Arch Bridges
ARCH´04
C L Brookes and P J Mullett (Eds)

SERVICE LOAD TESTING, NUMERICAL SIMULATION AND STRENGTHENING OF MASONRY ARCH BRIDGES

Carl L Brookes and Paul J Mullett

Gifford and Partners
Carlton House, Ringwood Road, Southampton, SO40 7HT, UK
Email: Carl.Brookes@Gifford.UK.com web page: www.gifford-consulting.co.uk

Key words: Masonry, Arches, Bridges, Strengthening, Discrete, Archtec, Serviceability

Abstract. *As part of the continuing development process of the Archtec masonry arch bridge strengthening system, a series of service load tests have been undertaken on a typically deteriorated bridge carrying a busy road in Lincolnshire, UK. Tests, undertaken before and after strengthening, were carried out using two ballasted vehicles during two separate road closures. The bridge, traversing the course of a redundant railway, is built from brick masonry and is typical of many constructed in the late nineteenth century and found frequently throughout the UK rail network. Consistent with other contemporary work on masonry arches and current assessment/design methods, verification and testing of Archtec strengthening has been focused primarily on ultimate strength predictions. These tests were designed to better understand Archtec strengthened bridge behaviour at the serviceability limit state and to show that there are no detrimental effects. The tests were instrumented to measure intrados strains, crack widths and anchor strains. In parallel with the experimental work, strain and displacement predictions have been made using numerical simulation based on the Finite/Discrete Element technique. This method, unlike limit state analysis which is generally used to assess arch bridge strength, permits the estimation of strength, displacement and their derivative quantities crucial to evaluate service condition and damage. Test results have shown that the strengthening reduces intrados strains and helps control pre-existing crack movement. Indications are that these characteristics would be of benefit under cyclic service loads. These quantities have also been predicted using Finite/Discrete Element modelling. It is concluded that Archtec strengthening can be designed not only for the ultimate limit state (strength) but also for the serviceability limit state (deflections, strains and stress ranges).*

1 INTRODUCTION

The Archtec system is a proprietary method of arch strengthening using retrofitted reinforcement. Developed in 1998, the system has now been used extensively throughout the UK with projects also in the USA and Australia. To date, over 130 bridges have been assessed or strengthened using the system.

Extensive verification of the Archtec method of strengthening[i] and the use of the Finite/Discrete Element (DE) analysis upon which strength assessments and designs are based, including several full-scale tests[ii,iii], has previously been undertaken. Consistent with other contemporary work on masonry arches, the verification and testing which forms the current design basis for Archtec has been focused primarily on predictions and comparison of ultimate strength. However, unlike more traditional methods of arch strength assessment such as mechanism analysis, the DE technique can also be used to predict bridge behaviour under service loads. Although some analytical work has been undertaken to investigate this it is with the advent of these tests that full verification with suitable data has been possible.

The primary objectives of the load tests was to demonstrate the efficacy of the Archtec strengthening system under service loads; to validate the use of the DE method to predict serviceability behaviour in unstrengthened and strengthened arches; to demonstrate that the retrofitted anchors contribute to the structural behaviour under service loads and that the effects are beneficial and measurable. By loading before and after strengthening, instrumenting the arch barrel and, in the case of the strengthened bridge, anchor strains, test and predicted results have been compared. This paper describes these tests, presents a selection of the results and discusses the findings in relation to the aforementioned objectives.

3 THE BRIDGE

Already placed on a strengthening programme, the bridge "Pop Bottle Bridge", located in Lincolnshire UK carries a 7.7m wide single two-way carriageway over a disused railway. Its construction and previous use make it an ideal representative of UK arch bridge stock and the disused and dismantled railway permitted easy access for test instrumentation. The bridge's principal dimensions are shown in Figure 1. Built from brick masonry and skewed at 25° the barrels of the two spans are formed from three rings laid to the English or Helicoidal Method.

