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ARCHTEC

POP BOTTLE BRIDGE SUPPLEMENTARY LOAD TEST

SUMMARY REPORT

Gifford
CONSULTING ENGINEERS

ARCHTEC

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SUMMARY REPORT

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POP BOTTLE BRIDGE SUPPLEMENTARY LOAD TEST

SUMMARY REPORT

C O N T E N T S

1. INTRODUCTION

Extensive verification of the Archtec method of strengthening masonry arch bridges and the use of ELFEN Finite/Discrete Element (DE) analysis upon which the assessment and designs are based, including several full-scale tests, has previously been undertaken.

Consistent with other contemporary work on masonry arches and current assessment/design methods, the verification and testing which forms the current design basis for Archtec is focussed primarily on predictions and comparison of ultimate strength.

However, unlike other methods of arch assessment/design, DE analysis also allows the consideration of arch behaviour in the elastic range under service loads and although some analytical work has been undertaken to investigate this it had not been possible to fully verify it in the absence of suitable test data.

In the course of discussions with the Bridge Owners Forum (BOF) – Masonry Arch Sub-group, regarding the more widespread adoption of Archtec, the benefit of a Supplementary Load test to investigate the behaviour of unstrengthened and strengthened arches under service loads was identified. Following a meeting of the BOF with Gifford and Partners and Cintec on 5 September 2003 Pop Bottle Bridge in Lincolnshire was identified as suitable for such testing.

Pop Bottle Bridge had already been allocated for strengthening using the Archtec system and the Client, Lincolnshire County Council, kindly agreed that Supplementary Load tests could be carried out in parallel with the strengthening works.

This report describes the Supplementary Load tests that were undertaken in the first quarter of 2004 before and after strengthening the bridge with the Archtec system

2. OBJECTIVES

Proposals for the tests were set out in Gifford Report, 'Proposed Monitoring of Pop Bottle Bridge'(1), October 2003, which were finalised in liaison with the BOF – Masonry Arch Sub-group.

The primary objective of the Supplementary Load tests was to demonstrate the efficacy of the Archtec strengthening system under service loads, namely:

- To validate the use of the ELFEN DE (DE) analytical method to predict serviceability behaviour in un-strengthened 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.
- And, if possible, to investigate potential long-term methods for monitoring the continued efficacy of the strengthening based on the dynamic response of the bridge and/or strain measurement of the anchors.

The bridge was to be loaded before and after strengthening using two 18 tonne lorries and instrumented to record intrados strains, vertical displacements and anchor strains in the strengthened bridge.

The optional possibility of investigating longer term methods for monitoring the health of masonry arch bridges based on dynamic response of the structure was identified in the proposal. However expediency of time and cost precluded such an investigation on this occasion.

3. POP BOTTLE BRIDGE

Pop Bottle Bridge, which had recently been transferred from the BRB Residuary Body to Lincolnshire County Council, was selected for these tests (details of the bridge are contained in Appendix A). Its construction and previous use make it an ideal representative of British arch bridge stock and the disused and dismantled railway permit easy access for test instrumentation.

An Archtec design, including a full geometric survey had already been prepared to strengthen Pop Bottle Bridge to a live load rating of 40/44 tonnes. Details of the Archtec strengthening are contained in Appendix B.

Pop Bottle Bridge (National Grid reference TF 439 223) is a skewed two-span brick masonry arch bridge. Each span is approximately 5.0m measured in the skew direction and rise at their crowns 2.3m. The barrel is built from three rings of brick with bricks laid to the English or Helicoidal Method and has a skew angle of 25˚. The overall barrel thickness is 355mm. The central pier is 800mm wide and approximately 2.1m high. Figure 2.1 shows an elevation and the road above.

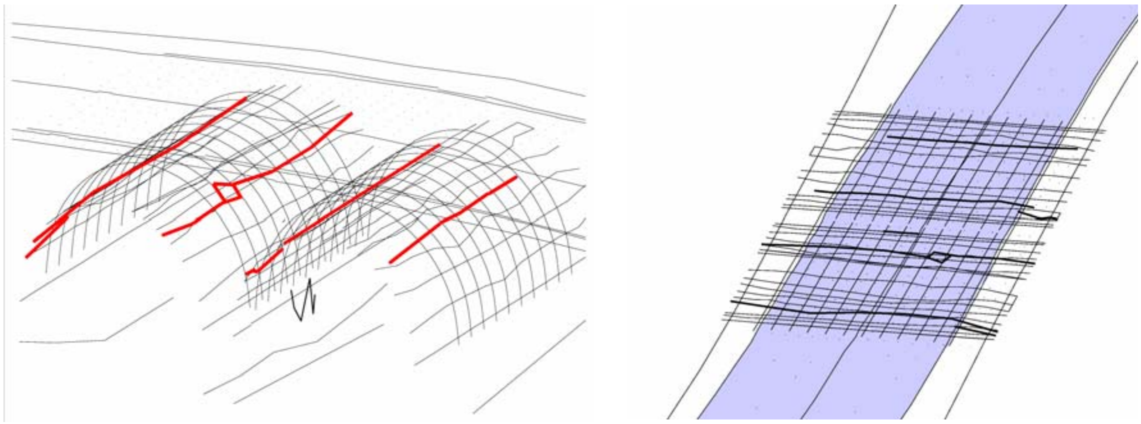
Part of side elevation

Road above

Figure 2.1 Pop Bottle Bridge

The bridge carries the B1359, a single two-way carriageway approximately 7.7 m wide, over a disused railway. The carriageway has a grass verge on one side and a footway on the other. The permitted traffic speed limit is 40 mph. There is clear access to the underside although a fence and trees prevent through passage.

Using modified MEXE and mechanism analysis the live load rating of the bridge was originally calculated to be 13 tonnes(2). Later, a Special Assessment was undertaken by Gifford(3) using the DE technique and the rating increased to 40/44 tonnes. However, the presence of four transverse cracks warranted the recommendation that an Archtec strengthening scheme be developed. These four transverse cracks at approximately quarter span locations are shown in Figure 2.2 together with the Archtec bridge survey contours. Cracks are shown in red.



Red lines mark crack positions in the barrel *Plan – Showing carriageway*

Figure 2.2 Archtec Survey Contours

The transverse cracks at the quarter points of the arches were of some concern and suggest the beginning of hinging failure at the quarter points due to live loading effects, possibly as a result of frequent heavy longitudinal braking. Archtec strengthening would provide additional capacity to achieve the required load rating and provide stitching across the hinge locations.

4. TEST ARRANGEMENTS

4.1 Overview

Pop Bottle Bridge was subjected to Supplementary Load tests both before and after strengthening with the Archtec system.

The tests were carried out in accordance with the guiding philosophy set out in BA 57/94 – Load Testing for Bridge Assessments.

The bridge and strengthening anchors were instrumented, as indicated in Figure 4.1 to monitor the following key parameters;

i) Vertical displacements at quarter points using LVDTs.

ii) Strains around the intrados of the arch barrel over 500mm gauge length using VW gauges with extensions.

iii) Displacement transducers at locations of specific cracks around the intrados.

iv) Electrical Resistance strain gauges attached to the strengthening anchors.

The bridge was closed to general traffic on the occasion of each test and the loading comprised two, ballasted, two axle lorries (18 tonne gross weight, 11.5 tonne on rear axle).

The first test, on the unstrengthened bridge, was carried out on the 12 January 2004 and the second test, following completion of the strengthening, on the 1 March 2004.

Invitations to witness the tests were extended to interested parties and a number of third parties were represented at the second test.

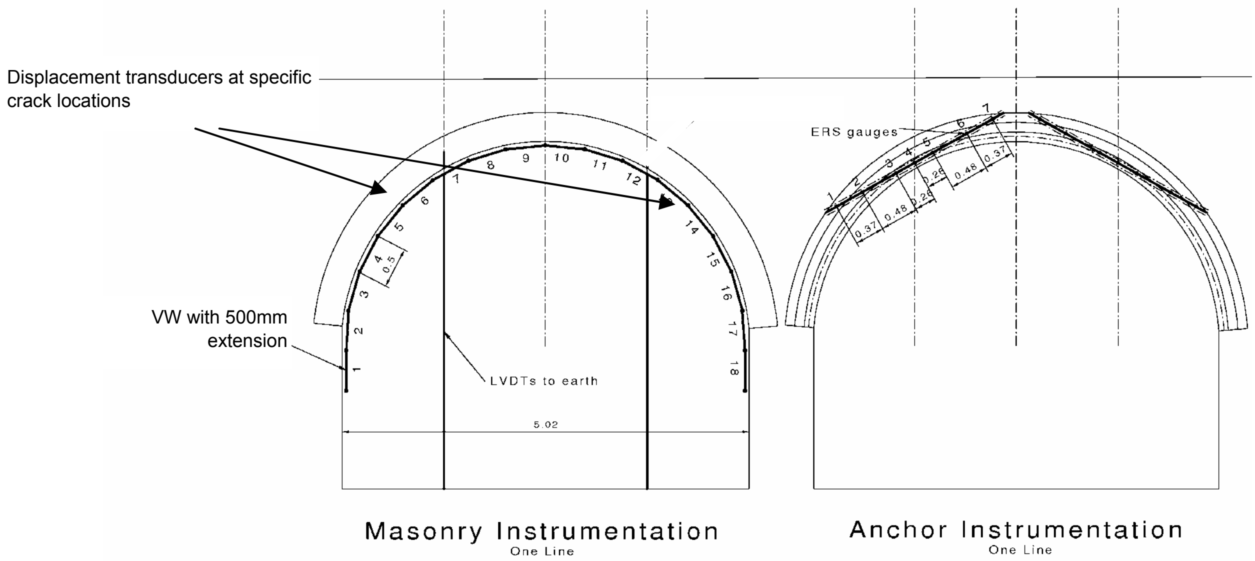


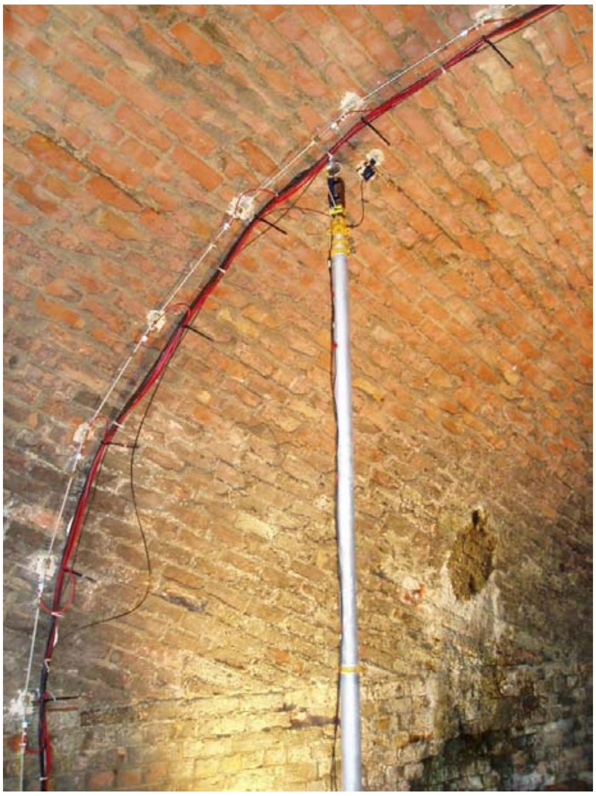
Figure 4.1 Instrumentation

4.2 Instrumentation

The instrumentation outlined above was installed on two longitudinal sections through the bridge as indicated in Figure 4.3. The instrumented sections were aligned under the centre of each of the two loaded lanes. Full details of instrumentation arrangements are contained on the drawing included in Appendix C and illustrated in Figures 4.2.

