# British Practice in Arch Bridge Assessment

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The bridge at Aldochlay in the Strathclyde region is small and is constructed from random rubble masonry. It shows no sign of distress, but it became clear very early in the assessment process that different types of analysis yielded very different answers for this structure. The example describes the root causes of some of these conflicts. Bargower is a semicircular bridge constructed from dressed sandstone and has a 10-m span. Its behavior is influenced by various effects of soil pressure and soil-structure interaction that are not well represented in many analytical approaches.

The bridges of Britain are not unique but are quite unusual in that a very large number of bridges with masonry arches are still in use on the highway system. India probably has a larger stock of arch bridges, but it is a much larger country. China certainly has an enormous stock, but knowledge of its assessment procedures is only just filtering out of the country. The ravages of land-based war in Europe means that the European mainland arch bridge stock is drastically reduced and nearly all bridges are modern.

In Britain we have a special interest in arch bridge assessment. Of the stock of approximately 70,000 arches on the highway system, by far the majority were built before the introduction of any loading standards. It is therefore clear that they were designed entirely empirically. There are considerable regional variations in the style of bridges and also more local variations in the quality of workmanship and the standard of design. Nonetheless, that proportion of the arches that did not collapse early in their lives and that are still carrying traffic has proved well able to sustain the steadily increasing loads imposed on them, provided they are reasonably well maintained. The Department of Transport regulations in Britain require a major inspection and assessment at least every 6 years for all trunk road bridges, and the same rules are usually applied to bridges on locally owned roads.

Interest in arch assessment techniques tends to run in 30-year cycles, with a long period of consolidation using the techniques that have been developed followed by a burst of effort. There has been a substantial amount of activity on arch assessment in Britain since 1980, largely influenced by Heyman's (*1*) work on the application of plastic theorems to arches. His proposals were incorporated in the Department of Transport's Departmental Standard BD21/84 (2) as an alternative to the long-established empirical method originally developed by Pippard in the 1930s. Working engineers were, on the whole, happy with the application of the MEXE method after 40 years of use, with no known failures of bridges that had passed assessments.

The new approach offered in BD21/84 (2) was slightly modified from Heyman's and attempted to present a limit state method of assessment. Engineers were much less confident that the limit state proposed would yield both safe and satisfactory results. Their concerns are heightened by the fact that after 10 years and probably nearly £2 million (£1 ≃ $1.60) worth of research work, the clauses on the use of Heyman's method were deleted from the updated version of the standard that

appeared in 1993. Indeed, an appendix casts considerable doubt on those computerized approaches that were based on Heyman's methods. It is perhaps surprising that similar doubts were also cast on the range of finite-element methods that have been developed since 1990.

Although this paper is written by the authors of one of the programs based on Heyman's techniques, an attempt will be made to present a reasoned view of the tools available for arch assessment and the way that they might be applied and to offer suggestions as to how further progress might be made.

## Process of Assessment

The assessment of the capacity of a masonry bridge requires three elements:

1. A field inspection,
2. A desktop study, and
3. Reflection and the application of judgment by a competent and experienced engineer.

There is considerable desire to remove the need for the third element, but it will be demonstrated that it is extremely unlikely that it will ever be possible to do so.

### Field Inspection

Three things are required. The first and most obvious is a geometric survey. Arch bridges depend on their shape for their strength to a greater extent than any other form of bridge. Ideally, the assessing engineer wants to know the basic geometry of the intrados or soffit of the arch, that is, the span, the rise, the shape of the curve, and the plan shape (whether square or skewed). The assessing engineer would clearly like to know the height and thickness of the abutments and the nature of the foundations on which those abutments stand. Knowledge of the thickness of the arch ring and any variation in that thickness over the span is also important, as is knowledge of the thickness and height of the spandrel walls and the depth and quality of the fill, which brings a steeply curved arch up to a reasonably level surface for the road.

Many of these details are completely hidden. In particular, it is extremely difficult to obtain dimensions for the abutment and ring thicknesses and for the nature and quality of the foundations. BD21/84 (2) and BD21/93 (3) avoid the most difficult of these problems by saying that if there is no sign of distress in the abutments, then they should be assumed to be adequate. This seems a very strange response in the light of the concern that is expressed about the performance of the arch itself.

