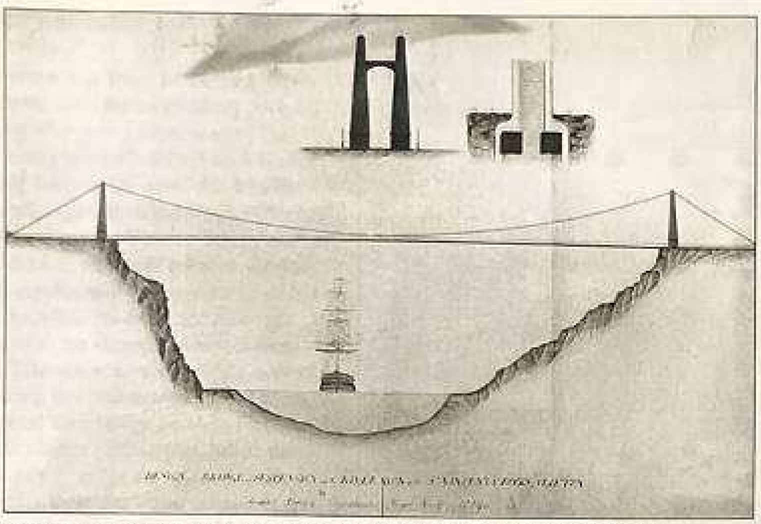


Fig. 9. Captain Brown's design for Clifton Bridge, Bristol, 1829-30[31] (Private Collection)

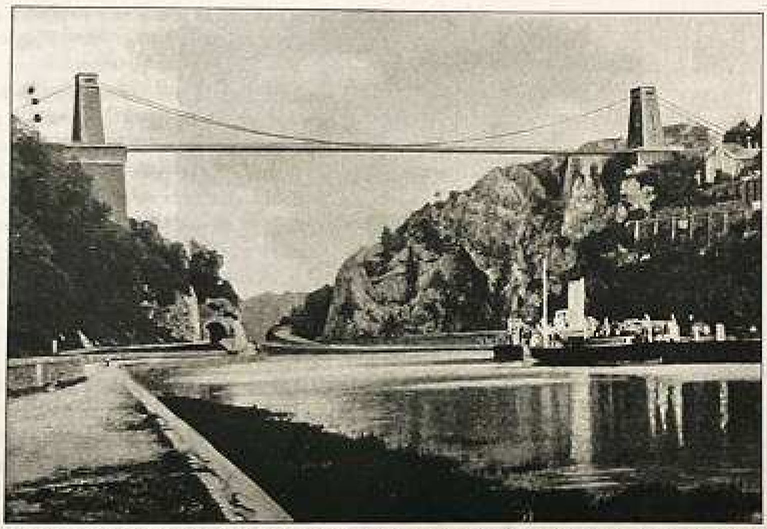


Fig. 10. Clifton Bridge-Brunel's design as completed by Hankshaw and Barlow in 1864[33] (W. Humber)

tee, as against Brunel's 4.20 tons in^2 [65 Nmm^2].[32] The implementation of Brunel's design (fig. 10)[33] exceeded the £45,000 available for the project which was exhausted by 1843, with another £30,000 being required to complete the project, was subsequently abandoned.[34] The bridge was completed from 1860-64, after Brunel's death, under the direction of John Hawkshaw and W.H. Barlow as a tribute to him.

British engineers considered wrought-iron bar links to be more dependable than wire cables for the main suspending members of suspension bridges. Their view was reflected in Sir John Rennie's comment that whilst Fribourg Bridge was "economical in the first cost, it requires constant attention, and it scarcely possesses sufficient durability for permanent structures."[35] In 1850 this preference was reinforced by the failure of a wire cable suspension bridge with considerable loss of life at Angers, France, where corrosion turned out to be a factor.[36] Cross-bolted rectangular links were still being used in the anchorages of Brooklyn Bridge in 1876 [1,520 no., 12 ft. x 3 in. x 8 in. [3.66 m. x 7.7 cm. x 20.4 cm.] eyebars][37] and in 1887 for the whole of the main chains in the reconstruction of Hammersmith Bridge. Chains of this type however had their limitations in terms of cost and appearance as spans approached 1,000 ft. [305 m.], and at about the mid-century the impetus in the progressive development of the suspension bridge returned to the United States.

