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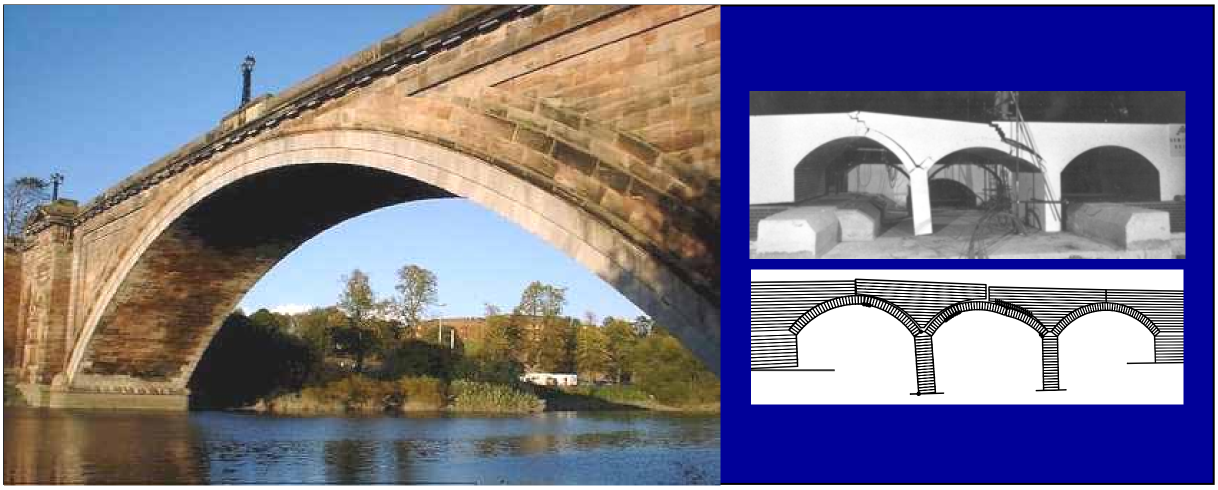

Masonry arch bridges

- construction, theory and recent UK research

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Masonry arch bridges: construction, theory and recent UK research

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1. Introduction

Masonry arch bridges have a special place in the hearts of many European citizens, as for example evidenced by the appearance of several fictitious 'historic' masonry arch bridges on the recently introduced Euro banknotes. As well as being of cultural and historic interest, in many European countries masonry arch bridges also form a vital part of the current transport infrastructure (e.g. approaching 50 percent of all bridge spans in the UK are masonry).

These notes briefly describe some of the key engineering aspects of masonry arch bridges: forms of construction, theories for how arches stand up (and fall down), and findings from recent UK research on the collapse behaviour of masonry arch bridges.

Whilst typically engineers are primarily concerned with repairing and maintaining the current bridge stock, their longevity and low maintenance requirements has, in 2003, led to the drafting of a new UK design code for new unreinforced arch bridges. This may lead to resurgence in their construction in the future.

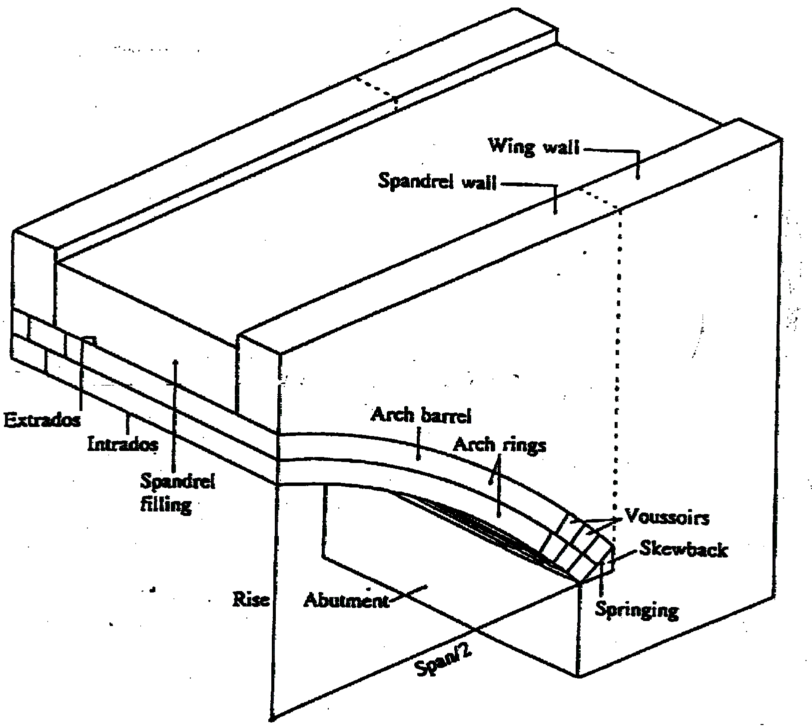


Figure 1 Arch bridge nomenclature

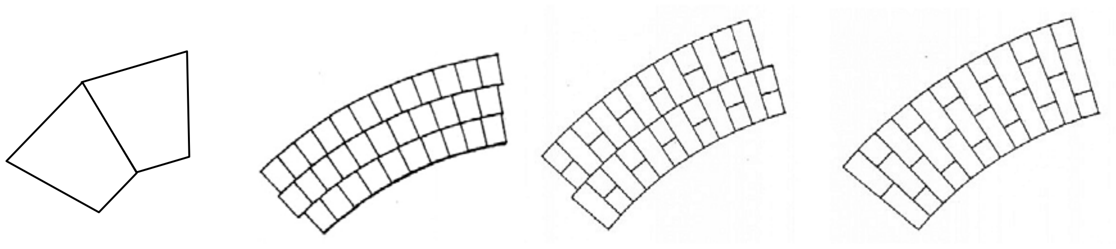
2. Arch bridge construction

2.1 Early bridges

It is probable that the earliest arches were built from brickwork rather than stone masonry, in the middle and near east[1]. These were almost certainly designed and built on a somewhat haphazard, trial and error basis (the term 'design' is used in a loose sense, to describe the thought processes which must have gone into the choice of the masonry required to form the arch), with 'rules of thumb' appearing over time to give the builder of a given bridge an idea of the proportions his bridge should be in order to successfully span the required distance. Such 'rules of thumb' would have had little to do with theoretical analysis as such, which came relatively recently, in the seventeenth century. For example, all major medieval masonry structures appear to have been built without reference to theoretical analysis, although there is evidence that in many cases small scale prototype models were built and tested first[2].