Once the basic geometry is noted, the engineer will proceed to consider the condition of the bridge. The masonry units, brick or stone, may have deteriorated with time, particularly if moisture has been allowed to penetrate from the road surface through the fill and into the masonry. Some of the poorest stones and bricks used in arches deteriorate progressively with time, even when they are kept in relatively benign condition. The mortar in the joints between the masonry units presents rather more problems. Although on some bridges that the authors have inspected it is still possible after 300 years to see the impression of the formwork on the mortar between the stones, on others the mortar has been completely eroded. It cannot be emphasized too strongly that the mortar is at least as important as the masonry units to the performance of an arch, not least because the forces must flow through the structure, and if there is a gap between two adjacent stones, then no force can pass between them.

Another important question for the field inspector is whether the bridge is cracked in any way. Cracks in the arch barrel are regarded as particularly important. Transverse cracks, except for a single crack very close to the crown, are very uncommon and are in any case unlikely to be particularly important. Longitudinal cracks, however, indicate some sort of breakdown of the structural system. They occur most commonly at the inside face of the spandrel walls and, particularly in railway bridges, between opposing traffic lanes. Cracks of this nature can hardly be caused by direct tension in the masonry. It is sometimes suggested that the pressure of fill on the inside of the spandrel walls will push the walls outward, and they will sometimes take the arch with them and crack it. Bearing in mind that the spandrels are supported off the arch usually with a very soft mortar, this mechanism seems extremely unlikely. However, the flexural stiffness of the spandrel walls and the fill behind them is very different, and the result is that the arch attempts to deform between the spandrel walls and is held in shape by the walls themselves, generating large displacements and enormous strains and stresses that cause longitudinal cracks. Once the cracks have formed, the broken edge of the arch may well push the spandrel walls outward. The spandrel walls may bulge as a result of pressure from the fill. This problem is not a matter for the present paper.

Diagonal cracks are rare but are of rather more concern since they can only occur as a result of some form of twisting deformation of the arch barrel. Whether the twist takes place between abutments that remain firm or whether the abutments themselves move is a matter for inspection and measurement. It is, of course, ex-

tremely important that the inspecting engineer exercise judgment, deciding what features of the structure are important and what can be ignored.

Some data that the inspecting engineer would very much like to have can only be obtained at considerable expense and probably by doing damage to the structure. It is possible to take cores from a bridge to ascertain unit strength and so develop the strength and, indeed, calculate elastic properties for the masonry. If these cores are well preserved and the assessing engineer is certain that they will not change with time, then coring may be justified. However, the enthusiasm some assessing engineers have shown for knocking holes in bridges in this way must be questioned.

A full understanding of the internal geometry of the structure and of the properties of the fill material can only be obtained by digging trial pits. Drilling or coring through the arch barrel or spandrel walls is notoriously ineffective in providing adequate, accurate data.

## Analysis

The constraints on the data available from a field survey must be borne in mind in deciding what analysis might be carried out. If a method demands particular items of data and these data are not available, reasonable estimates must be made. The sensitivity of the analysis to them must then be investigated. An engineer's experience in this is extremely important since the sensitivity of different shapes and sizes of bridge to different items of data will vary and a complete parameter study cannot be carried out on every structure that is assessed. In the end, an assessment is a matter of developing the confidence of the engineer in the structure that the engineer is assessing.

Most assessments are now carried out by consulting engineers who, because of the nature of their work, must carry insurance. The cost of that insurance is critically dependent on the engineer's success. At the same time, the engineer must submit to fee competition in obtaining work and therefore must minimize the amount of effort that he or she puts into a particular assessment.

The authors therefore suggest that analysis for assessment should be a matter of exploration and confidence building and should progress from simple, relatively understood techniques to more complex ones only if the engineer requires more support to improve his or her confidence. This approach is common practice among many engineers, but it is discouraged by the working of the design standard used.

### *Analytical Tools*

#### MEXE Analysis

The MEXE routine completes the assessment of an arch on one piece of paper. For most engineering groups this is on a standard form. The engineer begins by inserting leading dimensions. A nomogram or formula gives a capacity for a perfect arch (the Provisional Axle Loading, or PAL). A series of reduction factors is then applied to take account of

1. a less than "perfect" shape,
2. a ratio of ring thickness to fill depth that differs from the assumed value,
3. quality and geometry of the masonry units and the joints between them, and finally,
4. an entirely empirical condition factor.

The result of the final assessment may be 20 percent or less of the initial value extracted from the nomogram.

The nomogram itself is based on an elastic analysis. It was assumed that the arch is completely elastic and is supported on a pin at each end, that the fill only acts as dead load, that the critical position for a load is at the center of the span, and that the only important control is the compressive stress in the arch ring. With time it has become increasingly clear that this model does not in any way represent the true behavior of an arch. Nonetheless, the results obtained have proved satisfactory, although no one knows whether the actual factor of safety achieved is 1.1 or 11.