Deck Stiffening

Stevenson was amongst the first to publicise the need for longitudinal deck stiffening after being influenced by the "vibrating motion" of Dryburgh Bridge during a storm. He wrote that a more powerful agent than dead-weight "exists in the sudden impulses or jerking motion of the load...effects of gusts of wind... [and emphasised] the importance of having the whole roadway and side-rails formed in the strongest possible manner."[3] Brown was more confident about handling this problem and stated of his proposal for a bridge between North and South Shields [1825], that "while the ends of the bridge are secured in the masonry there can be no pendulous motion...no hurricane that ever blew could bend it edgeways."[38] Bridges and piers to his design were soon to demonstrate otherwise. Telford also underestimated the effects of oscillation at Menai Bridge.

A creditable early effort at deck stiffening by means of longitudinal trusses was made by Tierney Clark at Hammersmith Bridge [1824-27] (fig. 11).[39] Clark, influenced by the wind effects which became apparent at Menai in 1825-26, constructed a large-scale model of the bridge and exposed it to all weathers.[39] As a result of his investigations he incorporated timber and iron trussing along the full length of the roadway. Although this trussing was stated by Pasley to have been effective,[39] it had the drawback, pointed out by Rendel, that because the truss incorporated the suspenders its tendency was "on the passage of a heavy weight, to relieve four out of five suspending rods from their due proportion of load and to throw it upon the fifth rod."[40]

Rendel improved on this concept in his reconstruction of Montrose Bridge completed in 1840. He adopted four longitudinal timber trusses 10 ft. [3 m.] deep, 5 ft. [1.5 m.] above and 5 ft. [1.5 m.] below the deck, in pairs at each side of the bridge, and on either side of (but not incorporating) the suspenders (fig. 12).

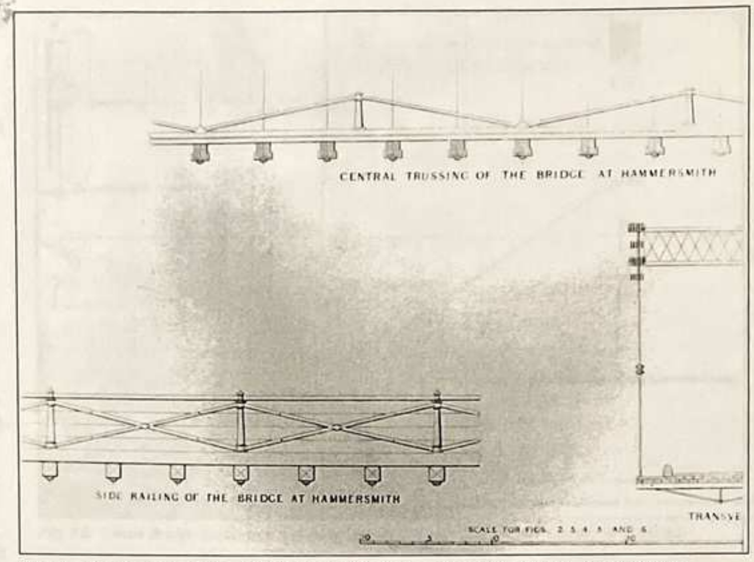


Fig. 11. Hammersmith Bridge 1824-27, longitudinal side and central trussing[39] (C.W. Pasley)

Fig. 12. Montrose Bridge 1829-1927, timber trussing showing below the deck (Davidson postcard)

He also adopted robust transverse trusses at frequent intervals. These measures, publicised in the *Proceedings of the Institution of Civil Engineers* in 1846, represented an important development in countering oscillation effects.[41] Clark adopted a similar arrangement at Budapest Bridge, completed in 1849.