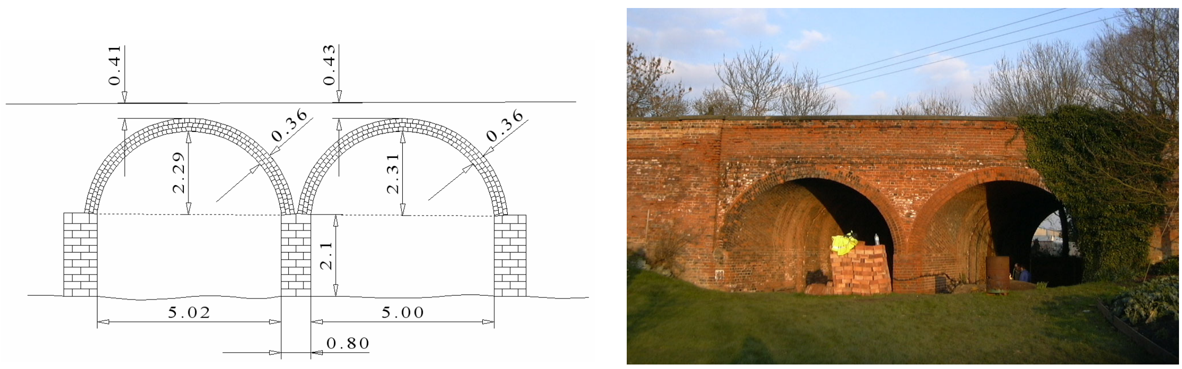


Figure 1 : Pop Bottle Bridge

Using modified MEXE and mechanism analysis the live load rating of the bridge was originally calculated to be 13 tonnes. Subsequently, a Special Assessment was undertaken by Gifford[iv], in accordance with BD 21[v], using the DE technique and the rating increased to 40/44 tonnes. However, concern over well developed cracks warranted the recommendation that a strengthening scheme be developed. The four cracks, that had developed possibly as a result of frequent heavy longitudinal braking, are shown in Figure 2.

4 ARCHTEC STRENGTHENING

The Archtec method of strengthening masonry arches[vi] involves retrofitting reinforcement (anchors) to increase the bending capacity of the arch barrel at critical positions. Critical positions are often where failures such as hinges are predicted to develop. The reinforcement, generally arranged longitudinally and installed from above, is installed using accurately positioned drilled holes as indicated in Figure 2. Manufactured in stainless steel, the reinforcement bars are grouted in place using a fabric sock grout delivery system to ensure consistent bond with the surrounding masonry. By placing reinforcement across cracks Archtec strengthening can also help stabilize crack growth.

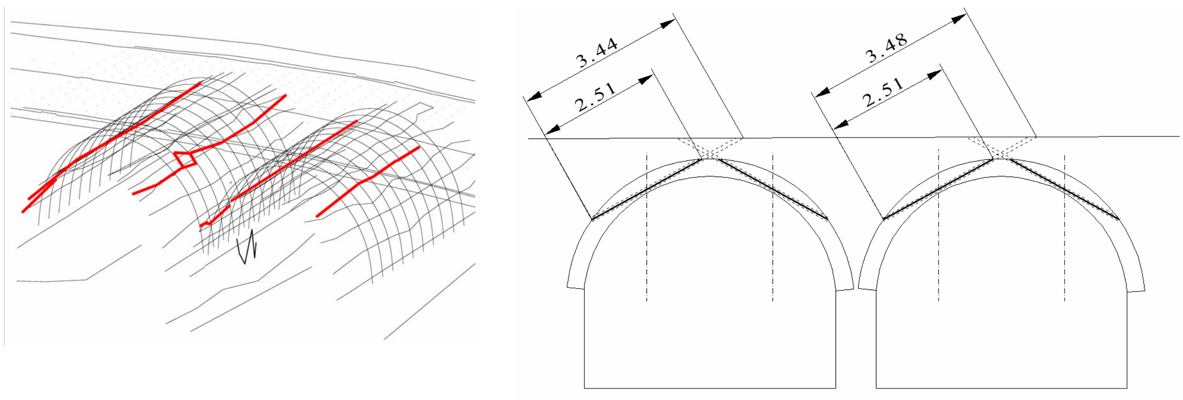


Figure 2 : Transverse Crack Locations and Proposed Archtec Strengthening

5 TEST ARRANGEMENTS

The bridge was subjected to load tests both before and after strengthening carried out in accordance with the guiding philosophy set out in BA 54/94[vii]. The first test, on the unstrengthened bridge, was carried out in January 2004 and the second test, following completion of the strengthening, in March.