In BD21/93 (*3*) an updated version of this procedure (described as the computerized Pippard/MEXE method) is recommended. It uses a frame analysis program to analyze an elastic arch ring on two hinges. The authors believe that the benefits of this procedure are wholly imaginary and that the dangers are considerable. The MEXE procedure has stood the test of time, whereas no attempt has been made to check that the new method either leads to results that are similar to those of MEXE or that it produces conservative results for a range of bridges that are demonstrably in sound condition. The system has, however, been calibrated against a series of full-scale tests to destruction that are of questionable value for this particular application.

#### Equilibrium Analysis

In 1676 Robert Hooke "solved" the problem of the functioning of a masonry arch (*4*). Essentially, he said that an arch works in the same way, but inverted, that a chain supports a system of loads. For 300 years now engineers have sought to find the pattern of the chain for a particular system of loads and thereby prove that,

the chain being contained with the depth of the arch material, the arch is sound. Through the 19th century and a large part of the 20th century many engineers have attempted to find a particular solution to this hanging chain problem. Barlow (*5*) in 1846 demonstrated that the attempt was doomed to failure but in any case was unnecessary. It is sufficient to show that a particular polygon or line of thrust can be contained within the arch without knowing precisely which line of thrust is used to carry the loads.

Pippard understood this well and knew that at the limit of arch behavior, a mechanism was formed involving alternate hinges on the intrados and extrados (Figure 1). Despite this and despite the historical context of his work exactly at the period at which Baker was advancing the plastic theorems, Pippard clung to inadequate elastic analysis for his assessment method. It was left to Heyman (*1*) to pick up French work from the 18th and 19th centuries on the collapse of arches and develop a limit state procedure based on the collapse mechanism. Heyman worked with Hooke's line of thrust and continued to treat the fill as unrealistic.

The present authors found that this had two disadvantages, one of which was picked up by the writers of the Departmental Standard. Using the line of thrust as a test for stability of a structure ignores the fact that stability can be destroyed by material failure. BD21/84 (*2*) required that the line of thrust never approach closer to the boundaries of the arch than half the width of a rectangular stress block capable of carrying the thrust at that point, whereas the authors took this one step further and drew a zone of thrust (*6,7*) which was at all points through the arch capable of sustaining the applied thrust (Figure 2). Applying this analysis to real bridges produced unacceptably low results, and it quickly became clear that the soil fill could exert an enormous stabilizing influence on the arch. Approaches that take account of this influence, however, lead to more complication in the analytical procedures and to a demand for more data. The approach taken in the ARCHIE program was therefore to allow an engineer to explore the limits of influence of various parameters in a very fast analytical cycle.

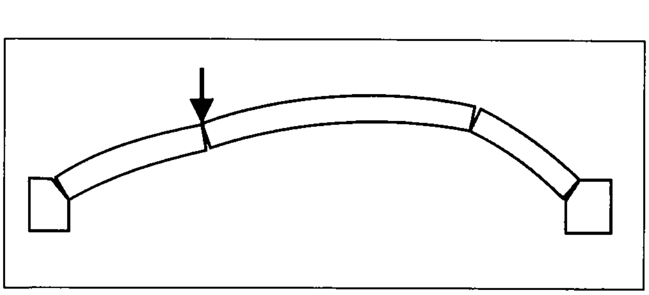

**FIGURE 1 Mechanism forming in a loaded arch.**

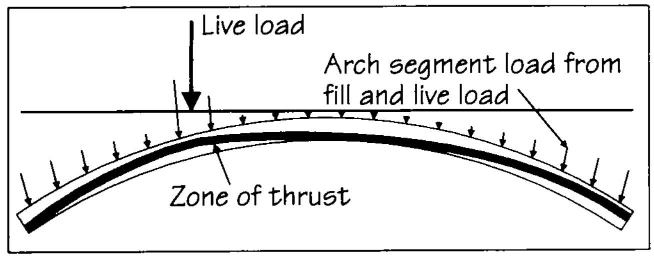


**FIGURE 2 Zone of thrust is the minimum arch capable of supporting these loads.**

## Finite-Element Analyses

During the late 1980s and early 1990s a number of workers developed specialized finite-element packages for analyzing masonry arch bridges. Crisfield (*8*) at the Transport & Road Research Laboratory adapted an existing program to treat the soil fill as a Mohr-Coulomb material to allow elasto-plastic cracking behavior in the arch ring and to take account of the ensuing changes in geometry. The program produces interesting and valuable results, but it takes several hours to produce a solution for a single load case and is therefore entirely impractical for assessment use.