a. South span

b. Typical crack and displacement LVDTs



c. Pole supporting displacement LVDT and part of a line of VW strain gauges

Figure 4.2 General View of Instrumentation

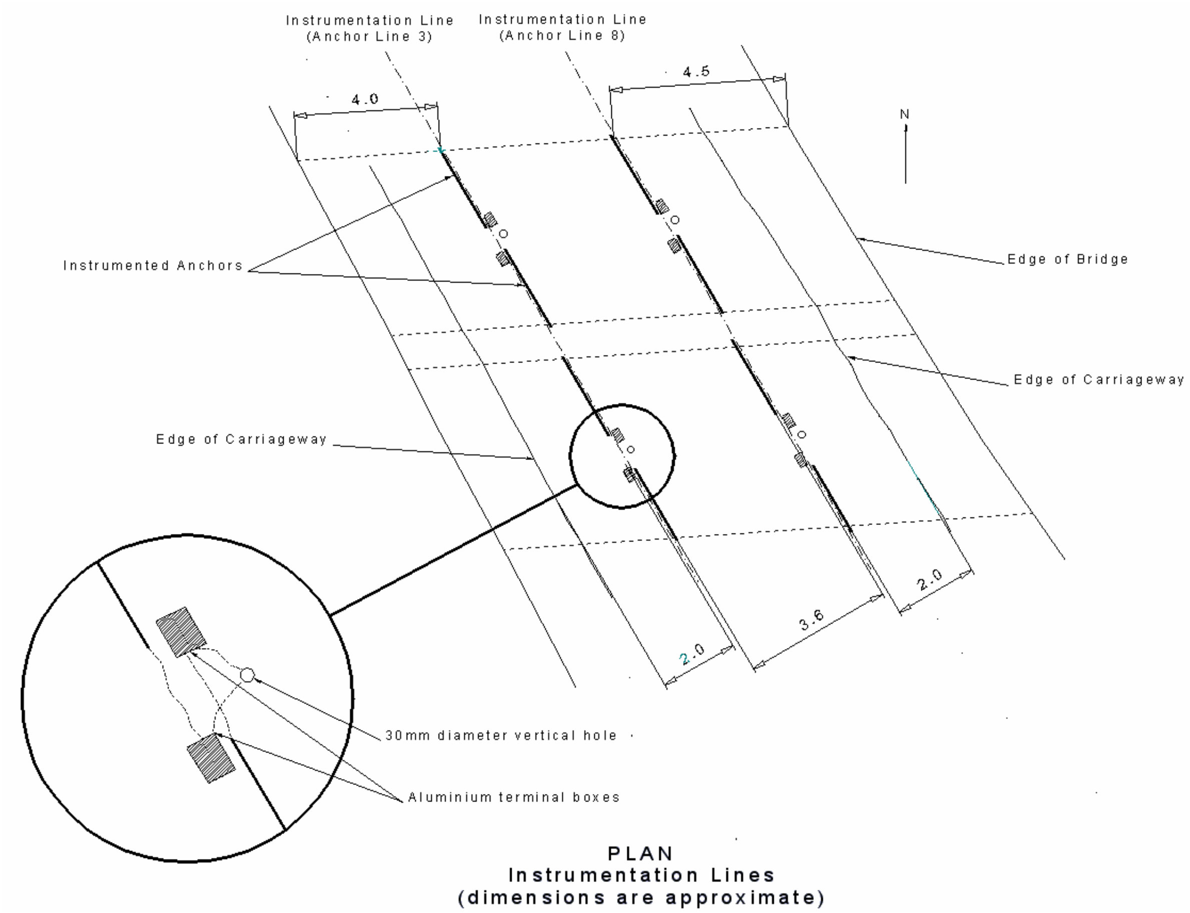


Figure 4.3 Plan of Bridge showing Instrumented Longitudinal Sections

The instrumentation that was installed is summarised in Table 4.1.

Table 4.1 Summary of Instrumentation

Method of Instrumentation				Accuracy	Range
Section	Description	Type	Number		
4.2.1	Vertical displacement at ¼ span positions	LVDT (pole mounted)	8	0.003 mm	±12.5mm
4.2.2	Intrados circumferential strain (macro) measurement	0.5m VW (glued to masonry)	72	0.3 με	±450 με
4.2.3	Crack monitoring	LVDT (glued to masonry)	8	0.0003 mm	±2.5mm
4.2.4	Anchor axial strain measurement	ERS (glued to rebar)	112	1 με	±200 με

Notes

i) LVDT is an abbreviation for Linear Variable Differential Transformer.
ii) VW is an abbreviation for Vibrating Wire strain gauges. 500mm extension pieces were provided to achieve the required gauge length.
iii) ERS is an abbreviation for Electrical Resistance Strain gauge.

Each part of the instrumentation is described in more detail in the following sections.

4.2.1 Displacement Measurements

Live load vertical displacements at each 1/4 span position were measured on two longitudinal sections for each span (eight gauges in total). Wire operated LVDTs were used with a potential reading accuracy of 0.001mm. However, after reviewing the data recorded during the first test, noting the very small displacements, and the likelihood of accidental movement of the support poles, these measurements were abandoned for the second test.

4.2.2 Intrados Strain Measurement

Eighteen VW strain gauges with extensions to provide a gauge length of 0.5m were set out on the intrados of each arch on each instrumented section as indicated in Figure 4.1. All 72 gauges were read for each live load position to build up a picture of intrados macro strains (average strains across bricks and mortar joints).

4.2.3 Crack Monitoring

As explained in Section 3.0 four notable transverse cracks exist across the full width of the intrados of the bridge and displacement transducers were installed across each of these cracks close to both the instrumented sections.

4.2.4 Anchor Instrumentation

ERS (Electrical resistance strain) gauges were attached to eight of the retrofitted anchors on or close to the instrumented sections. Fourteen gauges in pairs were positioned on each anchor at seven locations along the anchors so that one pair of gauges were at the centre of the anchor closest to the intrados and the remaining pairs at the centres of the three brick rings, see Figure 4.1. Gauges were mounted in pairs so that some redundancy existed should gauges be damaged during installation.

Although the primary role of the instrumented anchors was to measure anchor strains during the load test the wiring to these gauges has been left in place and they could potentially be monitored again in the future.

4.3 Loading

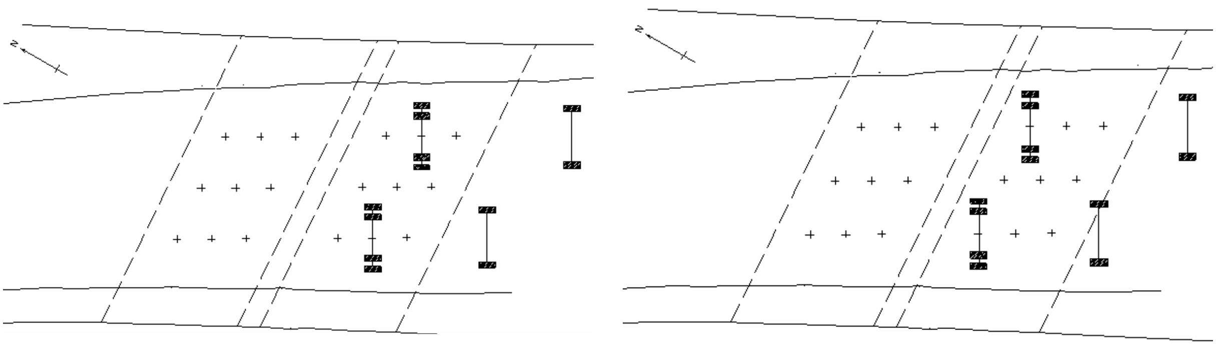
The loads were applied using two ballasted two axle lorries (18 tonne gross weight, similar to vehicle reference RE as defined in Annex D of BD 21/01). Only the rear axle, with a nominal weight of 11.5 tonnes, was of interest in each case. The front axles were far enough away to have no significant effect. Each axle load in the two axle vehicle is statically determinant and, as the axle loads were not varied in the tests they were measured before each test on a Department of Transport weigh bridge.

Trucks side by side – LC7 *Positioning of 11.5 tonne axle*

Figure 4.3 Application Of Test Loads Using Ballasted Lorries

The lorries were positioned in a variety of locations along the two instrumented lines, including cases where both vehicles were maintained effectively in line and others with only one vehicle on the bridge to provide data in relation to transverse distribution. In total 28 load cases were applied to both the unstrengthened and strengthened bridge. Photographs of typical load case arrangements are shown in Figure 4.3. Full details of all Test Load positions are provided in Appendix D.

This report focuses on the six most extreme load cases where the largest measured displacements and strains were recorded. These cases are those with two vehicles positioned side by side with the 11.5 tonne axles lined up in turn with span quarter points, (ie North span first quarter, mid span, second quarter and ditto South span) see Figures 4.3 and 4.4 for representative cases.



Load case LC8 – Pair of axles at mid-span *Load case LC9 - Pair of axles at the pier quarter point*

Figure 4.4 Selected Load Case Arrangements Used on the South Span

5. NUMERICAL MODEL

The numerical model that was developed by Gifford for the Special Assessment of Pop Bottle Bridge[(3)] has also been used to predict displacements and strains for comparison with the test results. Several minor modifications were made as follows.

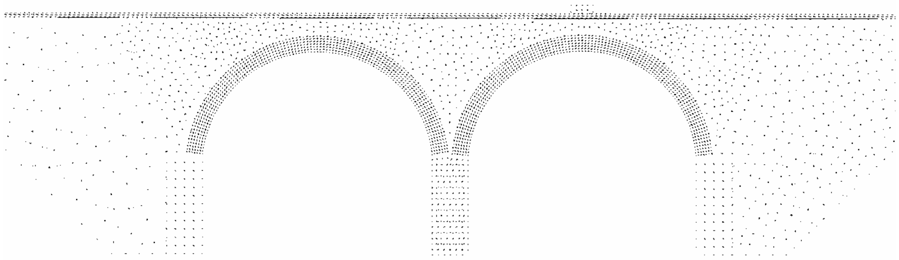
i) The four transverse cracks were explicitly represented in the modelled barrel by including a frictional cohesionless joint in the barrel through all rings at the crack locations. These cracks at approximately the 1/5 span, have been represented as being closed under permanent loads, prior to the application of the live load.

ii) In place of the codified live load cases used for assessment each of the 28 load cases used in the tests have been modelled by applying 11.5 tonne axles.