There were some notable developments in the art of masonry arch bridge building. For example, Perronet's[3] conceptual breakthrough regarding the thrusts in multi-span arch bridges resulted in modifications to the 'rules of thumb' and consequentially construction of such bridges with greatly reduced pier dimensions (he realised that the thrusts from adjacent spans could counter-balance one another).

Most of the ancient and medieval masonry arch bridges were constructed of stone masonry, the arch barrel being constructed of a number of rough or cut stone voussoirs, positioned in a single ring (Figure 2a).



(a) stone voussoir (b) brick: stretcher (c) brick: header/stretcher (d) brick: header

Figure 2 Typical masonry arch bonding patterns

Examination of stone blocks from the Stari Most indicated the use of iron dowels to fix blocks together. This seems to have been relatively common, since at least Roman times (Figure 3).

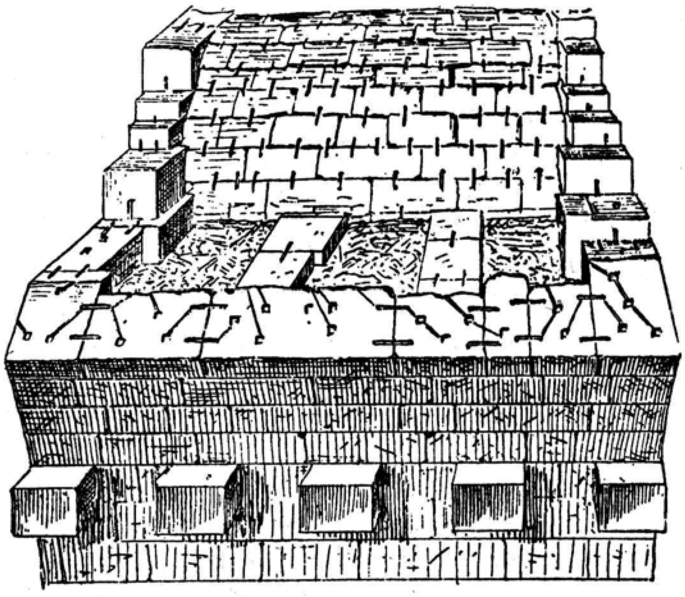


Figure 3 Reinforcement in extradas of Ponte Cestio, Rome[4]

It is interesting that the use of hand-worked iron rod when anchored in place using molten lead means that there is often little evidence of corrosion. More recently it is recorded that in the early nineteenth century 'hoop iron' was sometimes used to increase the strength of brickwork[5]. In the present day many researchers are investigating methods of retro-fitting reinforcement to existing bridges in order to increase strength. However, extreme caution must be exercised to ensure overall structural behaviour is not significantly changed.

2.2 Bridge construction in the industrial age

In the eighteenth and nineteenth centuries the urgent need for improved transport infrastructure in many European countries led to an unprecedented demand for new bridges. To reduce costs there was some standardisation, with many railway companies for example using a small number of standard designs for their bridges on a whole stretch of line[6]. Additionally canals and railways allowed the raw materials of bridges to be transported fairly long distances, sometimes resulting in the use of the same materials for a number of structures on a particular stretch of canal or railway.

Also in order to reduce the time and cost of building an arch many nineteenth century arch bridges were built of brickwork, rather than stone masonry (better mortars and more consistent bricks were being developed, and perhaps in the heyday of railway building, for example, there was a shortage of skilled stone masons). The barrels of brickwork arch bridges were often built up in rings. In the UK, because the use of 'header' bonded brickwork in arches (Figure 2d) generally requires that specially manufactured tapered bricks are used, the use of 'stretcher' bond was generally more popular (Figure 2b), the latter bond having no headers connecting adjacent sections of brickwork. Incidentally, the practice of building brick arches sequentially in courses, without interconnecting headers, is by no means a recent practice; ancient Egyptian[1] arches were sometimes built in this manner.

In the majority of relatively short, single span bridges, spandrel walls were built at the edges of the barrel, and the resulting spandrel void backfilled to provide a level surface for the road or railway. There is a good deal of evidence to suggest that the restraint afforded by the backfill material at either end of the span can significantly strengthen a given bridge. There is also evidence to suggest that the spandrel walls will often be able to provide additional restraint to the arch barrel under loading. These important points will be raised again later.

However, in order to reduce the dead weight of many long span or multi-span bridges the spandrel voids were often not backfilled. Instead internal spandrel walls (sometimes called longitudinal or 'sleeper' walls) were constructed to transfer applied loads onto the arch barrel. In the case of multi-span bridges these walls are likely to have the very important additional effect of propping apart the barrels of adjacent spans.

It has been stated[7] that the masonry arch was obsolescent by the mid-nineteenth century. However, whilst wrought iron, and later steel and reinforced concrete bridges did in turn become the state-or-the-art materials for bridges, numerous masonry arch bridges were built in the late nineteenth century and many in the early twentieth century. For example the Nairn viaduct over Culloden moor, the longest masonry arch bridge in Scotland and built of red sandstone, was not built until 1898.