Bridle and Hughes (*9*) at Cardiff developed a computerized version of Castigliano's analysis. They computed elastic and, indeed, inelastic deformation of the arch and progressively removed from the computation those zones of the material that were cracked, shifting the centerline of their elastic arch rib appropriately.

Choo at Nottingham used plane strain elements to represent the arch ring but tapered them progressively to remove from the calculations that part of the material that would be in tension. Unlike the Cardiff approach, his analysis did not take nonlinear geometry into account.

Both of these finite-element programs treat the soil fill as a set of horizontal-yielding elastic springs. The results obtained are obviously critically dependent on the spring constant used. The model used by Choo is not known, but it is known that Bridle and Hughes calibrated their soil springs to produce analytical results that match test failures as closely as possible.

## Examples

Two examples are presented. They show to some extent the problems of bridge assessment and also the limitations of the tools that are in use. In particular they will emphasize the role of judgment in bridge assessment.

## *Aldochlay Bridge, Strathclyde Region*

This small-span bridge had been repaired by guniting at some time. It is a trunk road bridge owned by the Scottish Development Department for whom the Strathclyde Regional Council acted as agent. The owners required an assessment to be carried out by the traditional MEXE approach but also for a rating to be produced for heavy vehicles, which necessarily involved more advanced analysis. The advanced analysis that was used involved a very simple version of the mechanism, or equilibrium analysis, and produced a result substantially lower than that yielded by MEXE. The authors were asked to carry out a review of the analysis and explain why these anomalies occurred.

The MEXE analysis (Figure 3) considers a load at midspan, whereas a properly constituted mechanism analysis (Figure 4) searches for the most critical position for a load. It was clear that the view of the geometry of the arch barrel that had been taken was simplistic. The masonry was hidden behind gunite, but it seemed likely that the stone was selected random rubble, and experience showed that the old masons tended to select bigger stones for the springings and smaller ones for the crown and then to hide this on the exposed spandrel face by carefully choosing stones of similar depth to express a parallel ring. Experience has shown that it is usually safe to assume that, provided the zone of thrust does not leave the arch until a point on the extrados vertically above the face of the abutment (Figure 5), the structure will be secure since there will be material to carry the thrust.

An excavation was carried out on site at a cost of some £2,500, and it was found that the arch was in fact much thicker than this near the springings, as shown by the broken line in Figure 5. This example clearly shows (a) the need for experience in bridge assessment, preferably backed up by regular observation of such bridge excavations and demolitions that take place, and (b) the need not to take analyses at face value.

Whatever form of rational analyses was applied to the structure, taking into account the additional material in the arch would produce a substantially higher result than ignoring it. The option of taking such material into account is not available in the MEXE method and is actually likely to have a detrimental effect in the computerized Pippard/MEXE method because taking a hinge at the centerline of the arch depth at the springings would in this case result in a much shallower arch curve without a corresponding increase in effective depth at the critical points.

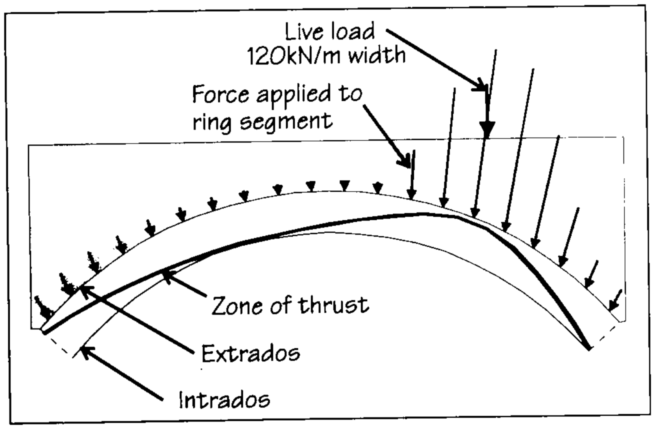


FIGURE 4 Zone of thrust with a single asymmetric load.

