MONITORING OF MONKSTOWN BRIDGE
A NOVEL FLEXI-ARCH BRIDGE SYSTEM

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ABSTRACT
Masonry arch bridges are one of the oldest forms of bridge construction and have been around for thousands of years. Brick and stone arch bridges have proven to be highly durable as most of them have remained serviceable after hundreds of years. In contrast, many bridges built of modern materials have required extensive repair and strengthening after being in service for a relatively short part of their design life. Sustainable construction is aimed at ensuring the economical use of finite raw materials and reducing, or mitigating against, the accumulation of pollutants and waste. This emphasizes the need to incorporate sustainability principles at an early stage in the design and development of construction projects. Hence, the development of a ‘flat-pack’ arch bridge system which requires no centering or steel reinforcement is timely in promoting sustainable bridge construction.

This paper presents the findings of monitoring of a flat profile, 10m span by 2m rise, flexi-arch bridge at full-scale and third-scale and follows the successful completion of a Knowledge Transfer Partnership between Macrete Ltd and the Centre for Built Environment Research at Queen’s University Belfast for which Dr Taylor was the Academic Supervisor. The bridge was monitored using a range of sensors including fibre optic sensors, electrical displacement transducers, electrical resistance strain gauges and vibrating wire strain gauges. The behaviour of the arch was also modelled using non-linear finite element analysis and ARCHIE and compared to the results of monitoring.

INTRODUCTION
It is no longer economically viable to construct masonry arch bridges in the traditional method due to the cost of the skilled labour required to build the accurate centring and to cut the masonry blocks. Progress on this type of work is usually slow and can be weather dependent. The ‘flexi-arch’ system is constructed and transported in the form of a flat pack by using a polymeric reinforcement to carry the self weight during the short lifting phase but it behaves as a masonry arch once in place. The voussoirs are pre-cast individually, laid contiguously horizontally with a layer of polymer grid material placed on top. An in-situ layer of concrete, 50mm deep in the case of the 10m Monkstown Bridge, is placed on top and allowed to harden to interconnect the voussoirs (Figure 1). The arch unit can be cast in convenient widths to suit the design requirement, site restrictions and available lifting capacity. When lifted, the wedge shaped gaps close, concrete hinges form in the outer most layer of concrete and an arch is formed. The arch ring is then placed on a pre-cast anchor block or seating units and the self-weight is then transferred from tension in the polymer to compression in the arch ring. This form of construction for low to medium span bridges is in line with advice given by the UK Highways Agency [1995b] where the arch form, plain structural elements and the elimination of corrotable reinforcement are recommended.

This paper presents the results of monitoring of the full-scale arch ring during backfilling and load testing of third-scale arch rings with concrete backfill. The arch system has also been analysed using ARCHIE and non-linear finite element analysis (NLFEA).
Figure 1: Construction of arch unit using pre-cast individual voussoir concrete blocks

2. ARCH PROFILE AND MONITORING

2.1 Arch system detail

The 10m flat profile arch details are given in Table 1 and the one-third scale arch was one-third of these dimensions, giving a clear span of 3.333m and rise of 0.667m. The full scale 10m concrete arch bridge system, as shown in Fig. 2, was monitored during concrete backfill operations and using the same procedure as the 5m arch prototype arch test outlined in a previous paper (Taylor et al, 2006). The third scale arch was also backfilled, using firstly granular material and then concrete, the arch was then monitored during load testing at third span. Control samples of concrete from the voussoirs and screed were tested. Advancements in composite material technology and the ability of the polymer in this system to be sufficiently strong yet flexible have provided the key to the success of the flexi-arch. Material tests were carried out on samples of the polymeric reinforcement and the average tensile strength is given in Table 1. The arch was lifted from flat to arch form by supports at the 7th, 13th and 17th voussoir positions.

