Small scale modelling of brickwork arch bridges using a centrifuge

T. G. Hughes, BSc, MSc, PhD, CEng, MICE, M. C. R. Davies, BSc(Eng), AKC, MPhil, PhD, CEng, MICE, FGS, and P. R. Taunton, BSc, MSc, CEng, MICE, MIQA, FGS

Proc. Instn Civ. Engrs Structs & Bldgs, 1998, 128, Feb., 49–58

Paper 11419

Written discussion closes 29 May 1998

This paper contains the results of a recent study undertaken to assess the viability of modelling soil–brickwork structure interaction problems, at small scale, using a centrifuge. A series of tests is reported that simulates the behaviour of a previously tested 3 m span brickwork arch bridge, at 1/6 full scale, at an elevated gravity equivalent to 6 g. The results show that not only are the overall mechanisms of failure correctly modelled but that the influence of both brickwork and soil strength can be properly investigated. The results are compared to predictions made using the current UK masonry arch assessment methods with particular consideration given to the influence of soil and masonry strength.

Keywords: brickwork & masonry; bridges; models (physical)

Introduction

Considerable effort has been expended in the UK in recent years in gaining a better understanding of the behaviour of masonry arch bridges. This work has been driven by the economic need to maintain the significant infrastructure which these bridges, many of them over 100 years old, constitute. In addition there is a move to construct new arch structures using new materials in an attempt to replicate the longevity of brickwork arches.

2. The load capacity of a masonry arch bridge generally lies not in the inherent strength of the material that forms the structure but in the requirement for the load to overcome the mass of the structure by the rotation of large elements of it against the force of gravity. In order to rotate the structure forms a series of hinges. The hinges, normally four, form by the opening of joints through the ring with some small element of the brickwork, the hinge, retaining its integrity. The true behaviour of both the brickwork, that makes up the arch barrel and spandrel walls, and the backfill, that both provides additional mass and also restrains the barrel, is stress-level dependent, with the stresses in the real structure being largely related to self weight. The structural behaviour of masonry arches is therefore dominated by gravity and so for correct physical modelling the influence of

gravity must be considered. This paper reports the first attempts to model these complicated interactions on a small scale in an increased gravity field.

3. The UK masonry arch research programme has three strands, the testing of real structures, laboratory modelling at a range of sizes and numerical modelling.¹ The large cost of testing existing real structures coupled with the difficulty of intrusive monitoring and sometimes the lack of arch definition has limited this programme of tests. Considerable effort has been expended in laboratory modelling with spans generally between 1 and 3 m. These tests have provided qualitative information by examining, for example, strengths with and without backfill. The principal problem with models at this scale is that they are not scale models of larger structures but are, in fact, small arches and so the stress levels both within the backfill material and the arch ring are low. This means that those phenomena that are relevant to larger arches such as brickwork crushing, ring separation and effective soil–structure interaction do not occur. At smaller scales at unit gravity it is understood that brickwork behaviour is not properly modelled and it is not considered necessary to use replica materials.²

4. Masonry arch bridges are generally investigated analytically as two-dimensional (2D) plane strain structures. Tests on laboratory arches with and without attached spandrel walls have shown that they can contribute to the strength and stiffness of the arch.³ Tests on real arches have produced differing structural responses. In some tests the spandrel walls parted from the arch barrel at an early stage of loading; in one test the spandrel walls fell away, while in another they remained standing after the test with the arch failing between them.⁴ In other tests the spandrel walls have remained an integral part of the structure through to collapse and contributed to its strength. The differing stiffness of the spandrel walls and the arch barrel, coupled with the frequent occurrence of seepage problems at the interface, questions the integrity of this joint in real structures. For the present study 2D plane strain modelling was adopted without spandrel walls, thus allowing the results to be compared directly to the current arch assessment procedures.