Elsewhere in Europe Sejourné's epic 85m span open spandrel Pont D'Adolphe bridge was opened in 1903, whilst in China very long span masonry arch bridges (up to 120m in span) have been constructed in the latter half of the twentieth century.

2.3 Notable UK masonry arch bridges

2.3.1 Pontypridd

With a span of approx. 45m, this was believed to be the longest stone span in the world when it was completed in 1755. However, perhaps the most interesting feature of the bridge was the present bridge was actually constructed immediately after two previous unsuccessful attempts by the builder, William Edwards, to bridge the gap – a river. An initial timber bridge solution was washed away in a flood whilst the next solution – a stone arch bridge - collapsed immediately following decentering, because it was of the wrong shape to carry the bridge self weight.

Edwards' eventual successful design, which stands to this day, incorporated openings over the haunches (Figure 4). This reduced the weight in these regions, and so allowed a line of thrust to be fitted entirely within the masonry. The bridge is quite slender (arch ring thickness : span ratio 1:56), and recently a scale-model of the bridge has been rebuilt and tested in a centrifuge, and the bridge also analysed using modern non-linear elastic finite elements[8].

2.3.2 Chester

The Grosvenor bridge in Chester was also believed to be the longest stone span in the world at approx. 60m when it was completed in 1833. The fact that this remains the longest stone span in the UK indicates that subsequently other materials and/or structural forms were favoured for such crossings of this size.

Figure 4 Engraving of Pontypridd bridge, Wales, UK[9]

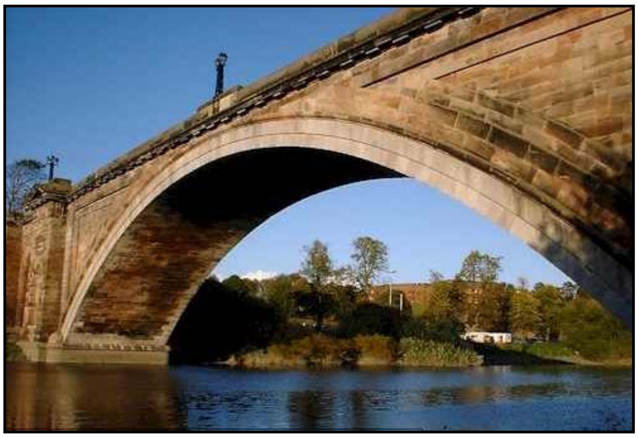


Figure 5 Grosvenor bridge, Chester (UK)

Figure 6 Ribblehead viaduct (UK)

2.3.3 Ribblehead viaduct

This railway structure, although perhaps famed as much for its location (across bleak moorland in the Yorkshire Dales region of the north of England) was completed in 1875. It is probably the most well known masonry arch bridge in Britain, and, despite its comparative modernity, is now a 'scheduled ancient monument'. The structure comprises 24 arches, is 400m long and up to over 30m high. It is however representative of many similar railway viaducts constructed from the early nineteenth century right through until the early twentieth century in the UK. Thus a stone façade hides stretcher bonded brick arches, internal spandrel walls are used instead of soil filling and the piers are quite slender (though every six spans larger piers are used to allow the arches in the structure to be constructed in groups of six and also to prevent all 24 arches from collapsing should one arch be removed). A major maintenance was recently carried out, with new waterproofing installed and some stone blocks in the piers being replaced and/or stitched together. The structure is now expected to last for many decades without significant attention.

3. Historical development of the theory of arches

3.1 Early theories

Robert Hooke appears to have been the first to properly understand how arches stand up. He noted in 1675 that 'as hangs a flexible line, so, but inverted, stands the masonry arch' (this was encrypted in the form of a latin anagram, apparently to avoid Hooke's great rival of the period, Isaac Newton, from stealing the discovery). The inference is that a hanging simple cable once frozen will carry its own weight if inverted (this ignores stability problems). If the inverted cable is symmetrically increased in thickness then it continues to carry its own self-weight without inducing any bending. Clearly the application of an external load will induce bending which can be idealised internally as an eccentric thrust. Provided this thrust line everywhere lies within the arch, then the arch will remain stable. Much later Barlow[10] demonstrated that there was no unique thrust line associated with a stable arch but that there were many possibilities.

In the late seventeenth century Hooke's finding must have confirmed what was probably already qualitatively appreciated by many, namely that to remain stable an arch must be correctly shaped in relation to the pattern of the applied loading. Hooke's theory was later famously used by Poleni to demonstrate the stability of the dome at St Peters, Rome, after the dome had become cracked into 'orange segments'. However, in less cosmopolitan parts of the UK the new theory appears not to have been universally used for many years (e.g. in the case of Pontypridd bridge – see section 2.3.1).

Additionally, using Hooke's theory it is easy to demonstrate that whilst in the case of a short-span bridge foreseeable live loading may deflect the profile of the line of thrust significantly (and hence has the potential to endanger the stability of the structure), in the case of a long span bridge foreseeable live loading is likely to deflect the profile of the line of thrust be a negligible amount, indicating that live loading is negligible in comparison to self weight effects (and hence will be unlikely ever to endanger the stability of the structure).

3.2 Application of elastic theory

Navier's work[11] showed that for linear elastic materials where plane sections remained plane, tension could be avoided by ensuring that the thrust line lay within the middle third of the section. Combining these two pieces of work led to the well-known middle third rule of design which aimed to eliminate tension in the masonry and thus avoid any cracking. It also afforded analysts the luxury of modelling the arch as an elastic continuum.