Table 1: 10m x 2m arch details

<table>
<thead>
<tr>
<th>Voussoir dimensions</th>
<th>480mm</th>
<th>250mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear span:</td>
<td>10.00m</td>
<td></td>
</tr>
<tr>
<td>Internal height or rise:</td>
<td>2.00m</td>
<td></td>
</tr>
<tr>
<td>Depth of arch ring:</td>
<td>0.250m (including 50mm top screed)</td>
<td></td>
</tr>
<tr>
<td>Width of arch ring:</td>
<td>1.00m</td>
<td></td>
</tr>
<tr>
<td>Polymer Reinforcement 200/100 Tensile strength (2 layers over middle 17 blocks and 1 layer for remaining outer 3 blocks)</td>
<td>110kN/m (15kN/m in scale arch)</td>
<td></td>
</tr>
<tr>
<td>Arch ring concrete compressive strength*</td>
<td>56N/mm²</td>
<td></td>
</tr>
<tr>
<td>Backfill</td>
<td>lean mix concrete to 0.4m above arch extrados (1/3 scale arch : granular and concrete backfill tested)</td>
<td></td>
</tr>
</tbody>
</table>

* is based upon control

2.2 Instrumentation and monitoring regime

The instrumentation set-up and for the full-scale stability test and the typical test arrangement for the third scale load tests is depicted in Figure 2. Deflection transducers were used to monitor both horizontal and vertical deflections; vibrating wire strain gauges were used to measure crack openings at the joints between voussoirs and discrete optical sensors were used in the arch extrados to measure strain.
Fibre Bragg Gratings (FBGs) for strain measurement have been used successfully in bridge monitoring and have many advantages over more conventional sensors (Grattan et al, 2006). FBG’s rely upon a change in strain causing a shift in the characteristic wavelength of light reflected back along (or through) the optical fibre where the grating is written. The equation used to determine the reflected wavelength, $\lambda_B$, is defined by the Bragg condition given in:

$$\lambda_B = 2n_e \Lambda$$  \hspace{1cm} (Equation 1)

where $n_e$ is the effective refractive index and $\Lambda$ the periodic spacing of the grating.

Monitoring of strain can provide useful information on the integrity of a structure and the scanning rate can be adjusted to suit requirements. That is, strain monitored over a short period of time can measure the effect of dynamic loads or over a longer period will assess strain due to quasi-static load conditions. In this case FBG sensors were attached to the extrados of the arch.
There are numerous advantages of FOS over current technology for this bridge monitoring including:
- wholly non-destructive, unlike many of the currently available systems
- can to be installed as part of the manufacturing process and thereby minimising setup complications
- increased sensitivity compared to other sensors
- unaffected by electromagnetic radiation
- allows the user access to a large amount of data which can be used to detect critical strain levels
- uses light-weight components, yet can be designed to deal with almost any environmental condition

3. STABILITY TEST AND MONITORING

3.1 Test procedure

The arch system is flexible (see Figure 5) and although a previous stability test on a 5m span by 2m rise arch ring (Taylor et al, 2006) showed the arch was stable during backfilling, this shallow profile arch could be less stable and experience greater uplift at the crown during backfilling operations. A stability test was conducted and the arch ring was monitored for horizontal deflections, vertical deflections and strain at the voussoir joints. Figure 2 shows the transducer positions. Shuttering for the backfill was set-up independently to the arch ring to allow free movement under backfill. In the full bridge system, the spandrel walls will retain the concrete backfill. The backfilling operation was carried out by placing approximately 250mm deep layers of concrete to each side of the arch to a distance of 1.7m from the back face of the anchor block. The distribution of load equally to either side of the arch ring was aimed at minimising the effects of asymmetric loading. The depth of the concrete was measured and readings were taken for each increment of backfill load. The height of the backfill was also measured for each batch of concrete to assess the height of backfill at each increment.

Figure 5: 10m span by 2m rise arch ring for Monkstown Bridge
3.2 Monitoring results and discussion

The readings showed a reasonably symmetric response to the backfilling operation. A summary of the deflections at various times throughout the backfill operation are given in Figure 6 and Figure 7 shows the exaggerated, x 50, deflected shape for a couple of the stages during backfilling. The maximum movement in the crown was 5.6mm upwards and occurred when the concrete backfill was at the height of the crown of the arch. This was due to the lateral pressure of the wet concrete creating an inwards movement at the sides of the arch which in turn caused the crown to rise. Subsequent load on top of the crown had a beneficial effect on the deflection. Additionally, as the concrete hardened, the lateral pressure reduced and the deflection response at the crown reversed in direction. That is, the upward movement ceased and the crown moved back toward its original position. Fifteen hours after the start of the backfilling operation, the cumulative deflection was 2.5mm above the original crown position.