T. G. Hughes, Senior Lecturer, University of Wales, Cardiff

M. C. R. Davies, Professor of Civil Engineering, University of Dundee, formerly Director, Cardiff School of Engineering, University of Wales, Cardiff

P. R. Taunton, Contracts Manager, Thyssen Construction Limited, Bristol, formerly Research Assistant, Cardiff School of Engineering, University of Wales, Cardiff

Centrifuge modelling

5. In essence, the use of a centrifuge simply increases the apparent density of the construction materials. In a 1/6 dimensional scale model the centrifuge is rotated at a speed that provides a centrifugal acceleration equivalent to $6g$, where g is the acceleration due to the earth's gravity field, thus making the brickwork and backfill materials appear effectively six times heavier. Therefore, at any particular depth in the fill the product of the scaled depth, the density and the increased g level produces a vertical self weight stress that is exactly equivalent to the full scale stress.

6. Because the response of arch bridges is non-linear any increase in effective density does not produce a linear increase in the load capacity. Fig. 1 shows a typical increase in failure load with increase in arch material density, demonstrating this effect using two different forms of analysis common in masonry arch bridge assessment, the mechanism and cracking-elastic analysis.^{5,6} Both methods of analysis show the failure load increasing at a less than linear rate in response to increases in material density.

7. For the mechanism approach this effect is caused by an increase in the size of the plastic

zone at the hinges as density increases. In order for a linear increase in load to occur it would be necessary for the depth of the plastic hinge to remain constant at different densities. In reality at higher densities the thrust in the arch is higher and so, also, must be the size of the plastic zone. This effect therefore reduces the increase in the collapse load. For the cracking-elastic analysis the effect is similar, only here it is related to stiffness. The higher densities generate larger deformations and larger axial thrusts and therefore the load at which snap-through occurs is reduced.

Fig. 1. Effect of the variation of density on the failure load using mechanism and elastic-cracking analyses

Table 1. Basic arch geometry and properties used in study

Test number	T5 and T6 (+prototype)	T3 and T4	T1 and T2
Geometry			
Intrados span: m	3.000	3.000	3.000
Rise at mid-span: m	0.750	0.750	0.750
Fill depth at crown (inc. road): m	0.300	0.300	0.300
Arch ring thickness: m	0.215	0.215	0.215
Arch shape	Circular	Circular	Circular
Width: m	2.835	2.835	2.835
Masonry			
Brick type	Class A Engineering – Accrington Nori	Common – Ibstock	Common – Ibstock
Brick compressive strength: N/mm ²	150	35	35
Mortar	1 : 2 : 9 BS 4451 (iv)	1 : 3 : 12 BS 4451 (v)	1 : 3 : 12 BS 4451 (v)
Brickwork bulk density: kN/m ³	23.7	16.0	16.0
Brickwork elasticity*: kN/mm ²	22.5	5.3	5.3
Brickwork compressive strength (BD 21/93)†: N/mm ²	15	7	7
Backfill			
Type	Limestone	Limestone	Sand
Fill bulk density: kN/m ³	20.5	20.5	18.6
K_a^*	0.17	0.17	0.30
K_o^*	0.30	0.30	0.47
$K_p^†$	3.5	3.5	1.95
Subgrade modulus*: MN/mm ³	31.0	31.0	13.0
Loading			
Position of load: m	0.750	0.750	0.750
Single axle loaded length: m	0.300	0.300	0.300
Distribution angle*†	1H : 2V	1H : 2V	1H : 2V

* Assumed value (cracking-elastic analyses).

† Assumed value (mechanism analyses).

8. Perhaps the largest non-linear effect is related to the backfill, of which both the strength and stiffness are highly stress-level dependent. The backfill provides both mass and lateral restraint to sway. This lateral behaviour is strongly stress-level dependent and cannot be modelled properly without the use of increased gravity.