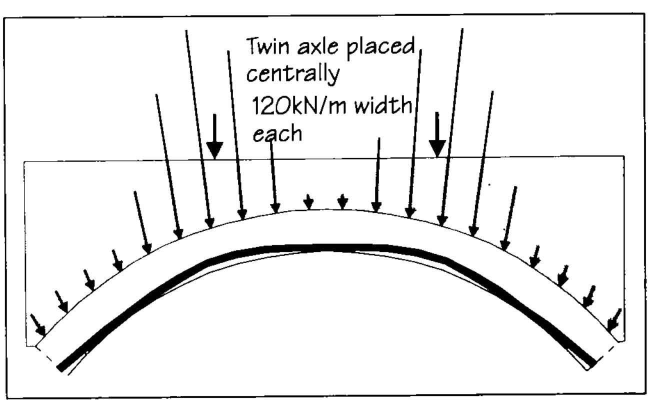


FIGURE 3 Zone of thrust view of MEXE analysis.

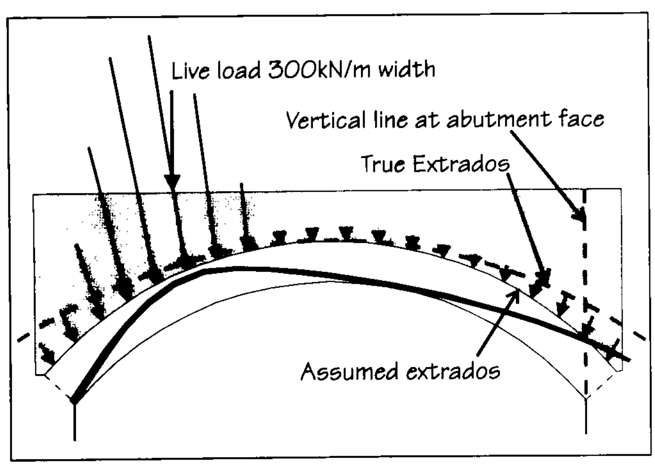


FIGURE 5 Increased capacity from a small amount of additional material.

## *Bargower Bridge*

The bridge at Bargower was one of a series tested to destruction in a program sponsored by the Department of Transport and carried out under the direction of the Transport & Road Research Laboratory. The bridge has a span of 10.54 m and is semicircular and apparently still in its true shape. It stands on abutments 5 m high and is slightly skewed, although not sufficiently to have any significant effect on the assessment. At the time of the test cracks were evident at the inside face of the spandrel walls over the middle half of the span. For this example it is worth running through the actual MEXE analysis.

### Assessment

The first step in the assessment is to determine the PAL (in metric tons). This can be found from either a nomogram or an equation (which would appear to be dimensionally incorrect). The following actual dimensions are used for the bridge at Bargower:

Span, 10.54 m ($L$)
Rise, 5.18 m ($r_c$)
Q pt rise, 4.49 m ($r_q$)
Ring thickness, 0.588 m ($d$)
Cover to crown, 1.71 m ($h$)

$$\text{PAL} = \frac{740(d + h)^2}{L^{1.3}} = \frac{740 \cdot (0.588 + 1.71)^2}{10.54^{1.3}} = 183T$$

The various factors that must be applied are then considered.

The span/rise factor ($F_{sr}$) makes allowances for the fact that steeper arches are stronger than flat arches. For arches for which the span/rise ratio is greater than 4 the value is read from a graph. For values of less than 4, as with the bridge at Bargower, for which the span/rise ratio is approximately 2, $F_{sr}$ is 1.

The profile factor ($F_p$) makes allowance for arches that do not conform to the ideal profile, which is assumed by this method to be parabolic. The value may be obtained from a graph or an equation.

$$F_p = 2.3 \cdot \left(\frac{r_c - r_q}{r_c}\right)^{0.6} = 2.3 \cdot \left(\frac{5.18 - 4.49}{5.18}\right)^{0.6} = 0.686$$

The material factor ($F_m$) is based on two other factors, barrel factor $F_b$ and fill factor $F_f$, which are obtained by reference to tables. The barrel factor ranges from 1.5 for built-in-course masonry to 0.7 for masonry in poor condition. The fill factor varies from 1.0 for concrete fill to 0.5 for weak materials. This would be the case if wheel tracking were evident. For the bridge at Bargower the values chosen were an $F_b$ of 1.5 and an $F_f$ of 0.7. $F_m$ is then obtained from the formula

$$F_m = \frac{(F_b \cdot d) + (F_f \cdot h)}{d + h}$$
$$= \frac{(1.5 \cdot 0.588) + (0.7 \cdot 1.71)}{0.588 + 1.71}$$
$$= 0.90$$

The joint factor ($F_j$) takes account of the joint width, mortar condition, and depth of mortar loss and is the product of three other factors, one for each of the elements given earlier. Wide joints, joints with missing mortar, and loose friable mortar all reduce the value of this factor.

$$F_j = F_w \cdot F_d \cdot F_{mo} = 0.9 \cdot 1.0 \cdot 1.0 = 0.9$$

The condition factor ($F_c$) is intended to take account of any cracking or deformation, which could affect the load capacity of the bridge and is perhaps the most subjective of all the factors. The bridge at Bargower exhibited longitudinal cracks under the inner edge of the spandrel walls. The suggested condition factor for this type of defect gives an $F_c$ of 0.8. However, it is unlikely that those cracks would have had any significant effect on the capacity of the arch, although they may have reduced the stability of the spandrel walls.