Castigliano[12] applied the theorems of minimum strain energy to the arch. The position of the thrust line was determined and then checked to see that it lay within the middle third; if this was not the case then the tensile zone was 'removed' and the calculation iterated until no tension was present at any point in the arch.

The main advantage of an elastic analysis is that stress levels and deflections can be calculated - how meaningful these are is open to much debate but it has to be conceded that they provide a 'feel' for their probable values. However, it is universally accepted that masonry arch bridges are cracked - even before the centering is removed. Thus at the opposite extreme of considering masonry as an elastic continuum is the analytical model of regarding masonry as a particulate assemblage of inelastic stones or bricks, the irregularities between which being filled with a mortar that will permit the transfer of compressive force whilst preventing sliding and not allowing tensile stresses to be transmitted. This approach is similar to that favoured by the present author (see section 4.4)

4. Recent UK studies of the behaviour of masonry arch bridges

In the mid 1980's in the UK the announcement that maximum vehicle weights to be allowed on the road network were to be increased led to fears that the strength of existing masonry arch bridges may be inadequate. Hence an extensive programme of research into the behaviour of masonry arch bridges was initiated. As part of this programme, a number of load tests to collapse on field bridges and full-scale model masonry arch bridges were performed, and, in parallel, the efficacy of various analysis methods were investigated.

4.1 Load tests to collapse on field bridges

The Transport and Road Research Laboratory (TRRL, now TRL) in the UK carried out, or contracted others to carry out, a series of 10 tests on redundant arch bridges in the late 1980's and early 1990's. It was intended that all the bridges would be tested to failure, though in the case of two of the ten bridges tested this was found not to be possible. Most bridges failed in four hinge mechanisms (Figure 7), though some of the bridges were reported as failing by 'three hinge snap through' or in 'compression' (material failure). It was likely that many of the bridges tested were restrained considerably by their attached spandrel walls and/or masonry backing. Further information on these bridge tests are provided in Page[13].

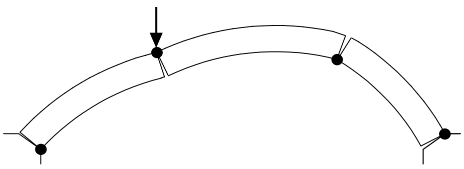


Figure 7 Four-hinge failure mechanism

With the benefit of hindsight, more pre-test investigation work should probably have been performed to better characterise the internal construction details and material properties. This would have been useful in providing a more comprehensive data set for use by analysts who have attempted to model the behaviour of the bridges under load (in the event analysts argued over the choice of realistic values for some of the poorly characterised parameters).

Additionally some critics questioned the choice of loading regime (use of a rigid beam across the full bridge width, typically at quarter span). The suggestion was that bridges with inadequate abutments may have been more susceptible to loading near the crown, and that use of a rigid loading beam across the full bridge width is unrepresentative of the concentrated vehicle wheel loadings found in practice (which might be more onerous).

4.2 Load tests to collapse on laboratory bridges

A significant advantage of laboratory bridge tests is that internal construction details and materials are known. This potentially makes laboratory tests more useful than field tests for testing the efficacy of alternative analysis methods. A possible disadvantage is that when designing such tests there is sometimes a temptation to simplify the construction details of the bridges to such an extent that the tests become completely unrepresentative of reality.

4.2.1 Bolton single-span bridge tests[14]

Melbourne and Gilbert reported tests on seven 3m and 5m span single-span bridges. The main objectives of the test programme were (i) to determine the effect on carrying capacity of the presence of through thickness bonding in multi-ring brickwork arch bridges, and (ii) to determine the effect on carrying capacity of the presence of spandrel walls. Thus all but one of the bridges were built with one or both of the (laboratory simulated) defects of ring separation (delamination) and spandrel wall detachment. An additional objective was to gain an appreciation of the behaviour of the backfill as the bridges were loaded. Figure 8 shows a typical 5m span bridge awaiting testing.

The defect of ring separation was achieved by using dampened sand in place of mortar between the rings. Spandrel wall detachment is a defect occurring in practice when the spandrel wall moves laterally, detaching itself from the barrel, usually taking part of the barrel with it and thus allowing the central portion of the barrel to move past the wall.

Figure 8 A 5m span laboratory bridge awaiting testing (in-situ stress testing equipment in foreground)

Failure modes are shown on Figure 9. The most important finding was that ring separation dramatically reduces bridge strength when a stretcher bonding pattern for the arch barrel is used (Figure 2b).