Model description

9. To validate the model test procedure it was decided to use as a prototype a well-documented laboratory test of a 3 m span arch,³ the geometric and material properties of which are given in Table 1. This approach, of trying to replicate a specific previous full-scale test, is the most onerous in small-scale modelling. Rather than requiring the model to replicate the genetic behaviour of full-scale structures of the type, it is necessary to obtain small-scale model equivalents of all the actual prototype materials used, including brick units, mortars and backfill materials.

Scale

10. A literature review undertaken on scale brickwork modelling revealed that testing at 1/2 or even 1/3 scale is quite common, with the smallest scale tests previously used being of the order of 1/10 full scale. Previous work suggests that compressive strength is well modelled but that the small-scale brickwork seems to be significantly less stiff than full scale.⁷

11. Space restrictions in the Cardiff centrifuge, which is large by international standards, limit the test package to approximately 1 m³. The largest scale for the 3 m structure was chosen as 1/6 full scale. This allowed the arch to be positioned asymmetrically within the strongbox, as shown in Fig. 2, such that the proximity of the strongbox end walls would not influence the behaviour of the backfill.

Brickwork

12. The prototype arch was constructed of brickwork composed of a strong engineering brick with a weak mortar.³

13. Model clay bricks can be produced either by firing small units or by cutting larger units. The features to be considered in cut bricks are the surface texture and the possible variation of properties through the brick. Tests undertaken on the prototype bricks indicated that this variation was not a feature of this particular brick. In considering surface texture there is the shear behaviour (a) between the arch barrel and the adjacent fill, (b) in the brick-mortar bed joint and (c) across the inter-ring mortar joint to be considered. The full-scale bricks have different finishes between the top and bottom, i.e. the bed joint faces, and the

Fig. 2. General arrangement of loading package and instrumentation

sides. The bed joint faces are wire cut and are relatively rough whereas the sides are extruded and are much smoother. It was considered that the surfaces obtained by cutting the bricks would have a similar surface texture to the brick sides, which was confirmed by shearbox tests on brick-fill interfaces, yielding angles of friction of 38° and 34°, respectively. The increased smoothness of the model bed joint face was not considered a significant problem for arch application, as bed joints are predominantly in compression. In addition the mortar to be used would be composed of finer aggregate and would provide sufficient adherence to the smoother model surfaces.

14. Firing of small bricks was considered to be significantly more problematic than cutting. It would be all but impossible to produce small-scale brick with the mechanical properties of the full size equivalents. In addition the mortar sand grading had to be changed because of the size of the model joints and the mortar brick interface would therefore anyway be different between the model and the full-size brickwork. It was therefore decided that cutting represented the best approach.

15. The full-size arch was constructed using bricks of dimension 215 × 102 × 65 mm with 10 mm bed and head joints. All these dimensions were scaled at 1/6 in the model studies. The mortar mix used is detailed in Table 1 for both

the model and the prototype. The mortar sand used has a maximum particle size of 0.25 mm.

Backfill

16. The prototype backfill was a crushed limestone. A local graded limestone was sieved to remove all particles greater than 6 mm to provide the model-scale backfill material. Apart from a slightly larger fraction at the fine end the particle size distribution of the model backfill was equivalent to a 1/6 scale prototype material.

Model package geometry

17. The model package was positioned on a swinging platform (gondola) which naturally rotates to the correct angle appropriate for the required acceleration. The package was positioned such that the front elevation, which remains in the vertical plane, appeared as Fig. 2 prior to spinning. As spinning commenced the model rotated vertically with the base of the model moving away from the centre of rotation of the centrifuge. There is, because of the different radii (approximately 2.4 m at the model centre), a small change in the effective gravity through the height of the model. In the present study the effective gravity varied between 97 and 106% of that at the model centre. In addition the soil and arch weights only act directly 'down' at the centre of the model with small lateral forces at other sections, these forces were larger at smaller scales where gravity itself represents a significant part of the force.