5.1 Instrumentation

The bridge and strengthening anchors were instrumented, as indicated in Figure 3, on two longitudinal sections aligned under the traffic lanes to monitor the following characteristics:

i) Vertical displacements at each 1/4 span position (eight gauges in total) using wire operated LVDTs (Linear Variable Differential Transformer).

ii) Strains around the intrados of the arch barrel were measured using VW (Vibrating Wire) strain gauges with extensions to provide a gauge length of 500mm. All 72 gauges were read for each live load position to record the distribution of intrados macro strains (average strains across bricks and mortar joints).
iii) Displacements across the transverse cracks were measured using LVDTs.
iv) Axial strains in the anchors were measured using ERS (Electrical Resistance Strain) gauges attached to eight of the retrofitted anchors. Fourteen gauges in pairs were positioned on each anchor at seven locations; one pair of gauges were at the centre of the anchor closest to the intrados; the remaining pairs at the centres of the three brick rings.

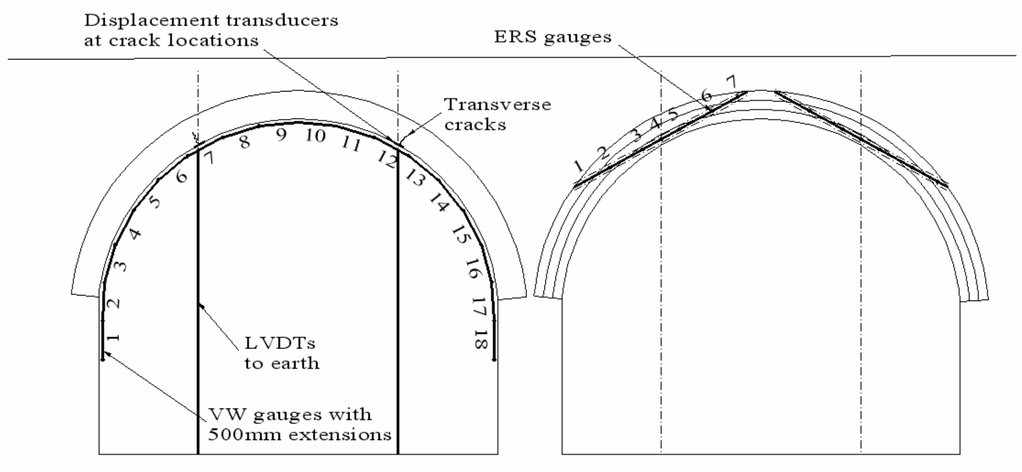


Figure 3 : Bridge Instrumentation (masonry – left span, and anchors – right span)

5.2 Loading

Live loads were applied using a pair of ballasted two axle trucks (18 tonne gross weight) with their 11.5 tonne rear axles on the bridge. During each test the lighter front axles remained off the bridge and far enough away to have no significant structural effect. Each truck arrangement involved positioning the heavy axles at designated locations along the two instrumented lines. Cases with vehicles side by side on both lines, back to back on the same line and single vehicles have been considered. In total, 28 load cases were applied to both the unstrengthened and strengthened bridge. This paper focuses on three of the most extreme load cases where the largest measured results were recorded; two vehicles positioned side by side with the 11.5 tonne axles lined up at the first quarter point (LC7), mid-span (LC8) and the second quarter point (LC9) of the south span. Figure 4 shows one such load arrangement.

6 NUMERICAL MODELLING

6.1 The Finite/Discrete Element Technique

Gifford have developed the application of the DE technique (2D plain strain assumptions), available in the explicit dynamic version of ELFEN[viii], for the analysis of masonry arches.