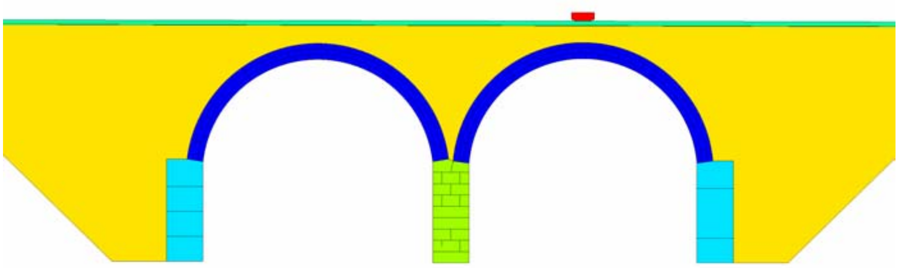
iii) The rules in BD 21 for distributing wheel loads transversely through the surfacing and fill have not been used. Instead axle loads have been applied over a 2.5m wide strip of barrel.

The extra strength and stiffness attributable to the effects of transverse load distribution, the effects of spandrel wall stiffening and the influence of skew have not been allowed for in the analysis. The analysis is based on a conservative two dimension plain strain representation and, therefore, is expected to give upper bound predictions for displacement and strain.

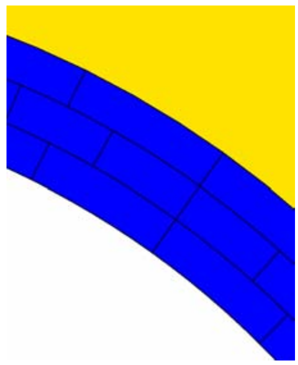
Figure 5.1 shows the DE model of Pop Bottle Bridge and the modelled transverse cracks.



Finite Element Mesh



Discrete Element Mesh



DE Mesh of Crack

Figure 5.1 DE Model of Pop Bottle Bridge used to Predict Test Responses

6. RESULTS

6.1 Tests

A significant amount of data involving 560 data sets has been obtained from both tests. Here each data set consists of all measured results for a particular quantity (crack displacement, intrados macro strain or anchor strain) on one instrumented line and for one span, see Figure 4.3. To avoid excessive amounts of data-processing the results described in this report focus on the six most extreme vehicle load cases as described in section 4.3, and for strain measurement comparisons, consider only those results taken for the South span. The basis for selecting these data sets is further explained below.

i) Load cases LC7, LC8 and LC9 over the South span and LC10, LC11 and LC12 over the north span have been considered for crack width displacement comparisons. Apart from generating the largest displacements and strains these load cases are also influenced the least in terms of transverse load distribution boundary effects; the applied load spreading laterally to unloaded parts of the bridge.

ii) Results corresponding to load cases LC8 and LC9 have been presented for discussion of trends based on Intrados macro strain measurements. More gauges worked successfully for the South span for both tests than the North span. These same load cases have also been used to assess the behaviour of the anchors in the second test.

The results of these representative load cases, six for crack measurement and two for strain results, applied to both the unstrengthened and strengthened bridge are set out here and discussed in section 6.

6.1.1 Vertical Displacement

Vertical displacements were recorded in the first test but, as anticipated, they were negligibly small under the applied serviceability loads and meaningful interpretation was difficult. Other monitored parameters produced much more useful data and thus detailed consideration of the vertical displacements is not presented here.

6.1.2 Crack Monitoring

Displacements across the four transverse cracks which were monitored during both tests are summarised in Figure 6.1. Positive values indicate cracks opening. Plan inserts showing the vehicle arrangement used for each load case are included, and viewed in plan the values for crack displacements appear adjacent to the approximate crack locations. Exact crack positions are shown in Figure 2.2. No results were obtained from the North East gauge in the South span.

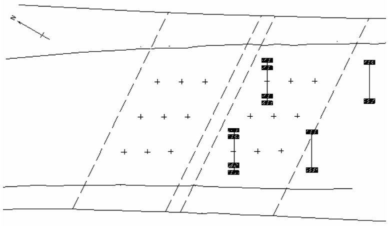
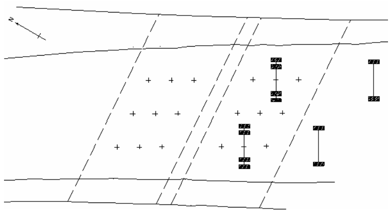
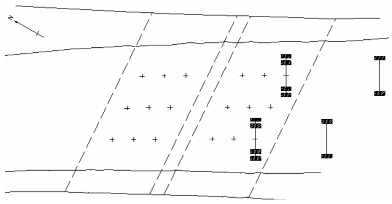
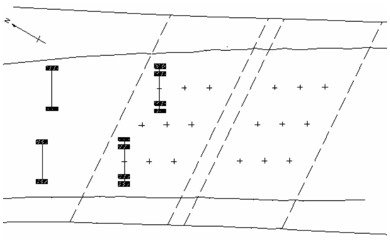
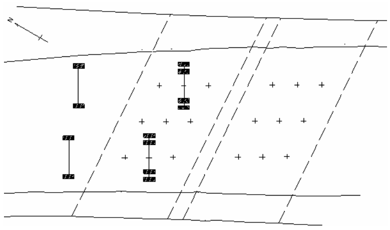
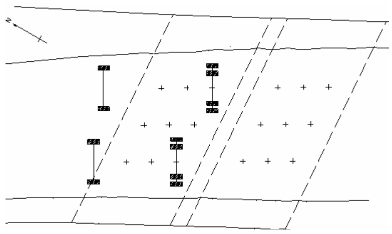
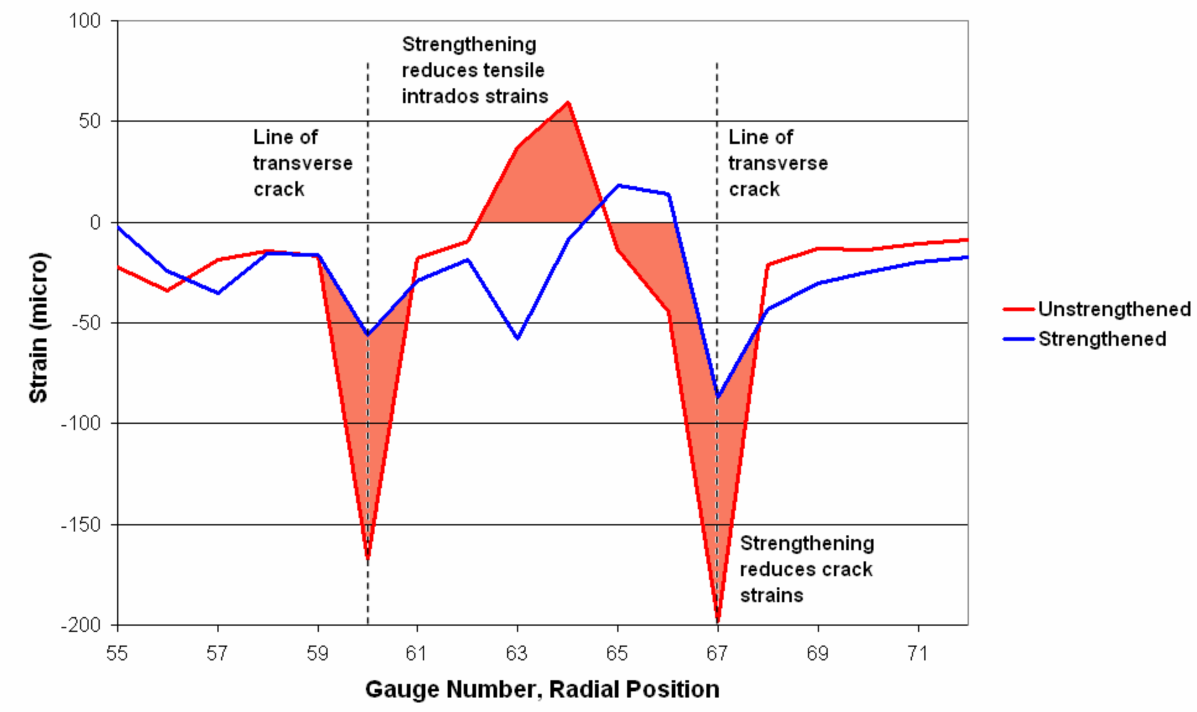
Crack Displacements [mm x 0.001] – RED Unstrengthened, BLUE Strengthened											
South Span – Double Vehicle Load Cases											
2	**1**		**2**	**2**	**0**		**3**	**2**	**0**		**2**
-3	**-2**		**-11**	**-2**	**-2**		**-27**	**-2**	**-2**		**-14**
 **LC 9**				 **LC 8**				 **LC7**			
-6	**-6**	**6**	**-12**	**-7**	**-6**	**-10**	**-24**	**-6**	**-5**	**-5**	**0**
2	**2**	**1**	**3**	**2**	**2**	**3**	**1**	**2**	**2**	**3**	**2**
North Span – Double Vehicle Load Cases											
-3	**3**		**6**	**9**	**4**		**2**	**6**	**2**		**2**
8	**-10**		**6**	**-46**	**-12**	**2**	**9**	**-28**	**-10**	**2**	**9**
 **LC 10**				 **LC 11**				 **LC 12**			
-19	**-7**	**2**	**8**	**-46**	**-12**	**2**	**9**	**-28**	**-10**	**2**	**9**
2	**2**	**1**	**2**	**4**	**2**	**1**	**2**	**3**	**2**	**1**	**1**