Anchorages, Towers and Saddles

Suspension bridge chains of this period were often anchored into large cast-iron ballast plates weighted down with boulders under the roadway approaches. For example, the north anchorage of Union Bridge where the ballast plate was said by Stevenson to be 24 ft. below the roadway.[3] An enterprising variant of this practice came to light in 1991 during the refurbishment of Gattonside footbridge, span 302.9 ft. [92.3 m.], erected in 1826. Two ship rudders had been pressed into service and loaded with tree trunks and large stones to form the anchorages. Bar chains were connected to each end of the rudder shaft. John S. Brown, partner of Redpath, Brown & Co., was responsible for the ironwork provision and erection [1782-1852].[42]

The earliest masonry and rock anchorages of significance in Britain were built at Union Bridge. Stevenson had in his possession an unexecuted drawing for Union Bridge signed by, although not necessarily drawn by, Capt. Brown for a masonry anchorage ribbed back to the tower (fig. 13).[43] The drawing seems too sophisticated to have been drawn by Capt. Brown himself, whose expertise was in the design and manufacture of chains. He usually had the advice of a bridge engineer on these elements of his projects. Brown's patent drawings of 1817 and 1818[44,45] are indicative of his sketchy knowledge of matters other than ironwork at that time. According to Stevenson, [3] the southern anchorage was connected into the rock by means of a masonry arch, an arrangement which may be attributable to John Rennie who advised on the towers.[46]

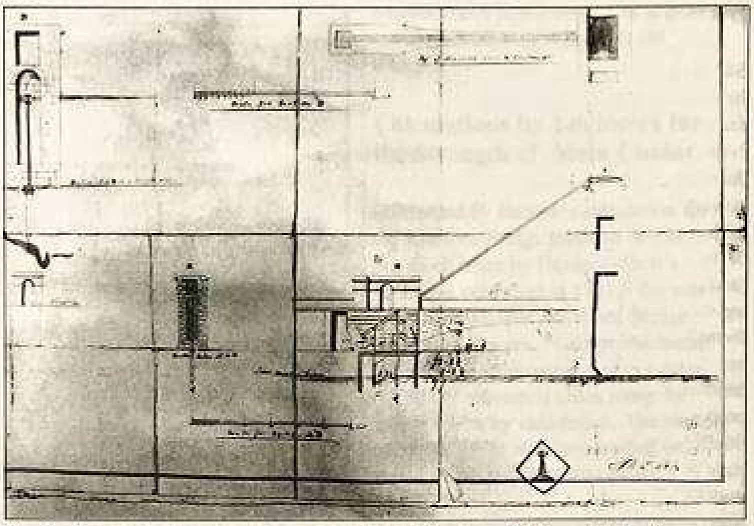


Fig. 13. Union Bridge anchorage and tower details c. 1819[43] (National Library of Scotland)

At Union Bridge, curved cast-iron saddle plates were incorporated into the towers over which it was intended that the chains could slide (fig. 13). In fact, it is doubtful if the chains ever did slide. Any temperature movements of the chains are now, and were probably always, accommodated by the deck rising and falling. An unusual feature of Gattonside Bridge was that no attempt was made to allow the chains to slide over the tower tops. The cast-iron tower capping plates, which came to a point, were surmounted by a bent link onto which the chains on each side were secured and positioned by means of tapering wedges (figs. 14 & 15). As far as is known, this arrangement had worked satisfactorily from 1826 until 1991, when it was replaced without any derangement of the tower masonry. The two pairs of chains of this bridge, one pair of which can be seen in fig. 14 were surmounted at frequent intervals by capping castings which enabled the hangers to transfer their load to the tops of the chains. A surprising finding from tests carried out by the author was that these capping castings which had served for 165 years were made of cast iron with a carbon content of 3.85% and an ultimate tensile stress of 188 Nmm^2 [12.2 tons in^2].