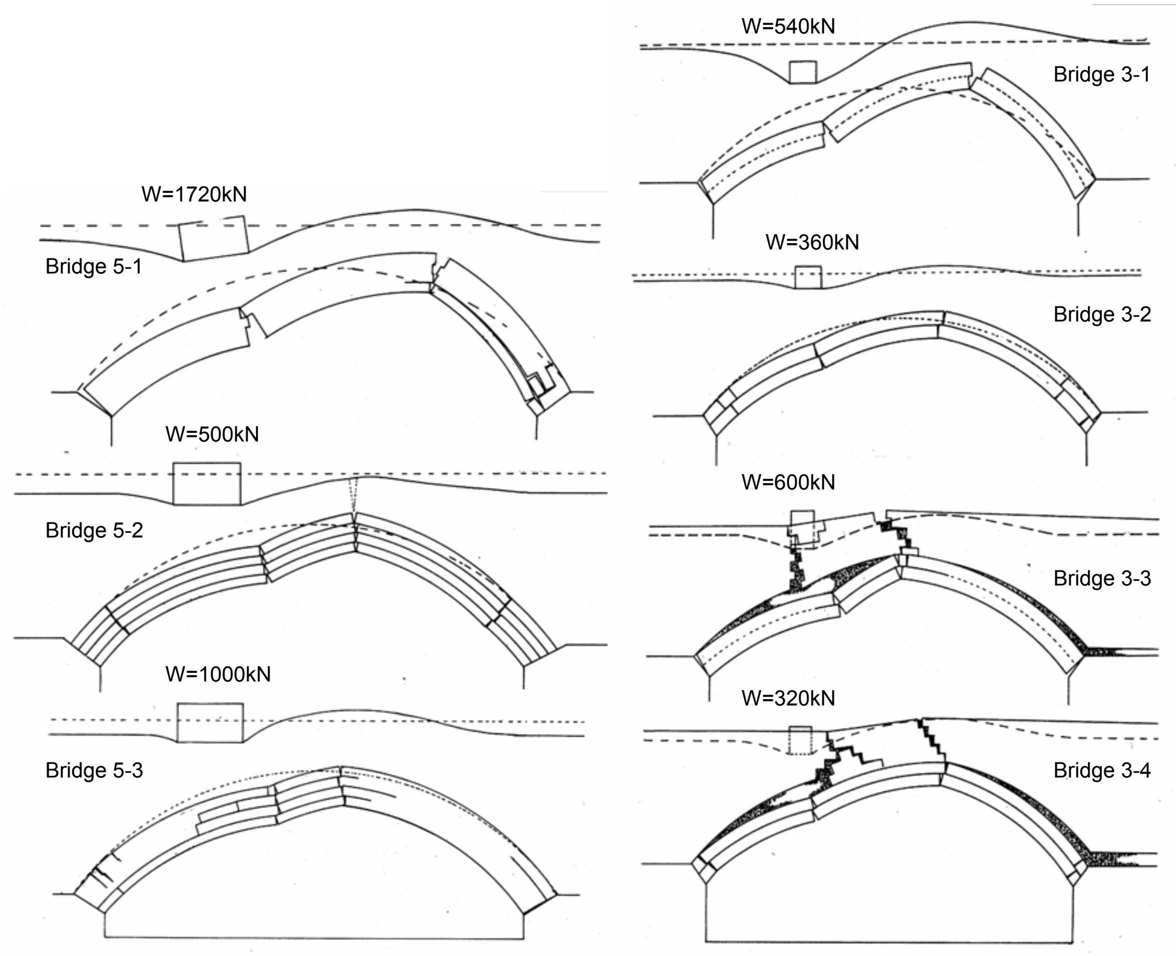


Figure 9 Single-span bridge failure modes

4.2.2 Bolton multi-span bridge tests[15]

The main objectives of the tests were thus (i) to determine the typical failure modes of multi-span arch bridges and (ii) to determine the multi-span carrying capacity compared with that of a single span bridge. Additional objectives were to determine the influence of spandrel walls and backfill on bridge behaviour. It should be borne in mind that because the likely failure mode for the multi-spans was not known prior to the initial tests, the critical loading position was obviously also unknown. For this reason the first two bridges were loaded at quarter span (of the centre span), to coincide with the critical position for most single span arch bridges.

The test programme comprised a series of three large-scale model multi-span bridges. Each bridge contained three 3m spans, each of the same geometry to 3m span single span bridges described in the previous section (span to rise ratio of 4:1, ring thickness=215mm). Bridge nos. 1 and 3 were nominally identical, both built with attached spandrel walls. Bridge no. 2 was built with walls which were detached from the arch barrels of each of the spans, but which were constructed on the same piers and abutments supporting the arch barrels. The intermediate piers were designed to have similar pier height : pier thickness and arch span : pier thickness ratios to those used in practice during the nineteenth century. Figure 10 shows the collapse mechanisms of the bridges.