Figure 6.1 – Displacements Across Cracks During Load Tests

6.1.3 Intrados Strains

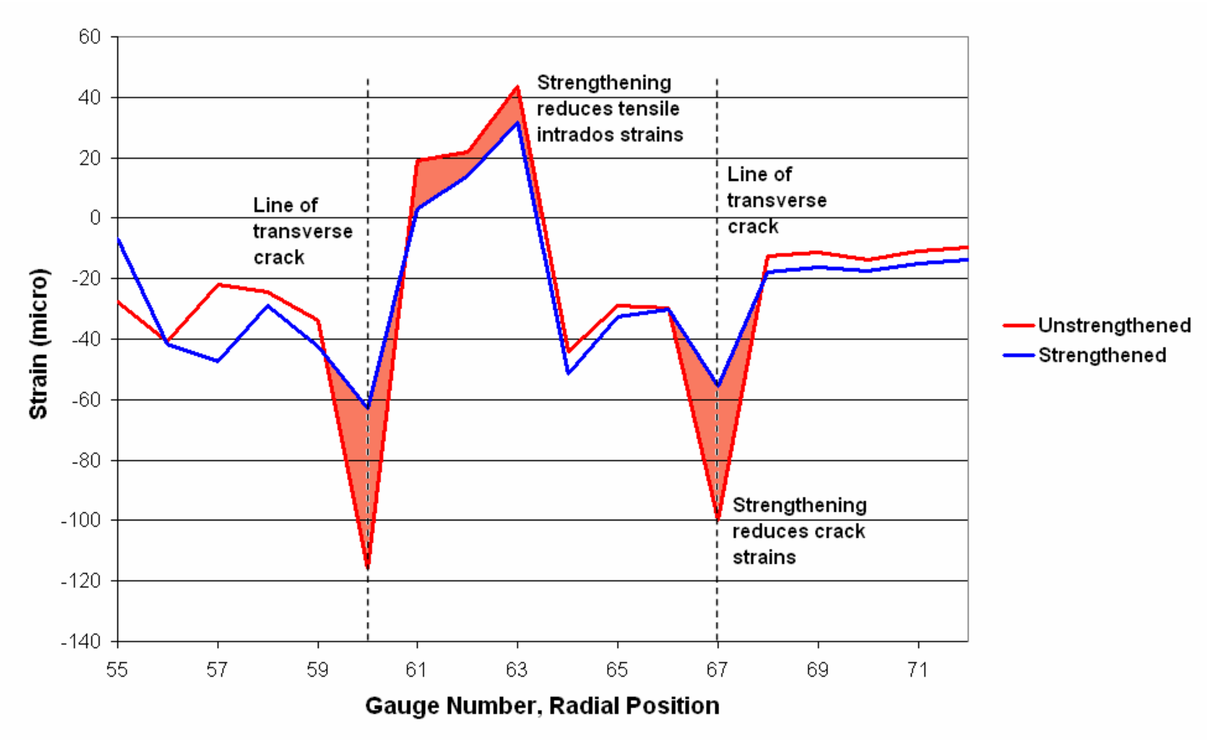
The intrados macro strain measurements, taken using 0.5m gauge lengths to record average strains resulting from both brick unit/mortar and crack behaviour, has provided data sets for both tests in most locations for all 28 load cases. Some gauges were lost due to water damage during the installation of strengthening; 28% on the North span, 5% on the South span.

Representative results are illustrated in Figure 6.2a and 6.2b for load cases LC8 and LC9 respectively. The results are presented for the eighteen gauges around the intrados of the south span on the east instrumented line; approximately along anchor line 8, see Figure 4.3. Macro strains are plotted against gauge number for the unstrengthened (red curve) and strengthened (blue curve) tests. 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. Vertical dotted lines mark the location of the cracks and indicate the gauges that would therefore span cracks.



Load case LC8 – A pair of 11.4 tonne axles at mid-span of the south span

Figure 6.2a Representative Intrados Macro Strains



Load case LC9 - A Pair of 11.5 tonne axles at pier quarter-span of the south span

Figure 6.2b Representative Intrados Macro Strains

6.1.4 Anchor Strains

Anchor strains along the length of the eight instrumented anchors were recorded for the strengthened tests and all 28 load cases. The 112 gauges were arranged in pairs and a total of 9% failed to operate correctly or were damaged during installation, leading to no significant loss in recordings. All measured results lay between a minimum value of -51 $\mu\varepsilon$ and a maximum value of 30 $\mu\varepsilon$.

Figure 6.3 shows the measured strains for the south span, east side corresponding to load cases LC8 and LC9. Micro strains are plotted against their relative position along the two anchors, see Figure 4.1; 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.

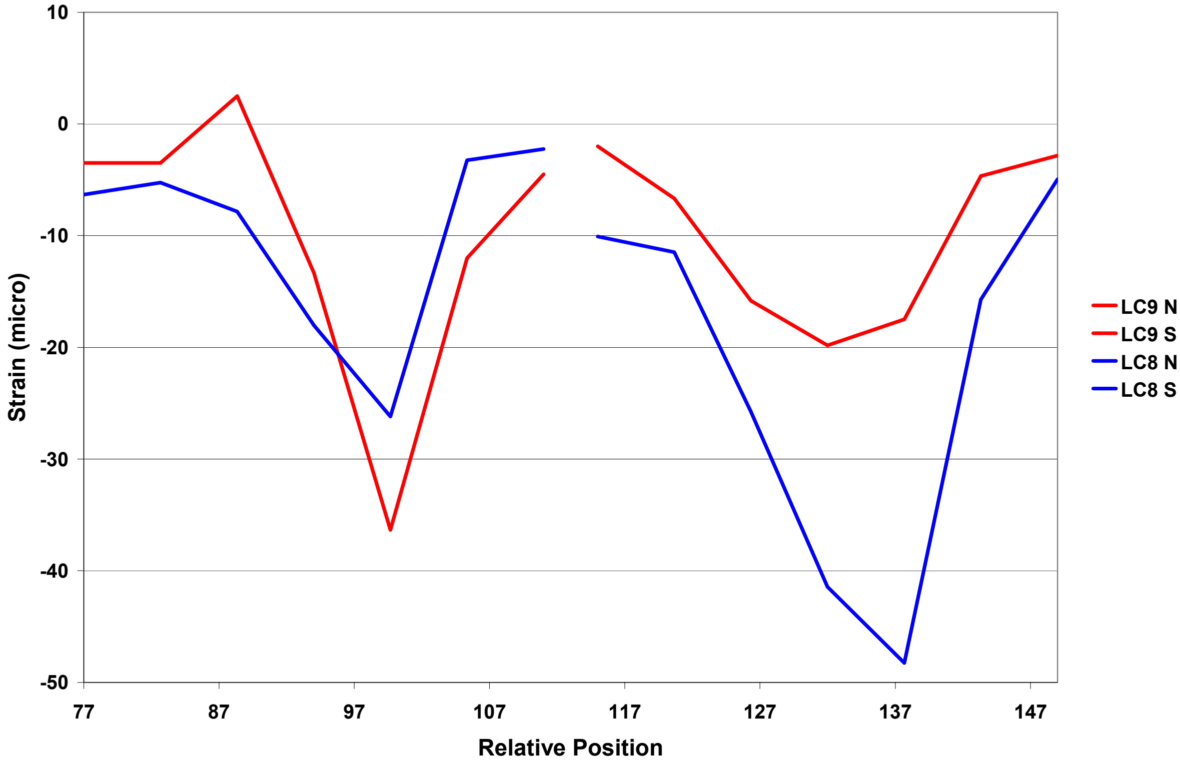
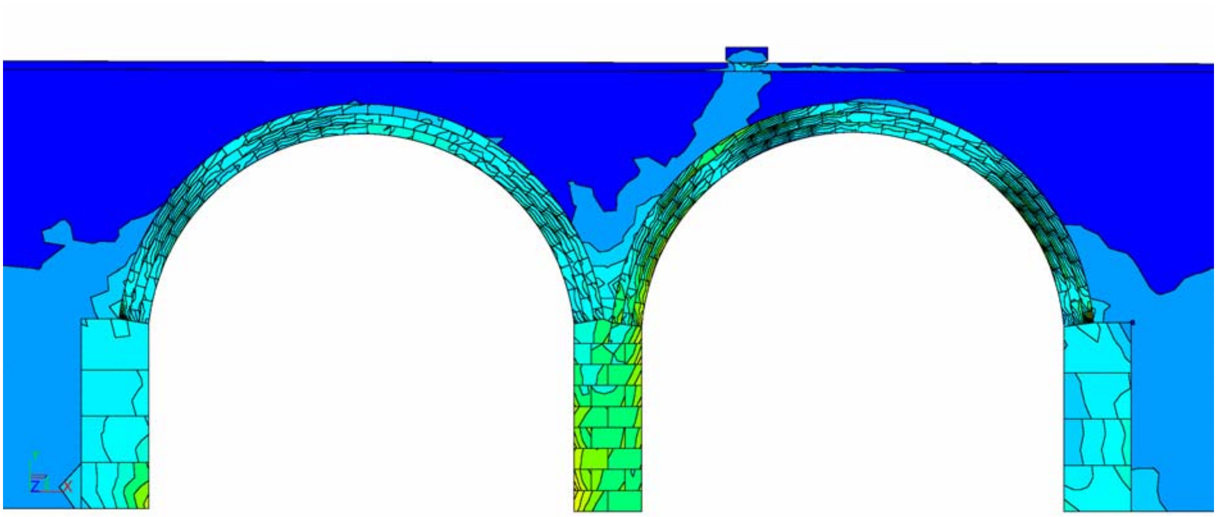


Figure 6.3 Representative Anchor Strains – East side of South Span

6.2 Predicted

Using the numerical model described in section 5, intrados macro strains and anchor strains have been calculated at similar locations to the instrumentation used in the tests. Average strains have been calculated over a 0.5m length for direct comparison with measured values and include the effects of elastic strain and mortar gaps opening. Although the cracks have been included as discontinuities in the model they have been modelled as initially closed for simplicity. Thus where application of live load has caused closure of previously open cracks these movements have not been predicted.

Figure 6.4 shows the results of a typical test simulation and in this case illustrates principal compressive stresses in the barrel and file.



Show principal compressive stresses; blue, green, yellow shoe increasing levels of compression

Figure 6.4 Typical Predicted Results – South Span, Load Case LC9

7. DISCUSSION

7.1 Test Results

7.1.1 Crack Monitoring

Significant movement across the four pre-existing cracks was recorded. Movement across the cracks was significantly reduced following the installation of the strengthening anchors demonstrating their effectiveness in tying the masonry across such discontinuities. Referring to Figure 6.1, the maximum absolute crack displacement for the unstrengthened case was 0.046mm compared with a corresponding value of 0.009mm for the strengthened case indicating a reduction of approximately 80%. At many locations strengthening virtually eliminated movement with readings close to the gauges resolutions.

The implications are that Archtec strengthening prevents significant movement of pre-existing transverse cracks under live load. The main 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.

7.1.2 Intrados Strains

Measurable strains were recorded around the intrados of the both arches and the data appears sensible and logical. As expected, significant strains were detected across the pre-existing transverse cracks and measurable strains were also recorded elsewhere. These macro strains resulting from the averaged effects of elastic and crack opening/closing behaviour following Archtec strengthening are significantly altered. Peak values, both compressive and tensile, are reduced. The representative results shown in Figure 6.2 illustrate this phenomenon; the shaded area marking where reduced strain has occurred. Strengthening appears to have influenced intrados strains in two ways as follows.

i) Strains measured across pre-existing cracks.

The largest reductions in strain occur in the vicinity of the cracks (the cracks were also monitored separately and discussed in section 7.1.1). Here the tendency is for strengthening to reduce compressive strains with the measured values being greatly influenced by crack movement. Again this supports the finding made by direct monitoring of the cracks, that by reducing crack movements Archtec strengthening helps limit further load cycle derived hysteretic damage.

ii) Strains measured away from cracks.

Away from the cracks there has been a general reduction in tensile strains. For the two cases illustrated in Figure 6.2, this trend is most notable at the centre of the span between the quarter points. This is particularly important as by reducing tensile strains Archtec strengthening lessons any susceptibility to further cracking and loosening of bricks which is clearly beneficial to the bridge's serviceability.