### Modified Axle Load

All of the various factors listed above are used to determine the modified axle load. This value is then multiplied by axle factors to get the safe load for a particular axle arrangement. For a bridge with a 10.5-m span such as the bridge at Bargower the single axle factor is 1.6. The safe axle load is given by

$$F_{sr} \cdot F_p \cdot F_m \cdot F_j \cdot F_c \cdot \text{PAL} \cdot A_f$$
$$= 1.0 \cdot 0.686 \cdot 0.9 \cdot 0.9 \cdot 0.8 \cdot 183 \cdot 1.6 = 130T$$

Exploring the performance of the structure by the equilibrium analysis presents some interesting difficulties. Figure 6 shows the effect of applying a load at the critical point on the span, roughly the third point for a semicircular arch of this scale, with the soil acting simply as vertical dead load. It is clear that under these circumstances the bridge is quite incapable of supporting a load. Figure 7 shows the same load but with the soil exerting a horizontal component of pressure with the at-rest coefficient, ($1 - \sin\phi$), for a value of $\phi$ of 35 degrees. Clearly, the performance is dramatically improved. Applying a proportion of passive pressure to that part of the arch that would rise as it failed (shown

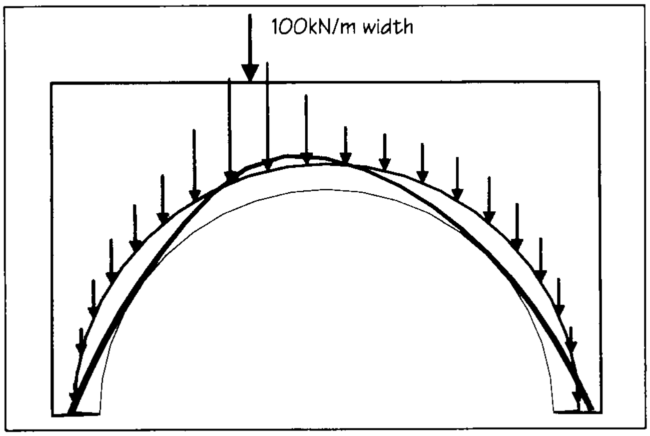


FIGURE 6 Fill acting as dead load only, negligible capacity, for bridge at Bargower.

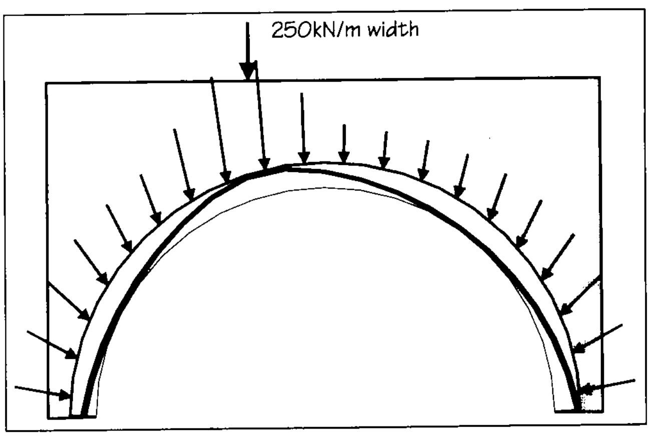


FIGURE 7 Fill exerting at-rest horizontal pressure for bridge at Bargower.

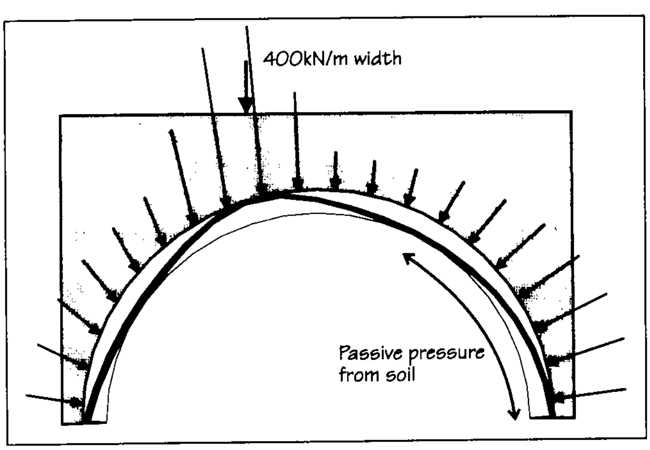


FIGURE 8 Fill exerting passive pressure for bridge at Bargower.

