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The foundations of Hexham Bridge

E. C. RUDDOCK*

The story of the construction and collapse of John Smeaton's bridge in 1777–1782 is told with liberal reference to contemporary practice, his own previous experience, and other attempts to bridge the Tyne at Hexham. The safety of the foundations against scour and against bearing capacity failure is assessed by modern analyses with the aid of a recent soil survey, and the contributions of skirts and girdles to the stability of the foundations is also investigated. The collapse is shown to have followed the classical pattern of failure caused by scour.

On présente l'histoire de l'établissement et de l'écroulement du pont de John Smeaton entre 1777 et 1782 en faisant souvent mention aux techniques contemporaines, à sa propre expérience, et aux autres essais de jeter un pont sur le Tyne à Hexham. La sécurité contre les affouillements et la capacité portante des fondations sont évaluées par des analyses modernes, en se servant d'une reconnaissance récente des sous-sols; on examine en plus les effets des 'jupes' et des 'ceintures' sur la stabilité des fondations. On montre que l'écroulement du pont se passa selon la manière classique d'écroulements par affouillements.

The collapse of John Smeaton's bridge over the Tyne at Hexham in March 1782 was the only important failure in his career. Although it has been mentioned in many books on engineering history, the fullest narrative of its design, construction and failure is still the *Memorial on Hexham Bridge* (*Reports* **3**, pp. 299–320) written by Smeaton himself in May or June 1783 and printed with other letters and reports concerning the bridge in his collected *Reports* in 1812. Twenty-two original drawings, or fair copies, are also extant in Smeaton's own collection of drawings, now in the library of the Royal Society of London (*Designs* **4**, pp. 131–148). The *Memorial* is the primary source of the historical description which follows and only facts derived from other sources are individually referenced. The dimensions used in the second part of the paper are derived from either the drawings or the *Memorial* unless otherwise noted.

HISTORY

Current practice and Smeaton's common methods

Several methods of making good foundations in water were in common use in Britain in the 1770s. The method most generally favoured was to form a watertight cofferdam, pump out the water and excavate about 1 m into the river bed, lay a timber 'grating' or platform at that level and the first course of masonry on it. Piles might be driven under the grating, and this was Smeaton's common practice, used for the bridges of Perth and Coldstream on bottoms where the gravel was deep and had no soft material underneath. He added what he called a 'fence' or 'casing' of timber sheet piles about 2·5 m long, driven just under the edge of the pier to ensure that the gravel under it would be retained whatever scour might happen elsewhere (*Reports* **1**, pp. 188–190 and **3**, pp. 236–239; *Designs* **4**, pp. 125, 125v, 157). It corresponds to what is nowadays called a 'skirt'. For Banff Bridge, which he designed in 1772 to replace one swept away by a flood after only three years' use (Cramond, 1891, pp. 289, 321), he contrived to make the casing serve as cofferdam as well (*Reports* **3**, pp. 349–352; *Designs* **4**,

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* Department of Architecture, University of Edinburgh.

p. 120v); the bottom was a similar gravel. In these, as in many other works in running water, he required the foundations to be surrounded by slopes or mounds of loosely dumped rubble stones, and that similar stones be dumped in any hollows which the flow of the rivers might make in their beds, because he had 'abundantly experienced that good quarry rubble would resist the action of a current to a greater degree than any kind of gravel' (*Reports* **3**, p. 309). He expected this rubble to fill up with gravel and finer particles and become, with such adjustment of shape as the flow itself caused from time to time, a stable and permanent bottom, needing only occasional inspection and minor replacement of material.

Other bridge designers often ordered unstable bottoms to be 'penned' or 'framed and sett'. These terms meant paving with heavy stone blocks or setts held within a frame of thick timbers laid flat on the bottom and tied down by short piles. This work might extend round the piers themselves or, alternatively, over the whole bottom between the piers and a short distance up- and downstream.[1] By preventing scour near the piers it removed the danger of the pier foundations themselves being underwashed.

The other method of founding was in caissons and it was most used where the ground was so permeable or the tides were so high that cofferdams could not be dewatered. The Tyne at Hexham, in a span of thirty years, was to call for all these methods.

'Gravel crust' bottoms

Every gravel river-bottom was subject to scour and Smeaton's defence had always been rubble stone; but a gravel crust over very weak material was especially difficult. Before his design for Hexham in 1777, Smeaton had recognized such conditions at least twice. In 1760 he bored the bed of the Clyde at seven different crossings by the town of Glasgow and found at all of them 'mud or sleech' underlying a gravel which varied from 0·3 to 1·6 m thick (*Reports* **1**, pp. 333–7). At the worst site of the seven, where the gravel was loose and 1 m thick, he thought it possible to found a bridge if the bed between the piers was protected by 'piling, setting and framing'. But he recommended that the bridge be built at another site where the gravel was more compact, and he was content to design his bridge there with a combined width of waterway through all the arches equal to the existing breadth of the river, so as to cause no increase of mean velocity of the current. The design shows piers only 4 m wide founded on timber platforms no more than 0·3 m below the surface of the river bed (*Designs* **4**, p. 114). There is a timber or stone 'girdle' laid on the projection of this platform tight against the side of the first course of masonry.

This design was submitted along with his proposals for development of the river, and these proposals involved considerable control works, including damming out the tides (Renwick, 1912, pp. 47–49) so he probably expected no violent currents to attack his bridge. The bridge and control works were both abandoned, but when another bridge was designed in 1768 by William Mylne, not for the site Smeaton chose but for the one he thought worst, Mylne drew just the same detail for the foundations of his piers (Mylne, 1768). They were built, however, not like that but with bearing piles and a surrounding sheet-pile skirt, an acknowledged copy of Smeaton's method at Perth Bridge then nearing completion (Ramsay, 1768). They gave some trouble just after they were finished and had to be protected by a 'dam' downstream, to raise and steady the water under the bridge.

The framing and setting which Smeaton mentioned here he never suggested again for a large bridge and he criticized the method at least once because decay of the timber made it impermanent (*Reports* **3**, pp. 345–348). What foundations he would have proposed, had there been no intention to control the river, we cannot tell.

[1] See Fig. 12.

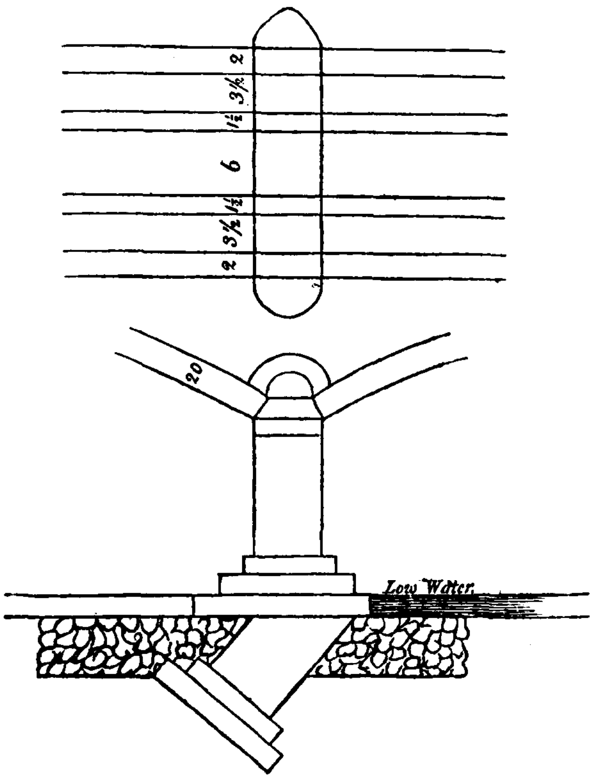

Fig. 1. Sketch of Dumbarton Bridge failure (Smeaton: *Reports* 1, p. 339)

At Dumbarton in 1768 he saw the failure of two arches of the newly-built bridge, caused by a gross tilt and settlement of the pier between them (*Reports* **1**, p. 339)—a movement which today would be called a bearing capacity failure, if his sketch is at all accurate (Fig. 1). He found the river bottom to be a crust of gravel 0·6 m thick over mud so soft that an iron bar sank in by its own weight to a depth of 12 m. That the failure was caused by simple overloading of the ground is confirmed by the fact that it happened 'without any flood, external violence, or previous notice' (*Reports* 3, p. 294). Smeaton recommended that the sunken pier be left as it was, the gravel crust made good, apparently with rubble stones (Fig. 1), and a new pier be built on top of the old one, making it and the spandrel over it much lighter than before. He believed this was done with success and recalled it when Hexham Bridge failed fourteen years later (*Reports* 3, pp. 293–294). Recent works confirm that the bridge had hollow spandrels (F. A. MacDonald and Partners, 1934; Babtie, Shaw and Morton, 1960).