7.2 Comparison of Predicted and Test Results

7.2.1 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 7.1. Again the representative load cases LC8 and LC9 acting on the south span have been used. The figure shows two graphs each with four curves;

- measured values (reproduced from Figure 6.2), labelled "Measured".
- measured values with the contribution attributed to movement across the significant pre-existing cracks removed, labelled "Ignore Crack".
- numerically predicted values, labelled "Predicted".
- numerical predicted values factored to make some allowance for 3D behaviour, labelled "Scaled Predicted". (As noted in Section 5, the 2D analysis is inherently conservative and, as anticipated, gives upperbound values with regard to strain, etc, since the 3D effects of transverse distribution, bridge skew and contribution of spandrel walls are not accounted for).

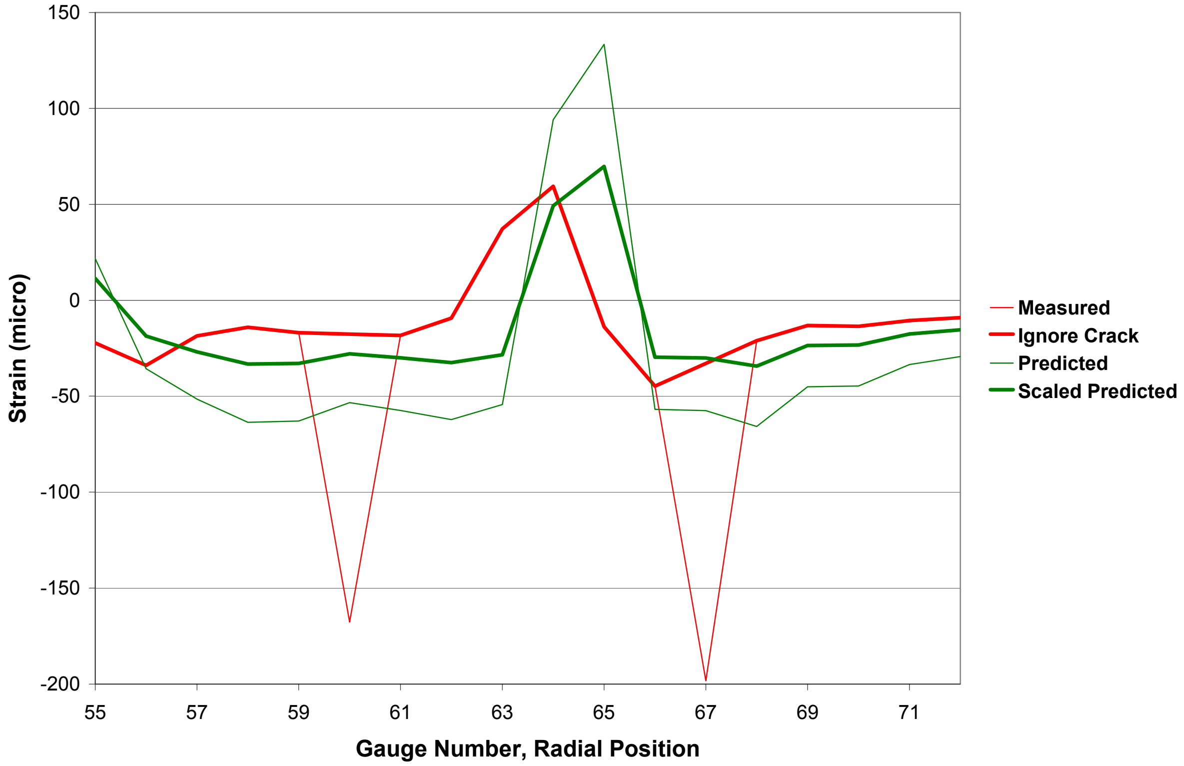
Macro strain results, positive values are tensile, are plotted against gauge numbers that mark the relative position around the intrados, see Figure 4.1.

Adjustment to the raw data both, measured and predicted is necessary to make valid comparisons between the two sets of results. As previously explained predicting the effects of closure of the pre-existing cracks is beyond the scope of the numerical analysis undertaken. In addition the 2D analysis was expected to be conservative and the need to factor the results to take account of 3D effects was anticipated. For the purposes of comparison, the predicted results have been reduced by a factor of 2 to illustrate correlation in the distribution of predicted and measured strains, even though the magnitudes have been conservatively over-predicted in the particular analysis undertaken. The adopted factor of 2 is considered a reasonable allowance for the 3D effects, in this case. It should be noted that the 2D analysis gives upperbound conservative values, as is appropriate for assessment and design work. Whilst 3D DE analysis of masonry arches is now feasible it has not yet been fully developed. Correlation of 3D analysis against the results of Pop Bottle test is beyond the stated objectives of the test, although the data could be of subsequent use for such verification in the future.

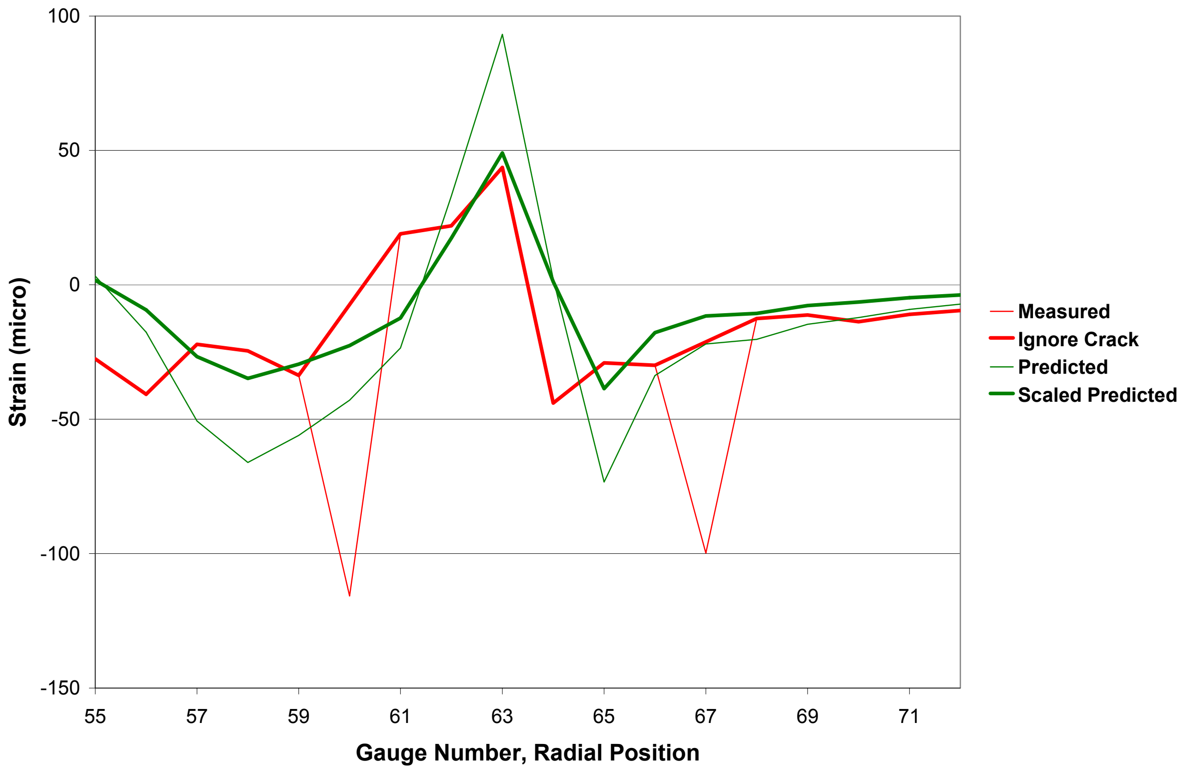
Once the above adjustments are made, for both cases the measured (curve marked “Ignore Crack”) and predicted (curve “Scaled Predicted”) results compare very well. Even before any adjustment is made to the predicted results (curve “Predicted”), the distribution of strain compares closely with that measured.

Figure 7.2 shows similar results for Archtec strengthened test. Again once adjustment are made to the raw data for the effects attributable to initial crack widths and full 3D behaviour measured (curve marked “Ignore Crack”) and predicted (curve “Scaled Predicted”) results compare very well. As for the unstrengthened case before any adjustment is made (curve “Predicted”) to the predicted results the distribution of strain compares closely with that measured.

These comparisons show that 2D DE simulations have been used to conservatively predict intrados strains in a defective arch barrel in its existing condition and after Archtec strengthening under working loads (serviceability limit state).