The DE technique is ideally suited to the analysis of non-homogenised materials such as masonry. The heart of the technique is concerned with automatic contact detection, including the calculation of friction and cohesion forces between thousands of parts and the use of material models to represent their linear and non-linear characteristics. Details of the DE technique and its application to masonry arches can be found in Owen[ix].

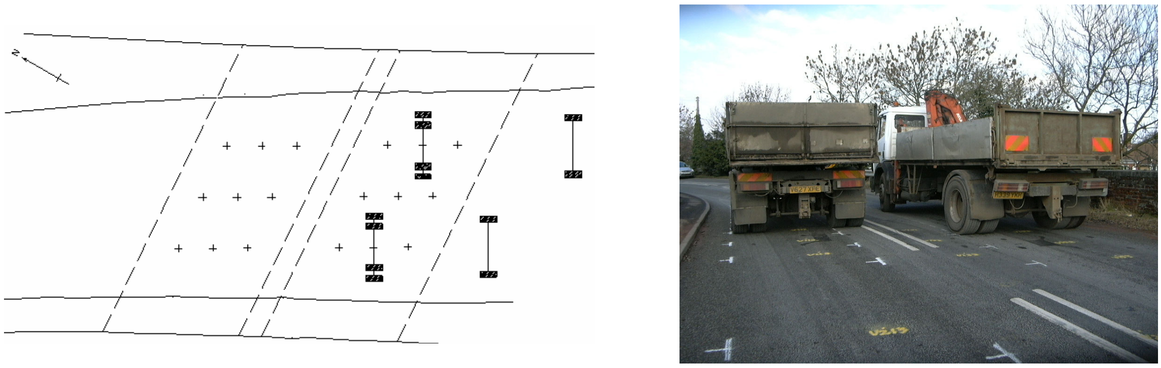


Figure 4 : Typical Load Case Arrangement

6.2 Test Simulation

The numerical model developed for the Special Assessment[iv] of the bridge was adapted to predict displacements and strains for comparison with the test results. Data from earlier site investigations, geometric surveys, masonry codes of practices[v] and verification studies[i] were used to define the model. Additionally, the four transverse cracks were explicitly represented by including a frictional cohesionless joint in the barrel, closed under permanent loads, through all rings at the crack locations. In the bridge these cracks are partially open, and during the tests the application of live load will cause movement, and possibly closure of previously open cracks, which cannot be predicted with this crack model.

The rules in BD 21[v] for distributing wheel loads transversely through the surfacing and fill were not used. Instead axle loads were applied over a 2.5m wide strip of barrel. The apparent extra strength and stiffness attributable to the effects of transverse load distribution, the effects of spandrel wall stiffening and the influence of skew were not allowed for in the analysis. The 2D analysis was, therefore, expected to give upper bound predictions for displacement and strain. Figure 5 shows the DE model of bridge.

Intrados macro strains were calculated at similar locations to the instrumentation used in the tests and averaged over a 500mm length for direct comparison with measured values. Anchor strains were calculated at gauge locations.