Earlier bridge work at Hexham

In making his design in 1777 (Fig. 2), Smeaton had several other precedents to consider. He himself had made the first known design for a bridge at Hexham in 1756, at a site between Tyne Green on the South and Hermitage on the North bank, some distance downstream of the West Boat (Fig. 3). What he knew of the form of the river bed and the nature of the material neither his drawing nor his estimate shows, but the estimate does show that he intended to lay the foundations, of 'rough block stones', several feet below the bottom. He allowed for

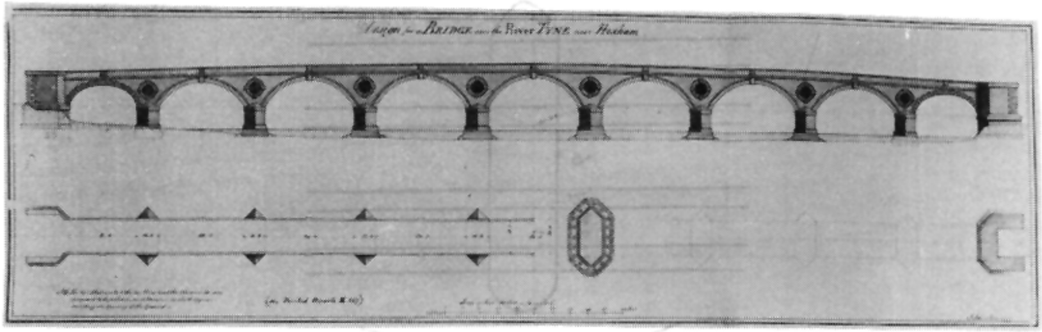

Fig. 2. Smeaton's design drawing, 1777 *Courtesy of the Royal Society*

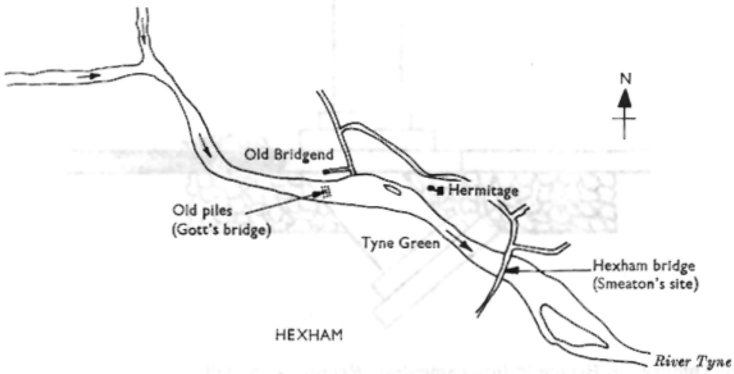

Fig. 3. Map of Hexham and the Tyne

cofferdams and pumping but no bearing piles. The bridge was to be of four long arches each 23 m span, with piers 3 m wide, the total width of waterway under the arches being 92 m (Smeaton, 1756).

He had a second invitation to design a bridge for Hexham, apparently in 1764, but declined to undertake it due to pressure of other work (Smeaton, 1764). The request came from Sir Walter Blacket who owned the estate of Hexham Abbey, and Sir Walter later engaged John Gott to design a bridge. This was built a little upstream of the West Boat, where Old Bridgend now stands on the North Bank (Fig. 3). Gott was well known to Smeaton as a bridge contractor in the West Riding and superintendent of works on the Aire and Calder Navigation, and Smeaton saw some of the building of the bridge when passing and heard more, certainly from Blacket and probably also from Gott. The piers were founded in 'broad cofferdams of earth' kept dry by chain pumps. There were tapered timber piles driven 3 to 4 m and supporting platforms of 0·3 m timbers 1 m below the river bed. On these platforms the masonry was built.

In the hours of darkness of 16–17 November, 1771, less than two years after its completion, all the superstructure of this bridge except one abutment was destroyed by the 'great inundation' of the Tyne which tore down six other bridges and left only one intact (Sykes, 1833). Because it happened at night the sequence of failure was not observed, but at least some people believed the bridge to have failed 'for lack of elevation and not for want of proper foundation'

Fig. 4. Platform for one pier of John Gott's bridge, photographed 1974

(*Errington Papers*, pp. 41–46). The likeliest meaning of this phrase would seem to be that, the whole bridge being low, the flood first filled all the arches to the crowns and then accomplished the destruction of the superstructure without disturbing the foundations.

Smeaton wrote in his *Memorial* in 1783 that he found this failure impossible to explain until he heard from Jonathan Pickernell in 1776–7 of later investigations of the river bed (described below), when he concluded that 'the violence of the water [had] taken off the crust of gravel, wounded also by the excavation for the piers, so as to let loose the quicksand' which he believed must extend under all the foundations. He was encouraged in this view by the knowledge that Blacket had paid a bond for £3000 to the County Justices rather than take the risk of rebuilding the bridge but absolved his undertakers from a similar bond, so sure was he that they could not have foreseen the violence of such a flood; and Smeaton was further encouraged by the sight of the platform which formed the base of one of the piers standing for several years after the collapse tilted sideways out of the water at about 45° to the horizontal.

Three years later, however, in his answers to a series of 'interrogatories' which were put to all the witnesses in the lawsuit between Errington and the County, he offered a different explanation in which he considered the tilted platform as possibly the only one disturbed by scour, and that at one side only, thus causing the tilt (*Errington Papers*, pp. 129–130). He referred also to the bridge's 'lack of elevation', but in the sense, not of a low profile of the whole bridge, but of a high ratio of span to rise in each arch with consequent large horizontal thrust which would cause a progressive collapse of all the arches once a single pier had moved.

Examination of the site today supports this later explanation. It is clear that the crust of gravel was not generally 'taken off', since most of the piles of two of the piers still stand erect and little displaced, protruding from 0·3 m to 1·0 m out of the river bed, while the platform of a third pier lies in its place, complete and only slightly tilted, and presumably with its piles intact under it (Fig. 4). There is a part of another platform tilted just like the one Smeaton described and, if it is the same one, it argues that the gravel at this site is in fact uncommonly difficult to 'take off'.

Smeaton was called to report on the destruction by this flood, not of the Hexham Bridge but the bridge at Newcastle (*Reports* 3, pp. 260–266). This he did in conjunction with the local engineer John Wooler, who was thereafter responsible for the reconstruction of the northern half of Newcastle Bridge. Wooler was also engaged by the Justices of the County in 1774 with regard to Hexham Bridge; first to approve a design by a certain John Fryer which local builders offered to build for £5000 (plus the materials of the former bridge) and later, when they withdrew and there was no answer to oft-repeated advertisements for another undertaker, to make his own design of a bridge to be built by direct labour under his supervision (*Quarter Sessions*, 1774–1775). Borings made by a 'surveyor' had shown, at a site about 50 m upstream[2] from the former bridge, 'a bed of clay laid at no more than four feet under the bed of the river', and so Wooler designed timber platforms to be placed on the top of the clay with piles driven below, the same sort of foundations as Gott had used. But when the resident supervisor Jonathan Pickernell dug for the first foundation in 1775 he found at 4 ft (1·2 m) depth not clay but 'quicksand full of bubbly springs, and of so loose a texture, that by hand only, a bar of iron entered into it 46 ft (14 m) without meeting any resistance; and that a trial pile of whole timber entered 26 feet (8 m), at 2 inches and a half (62 mm) per stroke of the ram without stopping'. In a letter dated July 1775 (*Errington Papers*, p. 3), Wooler pronounced the bridge impossible unless the whole river bed were paved with a 'wall' 2 m high and 13 m wide founded 1·6 m below the bed on timber 'sleepers' bedded in the quicksand at a close spacing of 0·3–0·45 m and protected by lines of closely-spaced timber piles driven 8 m into the quicksand along the whole of the upstream and downstream edges of the wall. This was much more than ordinary penning and would have made a true raft foundation for the bridge. It was obviously very expensive and never seriously considered.