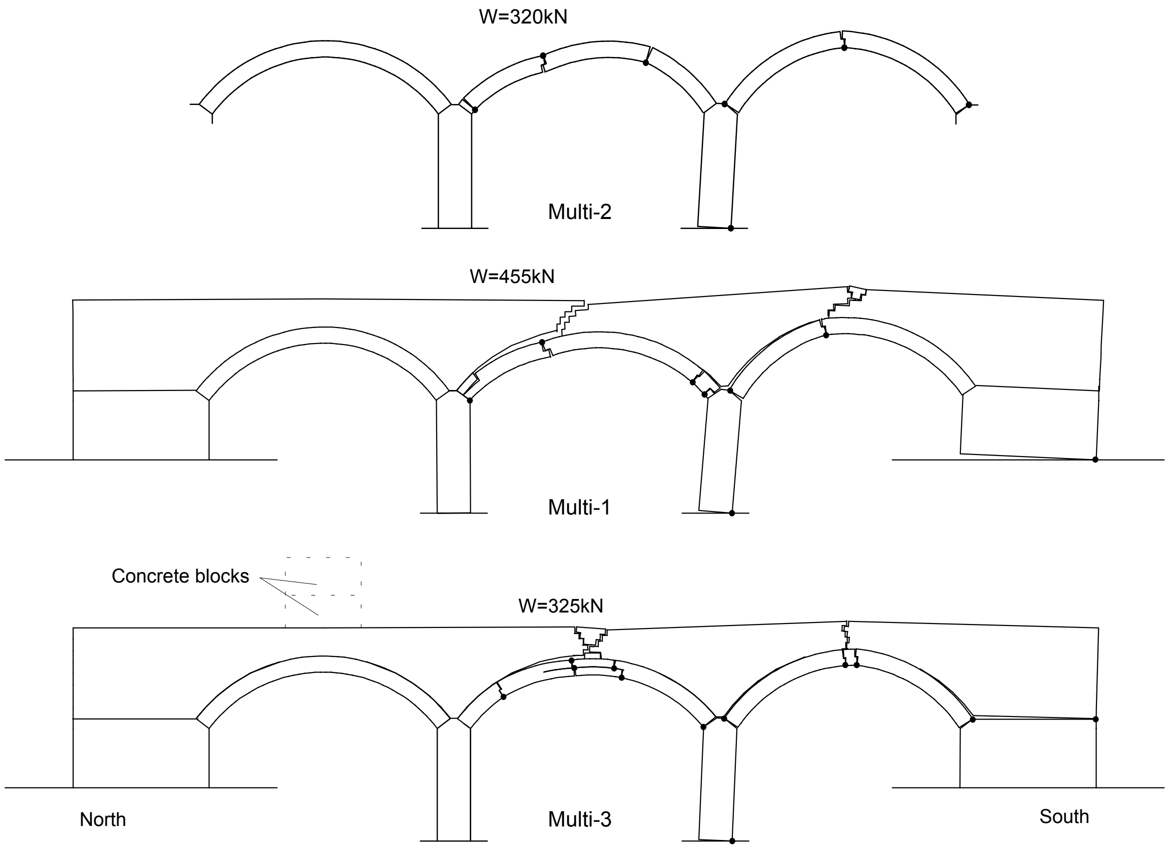


Figure 10 Multi-span bridge failure modes

Comparison of the failure loads of the bridges indicates that the critical loading position for multispan bridges will typically be in the vicinity of the crown. Furthermore, it is evident that the presence of spandrel walls may help enhance carrying capacity.

4.3 Limit analysis methods

Heyman[7] pointed out that despite the lack of plastic moment capacity of the sort steel and reinforced concrete structures possess, plastic methods of analysis can be applied to masonry gravity structures, such as piers and arches. To demonstrate this, compare and contrast the response of a masonry pier and steel tower both subject to a lateral load *P* (Figure 11).

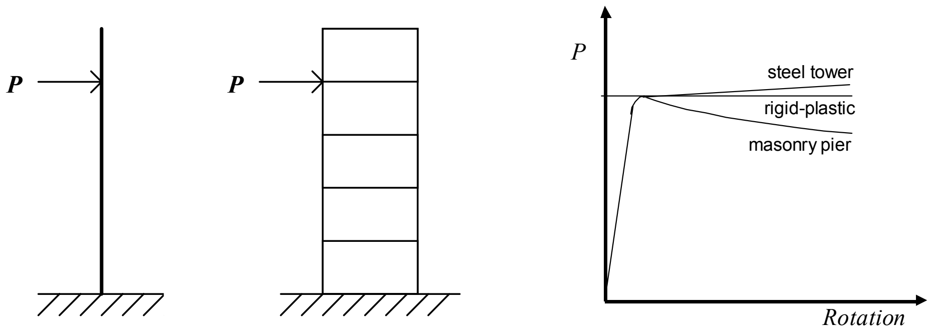


Fig 11 Laterally loaded (a) steel tower, (b) masonry pier, and (c) idealised response curves

The thickness and self weight of the pier mean that there clearly is some resistance against overturning. Thus the masonry pier could conceptually be considered as being identical to a thin steel tower but with a finite moment capacity (M_p) that varied with height. This variable M_p value must equal the normal force at a given cross section multiplied by half the pier thickness. However, as in masonry structures the moment of resistance effectively varies continuously, it is clear that bending moment diagrams can be difficult to interpret. Instead it is normally more useful to plot the eccentricity *e* of the compressive force *P*, or *thrust*, at each cross-section.

Thus in the context of masonry:

(i) The yield condition of plastic analysis may be deemed to be satisfied providing the line of thrust lies entirely within the masonry.

(ii) The mechanism condition of plastic analysis may be deemed to be satisfied providing the line of thrust alternatively touches the inner and outer edges of the masonry blocks a sufficient number of times.

Also, in the context of masonry the upper bound theorem of plastic analysis can be restated as: *if a line of thrust satisfies the mechanism and equilibrium conditions, then the applied load will be an upper bound on the true plastic collapse load.* Similarly the lower bound theorem can stated as: *if a line of thrust satisfies the equilibrium condition and also lies entirely within the masonry then the loading is a lower bound on the true plastic collapse load.*

4.4 Rigid block (limit) analysis method

In practice the geometry of masonry arch bridges is sufficiently complex as to preclude hand analysis in most cases. Whilst it is possible to automate hand-based plastic analysis methods, a more generally applicable computer-based method of analysis has now been developed. This method, sometimes referred to as the rigid block analysis method, additionally permits modelling of sliding failures.

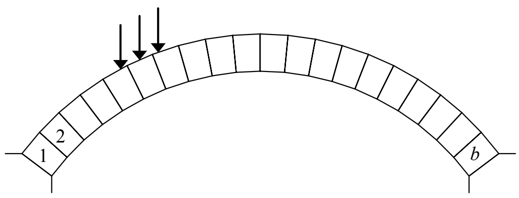


Figure 12 Loaded arch rib

The following is essentially the joint equilibrium rigid block analysis formulation put forward by Livesley. In comparison with other formulations (e.g. Gilbert and Melbourne[16]), the formulation produces a large number of constraints and variables, but the total number of non-zero elements will generally be relatively small, which means that it can be solved very efficiently using modern linear programming (LP) algorithms.