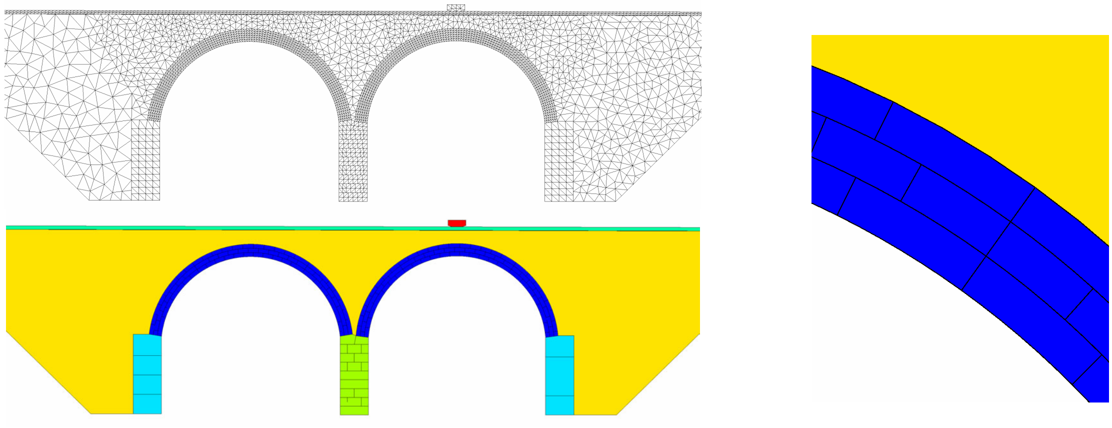


Figure 5 : DE Model of Pop Bottle Bridge used to Simulate the Load Tests

7 RESULTS AND DISCUSSION

A significant amount of data was obtained from the tests and this paper focuses specifically on the vehicle load cases described in Section 5.2. The tests are described more comprehensively elsewhere[x]. Test selection criteria used here has been based on those load cases generating the largest displacements and strains, load cases influenced the least by transverse load distribution effects and the span where most instrumentation worked successfully (a number of gauges where damaged by water ingress during strengthening installation). Hence, of the 28 load case arrangements tested LC7, LC8 and LC9 applied to the south span are discussed below. Predicted responses are discussed in Section 8.

7.1 Vertical displacements

Vertical displacements were recorded in the unstrengthened tests but, as anticipated, they were negligibly small and meaningful interpretation was difficult. Further discussion of results is not presented here as more useful data was obtained with the other instrumentation.

7.2 Crack monitoring

For each test, displacements (positive indicates cracks opening) across the four transverse cracks appearing adjacent to the approximate crack locations and the corresponding vehicle arrangements are summarised in Table 1. No results were obtained for one gauge.

Significant movement across the four pre-existing cracks was recorded which was significantly reduced following the installation of the strengthening. The maximum absolute crack displacement for the unstrengthened case was 0.027mm compared with a corresponding value of 0.003mm for the strengthened case, a reduction of approximately 90%. At many locations strengthening virtually eliminated movement.

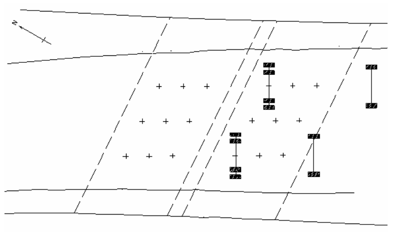
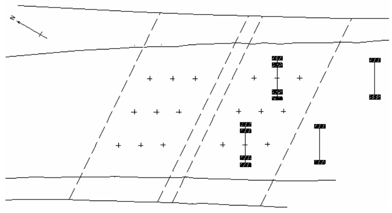
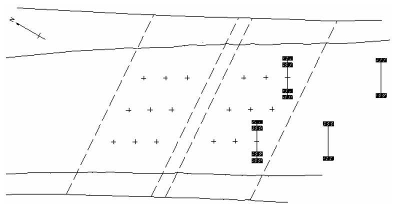
2	1		2	2	0		3	2	0		2
-3	-2		-11	-2	-2		-27	-2	-2		-14
 LC 9				 LC 8				 LC7			
-6	-6	6	-12	-7	-6	-10	-24	-6	-5	-5	0
2	2	1	3	2	2	3	1	2	2	3	2

Table 2 : Displacements Across Cracks [μm] during load tests (red - unstrengthened, blue - strengthened)

7.3 Intrados Strains

Macro strain results are illustrated in Figure 6 for the eighteen gauges around the intrados of the south span under the east side of the carriageway. Macro strains are plotted against gauge number for the unstrengthened (red) and strengthened (blue) for LC8 and LC9. The position of gauges 55 and 72 mark the north and south springings of the arch respectively and gauges 63 and 64 are at mid-span. Positive values indicate tensile strains and the vertical dotted lines show the gauges that span cracks.