The Justices, already committed by more than £3500 spent on preliminary works and materials (*Errington Papers*, pp. 164–165), continued to advertise for undertakers and continued to employ Pickernell. He had already made borings up and down the river, finding everywhere loose sand or quicksand under only a small thickness of gravel. One site he bored was a mile downstream from the former ones, a little below the East Boat, and it was an interest in this site, which adjoined his estate, that brought a suggestion from Henry Errington, Esq., that he would contract to build a bridge if John Smeaton should think it feasible, design it and direct the construction.

First design, 1777

Smeaton chose a line across the river near the downstream end of a still pool below the East Boat (Fig. 3) and made soundings himself from a boat, but only to a depth of 2·7 to 3·0 m. He used a pointed iron bar, counting the blows of a hammer required to drive it, which he thought to give a better estimate of the compactness of gravel than a boring tool.[3] He found that the bar was 'very considerably less resisted, and particularly in the main current, after it was

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[2] In his 1786 evidence Smeaton said 50 m downstream (*Errington Papers*, pp. 129–130), but as this is the only reference to a downstream site it is considered a mistake.

[3] As modern engineers use penetration tests instead of borehole samples in coarse-grained soils.

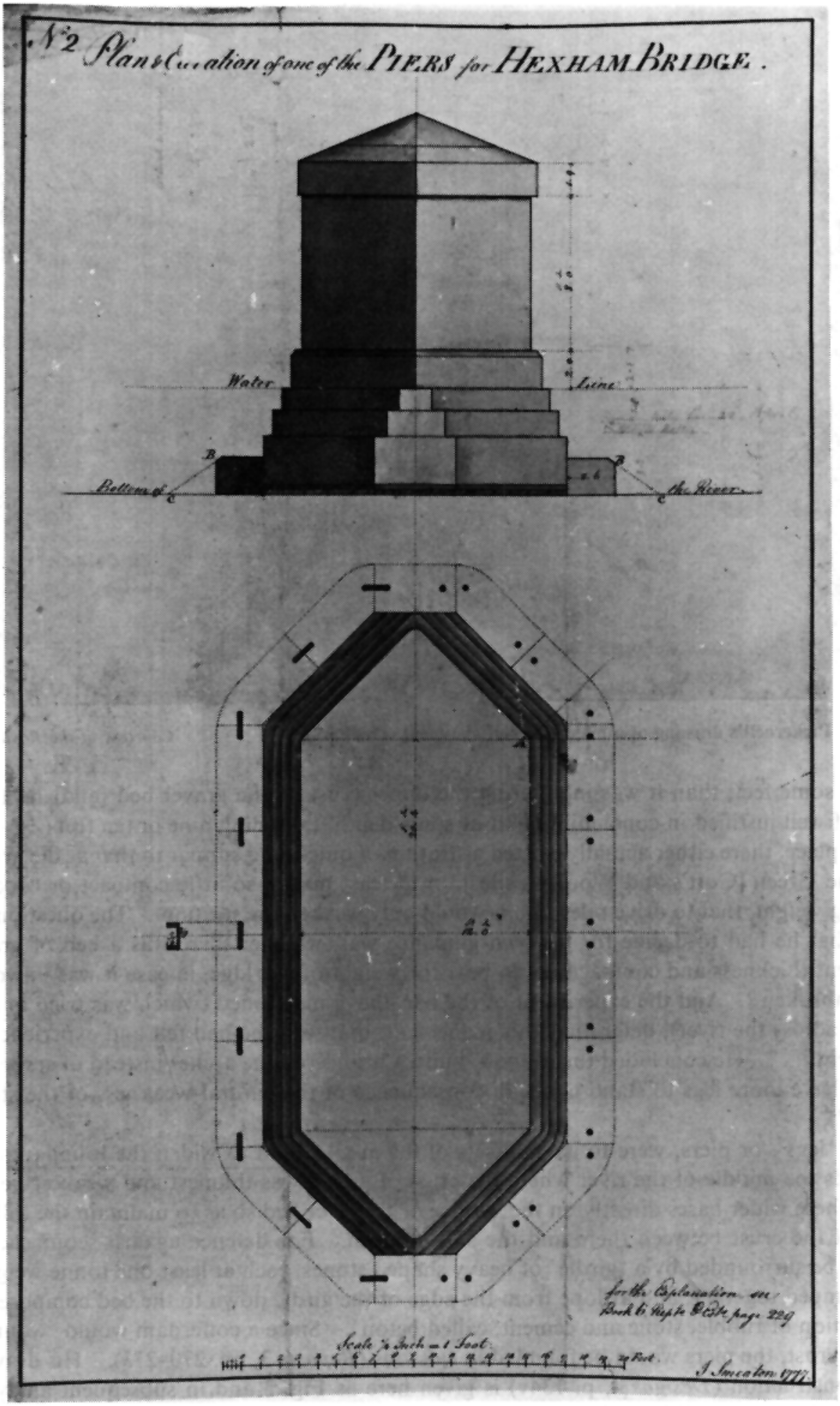


Fig. 5. **Smeaton's design for a caisson pier 1777 (Design 1)** *Courtesy of the Royal Society*

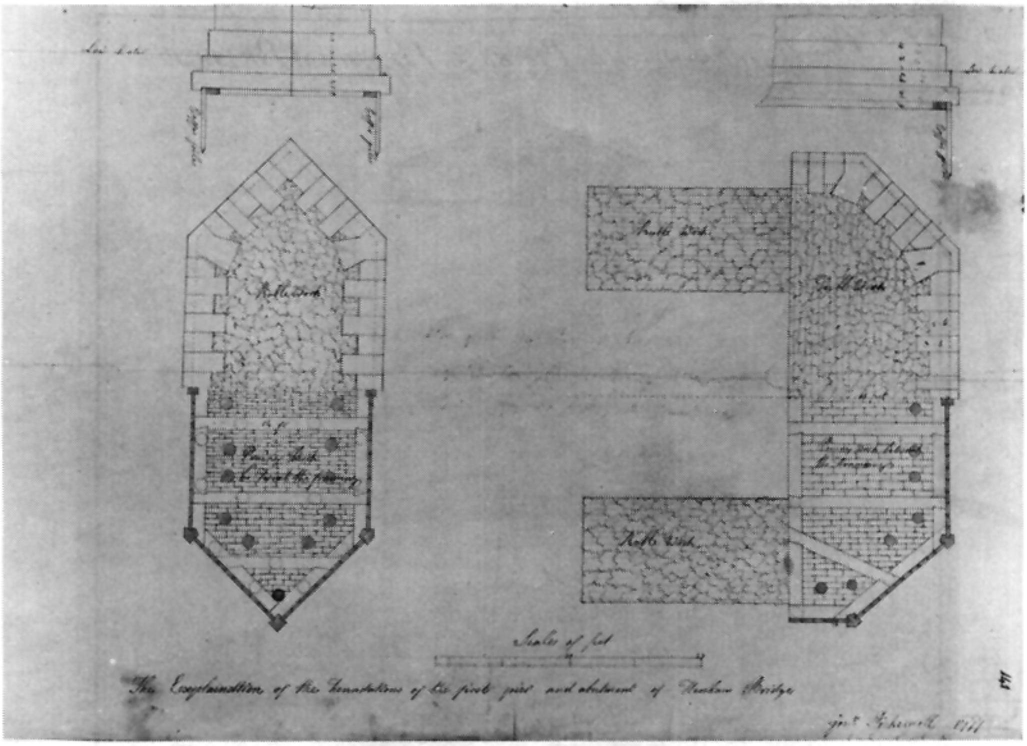
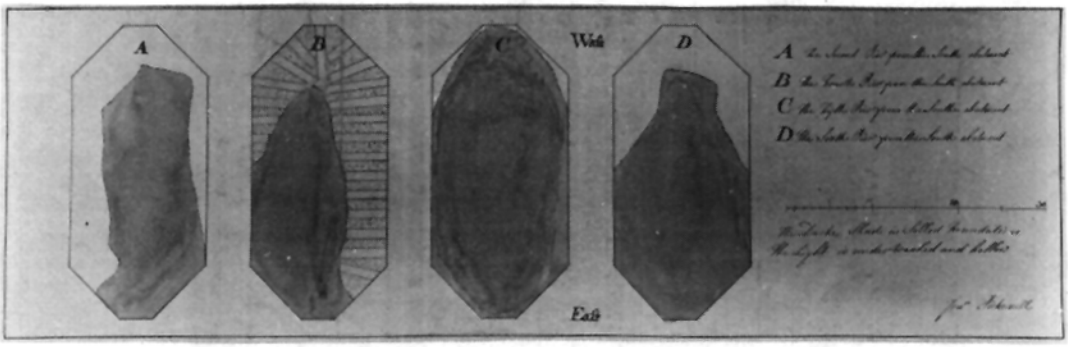