Instrumentation

18. A range of monitoring equipment, as detailed in Table 2, was installed in the model arches, located as shown in Fig. 2.

Pressure sensors

19. Two types were positioned radially between the extrados of the barrel and the backfill material to monitor the normal stress on the interface. These were 6 mm diameter, diaphragm pressure sensor (DPS) cells, manufactured by Kyowa (type PS-5KA, rated capacity 500 kPa), and force sensing resistors (FSRs), manufactured by Interlink Electronics (type FSR151NS, 9.5 mm active diameter). The latter gauges provide a highly non-linear response to load but are inexpensive and

expendable and proved useful for qualitatively detecting the general behaviour of interface stress distribution during testing. The DPS cells, although expensive, were quite robust and generally provided more satisfactory results.

Clip strain gauges

20. These were constructed from small pieces of pre-bent clock spring, with strain gauges bonded to both faces, forming a half-bridge circuit. Each 25 mm length clip was positioned radially on the intrados along the arch, so as to span two mortar joints, sprung between two small pins which were bonded to the brickwork surface by way of small flat plates.

Displacement monitoring

21. A range of high-resolution linear voltage displacement transducers (LVDTs), with full-scale deflections of 2–10 mm, were used. These were positioned radially, distributed throughout the intrados, measuring movements normal to the barrel.

Video images

22. In addition to providing qualitative images of the overall structural response, video images were used to monitor markers positioned in a grid on the exposed soil face. These images have been used, by way of image processing, to provide details of the soil movement. In each test an additional camera was positioned under the arch to view the development of cracking beneath the line load.

Loading

23. The load was provided by a gearbox through a 20 kN load cell. The load was transmitted to the soil by way of a column, articulated at each end, to a spreader beam across the full width of the model. Two LVDTs were used to monitor the vertical movement of the spreader beam.

Test procedure

24. The barrel brickwork was laid against an oiled steel formwork bolted to the abutments which were themselves bolted to a rigid baseplate. On completion of the barrel the instrumentation to be positioned under the intrados was approximately positioned and all

Table 2. Schedule of model tests and instrumentation

Test No.	Scale	Brick type	Backfill material	Mortar mix	LVDT	Clip gauge	FSR	Kyowa	Video	Peak load: kN
T1	1/6	Ibstock	Sand	1:3:12	9	8	10	1	2	260
T2	1/6	Ibstock	Sand	1:3:12	9	8	10	3	2	280
T3	1/6	Ibstock	Limestone	1:3:12	9	8	10	–	2	400
T4	1/6	Ibstock	Limestone	1:3:12	9	8	10	–	2	410
T5	1/6	Nori	Limestone	1:2:9	9	8	10	–	2	510
T6	1/6	Nori	Limestone	1:2:9	9	8	5	5	2	620

cabling passed under the abutments. The baseplate, containing the model, was then lifted into the strongbox. To prevent the arch barrel being restrained by contact with the Perspex faces of the strong box 2 mm gaps were left on both sides of the model. To prevent loss of backfill material, these gaps were sealed with grease contained within a roll of Clingfilm. The Perspex walls, which are 50 mm thick to provide adequate lateral restraint, were covered with small overlapping sheets of acetate, lubricated with silicone lubricant to minimize any side restraint.

25. The interface pressure sensors and accompanying wiring were positioned prior to the backfill material being placed and uniformly compacted in layers using a drop hammer. Once the filling was complete one Perspex side wall was removed to permit the formwork to be struck. The clip gauge location pins were bonded to the intrados brickwork and the clip gauges installed. The radial LVDTs were accurately positioned and the intrados video camera installed. White rigid plastic markers 6 mm in diameter were placed in a regular grid on the face of the fill. The Perspex side wall of the strongbox, complete with its lining of lubricated acetate sheets, was then replaced.

26. Once the model was complete the jacking system was fitted together with a loading column, the load cell, a spreader beam and the LVDTs to monitor the beam movement. The model 'package', as shown in Fig. 3, was then lowered into the gondola and the second video camera, to record the important side view of the test, installed.