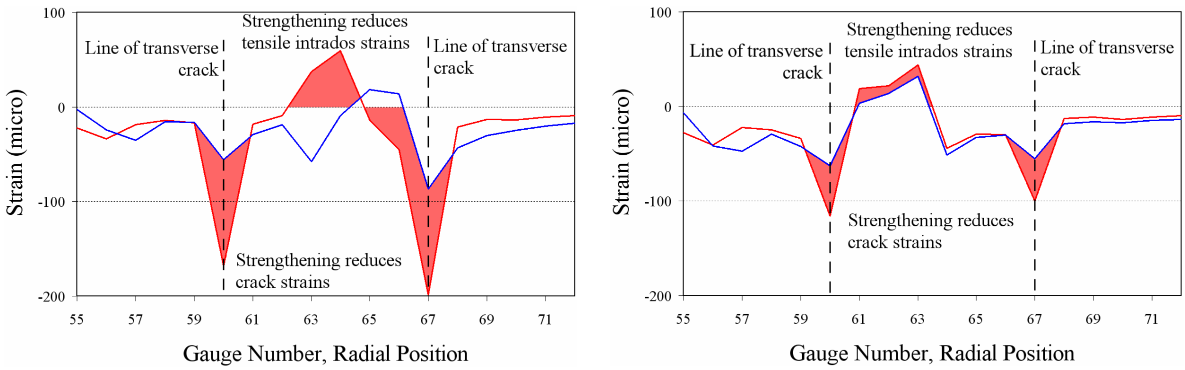


Figure 6 : Intrados Macro Strains for a Pair of 11.5 Tonne Axles (LC8 (see Table 1) – left, LC9 – right)

Significant strains were detected across the pre-existing transverse cracks and measurable strains were also recorded elsewhere. Macro strains resulting from the averaged effects of elastic and crack opening/closing behaviour following strengthening are significantly altered; peak values, both compressive and tensile, are reduced. In the figure the shaded area marks where strain reduction has occurred. Strengthening has influenced intrados strains in two ways depending on the location. Firstly, across pre-existing cracks, the tendency is for strengthening to reduce compressive strains with the measured values being greatly influenced by crack movement. Here the largest reductions in strain occur. Secondly, away from the cracks there is a general reduction in tensile strain. For the two cases illustrated this trend is most notable at the centre of the span between the quarter points.

7.4 Anchor Strains

Anchor strains were successfully measured along the length of the eight instrumented anchors indicating their participation in supporting live load. All measured results lay between a minimum value of -51 με and a maximum value of 30 με. The measured results are discussed further in relation to the predicted values in Section 8.

8 COMPARISON OF PREDICTED AND TEST RESULTS

8.1 Intrados Strains

Using the DE model and methodology described in Section 6.2, intrados macro strains have been calculated and are compared against measured values in Figure 7. The figure shows two graphs each with four curves; measured values (thin red dashed lines); measured values with the contribution attributed to movement across pre-existing cracks removed (red); predicted values (thin green dashed); predicted values factored to make some allowance for 3D behaviour (green). Strains (positive values are tensile) are plotted against gauge numbers that mark the relative position around the intrados, see Figure 3.

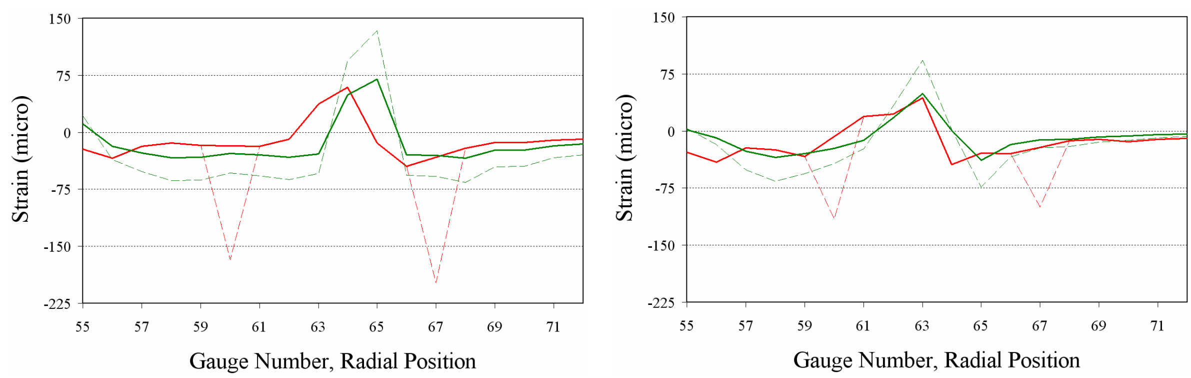


Figure 7 : Unstrengthened Test – Predicted Versus Test Intrados Macro Strains for LC 8 (left) and LC9 (right)