Fig. 6. Pickernell's drawing of a piled pier and abutment (Design 2) *Courtesy of the Royal Society*

driven some feet, than it was in entering the upper crust of the gravel bed [and] he thought himself well justified in concluding, that at some depth, exceeding nine or ten feet (2·7–3·0 m), at this place, there either actually existed a stratum of quicksand similar to that at the West end of Tyne Green [Gott's and Wooler's sites], or at least matter so little compact or capable of bearing weight, that to drive piles into it would only weaken the stratum. The question therefore, that he had to decide for his own guidance was, whether there was a bed of gravel of sufficient thickness and compactness to bear the weight of a bridge, in case it was unwounded and unbroken? And the experiment of the bar above-mentioned (which was tried in several places across the river), determined his judgement, that what he had felt and experienced was sufficient. . . . He concluded therefore to build a bridge of nine arches instead of seven that it might have more legs to stand upon, in consequence of the natural weakness of the stratum' (Fig. 2).

The 'legs', or piers, were to have offsets of 0·6 m all round to widen the foundations, and towards the middle of the river where the crust of gravel was thinnest and weakest he would place these wider bases directly on the surface of the river bed so as to maintain the full thickness of the crust between them and the soft stratum. For defence against scour each base would be surrounded by a 'girdle' of heavy shaped stones, each at least one tonne weight and all cramped together, and a slope from the edge of the girdle down to the bed composed of 'a deposition of rubble, stone and cement, called beton'. Since a cofferdam would 'wound' the gravel crust, the piers would be founded in caissons (*Reports* **3**, pp. 270–273). His drawing of this construction (*Designs* **4**, p. 134v) is given here as Fig. 5 and in subsequent analysis the proposed method will be called Design 1.



Courtesy of the Royal Society

Fig. 7. Pickernell's plans of the underwashed piers 1778. The dark areas are those remaining in contact with the ground, the white areas are underwashed

For the piers next to the abutments and the abutments themselves, where the gravel seemed harder and possibly deeper, Smeaton proposed founding by his common method in cofferdams, with bearing piles, grating, skirt of sheet piles about 1·5 m long, and rubble defending slope. This is shown in Fig. 6 (*Designs* **4**, p. 141). The gratings are shown by this and other drawings to have been laid at from 0·85 to 1·40 m below the assumed low-water level. The depths below the river bed are not recorded. This method of construction will be called Design 2.

The contract drawing is given in Fig. 2 (*Designs* **4**, p. 131). The middle arch had a span of 15·5 m and the total width of waterway through the nine main arches was 129 m, with a further 16·5 m of 'land arches' through the south approach. The width obstructed by the piers was 29 m. On the back of a note of these dimensions amongst Smeaton's drawings (*Designs* **4**, p. 130v) is a second list of figures which seem to be the dimensions of the arches of Gott's bridge and these would show that that bridge had a waterway of only 113 m; the middle arch was certainly 21·4 m in span. Two drawings made after the collapse in 1782 (*Designs* **4**, p. 148; *Errington Papers*, p. 7) show that the spandrels of Smeaton's bridge were built hollow to lighten the superstructure.

Changes in design during construction (1778–80)

Use of the cofferdam method (Design 2) was extended to the second pier from the north end as well as the first from each end. The founding of the other five piers began in the spring of 1778 and Smeaton was present to see the first one founded. By the end of July three more were founded and built up above water level. The placing of the girdle and beton slope, however, had begun only at one pier when a rapid flood scoured the bed from under all four at their upstream ends and along the sides to an average width of 0·4 m and they tilted and settled by up to 0·45 m (Fig. 7).

Smeaton decided that the girdle and beton were not enough protection and ordered sheet piling to be driven at about 1 m distance from all the faces of each pier, with a 'screw clamp' or waling to tie it just below low-water (*Reports* **3**, pp. 275–278; *Designs* **4**, p. 139). He hoped it could be used as a cofferdam and pumped dry while small stones, gravel and sand were packed by a hand ram into the voids to underpin the pier, but this proved impossible and a small diving bell was used for the underpinning. The piles were cut off just above the waling at about low-water level and the space within the casing filled up with rubble and sand, paved on

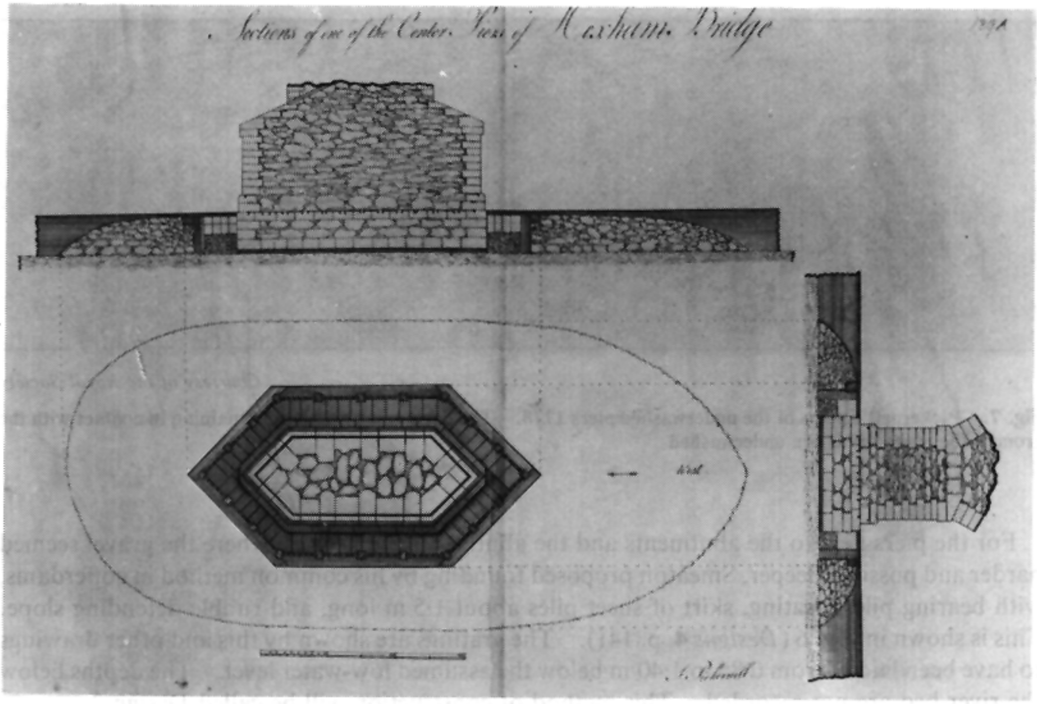


Fig. 8. Pickernell's drawing of a pier as built 1779 (Design 3) *Courtesy of the Royal Society*

top with large squared stones. Outside the piling a slope of coarse rubble was dumped, extending about 3 m out at the sides of the pier, 10 m upstream and 7 m downstream. Pickernell's drawing of this (*Designs* **4**, p. 139A) is reproduced as Fig. 8. The masonry was completed with adjustments for the tilting and extra cramps and tie courses. He ordered the sheet piling to be driven 3 m into the gravel at the upstream end of each pier, reducing to 2·1 m at the downstream end, but the piles at one of the middle piers would drive only 1·5–2·1 m and at another, the seventh from the north, they were badly deflected by large stones in the gravel. He assumed that the remainder were driven as ordered. In subsequent analyses this construction, with the piles penetrating 1·83 m into the existing river bed, will be called Design 3.

