ARCHTEC

POP BOTTLE BRIDGE SUPPLEMENTARY LOAD TEST

SUMMARY REPORT
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CONTROLLED DOCUMENT

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Revision Record

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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](image1)

![Road above](image2)

**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](image1.png) ![Plan – Showing carriageway](image2.png)

**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.

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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.
Figure 4.2 General View of Instrumentation
The instrumentation that was installed is summarised in Table 4.1.

Table 4.1  Summary of Instrumentation

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Method of Instrumentation</th>
<th>Type Description</th>
<th>Number</th>
<th>Accuracy Range</th>
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<tr>
<td>4.2.1</td>
<td>Vertical displacement at $\frac{1}{4}$ span positions</td>
<td>LVDT (pole mounted)</td>
<td>8</td>
<td>0.003 mm ±12.5mm</td>
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<tr>
<td>4.2.2</td>
<td>Intrados circumferential strain (macro) measurement</td>
<td>0.5m VW (glued to masonry)</td>
<td>72</td>
<td>0.3 $\mu$ε ±450 $\mu$ε</td>
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<tr>
<td>4.2.3</td>
<td>Crack monitoring</td>
<td>LVDT (glued to masonry)</td>
<td>8</td>
<td>0.0003 mm ±2.5mm</td>
<td></td>
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<tr>
<td>4.2.4</td>
<td>Anchor axial strain measurement</td>
<td>ERS (glued to rebar)</td>
<td>112</td>
<td>1 $\mu$ε ±200 $\mu$ε</td>
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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.

Figure 4.3  Plan of Bridge showing Instrumented Longitudinal Sections
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.
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

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<td>-19</td>
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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.

![Figure 6.2a Representative Intrados Macro Strains](image-url)

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

*Figure 6.2a Representative Intrados Macro Strains*
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 με and a maximum value of 30 με.

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.
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.
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 lessens 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 $\mu\varepsilon$ 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.

![Graph showing anchor strains](image)

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


Appendix A

General Arrangement of Pop Bottle Bridge
Appendix B

Detail of Archtec Strengthening
Appendix C

Arrangement of Instrumentation
Appendix D

Test Load Positions