27. A full set of the readings from all 31 channels of the instrumentation were taken at 6-sec intervals throughout the test. The centrifuge was initially accelerated to 6g in 2g increments with the speed being held at each increment to allow the readings to stabilize. The spreader beam was then lowered to the soil surface and loading applied at a uniform rate equivalent to 18 mm/h. A bedding-in load was applied by increasing the load up to an estimated 25% of the expected peak load and then removed at the same rate. The load was then reapplied and taken through to failure. The test was monitored on both video cameras. The test was terminated at deflections equivalent to 60 mm on the prototype, the load withdrawn and the centrifuge stopped.

28. Following removal from the centrifuge the model was carefully dismantled and the bulk density and moisture content of the backfill material determined. The location of cracking in the arch barrel and discontinuities in the soil were recorded.

Prototype/model comparison

29. The failure modes of the prototype and models both involved the formation and rotation

Fig. 3. Strong box incorporating 1/6 scale model, together with loading frame and gear box

of four distinct hinges. Included in Fig. 4 are details of the position of the through-ring cracks for both the model tests, T5 and T6, and the prototype. The positions of the hinges are generally the same except that in model test T5 the hinge that was remote from the load did not form at the abutment but in the fourth brick course up to the intrados. In test T6 hinge cracking was also apparent at this location although the main hinge did form at the abutment.

30. Figure 5 shows the load-deflection plots for the prototype and for the model tests following the unload-reload loop. The full-scale response is generally stiffer than the model. Separate tests undertaken on stacks composed of six model bricks indicated stiffnesses of 8–14 kN/mm² compared to reported prototype stack results of 9–25 kN/mm². It is a widely reported feature of scale effects in the fracture of concrete materials that smaller models produce higher strengths and increased ductility. This effect is associated with the increased relative size of the non-linear fracture process zone in small tests. However the present results suggest a general softening of the response without an overall loss of strength, with the peak loads of the model tests, T5 and T6, at 510 kN and 620 kN, straddling the prototype peak load of 540 kN.

31. Figure 6 shows a comparison between

the prototype and model results for the overall radial displacement of the arch ring, normalized by displacement under the load, at different stages of loading. The results importantly show that the soil movement in the passive zone, at between 70 and 100% of the arch span, is correctly related to the movement under the load. There is a poorer comparison at the crown, with the model results showing little radial movement whereas the prototype has a large movement. The prototype results presented are misleading in that the rate of change of deflection is at its peak at this location and the position of zero radial movement is likely to be much closer to the centre point than indicated by the linear interpolation shown in Fig. 6; indicating it to be at about 35% of the span.

32. The most contentious issue in arch bridge assessment, relating to the behaviour of the backfill, is the extent of the development of passive pressures remote from the applied load, as the arch sways into the backfill material. Assessment models have been developed that allow the pressure to build up linearly with the displacement. Views have also been expressed that the arch movement is likely to be insufficient to develop full passive pressure and assessment methods have been developed that reflect this. The ability of the scale models to properly replicate prototype behaviour is therefore important. Fig. 7 includes the variation of soil pressure with applied load, normalized by the peak load, at the quarter point remote from the point of applied load, for model T6 and for the prototype, in which three cells were located across the width of the arch. The results of both the prototype and model show a gradual increase in soil pressure with applied load, with an initial stiffer response in the model. There is then a significant reduction in the soil pressures of both the prototype, and the model, as the applied load increases. The model test shows the phenomenon at a lower percentage of the applied load than the prototype and the situation is confused by the unload-reload cycle. However, there is a clear reduction in both model and prototype which is at odds with previous considerations of passive behaviour. The limited number of pressure cells both in the prototype and the model do not allow the extent of the region experiencing this phenomenon to be quantified at present. Whilst the quarter point experiences the maximum horizontal movement and thus may be seen as a suitable point to monitor the build up of passive resistance the situation is complicated by the large vertical movement.