The maximum movement at the sides of the arch occurred in the transducers at the quarter span (equivalent to 1.6m above the height of the springing). The movement was inwards due to the lateral pressure from the wet concrete. At the end of the backfill operation, the maximum horizontal and vertical deflections were 2.6mm and 3.4mm respectively giving a total inwards movement of 4.3mm at the quarter span. The results of the monitoring showed that little movement had occurred in the arch ring and it was concluded that the arch was stable during backfill operations.
4. LOAD TESTS ON THIRD SCALE ARCHES WITH CONCRETE BACKFILL

4.1 Test and monitoring procedure

The third scale arch ring was made in accordance with the procedure for the full scale arch ring and an appropriately scaled strength polymer used over the voussoirs. The concrete for the screed was also a third scale mix (Fig. 8). After curing, the arch was lifted into position on the structural floor and formwork placed at the sides, independently to the arch ring, to contain the wet concrete backfill. After three days the arch was tested in accordance with the requirements of the relevant bridge category and following the guidelines in BS8110 (1985) for the testing procedure. A simulated static axle load was applied at the third span of the arch ring. It is recognized that, in practice, there will be a dynamic amplitude factor above the static load. The single wheel load and factors of safety which will be used for the full-scale arch are based upon the requirements in BD91/04 (Highways Agency, 2004). Bridges are designed under static load conditions with factors of safety applied to these loads. An impact factor of 1.8 is recommended in BD91/04 and this takes into account dynamic amplitude effects on an arch bridge.

A 25mm knife edge load was applied at the loading position via a stiff steel beam bedded on soft board. The application of load was from an accurately calibrated 600kN capacity hydraulic actuator and the test rig was assembled with the top beam horizontal about both axes thus minimizing eccentricity effects. The simulated static wheel load was applied a third span position as illustrated in Figure 2. The load was applied incrementally and the deflection and strain measurements were recorded at each increment of load. A service test load of 20kN was applied twice prior to the application of the full test load.

Figure 8: Third scale arch ring construction
4.2 Monitoring results

No cracking was evident under the two test loads and the first cracking was visible at an applied load of 75kN and occurred in the concrete back-fill under the load position. Under further loading the crack propagated towards the load. Below an applied load of 110kN, no opening of the joint between voussoirs was observed. However, between 110kN and 120kN applied load, openings were observed in the joint directly under the load and the joint to the right of the midspan. The opening under the load position shows the most likely position of a hinge and the collapse mechanism of the arch.

The load vs. deflection results, including the vector sum of the horizontal and vertical deflections have been summarized in Figure 10. It can be seen that the maximum deflection under the third span loading was 10mm. A deflection of 10mm is equivalent to a span/333 and within acceptable limits for deflection. It can be seen from load versus deflection results that, at an applied load of 110kN, the rate of deflection increases for similar load increments. This was due to cracking in the backfill and opening of the joints in the tension face. The arch was not loaded until failure due to the arching thrust at the left hand wall support and a recovery of these deflections is noticeable in Figure 10. Figure 11 shows the exaggerated deflected shape at maximum applied load. The outwards deflection at the opposite side to the load region is clearly visible in this graph.
Figure 11: Exaggerated deflections of the arch at maximum applied load

5 ANALYSIS OF THE ARCH SYSTEM

5.1 NLFEA Analysis

Other researchers, such as Fanning and Boothby (2001), have shown the benefits of modelling arch behaviour using non-linear finite element analysis (NLFEA). ABAQUS NLFEA was used to analyse the arch with the concrete in both the ring and backfill being modelled using a plastic material model. The ring was assumed to be homogeneous. The use of the plasticity model allowed the formation of cracks in the concrete to be simulated and as a consequence the approach will be able to predict the formation of the hinges within the arch. Figure 12 shows the mesh arrangement with the arch ring being modelled using a structured 8 node quadrilateral mesh while the backfill was modelled using an unstructured 6 node triangular mesh. Different concrete properties were specified for the ring and backfill reflecting the different strengths of the two materials. The analysis was carried out using two load steps, initial the gravity loading was applied and Figure 13 shows the stresses and deformed shape under the self weight of the structure.

Figure 12: Finite element mesh

Figure 13: Stresses and deformed shape of arch under self weight
As expected, this step produces an elastic response from the structure with no plastic deformation occurring at this stage. The second step applied a point load of 250kN at the third span; in contrast to the first step this step was highly nonlinear. Figure 14 shows the deflection of the arch under the applied load. The figure indicates a sudden failure of the structure at an increment time of 1.98 where a time of 1 corresponds to the start of the step and time 2 to the application of the full 250kN load. Therefore the failure at time 1.98 corresponds to a load of 245kN. A contour of the plastic deformation is shown in Figure 15.