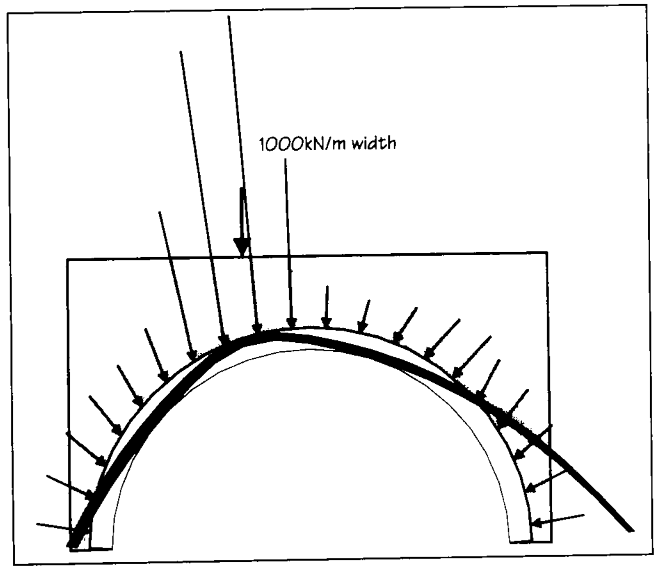


FIGURE 9 Assuming masonry backing to quarter point for bridge at Bargower.

by the arrow in Figure 7) increases the load capacity by a factor of 2 (Figure 8).

Allowing the thrust to escape from the arch at roughly the quarter point (Figure 9) increases the capacity by an additional factor of 2.5. This is a realistic scenario because on demolition it was found that the bridge had masonry support to the arch ring up to this depth. Even assuming a depth of backing as suggested above increases the load capacity to 400 kN/m of width (Figure 10).

It is clear from this example that a detailed survey of the bridge is necessary and that if a realistic assessment

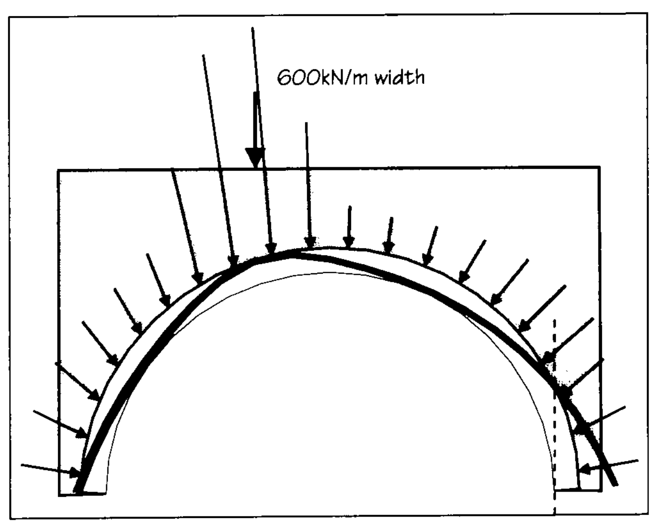


FIGURE 10 Assuming minimum masonry backing for bridge at Bargower.

is to be made, estimates are required for the complex geometry of the bridge interior. It is hoped that this also indicates the value of taking an exploratory approach to analysis for bridge assessment, progressively exploring more complex geometries.

## Analysis in Application

It is clear that the MEXE analysis, if analysis it can be called, is very simple to use and has the confidence of engineers. Any approach that is more difficult to use is unlikely to be welcome. Thus, until such time as someone writes a program to speed its application, the computerized Pippard/MEXE method recommended by the Department of Transport is unlikely to come into regular use. There are other pieces of software known to the authors, but ARCHIE, the Cardiff program CTAP, and the Nottingham program MAFEA are all characterized by specially written data input modules that speed operation.

Nonetheless, all three have major drawbacks. Perhaps the most important, shared by all of them, is the fact that they are essentially two-dimensional analyses of three-dimensional structures. They all ignore the potential stiffening effect of spandrel walls and can only take account of load distribution by the use of empirical effective widths.