33. Figure 8 shows vectors of the movement of the arch fill material as determined by image processing of a video recording. Since displacements near the boundaries are negligible the results confirm that the walls of the strongbox are sufficiently far from the arch so as not to

Fig. 4. Arch cross-sections for each test showing the position of hinges and the location of any ring separation

influence the failure. The results also indicate a general sway movement with large vertical upward movements between the crown and the remote quarter point. The soil under the applied load moves downwards almost vertically with very little sway. The points between the near springing and the loading point indicate a general downwards movement towards the springing. This agrees with the results of previous model tests at unit gravity² but contradicts most analytical models which contain sway movements.

Brickwork strength influence

34. The latest UK assessment standard^{8,9} includes the use of an assessment technique based on the analysis of the arch as a two-pinned elastic structure with compressive strength as

Fig. 5. Load-deflection response for full scale and equivalent 1/6 scale model tests

Fig. 6. Normalized deflections along the arch at various loads for full scale and equivalent 1/6 scale model test (---, model response; —, prototype response) T5

Fig. 7. Variation of soil pressure at quarter point with applied load

Fig. 8. Deformation vector plots showing overall backfill movement (deformation $\times 3$) test T5

Fig. 9. Failure of brick during test T3

the load determining criterion. The model experiments were repeated with the same geometry and backfill material, only with the prototype bricks and mortar being replaced with weaker bricks and mortar, as detailed in Table 1. The bricks were again cut from full size units.

35. The load deflection plots for the two tests, T3 and T4, detailed in Fig. 5, demonstrate a remarkable self-consistency, with equivalent peak loads of 400 and 410 kN for the two tests. The failure modes were similar, with the formation and rotation of hinges, the position of which are detailed in Fig. 4. However, for these tests there were two additional features, ring separation and brickwork crushing.

Ring separation

36. Ring separation is known to occur in real arches and there are a number of postulated causes including fatigue coupled with continued wetting and drying. Its influence on strength has previously been investigated in the prototype by

repeating the test on an arch which was artificially weakened, by incorporating a sand layer between the two arch rings, to simulate the fault.³ There are a number of reasons for the formation of ring separation in multi-ring arches but it generally stems from a lack of through-ring bonding in the brickwork. This bonding, which is an integral part of all the brickwork walls, is not generally included in multi-ring brickwork arches because adjacent bed joints only occur infrequently. However, in many arches the ring separation is only partial and can only be identified as a hollow sound when struck. In this situation it is more likely that composite action between the rings will be retained. In the current study ring separation occurred, in both test T3 and T4, under live load, at the positions detailed in Fig. 4. The separation occurred naturally as part of the loading process and, although the two tests had different ring separation locations, their position did not appear to influence the failure load.

Brickwork crushing

37. Failure of brickwork walls in axial compression is normally considered as a compressive failure, although detailed investigation of the process suggests a different view.¹⁰ Compressive failure of the brickwork in the real bridge tests has been suggested but the evidence for this is weak because the principal location, in the extrados under the applied load, is obscured during the test and the damage which occurs during the final collapse of the full scale test tends to destroy the supporting evidence. Elastic-cracking analysis suggests that the stresses in the extreme fibres reach levels at which the brickwork would, in normal situations, fail. However, the stress regime, particularly under the point of loading, is extremely complex and it has previously been suggested¹¹ that allowable stresses, higher than normally considered for brickwork, are appropriate largely because of the very limited area of the highly stressed region. In test T3, failure of the brickwork units was identified during the test post-mortem, at course 15 shown in Fig. 4. Fig. 9 shows the complete delamination of the surface of the units taken from under the load line. This delamination removed a slab of 1–2 mm thick over the entire extrados face of the unit. Every unit under the applied line load suffered a similar fate, strongly supporting the pure 2D plane strain nature of the test regime. This failure suggests a secondary tensile fracture caused by the high compressive stresses as the line of the thrust moved to the extreme edge of the barrel. It is not considered a coincidence that the thickness of the failure plate corresponds to that of the mortar bed joint. It is considered that the failure of a portion of the mortar at the extreme edge of the joint, with the mortar breaking out into the backfill, pre-empted the brick failure.