On 12 December, 1778, when the piling and most of the underpinning had been completed, but not the rubble slopes, a flood rose to within 0·22 m of the top of the impost (top course of the piers) and Pickernell reported that there was a fall of 0·69 m from the upstream to the downstream end of the piers. In 1779, when most if not all of the rubble had been placed, a fall of 1·15 m was reported during another flood. Both these floods were 'sufficient to tear up and remove the natural bed of gravel . . . wherever there was a particular set upon it', but caused no distress to the piers, and the rubble slopes were found to have survived, being usefully filled up and covered with gravel in some places.

In 1780, as the bridge was being finished, it was reported that the bottom was wearing deeper in the channels between the piers and Smeaton advised Pickernell that the rubble slopes might be allowed to settle until their tops were 0·92 m down the face of the sheet piling if their surfaces were at a gradient of no more than 1 in 2 and the bottom of the channels no more than 2·44 m below low-water (*Reports* **3**, pp. 291–293). This settling of the surface of the slopes would be

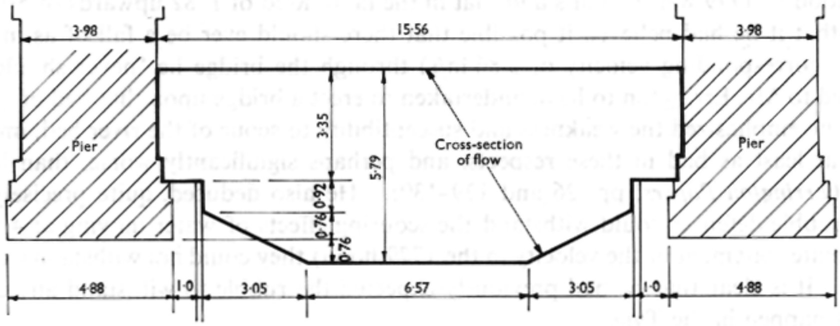


Fig. 9. Waterway and piers of middle arch. Limiting safe conditions envisaged by Smeaton (Design 4)

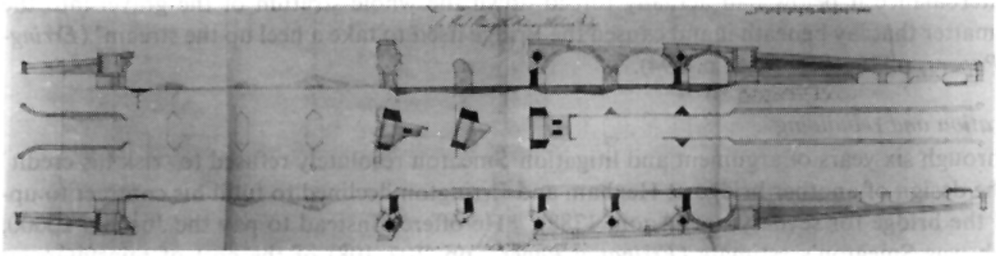


Fig. 10. Smeaton's drawing of the ruins 1782 *Courtesy of the Royal Society*

caused not by the current removing the rubble but by scour of the natural gravel bed near and under the toe of a slope, with the rubble rolling down the slope as well as settling and compacting. Fig. 9 shows the limiting condition which Smeaton was describing and this will be called Design 4 in the analyses given below.

The collapse, 1782

The facts of the bridge's collapse on the morning of 11 March, 1782 were briefly told by Errington's agent John Donkin who watched it (*Errington Papers*, pp. 24–25). He noted the levels of the water at both ends of the piers, which differed by between 1·22 and 1·53 m, and probably about 1·37 m. Particles of lime first began to fall from the fourth arch from the north end; this increased for about a minute, then cracks spread through the arch and spandrels for about another minute and then the third pier and the third and fourth arches collapsed together. Five other arches fell within the next half-hour, leaving only the first three at the south end, but large parts of two of these fell shortly afterwards. The two middle piers stood tilted at 10° or more, their west (upstream) ends sunk in the river bed and their east ends lifted clear above it (*Designs* **4**, p. 148). The piers founded on piles and gratings fared no better, two at the northern end being reduced to mounds of stones (Fig. 10).

Smeaton's explanations of the failure

In several explanations during the ensuing years Smeaton gave three reasons for the failure. The first and most important was that the floods in the River Tyne had been shown capable of a degree of violence far greater than he had thought possible (*Errington Papers*, p. 26). He calculated from the heights of fall reported that the velocity of the water through the bridge during the flood of 1779 was 4·73 m/s and that in the fatal flood of 1782 upwards of 5·10 m/s. He insisted that if he had believed it possible that there should ever be a fall of as much as 0·76 m (with corresponding velocity of 3·86 m/s) through the bridge he 'never should have recommended to Mr. Errington to have undertaken to erect a bridge upon that bed of gravel'.

Secondly, he emphasized the weakness and susceptibility to scour of the river bed, implying that it was at least as bad in these respects, and perhaps significantly worse, than he had anticipated (*Errington Papers*, pp. 26 and 129–130). He also deduced, quite precisely, that though the rubble defences could withstand the scouring effects of water moving at 4·57 m/s (an approximate statement of the velocity in the 1779 flood) they could not withstand a current of 5·10 m/s. It is clear that he had previously expected the rubble to withstand any current which might happen in the Tyne.

Thirdly, he expressed twice in the year after the failure the idea that 'the incumbent weight of a body of water of 5 ft (1·53 m) in thickness above the bridge superior to what was to counterbalance it below had actually forced down the whole stratum of the gravel into the soft matter that lay beneath it and caused the Bridge itself to take a heel up the stream' (*Errington Papers*, p. 26; *Reports* **3**, p. 294).

Litigation and rebuilding

Through six years of argument and litigation Smeaton resolutely refused to 'risk his credit' on the design of another bridge at Hexham and Errington declined to fulfil his contract to uphold the bridge for seven years (Anon, 1788). He offered instead to pay the Justices £3000, which was Smeaton's estimate (*Errington Papers*, pp. 107–108) of the cost of reinstatement according to the original design, to which the contract referred; a jury eventually awarded the Justices £4000 damages in 1788 (Anon., 1788). The Justices engaged Robert Mylne, the designer of Blackfriars Bridge, as their technical witness and as part of his survey of the ruins in 1783 he had a single boring made near the second pier from the north end, which was one of the piers built on piles because Smeaton thought the gravel deep enough there. Mylne's 'borer' drilled to a depth of 7 m and reported the ground to be 'uniformly a composition or congeries of roundish and flat stones, gravel and sand, of equal quality and consistence in the whole of that depth' (*Reports* **3**, pp. 296–298).

Mylne considered that a bridge could be built safely on this bottom and when the case was concluded the Justices duly built one on the same, or very nearly the same, site by direct labour under the direction of their Bridge Surveyors, William Johnson and Robert Thompson (*Quarter Sessions*, 1788–1796). How much help these men received from Mylne is not clear, but it was almost certainly limited to the foundation design, for the superstructure (Fig. 11) is a replica of Smeaton's bridge, and must include much of the stone of it, salvaged from the river after the collapse.

The type of foundations on which the bridge now stands is shown by an isometric drawing at the County Surveyor's office, which is reproduced as Fig. 12, and there is a small-scale plan and elevation drawn and signed by William Johnson in 1796, now in the Drawings Collection of the Royal Institute of British Architects, which confirms that these are the original foundations. The piers stand on timber piles and platforms and the whole river bed is framed and sett both under the arches, for 1 or 2 m upstream and for almost 8 m downstream from the ends of the piers. The framing is tied down by no less than five lines of closely-spaced piles extending right across the river. The cost to the county was just over £8000 (*Quarter Sessions*, 1788–1796), compared with a contract price of £5700 for Smeaton's bridge.[4]

[4] Evidence by John Donkin during the legal proceedings shows that Errington's actual expenditure was about £6060, including an allowance for the services of his agent, etc.