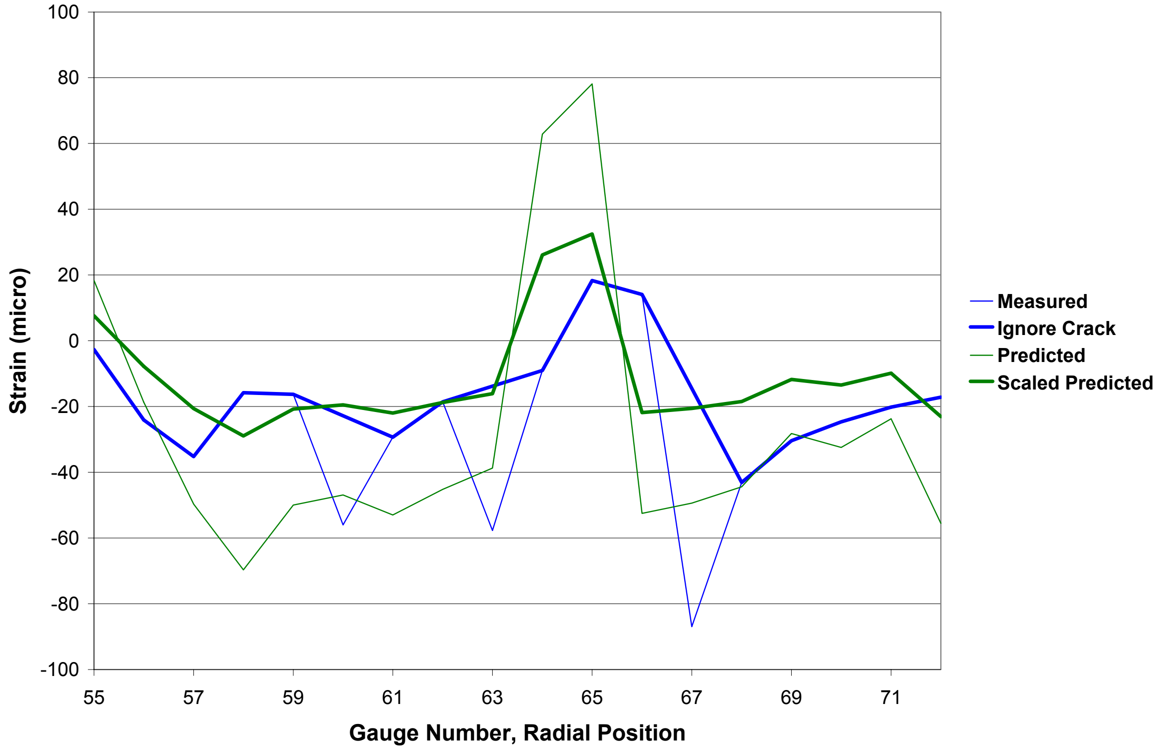


Load case LC8 – A pair of 11.4 tonne axles at mid-span of the south span

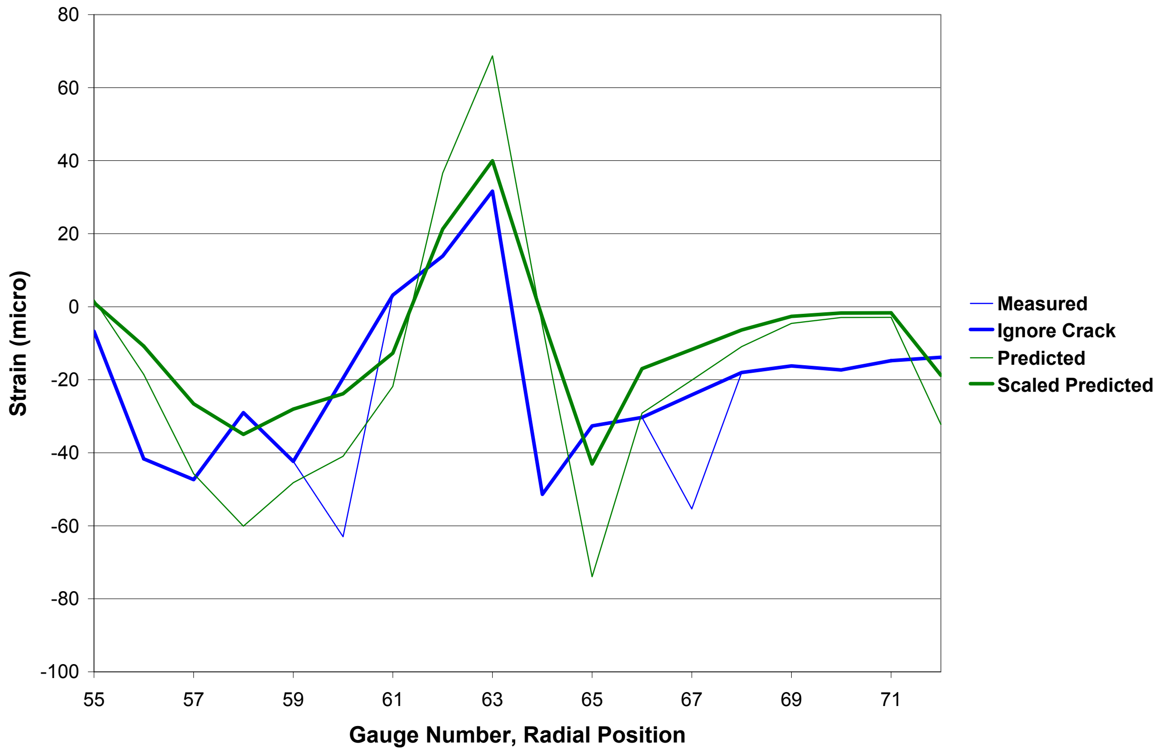


Load case LC9 - A Pair of 11.5 tonne axles at pier quarter-span of the south span

Figure 7.1 Unstrengthened Test – Predicted versa Test Intrados Macro Strains



Load case LC8 – A pair of 11.4 tonne axles at mid-span of the south span



Load case LC9 - A Pair of 11.5 tonne axles at pier quarter-span of the south span

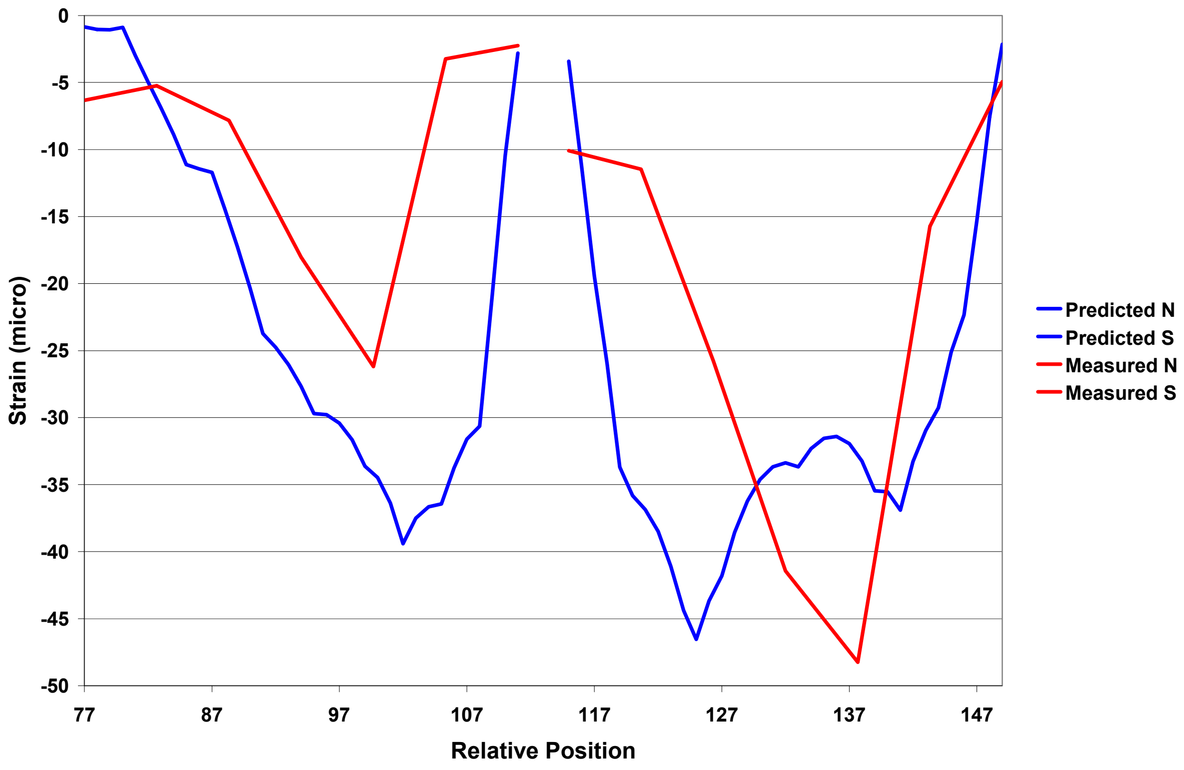
Figure 7.2 Strengthened Test – Predicted versa Test Intrados Macro Strains

7.2.2 Anchor strains

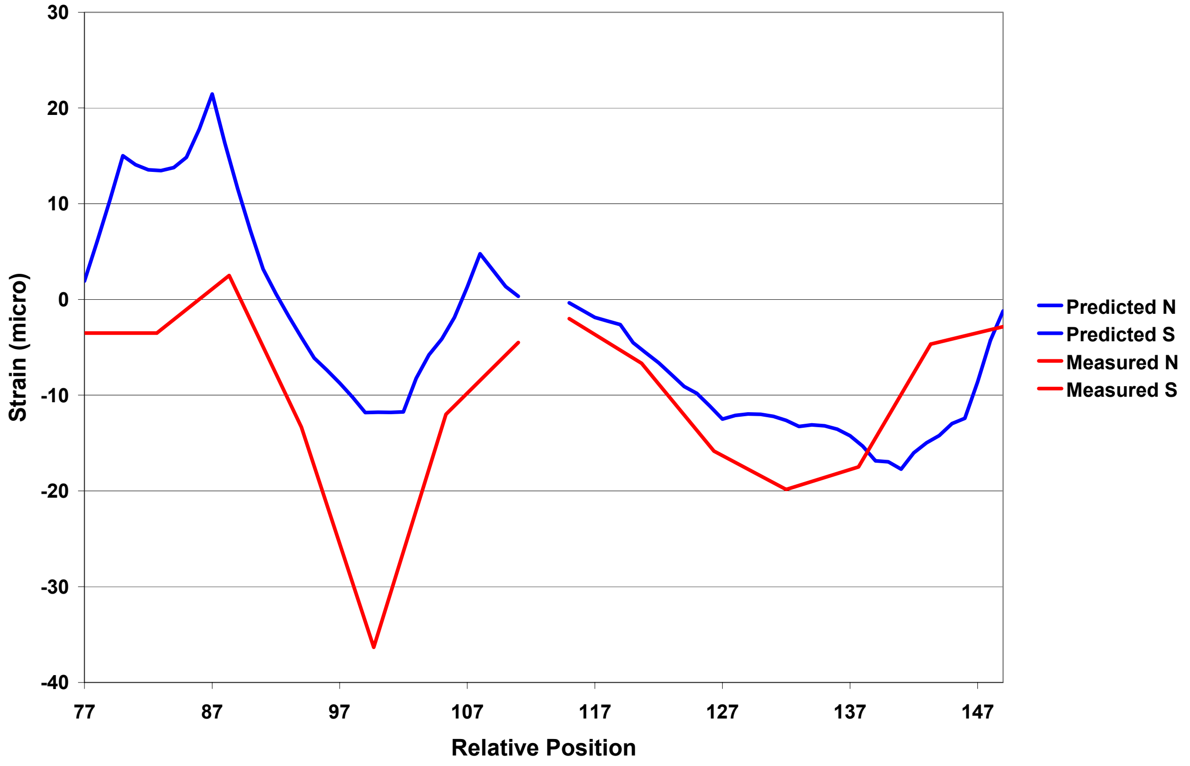
Using the DE model including Archtec strengthening and the methodology described in section 6.2 anchor strains have been calculated and are compared against the measured values in Figure 7.3. Again the representative load cases LC8 and LC9 acting on the south span have been used. The figure shows two graphs each comparing the predicted and measured strains along the relative length of each anchor; measured values (reproduced from Figure 6.3) and numerically predicted values. Positive strains indicate tension with 50 με indicated a bar stress of 10 N/mm^2.

Bearing in mind the relatively small strains that are being compared both the distribution and magnitude of the results compare well. Some 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 which do not allow for pre-existing open cracks.

These comparisons show that 2D DE simulations have been used to predict anchor strains in a defective arch barrel that has been strengthening using Archtec under working loads (serviceability limit state). Consequently simulations can be used to assess the level of stress in the bars.



Load case LC8 – A pair of 11.4 tonne axles at mid-span of the south span



Load case LC9 - A Pair of 11.5 tonne axles at pier quarter-span of the south span

Figure 7.3 Predicted and Test Anchor Strains

8. CONCLUSIONS

The following general conclusions can be drawn from the results of the two load tests, on the bridge in its unstrengthened condition and after being Archtec strengthened, and from predictions of their behaviour using numerical simulations.