Backfill type influence

38. Experiments have been performed on arches with and without backfill to try to quantify its influence.¹ However, the soil supports the arch in at least two distinct ways, firstly by the increased mass that has to be overcome at failure and secondly by the lateral support provided against sway. To better quantify the influence of the soil, model tests were repeated with a sand backfill. Because the softer brickwork is more typical of real arches this brickwork was retained for these tests, the full details of which are given in Table 1.

39. The failure modes of the two tests, T1 and T2, were similar with the hinge positions as detailed in Fig. 4. Test T1 however developed ring separation whereas T2 did not. The load deflection plots, detailed in Fig. 5, indicate T1 providing an initially softer response, although the peak loads are similar. The results indicate a further one-third reduction in the overall

strength of the arch resulting from the use of the weaker and less dense backfill material.

40. The results of the use of the weaker backfill in tests T1 and T2 are also included in Fig. 7. The results show that the overall pattern with the initial rise in pressure followed by a gradual reduction as peak load is approached is repeated although the magnitudes of the test T2 pressures are significantly lower.

Assessment analyses

41. There are currently four generically different methods of arch bridge assessment, all of which are detailed in the UK highway arch assessment standard. These are the modified MEXE method, the Pippard/MEXE method, the mechanism method and the cracking-elastic method.^{8,9} The response of each to the combinations of backfill and brickwork used in the centrifuge experiments are considered.

MEXE

42. The method predicts a provisional axle load (PAL) based on the span and combined fill and barrel thickness of the arch.^{8,9} For the arch detailed in Table 1 the PAL is 47 t. This is modified by a number of factors related to the geometry, material and condition of the arch. For the prototype brickwork and backfill this produces a modified single axle load of 35 t which, for this span, is the critical axle pattern. MEXE is an allowable load method and does not predict an ultimate load. For the other assessment methods, which are based on ultimate load analysis, the appropriate partial load factor, based on test reports, is 3.4.⁸ Therefore to translate the MEXE allowable load into an ultimate load, permitting comparison with other methods, it is considered appropriate to multiply it by 3.4. This may overestimate the strength of the arches since the actual partial load factor for the method is unknown. The width of the prototype corresponds approximately to that of a normal axle and therefore the MEXE method suggests an ultimate prototype load of about 1150 kN. For the weaker bricks of tests T3 and T4 the arch barrel factor is reduced from 1.2 to 1.0 and, since

Fig. 10. Variation of model and predicted assessment loads with masonry and soil type

all other factors are the same, the predicted ultimate load reduces to 1050 kN. In relation to backfill material, fill is considered only as being well compacted or weak and the fill factor was therefore reduced from 0.7 for the limestone to 0.5 for the sand. With all other factors the same the predicted ultimate loads for tests T1 and T2 were therefore reduced to 900 kN.

Pippard/MEXE

43. The allowable compressive strength of the prototype brickwork composed of the 150 N/mm² unit with a grade (iv) mortar is of the order of 15 N/mm². Using this compressive strength, a two-pinned elastic analysis on the prototype, undertaken in accordance with the current standard, produces a predicted failure load of 1810 kN. For the weaker brickwork of tests T3 and T4 the compressive strength of the brickwork is approximately 7 N/mm². This reduces the allowable load to 830 kN. The new Pippard/MEXE method makes no specific allowance for the soil type and therefore the predicted collapse load for tests T1 and T2 is the same as for tests T2 and T4.