Adjustment to the raw data, both measured and predicted, is necessary to make valid comparisons between the two sets of results. Closure of pre-existing cracks cannot be predicted with the DE model and so the respective results have been removed from the test data. Additionally, the 2D analysis was expected to yield upper bound predictions and factoring to take account of 3D effects was anticipated. For the purposes of comparison, the predicted results have been reduced by a factor of 2.0 to illustrate correlation in the distribution of predicted and measured strains. With these adjustments the measured (red) and predicted (green) results compare very well.

Figure 8 shows similar results for the Archtec strengthened tests. Again with adjustments made to the raw data for the effects attributable to initial crack widths and full 3D behaviour, measured (blue) and predicted (green) results compare very well.

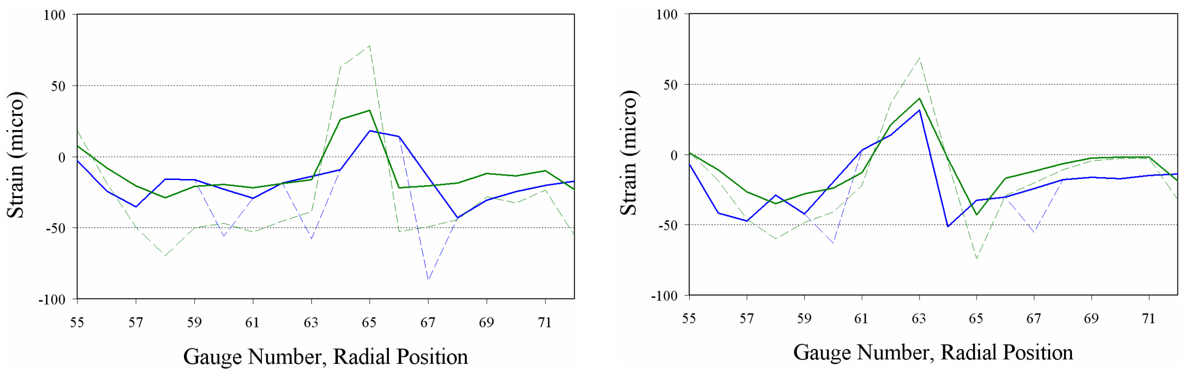


Figure 8 : Strengthened Test – Predicted and Test Intrados Macro Strains for LC8 (left) and LC9 (right)

8.2 Anchor strains

Predicted anchor strains and test values are compared in Figure 9. The figure shows predicted (green) and measured (blue) strains (measured strains for the east side of the carriageway only), corresponding to load cases LC8 and LC9. Micro strains are plotted against their relative position along the two anchors (see Figure 3); numbers 77 to 111 relate to the anchor in the north half of the span, numbers 115 to 149 to the anchor in the south half. Positive strains indicate tension with 50 με indicating a bar stress of 10 N/mm^2.