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 macro strains and, therefore, reduces the likelihood of loosening masonry under cyclic live loads.

iii) Direct instrumentation of cracks and intrados macro strain measurements have demonstrated that Archtec anchors positioned across transverse cracks reduce cyclic opening and closing under repeated live loads. The main 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 conservative because of skew behaviour, transverse load distribution and spandrel wall stiffening.

v) It has been demonstrated that Archtec strengthening can be designed not only for the ultimate limit state[4] (strength) but also for the serviceability limit state (deflections, strains and stress ranges).

In summary, the two principal objectives of the tests;

- to validate the use of the ELFEN DE analytical method to predict serviceability behaviour in un-strengthened and strengthened arches, and,
- to demonstrate that the retrofitted anchors contribute to the structural behaviour under service loads and that the effects are beneficial and measurable,

have been achieved.

9. REFERENCES

1. Gifford and Partners. October 2003*, Proposed Monitoring of Pop Bottle Bridge*

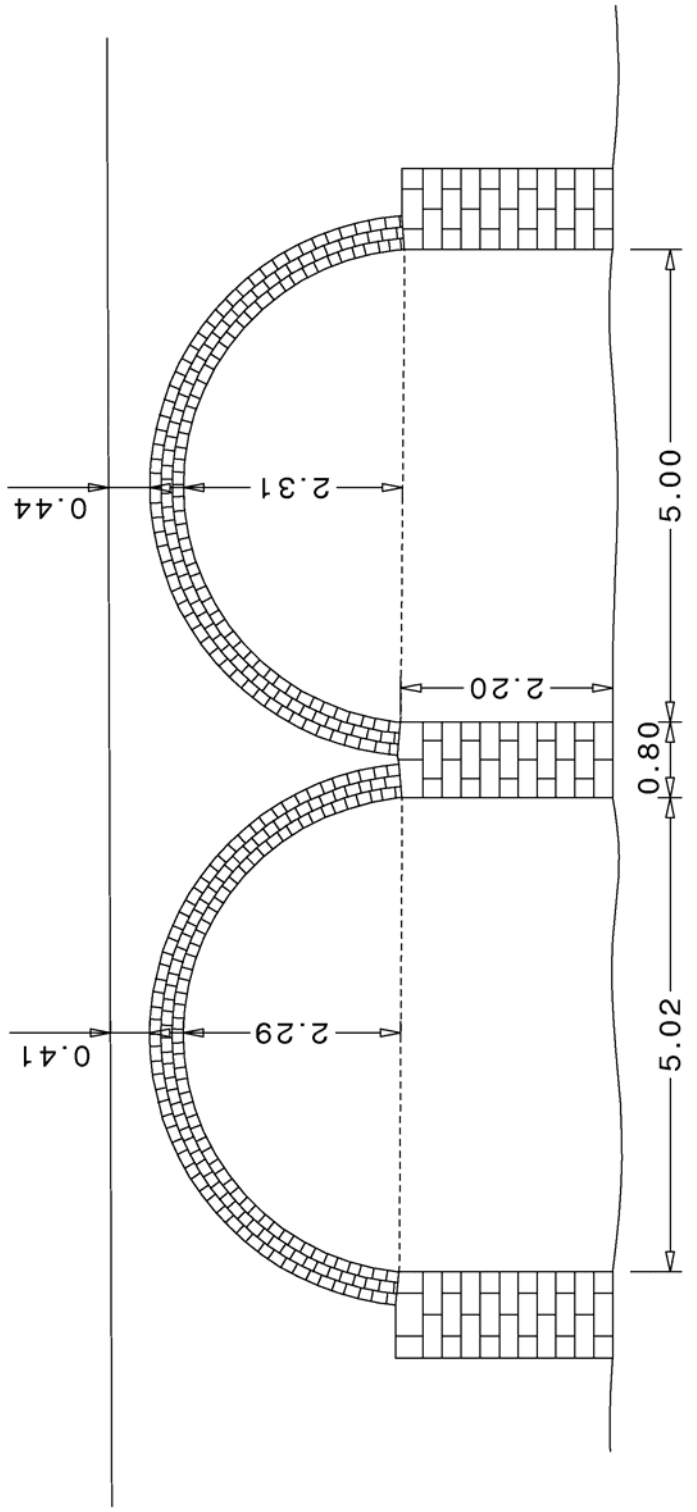
2. Gibb Ltd for Rail Property Ltd. March 2001. *00/01 BE4 Assessment Programme – Assessment and Inspection Report.*

3. Gifford and Partners. March 2002*. Pop Bottle Bridge – Arch Barrel Special Assessment. Report number B1660a/214/R01.*

4. Gifford and Partners. August 2003. *Archtec – Verification of Structural Analysis. Report number B1660A/V10/R02 Revision C dated 6 April 2004.*

Appendix A

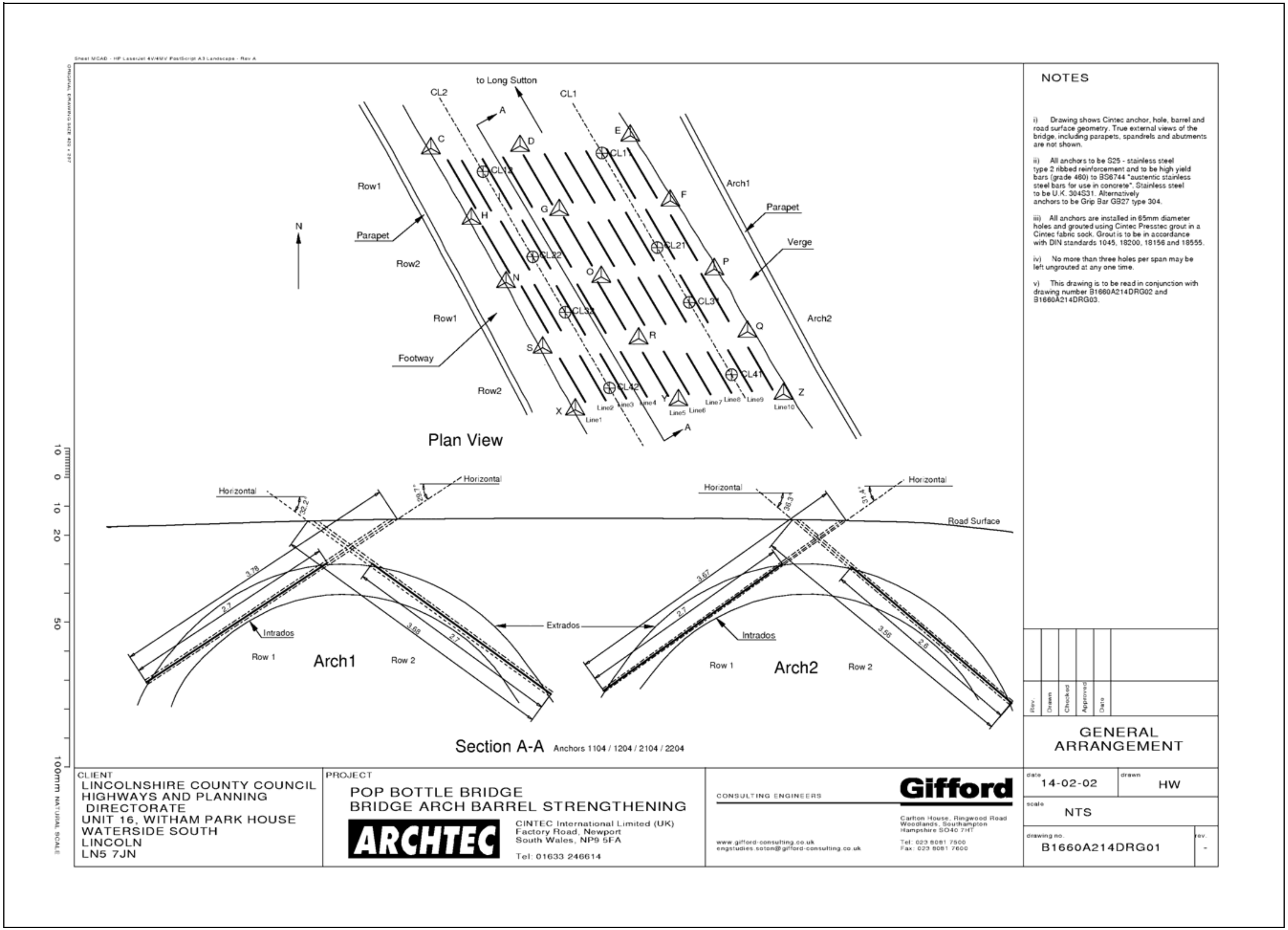
General Arrangement of Pop Bottle Bridge