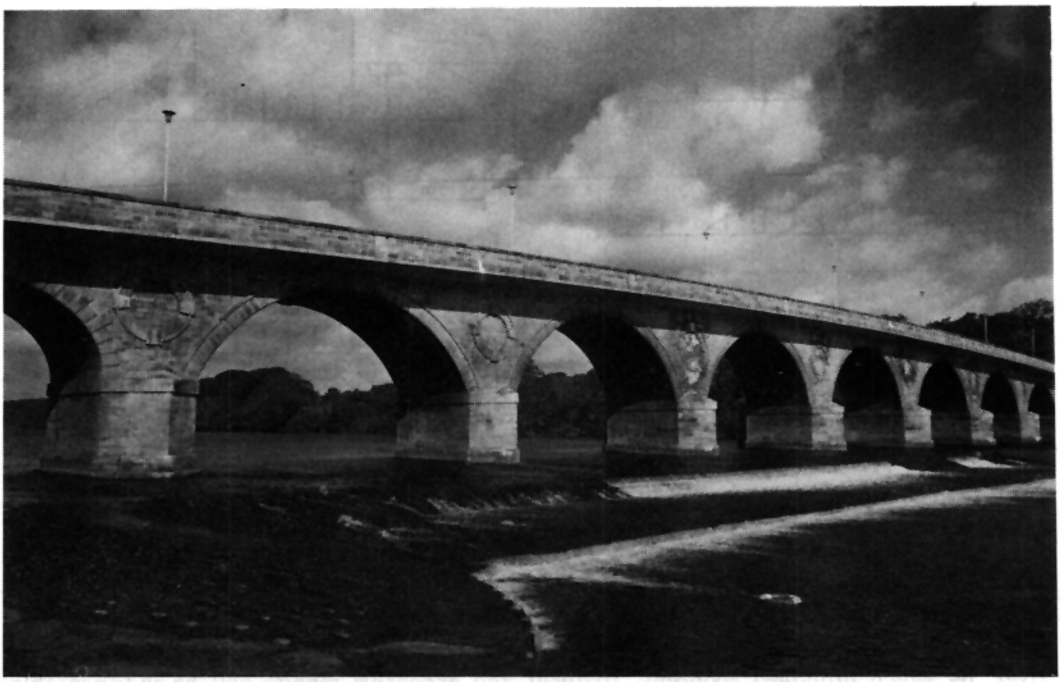
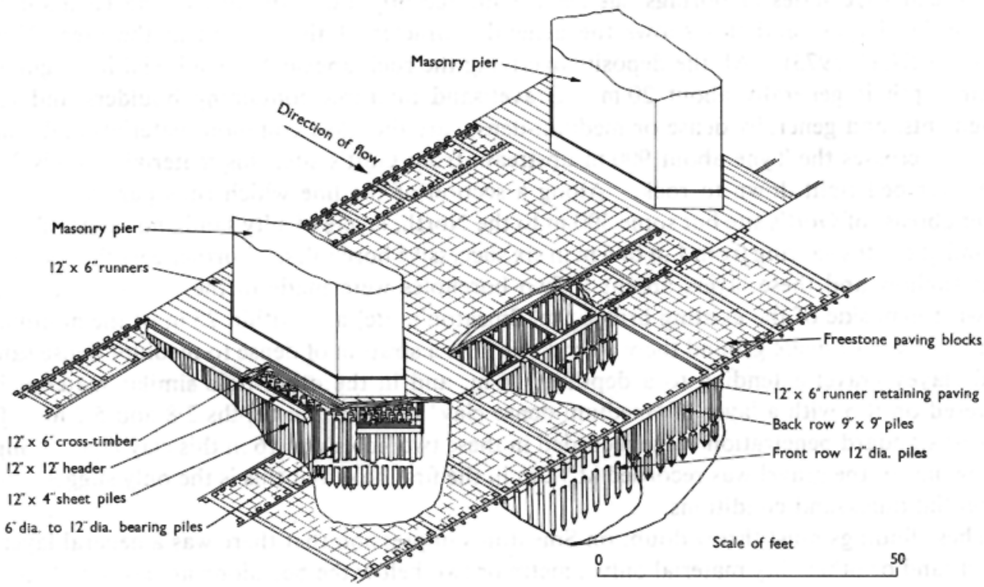


Fig. 11. Hexham bridge today. The cantilevered footways were added in 1964



Courtesy of Northumberland County Surveyor

Fig. 12. Drawing of foundations, twentieth century

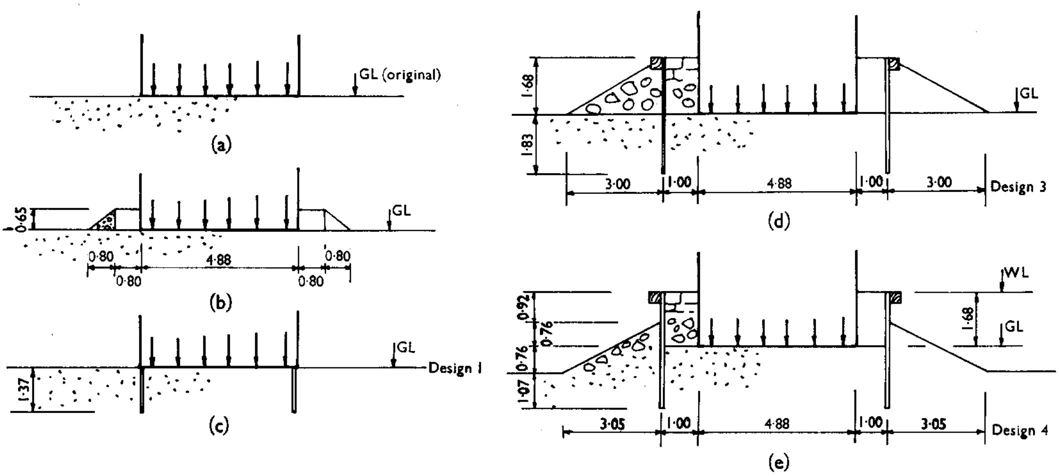


Fig. 13. Sections of pier foundations for bearing capacity calculations

Later history and recent soil survey

In 1831 the County Surveyor, in a review of bridges under his care, commented that 'the foundation masonry of the abutments and piers has not been laid at a proper depth below the river, by which ill-judged economy, frequent and expensive repairs will be indispensable' (*Surveyors Bridge Book*, 1831). Major repairs, mostly to the 'penning' cost £644 in 1831–1832, £355 in 1867, and £92 in 1883–1885 (*Expenditure*, 1831–1885). In this century the penning has been concreted over and the concrete extended to form an apron stretching at least 13 m downstream over the whole width of the river.

An extensive series of borings has been made recently along the line of the Hexham and Corbridge Bypass and this shows the general character of the ground in the area (North Eastern RCU, 1973). All the deposits overlying the rock appear to be alluvial in origin and their depth is generally about 20 m. Gravel/sand mixtures containing boulders and rock fragments, and generally dense or medium-dense, are the most common materials and where the line crosses the Tyne about 900 m upstream from Gott's site, this material extends from the river bed right down to rock. Along a section of the line which runs parallel with the river abreast of Gott's site but some 300 m to the North, layers of silty sand and sandy silt were found at depths varying from 1 m to 6 m in six successive boreholes. Further east the rock level was shallow and these layers absent. Two boreholes were made in the north bank on the downstream side of the existing bridge (i.e. Smeaton's site) and within 25 m of the abutment. Under 3–4 m of made ground there was in one hole a stratum of dense to medium-dense sandy and clayey gravel extending to a depth of 16 m, and in the other hole similar material but covered on top with a layer of 'medium-dense clay' between the depths 3·8 and 5·2 m. The lowest standard penetration value recorded in these two holes was 16 in this clay layer. Slight 'blowing' of the gravel was recorded at 12 m in the first hole, but this is the only suggestion of potential quicksand conditions.