The contour plot shows a localized failure in the region of the loading. This mode of failure resulted from the use of concrete as the backfill material with the sides of the arch being extremely strong. In contrast finite element analyses of arches without backfill or with backfill consisting of granular material have indicated the formation of a series of hinges in the arch ring allowing a mechanism to form. The use of concrete backfill would seem to prevent this from happening. The figure would suggest that the plastic deformation is limited to small region near the loading; this is not actually the case. The plastic strains near the loading are of an order greater than the plastic strains occurring in other parts of the structure, plastic strains with a much smaller magnitude occur over a significant area of the arch.

The finite element failure load of 245kN is higher than that 160kN load applied in the test but this is due to the test load being stopped prematurely and prior to the arch ring failure (due to problem with the end support). A sensitive study into the influence of the material parameters on the failure strength predicted by the analysis is ongoing but initial findings suggest that the failure load is sensitive the tensile strength of the concrete and it should be possible to reduce the failure load predicted from the analysis by the selection of appropriate material parameters. The deflections obtained from NLFEA are extremely small with a maximum of 1mm deflection occurring before the rapid collapse of the arch. The measured deflections were small but the deflections from the analysis were considerably smaller than those measured in the tests. This is due to the modeling of the arch ring and concrete tensile strength but could also be due to imperfections in the real arch allowing the arch to settle between voussoirs and at the supports.

5.2 ARCHIE Analysis

The 10m arch was also analysed using ARCHIE (Obvis, 2006), a numerical analysis package which takes into account the arch backfill and assumes a collapse mechanism type failure (as opposed to ring separation). The analysis assumes that there is some dispersal of load through concrete backfill but negligible dispersal through granular backfill. It is important to note that this software is used by the DRD Road Service in Northern Ireland for load assessment and analysis of their masonry arch bridges. Therefore, assessment of the load carrying capacity of the flexi-arch system was a critical task in the
acceptance of the arch system and particularly this flat profile arch. The arch unit was analysed under different wheel loading conditions but the SV load (EU vehicle axle load from BD91/04) was the most critical. A line of thrust is indicated in Figure 16. Under the design loading, the position of the thrust line in the arch unit gave information about the stability of the unit. Furthermore, ARCHIE was able to demonstrate the change in the thrust line by changing the height of the Backing Material (BM) at the springing level and the effect of changing the Passive Pressure (PP). Therefore, for a particular loading condition, arch ring depth and appropriate passive pressure the load capacity of the bridge was established. A similar deflection response was given in the ARCHIE output as measured and shown in Fig. 11. However, the predicted ultimate capacity was conservative based on the actual load carrying capacity of the arch system.

![Figure 10: ARCHIE analysis output for SV vehicle](image)

CONCLUSIONS

Experience has shown that arch bridges are highly durable structures requiring little maintenance in comparison with other bridge forms. Thus, the objective of the new Highway Agency Standard (BD91/01, 2004) is welcomed especially if it encourages a renaissance in arch building using unreinforced masonry materials. The monitoring of the flat profile arch ring for Monkstown Bridge has shown that the arch ring is stable during backfilling and that it has a strength capacity far in excess of that required for this category of bridge.

The novel arch system has been demonstrated to be a viable alternative to long established methods of construction and the following advantages have been identified:

1. As the arch system is cast horizontally it can conveniently be transported to site in a “flat pack” form and as centring is not required during installation this greatly simplifies the process and enhances the speed of construction.
2. As there is no corroding reinforcement the long term durability should be assured.
3. Structural monitoring has proved the integrity of the system.
4. Initial estimates would indicate that the system is cost competitive with alternatives such as RC box culverts which do not share the aesthetic benefits or the longevity of an arch.

The monitoring of this flat profile arch showed that the arch had strength far in excess of the current load requirements and that NLFEA was able to predict the behaviour of the arch under third span loading. The NLFEA gave more accurate predictions compared to ARCHIE which gave conservative values for the
arch bridge strength. There is further refinement needed of the NLFEA model and this is on going research.

Current data indicates that in the UK alone almost 3500 bridges with spans between 3m and 10m need to be strengthened or replaced at an estimated cost of £80 million. Thus there is obviously a potential market for this short span bridge system that guarantees high quality pre-cast concrete units, ease of transport, simplicity of erection and exceptional durability. From all view points the system represents a very sustainable alternative for the future.

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REFERENCES