It is clear from the nature of the Cardiff and Nottingham programs that although they are capable of providing rather more information at loads less than the ultimate limit, once the limit is reached, the results should be very similar to those produced by the much simpler ARCHIE program. Only in the case of very large flat arches in which substantial elastic deformation takes place before failure will the CTAP program produce noticeably different results for the ultimate limit, and then they might be substantially lower.

## Conclusions

1. Arch bridge assessment is a complex and difficult operation made more difficult by the lack of firm data on which to base an analysis.
2. Exploratory analysis investigating the effects of various parameters is extremely valuable, increasing an engineer's confidence in his or her results.
3. Such exploratory analysis would be too expensive to carry out without efficient, specially designed software.
4. Analysis based on unreasonable structural theory is unlikely to lead to confidence in the output.

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## Appendix

### MEXE Method

The MEXE approach began with a series of assumptions. The arch is assumed to be parabolic to ease the calculations, it is assumed to be elastic but supported on pins, the span/rise ratio is assumed to be 4, and the ratio of ring thickness to fill depth is assumed to be 1. All loads are assumed to act vertically on the arch. The live load is a twin axle placed centrally. With this layout a formula can be written for the value of live load axle that will produce a compressive stress of 200 lb/in.$^2$ or 1.4 MPa at the crown. This calculated stress was found empirically to correspond to the first crack in a series of real arches tested in Britain in the 1930s and 1950s. The approach therefore takes empirical account of such factors as load distribution and stiffening of the arch by the spandrels. For ease of use a nomogram (Figure 11) was produced, relieving military engineers working under stress from the need to carry out calculations.

Clearly, arches are not all parabolic, nor do they all have a span/rise ratio of 4 or equal depths of ring and fill. Modification factors were computed to take account of each of these changes. It is worth noting that however the factors were computed, they are demonstrably wrong since the shape factor indicates the para-

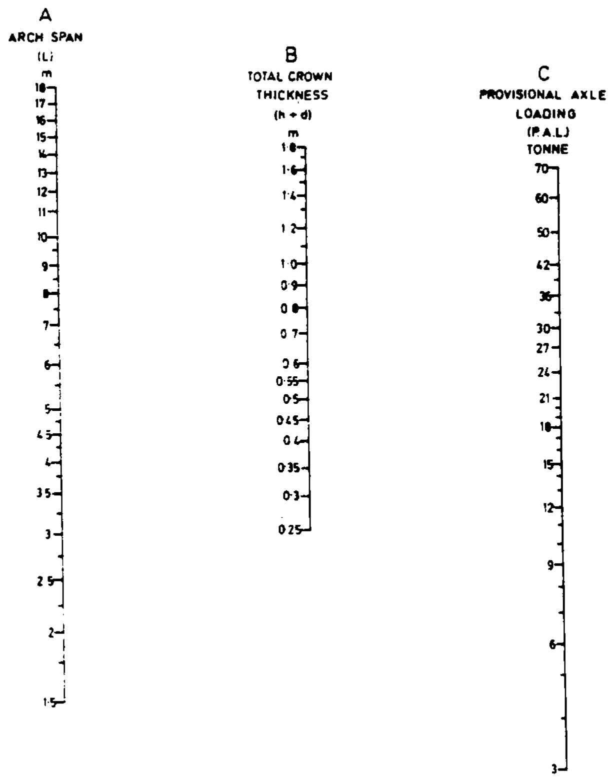


FIGURE 11 Nomogram for determining provisional axle loading of masonry arch bridges before factoring.

bolic shape to be the best, whereas for a typical real arch with realistic soil pressures, the best shape will vary considerably with the span and rise. There are additional factors to take account of the strength and stiffness effects of different materials, a lost section as a result of loss of mortar in the joints, and finally, a general condition factor for the bridge. The condition factor is entirely a matter of judgment, although guidance is provided in the Department of Transport standard by the use of photographs. There is no indication of how the factors were actually derived, nor is there any evidence that the loss of capacity indicated has been checked against test results. The nomogram for the various factors is presented below.

## Computerized Pippard-MEXE Method

The computerized Pippard-MEXE method is essentially a modernized basic MEXE method. The arch is divided into a number of segments and is analyzed as a rigid jointed frame with pinned supports. All loads are assumed to be vertical and concentrated at the nodes, but the true shape of the arch and the correct loading can be used. Despite these improvements the method remains empirical since it is recognized that the analytical model does not represent the real behavior of the arch.