These findings must throw doubt on Smeaton's impression that there was a general layer of quicksand or other silty material only a metre or two below the bed along at least a mile of the river; but with regard to the actual site on which he was building the two borehole logs do not conflict with the findings of his probing survey, and there is no check on his findings about the river bed at midstream.

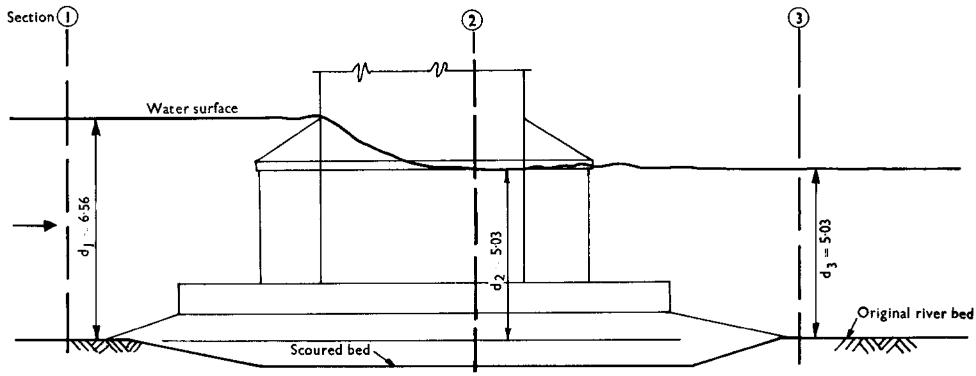


Fig. 14. Profile of flow through middle arch (Design 4)

ANALYSIS

There are several small discrepancies between dimensions given in Smeaton's design drawings and what are apparently record drawings made by Pickernell at the site, and where these occur Pickernell's values are preferred. The normal or low-water level is taken as 1·68 m above the (original) natural surface of the river bed at the caisson piers. This conforms to the level of water shown in Pickernell's drawing of one of the middle piers (Fig. 8). His drawing of an end pier and abutment (Fig. 6) shows that both the river bed and the base of the masonry were higher there, being only 0·4 m and 0·7 m respectively below low-water.

Hydraulic calculations and assessment

Smeaton's calculations of velocities of flow from the observed fall are considered in relation to Design 4, since they were made after he had advised that Design 4 would be safe. Design 4, as it applies to the middle arch and its piers, is illustrated by Figs 9 and 13(e).

His results all correspond to the simple relationship $h = v^2/2g$, where h is the loss of potential energy head in passing the bridge—or the 'fall',—and v is the velocity gained. This, as he said, was a 'rule' stated in the hydraulic textbooks of the time, but it was proved for, and properly only applied to, the discharge from a static tank through a small orifice. It could not be applied with any accuracy to the discharge through the arches of a bridge because there would be a considerable velocity of approach. The amount of error is shown by calculating the velocities through the middle arch by a modern textbook method (Webber, 1968, pp. 202–204).

The longitudinal profile of the water is known to be of the general form shown in Fig. 14. Webber finds that

$$\frac{v_2^2}{2g} - \frac{v_1^2}{2g} = d_1 - d_2 = 1·53 \text{ m} \qquad (1)$$

For continuity of flow,

$$A_1 v_1 = A_2 v_2 = A_3 v_3 \qquad (2)$$

The appropriate width of flow area at cross-sections (1) and (3) is that of the middle arch plus the half-widths of the adjacent piers, i.e. 19·54 m. A_2 is the area of flow shown in Fig. 9. Inserting values, the equations yield $v_1 = 4·05$ m/s; $v_2 = 6·84$ m/s; and $v_3 = 5·40$ m/s.

Smeaton's calculation gave a single velocity, applying correctly to section (2), of about 5·10 m/s. Although the difference between this and the modern result is not large it could be quite disastrous in view of the fact that the transporting power of a current, and therefore the onset of scour, is found to be proportional to the fifth or sixth power of the velocity (Webber, 1968, pp. 164–6). Smeaton's statement that the river bed was stable under a current of 4·57 m/s and yet completely torn up by a current of 5·10 m/s is much more reasonable in the light of this modern knowledge than it must have seemed in his own day.

However, even the velocity of 3·86 m/s which he said would have dissuaded him, could he have foreseen it, from ever undertaking the bridge, is very much higher than the velocity which would nowadays be considered safe over even the coarsest of gravel (Neill, 1973, pp. 90–94; Webber, 1968).

The location of maximum scour agrees well with modern experience. The worst scour during the flood of 1778 took place near the shoulder angles of the upstream ends of piers (Fig. 7) and on the day of the collapse two piers in the middle of the river developed a steep tilt upstream (Fig. 10), clearly due to concentration of scour at their upstream ends. From the evidence of observations at full scale and in hydraulic models (for instance, Neill, 1973, pp. 76, 94, 98; Terzaghi and Peck, 1948, pp. 409–11) this is now considered the 'classical' pattern of scour round bridge piers. The sharp 135° shoulder angles used by Smeaton in the plan of the piers cause more eddying and therefore greater scour at the upstream ends than would rounded surfaces. He was well aware (*Reports* **1**, pp. 99–102) of the use of smoothly-curved shoulders by the French engineers and of the hydraulic arguments for them, but in most if not all of his bridges he adopted shoulder angles of 135°, with the point of the cutwater forming a right-angle.

Foundation methods and bearing capacity

A precise analysis of the final mechanism of collapse is not attempted because it would require accurate knowledge of the properties of the soil and the shape of the river bed surface just before the collapse. Such data are not available. The following verbal arguments and calculations attempt, firstly, to assess the effects on the stability of the foundations of the two expedients which Smeaton used to protect the piers from the effect of scour. There is little doubt that he expected these devices, here as well as in other bridges, to increase the bearing capacity and reduce the settlement as well as fulfilling their primary function. The two devices were the girdle of masonry and/or loose rubble and the skirt of sheet piling. The second purpose of the analysis is to determine the approximate values of the factors of safety against a failure of bearing capacity at various stages of the design and construction.

The five designs considered are shown in Fig. 13 (a)–(e). Fig. 13(a), which Smeaton did not propose at all, is included as a control case with no girdle or skirt. Fig. 13(c) differs from Smeaton's Design 2 in omitting both the bearing piles and the rubble slope; this is done to isolate the effect of the skirt. The length of the skirt, 1·37 m, corresponds to Pickernell's drawing (Fig. 6).

Table 1. Footing on river bed (Fig. 13(a))

Assumed ϕ'	30°	35°
N_γ	22·4	48·0
Ultimate bearing capacity q_0, kN/m²	437	937
F_b (load factor for pressure 240 kN/m²)	1·82	3·90
N_γ for failure under 240 kN/m²	12·3	12·3
Hence, ϕ' mobilized by 240 kN/m²	25·9°	25·9°
and $F_S = \left(\dfrac{\tan\phi'}{\tan\phi'_{\text{mobilized}}}\right)$	1·19	1·44

Effect of skirts. The effect of a skirt can be considered in relation to each of the recognized modes of failure of shallow footings (Vesić, 1973). In the case of general shear, a skirt which extended through the zones of plastic equilibrium and some distance into the unsheared soil below would offer some resistance to the formation of the shear zones and so increase the bearing capacity. But since each of the skirts in Fig. 13(c)–(e) would lie entirely within the zones of plastic equilibrium they would not add anything to the soil's resistance to general shear failure. Because they are aligned more or less with planes of shear in the (theoretical) zones of radial shear and the friction between timber and soil is usually less than the internal friction of the soil itself, they would be more likely to reduce the resistance to general shear.

If the failure were by punching shear, the primary surfaces of shear (Vesić, 1973) would be more or less vertical and close to the planes of the skirts; and so the skirts would be likely again to decrease rather than increase the resistance to failure.