The early attempts to accommodate chain sliding over the towers of suspension bridges, particularly at Menai Bridge, influenced the use of saddles on rollers in most large-span bridges, particularly those by British engineers. With hindsight the provision of rollers seems to have been only marginally beneficial and they are not provided in modern steel towers. When Menai Bridge was examined, after serving for nearly a century, the rollers did not rotate and the sliding movement was only about ¼ in.[6 mm.] with a considerable temperature differential.[47] To give Telford his due, he catered for any possible chain or deck movements deranging the towers by the prudent provision of dowelled masonry throughout the tapering towers above deck level (fig. 16).[48] The original towers remain in very good

Fig. 14. Gattonside tower–chain-bar connection over rigid saddle, 1991 (Travers Morgan)

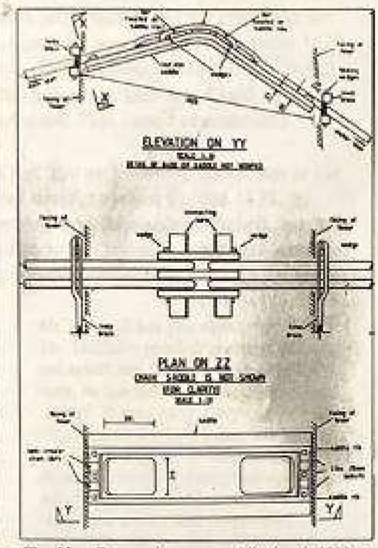


Fig. 15. Gattonside tower–saddle detail, 1991 (Travers Morgan)

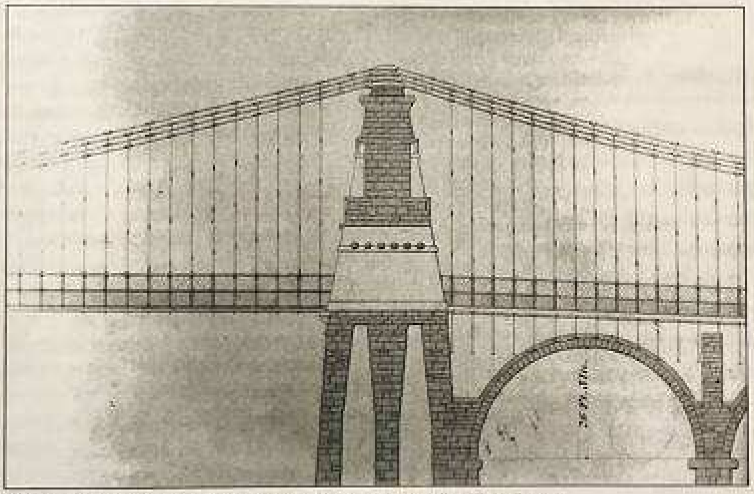


Fig. 16. Menai Bridge, 1819-26–cross-section through tower[41] (W. A. Provis)

condition and in full service as at present there is no weight limit on the bridge.[49]

Calculations by Engineers for the Strength of Main Chains

Influenced by Barlow's calculation for the Runcorn Bridge proposal of 1817, but much more by Davies Gilbert's equations published in 1821,[50] the catalyst for which was the Menai Bridge project, Stevenson,[3] George Buchanan[51] and other engineers increasingly determined the maximum chain strain for their projects by calculation. The various approximations that they applied produced reasonably reliable results.

An insight into various methods of calculation is provided by Stevenson's former assistant James Slight [1765-1854] who went to work for Brown for several years from 1825 to provide a civil engineering input to his projects. These calculations relate to a determination of the maximum chain stress of the Wellington or Craiglug Bridge, Aberdeen, 1829-31 (fig. 8), which Slight considered "the strongest bridge that Capt. Brown has yet erected."[52] It is still in use with its original chains.