Section Through Pop Bottle Bridge

Appendix B

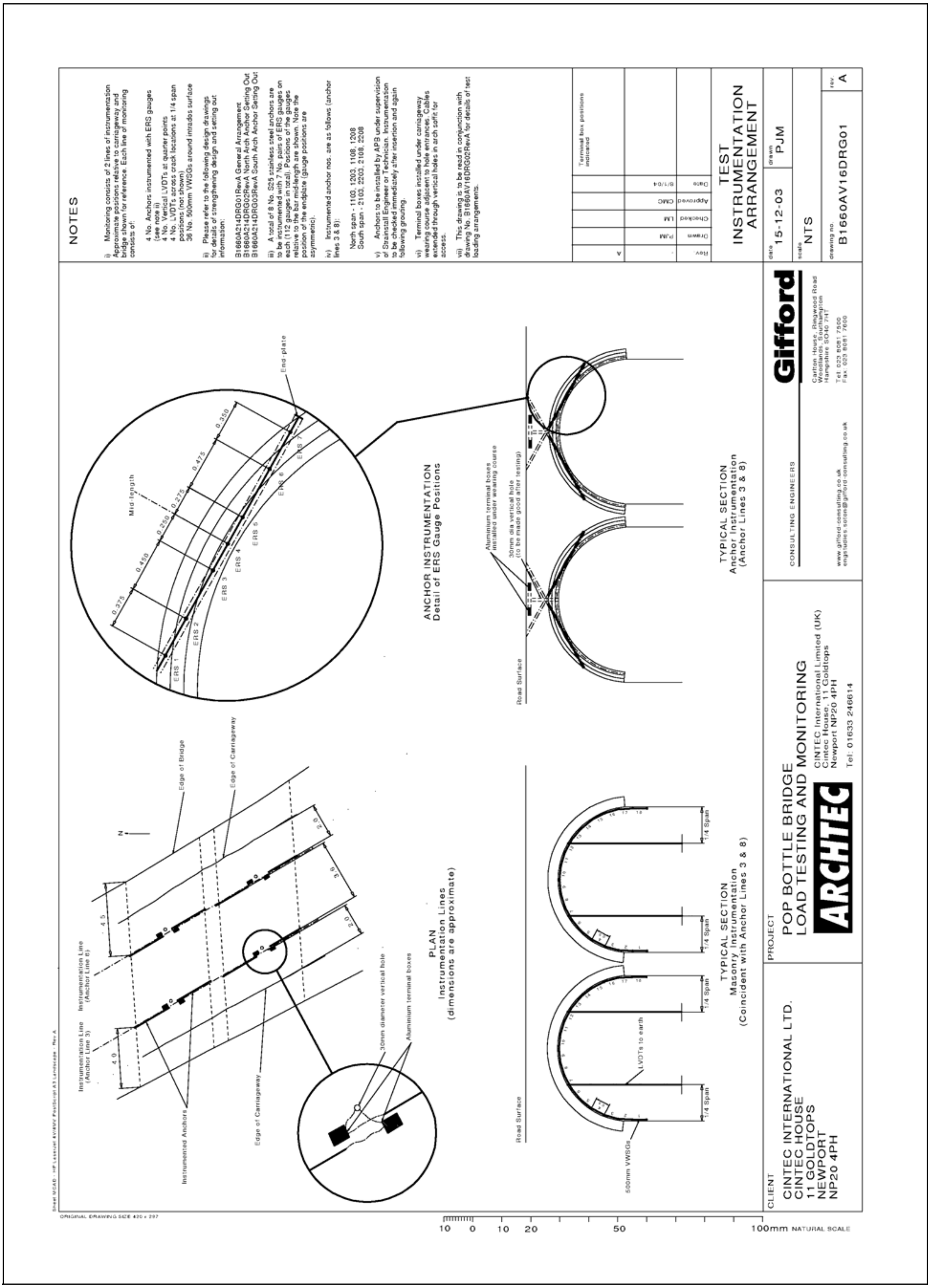
Detail of Archtec Strengthening



NOTES
i) Drawing shows Cintec anchor, hole, barrel and road surface geometry. True external views of the bridge, including parapets, spandrels and abutments are not shown.
ii) All anchors to be S25 - stainless steel type 2 ribbed reinforcement and to be high yield bars (grade 460) to BS6744 "austentic stainless steel bars for use in concrete". Stainless steel to be U.K. 304S31. Alternatively anchors to be Grip Bar GB27 type 304.
iii) All anchors are installed in 65mm diameter holes and grouted using Cintec Presstec grout in a Cintec fabric sock. Grout is to be in accordance with DIN standards 1045, 18200, 18156 and 18555.
iv) No more than three holes per span may be left ungrouted at any one time.
v) This drawing is to be read in conjunction with drawing number B1660A214DRG02 and B1660A214DRG03.
to Long Sutton
CL1
CL2
Row1
Row2
Parapet
Footway
Arch1
Arch2
Verge
N
Plan View
Horizontal
Road Surface
Extrados
Intrados
Row 1
Row 2
Arch1
Arch2
Section A-A Anchors 1104 / 1204 / 2104 / 2204
Rev.
Drawn
Checked
Approved
Date
GENERAL ARRANGEMENT
CLIENT
LINCOLNSHIRE COUNTY COUNCIL
HIGHWAYS AND PLANNING DIRECTORATE
UNIT 16, WITHAM PARK HOUSE
WATERSIDE SOUTH
LINCOLN
LN5 7JN
PROJECT
POP BOTTLE BRIDGE
BRIDGE ARCH BARREL STRENGTHENING
ARCHTEC
CINTEC International Limited (UK)
Factory Road, Newport
South Wales, NP9 5FA
Tel: 01633 246614
CONSULTING ENGINEERS
Gifford
Carlton House, Ringwood Road
Woodlands, Southampton
Hampshire SO40 7HT
Tel: 023 8081 7500
Fax: 023 8081 7600
www.gifford-consulting.co.uk
engstudies.soton@gifford-consulting.co.uk
date 14-02-02
drawn HW
scale NTS
drawing no. B1660A214DRG01
rev. -
100mm NATURAL SCALE

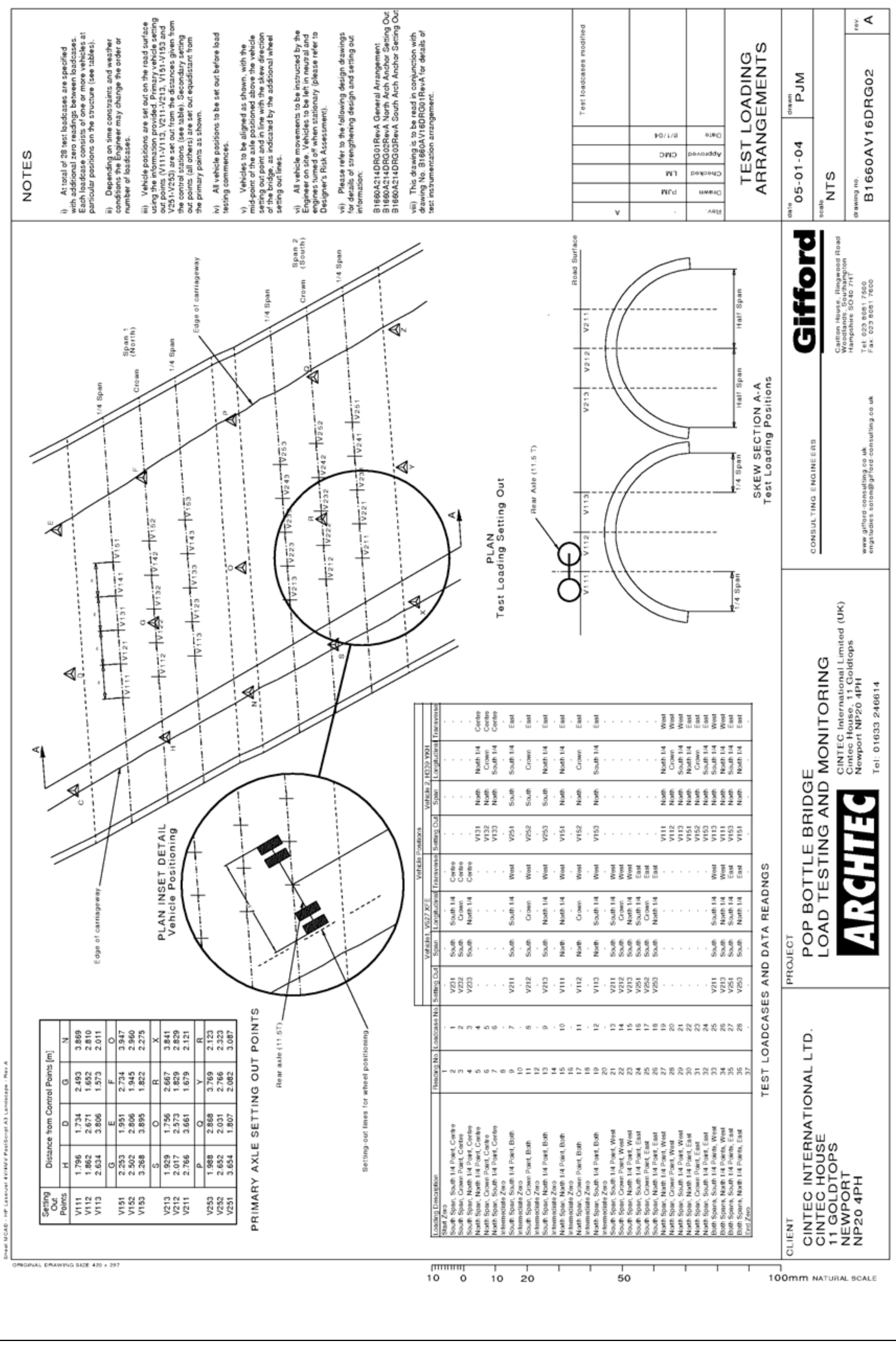
Appendix C

Arrangement of Instrumentation


NOTES
PLAN
Instrumentation Lines
(dimensions are approximate)
Edge of Bridge
Edge of Carriageway
Instrumentation Line (Anchor Line 3)
Instrumentation Line (Anchor Line 8)
Instrumented Anchors
30mm diameter vertical hole
Aluminium terminal boxes
ANCHOR INSTRUMENTATION
Detail of ERS Gauge Positions
Mid-length
End-plate
TYPICAL SECTION
Anchor Instrumentation
(Anchor Lines 3 & 8)
TYPICAL SECTION
Masonry Instrumentation
(Coincident with Anchor Lines 3 & 8)
Road Surface
1/4 Span
LVDTs to earth
500mm VWSGs
CLIENT
CINTEC INTERNATIONAL LTD.
CINTEC HOUSE
11 GOLDTOPS
NEWPORT
NP20 4PH
PROJECT
POP BOTTLE BRIDGE
LOAD TESTING AND MONITORING
ARCHTEC
CINTEC International Limited (UK)
Cintec House, 11 Goldtops
Newport NP20 4PH
Tel: 01633 246614
CONSULTING ENGINEERS
Gifford
TEST
INSTRUMENTATION
ARRANGEMENT
15-12-03
PJM
NTS
B1660AV16DRG01
A
ORIGINAL DRAWING SIZE 420 x 297
100mm NATURAL SCALE

Appendix D

Test Load Positions



NOTES
i) At total of 28 test loadcases are specified with additional zero readings between loadcases. Each loadcase consists of one or more vehicles at particular positions on the structure (see tables).
ii) Depending on time constraints and weather conditions, the Engineer may change the order or number of loadcases.
iii) Vehicle positions are set out on the road surface using the information provided. Primary vehicle setting out points (V111-V113, V211-V213, V151-V153 and V251-V253) are set out from the distances given from the control stations (see table). Secondary setting out points (all others) are set out equidistant from the primary points as shown.
iv) All vehicle positions to be set out before load testing commences.
v) Vehicles to be aligned as shown, with the mid-point of the axle positioned above the vehicle setting out point and in line with the skew direction of the bridge, as indicated by the additional wheel setting out lines.
vi) All vehicle movements to be instructed by the Engineer on site. Vehicles to be left in neutral and engines turned off when stationary (please refer to Designer's Risk Assessment).
vii) Please refer to the following design drawings for details of strengthening design and setting out information:
B1660A214DRG01RevA General Arrangement
B1660A214DRG02RevA North Arch Anchor Setting Out
B1660A214DRG03RevA South Arch Anchor Setting Out
viii) This drawing is to be read in conjunction with drawing No. B1660AV16DRG01RevA for details of test instrumentation arrangement.
PRIMARY AXLE SETTING OUT POINTS
PLAN INSET DETAIL
Vehicle Positioning
PLAN
Test Loading Setting Out
SKEW SECTION A-A
Test Loading Positions
TEST LOADCASES AND DATA READNGS
Test loadcases modified
TEST LOADING ARRANGEMENTS
05-01-04
PJM
NTS
B1660AV16DRG02
A
Gifford
CONSULTING ENGINEERS
CLIENT
CINTEC INTERNATIONAL LTD.
CINTEC HOUSE
11 GOLDTOPS
NEWPORT
NP20 4PH
PROJECT
POP BOTTLE BRIDGE
LOAD TESTING AND MONITORING
ARCHTEC
CINTEC International Limited (UK)
Cintec House, 11 Goldtops
Newport NP20 4PH
Tel: 01633 246614
ORIGINAL DRAWING SIZE 420 x 297
100mm NATURAL SCALE