The third mode of failure, by local shear, involves considerable settlement of the footing with the initial shear flow and lateral expansion of the soil confined to zones close under the edges of the footing. There is no doubt that such shear strains and the resulting settlement would be reduced by the presence of even a short skirt, particularly if it were restrained from lateral movement at its top as Smeaton's skirts were. The effect of the skirt would be to prevent lateral expansion of the soil in the critical zones and also to reduce the vertical pressure on these zones by carrying some of the pier load directly down to lower levels.

Factors of safety and the effects of girdles. The properties of the soil are assumed to be: $\gamma' = 10$ kN/m^3, and $\phi' = 30°$ or $35°$. The actual friction angle of the gravel at and near the surface of the river bed would almost certainly lie within these limits. Weaker material may have been present at greater depth, as Smeaton believed, but the recent site investigations make it virtually certain that such material would be, at worst, a silt or sandy clay in which pore-pressures would dissipate as quickly as the load was applied (two years for the whole dead load); and the assumption of a cohesionless material with $\phi' = 30$–$35°$ at all levels is therefore justified.

From Smeaton's initial drawings the volume of solid work carried by one of the middle piers is found to be 578 m^3. Since this includes ashlar, rubble masonry and gravel fill, an average density of 2000 kg/m^3 is assumed. For the bearing area of 45·1 m^2 given by Fig. 5, the bearing pressure is found to be 254 kN/m^2. With the water at the assumed low level, buoyancy reduces the effective pressure to 240 kN/m^2. This pressure, and the dimensions of one of the middle piers, are used in all the calculations to facilitate comparisons, although Design 2 was applied only to end piers where the pressure and dimensions were somewhat different.

For a footing on the river bed (Fig. 13(a)) of width 4·88 m and length/width ratio about 2, calculations using Vesić's (1973) tables of bearing capacity factors give the results shown in Table 1.

As bearing capacity theory can take account only of uniform surcharge loads outside the footing, the effect of the girdle in Smeaton's Design 1 (Fig. 13(b)) cannot be quantified. And to obtain an approximation to the effect of the girdle in Design 3 (Fig. 13(d)) it is necessary to assume that its actual weight (assuming $\gamma' = 10$ kN/m^3) is distributed uniformly over the half-width of the opening between the piers, namely 7·33 m (from Fig. 9).

Calculations following Vesić then give the results shown in Table 2.

Hence the girdle in Design 3 gives a useful increase in the factor of safety. The effect of the girdle in Design 1 would clearly be much smaller. The change from Design 3 to Design 4 is a lowering of the river bed by 0·76 m and of the surfaces of the slopes by 0·92 m, while the surfaces inside the skirts are unchanged. Considering this to be equivalent to a reduction of the uniform surcharge of 0·84 m, the calculations can be extended to give the values shown in

Table 3. Since the footing base is now higher than the river bed, these calculations are not strictly justified but they provide a rough measure of the difference between Designs 3 and 4. Design 4 clearly represents a condition approaching failure.

Stability of skirts as anchored bulkheads. The simple stability calculations commonly applied to sheet pile bulkheads would show that the skirts in Designs 2, 3 and 4 are all unstable. This is because the calculations ignore friction between the soil and the bulkhead and between the soil and the surcharge (in this case, the bottom of the pier). In practice, the skirt of Design 2 would be perfectly stable. The skirt of Design 4 comes nearest to the conditions of a common anchored bulkhead, and a simple 'free earth support' calculation gives a factor of safety of 0·82 when the passive resistance of the slope is ignored but a uniform surcharge of gravel is assumed, of the full height of the slope (1·52 m), resting on the gravel of the passive zone below. This suggests that there was a serious risk of outward rotation of the skirt if the river bed and slopes were scoured to the shape drawn as Design 4.

Discussion

Skirts and girdles. A girdle as big as that of Design 3 gave a considerable increase in the bearing capacity but it also reduced the area of flow and so increased the intensity of scour. A skirt under the edge of the pier, as in Design 2, could have no such bad effect but it, too, proved an inadequate defence against underwashing; it is unlikely to have increased the bearing capacity.

Scour and the sequence of failure. Whether the condition of Design 4 had been reached, or had actually been passed, at any pier before the fatal flood is not known; it exists only as a drawing of what Smeaton thought to be safe. The calculations, both for bearing capacity and bulkhead stability, suggest that the margin of safety was, at best, small. Taking account of the ease with which the river bed was known to scour (after the flood of 1778) it was not right to call Design 4 safe at all.

Table 2. Design 3 (Fig. 13(d))

Assumed ϕ'	30°	35°
Surcharge pressure, kN/m^2	5·7	5·7
N_q	18·4	33·3
Shape factor (for $L/B = 2$)	1·29	1·35
Depth factor (for $D/B = 0{\cdot}12$)	1·03	1·04
Additional bearing capacity, kN/m^2	139	264
Total bearing capacity	576	1201
Revised F_b	2·40	5·00
Revised F_S	1·33	1·61

Table 3. Design 4 (Fig. 13(e))

Assumed ϕ'	30°	35°
Change of surcharge pressure, kN/m^2	−8·4	−8·4
Reduction of bearing capacity, kN/m^2	−204	−389
Residual bearing capacity, kN/m^2	372	812
F_b	1·55	3·38
F_S	1·13	1·37

The fact that scour is never uniform makes it almost certain that the collapse of each pier began with a yield of the ground at the toe of the sheet piling wherever the scour was deepest (usually near the upstream end); the release of lateral restraint would then allow a sudden settlement of the pier with consequent cracking of the arches and spandrels and a rapid progress towards collapse. The fact that five or six piers gave way in half an hour is surprising, however, and can only be explained by supposing either that the velocity of the current was still increasing rapidly or that the obstruction of the fallen arches caused such an increase of velocity round the remaining piers.

A bearing capacity failure. The conditions at Dumbarton Bridge present an important contrast. Babtie, Shaw and Morton (1960), from borehole surveys for the new bridge upstream, reported soft river alluvium of great depth and they had experience of similar material through the whole town area east of the old bridge. Allowable bearing pressures are generally assessed at no more than 67 kN/m^2, suggesting ultimate bearing capacity of little more than 160 kN/m^2. This applies only near ground level, the allowable pressure reducing with depth. Smeaton's report of a stronger 'crust' is therefore confirmed. If there were no offsets at the feet of the piers the actual pressure under them would be about 160 kN/m^2. As there are offsets, the actual pressure is less, but the width of the offsets is not known and so neither is the value of the pressure. There is also extensive protection of the river bed and the piers by piling and setting. Babtie, Shaw and Morton found serious settlement only at one pier in 1960, the evidence being a downstream tilt of 125 mm in the length of the pier.

The bearing pressures at Dumbarton have therefore always been much higher than modern design would permit but there has been no shear failure since the partial collapse in 1768. Unfortunately there are few records from 1768 and the amount of load applied before the collapse and its rate of application are not known. The small uniform surcharge provided afterwards by setting on the river bed appears to have been carefully maintained. The foundations at Hexham almost certainly had higher factors of safety against shear failure, but shear failure occurred after the sudden scouring of the river bed and rock slopes, which destroyed the surcharge of the girdles and probably also the toe support of the skirts.

Science and design. The only quantitative criterion of safety Smeaton quoted was a height of 'fall' through the bridge and the only parameters he calculated were velocities of flow. He made these calculations after the failure by a rough rule and, as there were better methods available which, with his undoubted skill in fluid mechanics, he could have used, we must conclude that he did not think bridge design worthy of, or amenable to, accurate quantitative predictions.

For assessing the strength of foundations there was no quantitative theory. Like other engineers of his day, Smeaton judged the type and condition of the soils with simple tools and then designed on the basis of his own previous experience. He took full advantage of the opportunity, which the contemporary speed of construction afforded, to change his design in response to observations of foundation behaviour during the construction period. At Hexham this was not enough to compensate for the error in his initial judgement that the 'gravel crust' could resist the erosive power of the river.

In his idea that the difference of water level upstream and downstream would tend to overturn the bridge there is an interesting hint of the slip-circle mode of collapse, now so important in soil mechanics theory; but on permeable soil there could be no overturning moment because the effective stresses in the ground would be independent of the height of water over it. The reason the bridge 'took a heel upstream' is to be found solely in the well-known concentration of scour at that end of the piers.

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