Thus assuming there are *b* blocks and *c* contact surfaces, the problem may be stated as follows:

$$\text{Max } \lambda$$

subject to

$$\mathbf{Bq} - \lambda \mathbf{f}_L = \mathbf{f}_D$$

$$\left.\begin{array}{l} m_i \le 0.5 n_i t_i \\ m_i \ge -0.5 n_i t_i \\ s_i \le \mu_i n_i \\ s_i \ge -\mu_i n_i \end{array}\right\} \quad \text{for each contact, } i = 1,\ldots,c \tag{1}$$

where λ is the load factor, **B** is a suitable ($3b \times 3c$) equilibrium matrix derived from the geometry of the structure and **q** and **f** are respectively vectors of contact forces and block loads. Thus $\mathbf{q}^{\mathbf{T}} = \left\{ n_1, s_1, m_1, n_2, s_2, m_2, \ldots n_c, s_c, m_c \right\}$; $\mathbf{f} = \mathbf{f}_D + \lambda \mathbf{f}_L$ where $\mathbf{f}_D$ and $\mathbf{f}_L$ are respectively vectors of dead and live loads. Contact and block forces, dimensions and frictional properties are shown on Figure 13. Using this formulation the linear programming problem variables are clearly the contact forces: n_i, s_i, m_i (where $n_i \ge 0$; s_i, m_i are free variables).

In the above formulation sliding is modelled as associative, or 'saw-tooth' type friction (that is to say, dilatancy accompanies separation). Though friction is often not particularly important in 2D arch bridge problems (as hinging failure modes often predominate), various workers have recently modified the above mathematical formulation so as to treat non-associative friction problems[17,18].

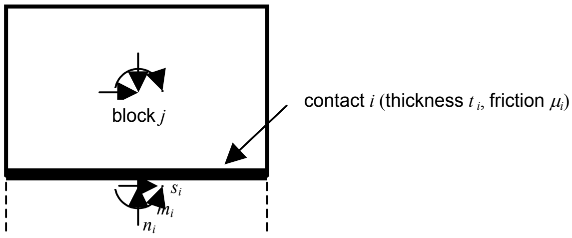


Figure 13 Block and contact forces

4.4.1 Soil-structure interaction

In practice the strength of masonry arch bridges backfilled with soil is enhanced (a) by dispersion of the applied load through the fill, and (b) by horizontal passive type restraining pressures. The latter are often assumed to increase linearly with depth, with the coefficient K_p determining the ratio of horizontal to self weight pressures. It has been found that in complex geometrical assemblages an effective, although rather crude, method of incorporating horizontal fill pressures in the rigid block method of analysis is to introduce so-called fill elements (Figure 14).

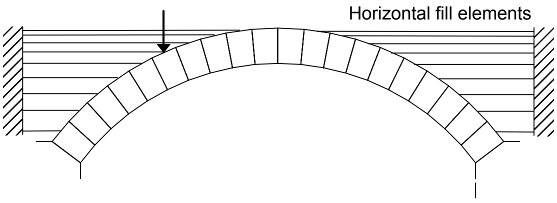


Figure 14 Arch restrained with uniaxial fill elements

These fill elements compress at constant force (equal to K_p × vertical pressure × area), but are unable to extend. This ensures that pressures are mobilised in the correct sense ('active' pressures are usually relatively small, and so are often neglected). Fill elements are readily added to the LP problem formulation (1). However as in practice fill pressures may only be mobilised when structural deformations are large, it could be argued that a gross displacement analysis[19] is required (the analysis described to date assumes infinitesimal deformations). The issue of soil-structure interaction is a field which requires further study.

4.4.2 The RING software[20]

Despite the advantages of the rigid block method, until recently few researchers or practitioners have had access to software based on the rigid block analysis formulation. To remedy this, rigid block analysis software developed by the author for personal research use was recently developed into usable engineering software. The resulting software has been named RING, and is now freely available via the web (`www.shef.ac.uk/ring`). Figure 15 shows a sample screen display.

RING was originally developed to assist with the interpretation of the results from laboratory tests described in section 4.2. Since the original publication of the work in The Structural Engineer[14,15,16], the program has been enhanced to include, amongst other things, material crushing[21] and more realistic models of the dispersion of the applied load through the backfill. In Table 1 sample updated RING analysis results are presented alongside experimental test results (for bridges with detached spandrel walls). To obtain these latter RING results a standard coefficient of lateral earth pressure has been specified (rather than the back-substituted experimentally recorded pressures, as used in the original publications). This assumption potentially leads to an over-prediction of the magnitudes of the local pressures realisable in zones of the soil mass where strains small (e.g. near to the hinge at the extrados springing in a single span bridge) but it is clear from Table 1 that, using a single K_p value, reasonable results can be obtained for a variety of different bridge geometries.