The dimensions of Wellington Bridge as given by Slight were: Span 220 ft. [67m.], width 20 ft. [6.1m.], length of catenarian curve 229 [224] ft. [69.9 (68.3 m.)], deflection 18½ ft. [5.6 m., nearly 1/12th of the span] and weight 219 tons [222.5 tonnes]. Reference is made to a live load of 74 tons [75.2 tonnes] i.e. 1,100 men each occupying 4 ft^2 [0.37 m^2] and weighing 150 lbs.[68 kg.]. Slight used Professor John Leslie's formula for one of five calculations for determining the chain strain. This gave the strain at the point of support as $L^2/8d + d/2 =$ 363.55 ft. [110.8 m.], i.e. the length of chain which if suspended perpendicularly would produce a strain at its point of suspension equal to that of a similar chain suspended in the catenarian curve. Then by simple proportion, as 229:363:: 219:347, thus giving 347 tons [352 tonnes] as the ultimate strain.

Slight had determined this strain using "multipliers determined from experiment" applied to the total suspended weight and used one of "1.6 for a bridge of Craiglug proportions", i.e. 219 x 1.6 = 350.4 tons [356 tonnes] ultimate strain. He also determined a multiplier of 1.617 by experiments, presumably from model tests, giving an ultimate strain in this example of 354 tons [359.7 tonnes]. Brown had used a slightly different method [not stated], of which Slight commented, "I suspect that Capt. Brown instead of taking the span ought to take the *length of the chain* which gives a result of 339 instead of 331 tons for the strain."[38] Slight averaged the five results (347, 350, 354, 331, 339) at 344 tons [349.5 tonnes]. With twelve 3 x 1½ in. chains of total cross-sectional area 40.5 in^2 [261 cm^2], the chain stress at the points of support calculates at 8.49 tons in^2 [130.5 Nmm^2], or 8.17 tons in^2 [126.2 Nmm^2] by Capt. Brown's method, well within the elastic limit of the iron. It was speci-fied that all the bars be proved at 9 tons in^2 [139 Nmm^2] of section.

Applying other theoretical methods used by engineers at this time, Davies Gilbert's formula and William Chapman's approximation,[52] which assumed that the chain stress at the supports and at the mid-span were equal, yields maximum strain values of 343 tons [347 tonnes] and 326 tons [331 tonnes] respectively. By the author's calculation these figures compute at 343.5 tons [349 tonnes] for the maximum strain and 325.5 tons [330.7 tonnes] at mid-span.[25] The value by Chapman's method is about 6% too low. For flatter curvatures the use of Chapman's approximation gives a more accurate result. This explains why there was little enthusiasm amongst engineers for implementing a catenary of uniform strength, as suggested by Davies Gilbert. It would have been difficult to have achieved varied the cross-sectional area of the chains and the saving in iron would have been small. The degree of curvature adopted had a much greater effect. For example, at Craiglug Bridge, for a dip of 1/10th span, the maximum strain would have reduced to about 288 tons [292.6 tonnes]; for 1/20th span it would have almost doubled to about 558 tons 567 tonnes]; and for 1/50th span increased by nearly six-fold to about 1619 tons [1645 tonnes]. Another significant factor was the amount of loading adopted. This varied considerably.

Capt. Brown informed Stevenson of his bar testing practice in June 1830, by which time Alexander Mitchell, brother of Joseph, (Telford's assistant) had replaced Slight as assistant:

> Mr Mitchell has just mentioned that Mr. Duffus's proving machine is not warranted to carry more than 65 tons, he has however frequently carried 70 tons on it. I have no objection to the bars being proved in pairs as high as 70 tons. It would however occasion disappointment and delay if the machine should break. I consider that from 10-11 tons to the square inch will be quite satisfactory and this strain you will find will elongate the bars from 1/16 to 1/6th of an inch, but then they would contract after the strain was removed."[38]

This statement reflects Brown's knowledge of the "elastic limit" by, and probably well before, 1830.