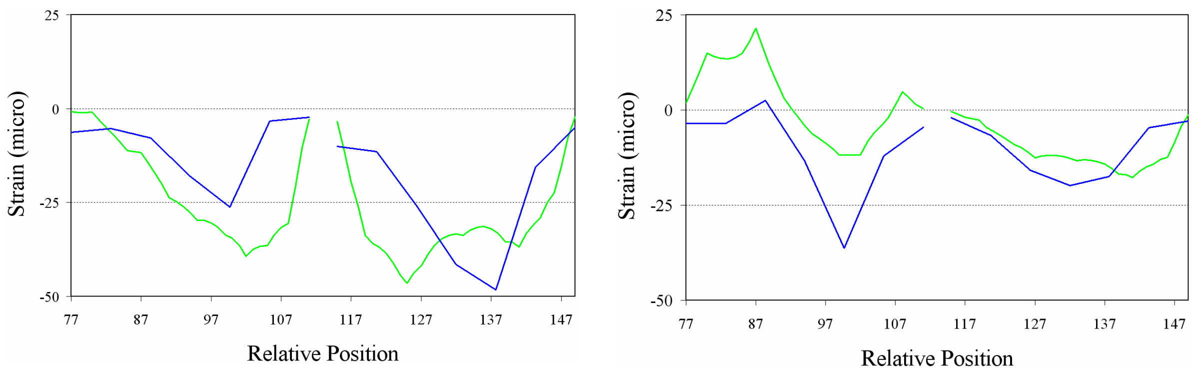


Figure 9 : Predicted and Test Anchor Strains for LC 8 (left) and LC9 (right)

Both the distribution and magnitude of the results was found to compare well. Further adjustment might be warranted to cater for finite crack widths and 3D behaviour but the approach to use is less clear than for intrados strain results. A pattern is discernable which suggests that measured strains do exhibit additional compression in the proximity of the cracks (close to 97 and 127 on the abscissa) compared with predicted values.

9 CONCLUSIONS

The principal conclusions drawn from the unstrengthened and strengthened bridge tests, and from the accompanying predictions of bridge behaviour using the DE technique are as follows:

i) Based on strain measurements, the Archtec anchors used to strengthen the bridge are stressed under working loads and are contributing to the bridge's stiffness.
ii) Archtec strengthening reduces tensile intrados strains and, therefore, reduces the likelihood of loosening masonry under live loads.
iii) Intrados strain measurements have demonstrated that Archtec anchors positioned across transverse cracks reduce their closure under live loads. A benefit of this behaviour would be the reduction in load cycle derived hysteretic damage; opening and closing of cracks under traversing traffic. Reducing this type of damage will almost certainly be beneficial to the bridge service life.
iv) Predictions of strain and displacement made with DE numerical simulations agree well with measured values, both masonry and anchors. Results are upper bound because of skew behaviour, transverse load distribution and spandrel wall stiffening.
v) It has been demonstrated that Archtec strengthening can be serviceability limit state (deflections, strains and stress ranges).

10 REFERENCES

[i] C.L. Brookes, "Archtec – Verification of Structural Analysis", *Gifford and Partners*, Unpublished report B1660A/V10/R02 Rev C, (2004).

[ii] S.K. Sumon, "Load test to failure on a ring-separated arch repaired using Cintec Anchor Systems", *Transport Research Laboratory*, Unpublished project report PR/CE/61/98, (1998).

[iii] A. Sexton and G.I. Crabb, "Loading to failure of an Archtec strengthened brick arch using Cintec Multi-bar anchors", *Transport Research Laboratory*, Unpublished report PR/IS/59/01, (2001).

[iv] L. Mabon, "Pop Bottle Bridge – Arch Barrel Special Assessment", *Gifford and Partners*, Unpublished report B1660A/214/R01, (2002).

[v] The Highways Agency, Standard BD 21/01 "The Assessment of highway bridges and structures", *Design Manual for Roads and Bridges*, Vol.3, Section 4, Pt. 3, (2001).

[vi] C.L. Brookes and G.P. Tilly, "Novel Method of Strengthening Masonry Arch Bridges", *8th International Conference on Structural Faults and Repair*, (1999).

[vii] The Highways Agency, Advice Note BA 54/94, "Load testing for bridge assessment" *Design Manual for Roads and Bridges*, Vol.3, Section 4, Pt. 8, (1994).

[viii] Rockfield Software Limited, ELFEN version 3.0.4 (Elfendyn v3.3.24) Archtec version, University of Wales Swansea, (1997-2004).

[ix] D.R.J. Owen, D. Peric, N. Petrinic, C.L. Brookes and P.J. James, "Finite/Discrete Element Models for Assessment and Repair of Masonry Structures", *2nd International Arch Bridge Conference*, (1998).

[x] C. L. Brookes, "Archtec - Pop Bottle Bridge Supplementary Load Test Report", Gifford and Partners, Unpublished report B1660A/V16/R03, (2004).