Mechanism

44. Using the compressive strengths, as for the Pippard/MEXE method above, and the passive soil coefficient, K_p , as detailed in Table 1, results in a predicted ultimate load for the prototype of 440 kN. For the weaker brickwork in T3 and T4 the reduced compressive strength reduces the predicted collapse load to 360 kN. The reduction in soil strength for T1 and T2 further reduces the predicted mechanism load to 270 kN.

Elastic-cracking

45. Using the brickwork and soil prototype properties detailed in Table 1 the predicted collapse load is 320 kN. This reduced to 280 kN for the analysis of tests T3 and T4 and further to 210 kN for tests T1 and T2.

46. The results of the analyses are summarized in Fig. 10, which shows the average results of each pair of models together with the predicted ultimate load using each of the four assessment methods. The figure shows that both the MEXE and Pippard/MEXE methods grossly overestimated the strength of all the arches. However, in relative terms, the MEXE method is able to distinguish properly between the different masonry and backfill types. The Pippard/MEXE method has no allowance for different backfill materials and was incapable of properly distinguishing even the relative effects of masonry strength. The mechanism and elastic cracking analyses both appear to produce good safe predictions.

Conclusions

The following conclusions can be drawn from

this study:

- (a) Small-scale modelling of brickwork arch bridges under increased gravity produces consistent results and replicates all the features of full-scale behaviour.
- (b) The strength and stiffness of the arch brickwork has a significant but not dominant effect on the collapse load of the prototype geometry arch.
- (c) The strength and stiffness of the backfill material has a significant but not dominant effect on the collapse load of the prototype geometry arch.
- (d) Comparison of the test results with both the MEXE and Pippard/MEXE assessment methods indicates that the methods significantly overestimated the strength of the arches.
- (e) The mechanism and elastic-cracking analyses produced good safe predictions and were properly able to reflect the influence of both brickwork and soil type.

Acknowledgements

The authors wish to acknowledge the financial support of EPSRC under grant GR/K04262.

References

1. PAGE J. *Masonry arch bridges—TRL state of the art review*. HMSO, London, 1993.
2. FAIRFIELD C. A. and PONNIAH D. A. Model tests to determine the effect of fill on buried arches. *Proc. Instn Civ. Engrs Structs Bldgs*, 1994, **104**, 471–482.
3. GILBERT M. *The behaviour of masonry arch bridges containing defects*. Manchester University, PhD Thesis, 1993.
4. PAGE J. *Load tests on two arch bridges at Torksey and Shinafoot*. Transport Research Laboratory, Crowthorne, 1988, TRRL Research Report 159.
5. CRISFIELD M. A. *Finite element and mechanism methods for the analysis of masonry and brickwork arches*. Transport Research Laboratory, Crowthorne, 1985, TRRL Research Report 19.
6. BRIDLE R. J. and HUGHES T. G. An energy method for arch bridge analysis. *Proc. Instn Civ. Engrs*, 1990, **89**, 375–385.
7. EGERMANN R., COOK D. A. and ANZANI A. An investigation into the behaviour of scale model brick walls. *Proc. 9th Int. Brick/Block Masonry Conf.*, Berlin, 1991, 628–635.
8. DEPARTMENT OF TRANSPORT. *The assessment of highway bridges and structures*. Department of Transport, London, 1993, Department Standard BD21/93.
9. DEPARTMENT OF TRANSPORT. *The assessment of highway bridges and structures*. Department of Transport, London, 1993, Department Advice Note BA16/93.
10. PANDE G. N., KRALJ B. and MIDDLETON J. Analysis of the compressive strength of masonry given by the equation $f_k = K(f_b)^{\alpha}(f_m)^{\beta}$. *Struct. Engr*, 1994, **71** (14), 7–13.
11. HENDRY A. W. *Masonry properties for assessing arch bridges*. Transport Research Laboratory, Crowthorne, 1990, TRRL Contractor Report 244.