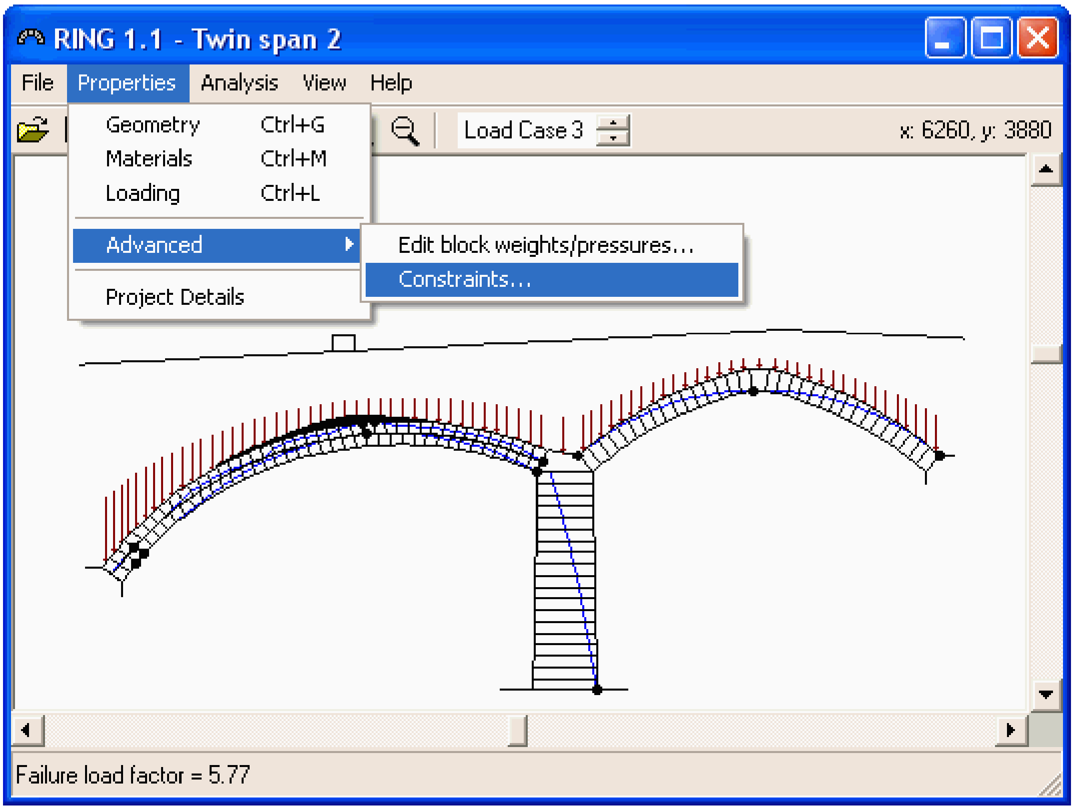


Figure 15 RING software: sample screen display

Bridge	Description	Expt. collapse load (kN)	RING analysis			Theoretical / Expt. collapse load
			Limiting load dispersion angle (degrees)	Coefficient of passive earth pressure, K_p	Theoretical collapse load (kN)	
3-1	3m single span	360	45	4.5*	243	0.68
3-2	3m single span; debonded arch rings	540	45	4.5*	550	1.02
5-1	5m single span	1720$^+$	45	4.5*	2238	1.30$^+$
5-2	5m single span; debonded arch rings	500	45	4.5*	482	0.96
Multi-2	3m triple span	320	45	4.5*	358	1.12

*approx. 1/3 of the full classical passive pressure coefficient indicated by the measured ϕ value of 60°

$^+$the experimental collapse load of this bridge was reduced by the sudden onset of partial ring separation

Table 1 Sample comparison between Bolton laboratory and RING collapse loads

It should perhaps be noted that RING is not capable of identifying the onset of ring separation, a quasi-brittle phenomenon (hence the over-prediction of the strength of bridge 5-1; see Table 1). A more advanced analysis technique is required in this case (e.g. non-linear finite element analsyis). However, given that the nature of the adhesion between rings will normally be unknown in the case of real bridges, it is doubtful that application of highly complex numerical models will be justifiable in most cases.

5. Conclusions

These notes have outlined some of the main issues related to structural engineering aspects of masonry arch bridges. Recent research has improved our understanding of how single and multi-span bridges perform under load. However, we still have much to learn. Currently there are bridges in the field which, when analysed, appear to be unable to carry even light vehicles, yet in practice show no visible signs of distress even when carrying heavy vehicles. This is probably principally a result of our current poor understanding of the way arches interact with surrounding fill material. Nonetheless, problems with applying new scientific methods of analysis has meant that, despite all the improvements in analysis methods which have taken place since the mid 1980's, UK highway and rail authorities still advocate the use of a semi-empirical method of assessment developed during World War II (the so-called MEXE method).

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Brief Curriculum Vitae

Matthew Gilbert graduated with a first class honours degree in Civil and Structural Engineering from the University of Sheffield in 1989. Following a brief period spent with a civil engineering contractor he returned to academia, studying the behaviour of masonry arch bridges (awarded PhD by the University of Manchester, 1993). He then joined the University of Sheffield, employed initially as a Research Associate and subsequently as John Carr Lecturer. In 2003 he was awarded a prestigious 5-year EPSRC Advanced Research Fellowship (the first time this has been awarded to any member of a UK 'Civil Engineering' department for many years).

He has a strong interest in the static and dynamic performance of masonry structures, particularly masonry arch bridges. Additionally for about a decade he has been researching the impact behaviour of masonry walls and the dynamic fracture characteristics of masonry joints. In total he is the author or co-author of more than 60 research papers, reports and book chapters on masonry structures.

As part of his EPSRC Advanced Research Fellowship work he will focus on developing novel computational limit analysis and design synthesis methods for application to a wide range of structural types (including masonry structures). The situation is that at present to quickly estimate the ultimate strength of a structure, practitioners often have to rely on either highly simplistic hand type calculations or on highly complex non-linear elastic modelling techniques, with no rational 'middle way'. Additionally, the initial design stage for structures such as canopies and bridges tends to be carried out in an ad-hoc manner, with individual engineers' intuition typically being used to initially determine optimum member layouts. Both these issues are being addressed.

He is also a chartered civil engineer and a recipient of the Institution of Structural Engineers' Husband Award for a paper on multispan masonry arch bridges published in their journal. Organisations sponsoring his research have to date included EPSRC, Network Rail and Buro Happold (recipient of research awards totalling >£650K since 1998).

Selected recent publications:

Gilbert, M. and Tyas, A., 'Layout optimization of large-scale pin-jointed frames', *Engineering Computations*, Vol. 20, No. 8, pp. 1044-1064, 2003.

Gilbert, M., Hobbs, B, and Molyneaux, T.C.K., 'The performance of unreinforced masonry walls subjected to low-velocity impacts: experiments', *International Journal of Impact Engineering*, Vol. 27, pp. 231-251, 2002.

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