Conclusion

To sum up, the largest single influence in the progressive development of the long span suspension bridge in Britain from 1814 to the mid-1830s, and later in Britain, was the Menai Bridge project. It is gratifying to find support for this conclusion in the comment of the eminent suspension bridge engineer John R. Roebling, who wrote in 1867:

> Telford's successful accomplishment of the old Menai suspension bridge was the great feat of those days. But by the modern railway traffic greater strength and stability were demanded and Telford's great achievement was then mistakenly left unappreciated and greatly undervalued.[53]

Authors' Credentials

Roland Paxton is an honorary professor at the civil and offshore engineering department of Heriot-Watt University in Edinburgh, is chairman of the ICE panel for historical engineering works and a member of the Royal Commission on the Ancient and Historical Monuments of Scotland. He has a M.S. and Ph.D. in Civil Engineering. Paxton is a Member of the British Empire Queens Honours List, a Chartered Engineer (United Kingdom), a Fellow of the Institution of Civil Engineers, and a Fellow of the Royal Society of Edinburgh.

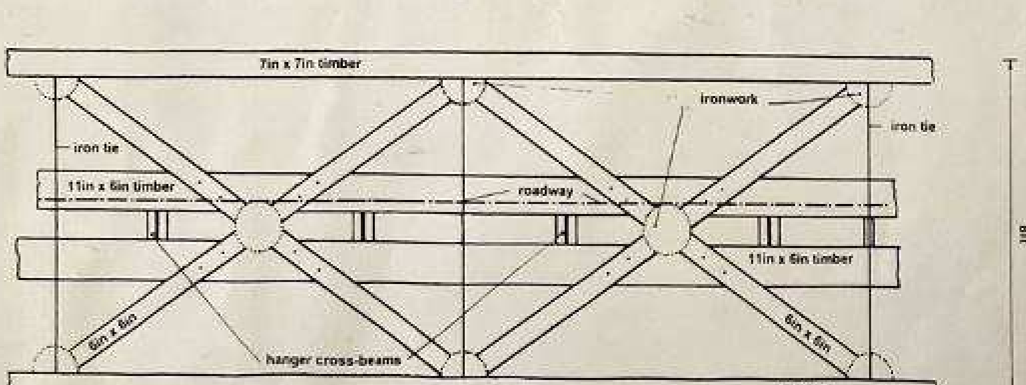
[Circulated at the Conference]

ADDENDUM TO PAXTON'S PAPER *'ON THE DEVELOPMENT OF THE LONG SPAN SUSPENSION BRIDGE IN BRITAIN 1810-40'* [15 October 1999]

Since preparing this paper the author has located at Angus Archives, Montrose, drawings by J.M. Rendel [1799-1856] showing long-lost details of the two longitudinal timber deck-trusses [not four, as previously indicated from the ambiguous wording in Rendel's 'Memoir'[41]] with which he strengthened Montrose suspension bridge [1829-1930] in 1839-41. Details will be given in the presentation. In elevation, each lattice truss was 8 ft. deep [rather than 10 ft. as previously quoted from Rendel's 'Remarks'[40]] and secured to the deck by means of pairs of beams immediately above and below the cross-beams to which the hangers were attached [see figure]. This arrangement was novel, being more substantial than that adopted in previous suspension bridge practice, and also, because the trusses were completely independent of the hangers which were positioned about 6 ft. outside the line of each truss. The diagonal bracing and ties were attached to the top and bottom members and at the centre by means of 'cast iron boxes' with tightening wedges.[41]

Rendel's deck-stiffening work at Montrose, represents a landmark in the progressive development of suspension bridge design. He had been first involved in suspension bridge deck design when working for Telford on the Runcorn Bridge proposal in 1817 and would have known of the measures taken to counter undulation problems which arose at Menai Bridge in 1825-26. His competent Clifton Bridge proposal of 17 November 1829 indicates that by then he was well aware of the need for substantial longitudinal deck stiffening in suspension bridges.

Other additions to the paper in the author's presentation include, a slide from an original drawing at the Institution showing the robust anchorage proposed by Telford for Runcorn Bridge in 1817, and the substantial rock anchorages implemented by him at Menai Bridge.



Montrose Suspension Bridge – Sketch elevation of longitudinal truss designed by Rendel