BEHAVIOUR OF BACKFILLED MASONRY ARCH BRIDGES SUBJECTED TO CYCLIC LOADING

L. Augusthus-Nelson[1], G. Swift[2], C. Smith[3], M. Gilbert[3], C. Melbourne[1]

[1]University of Salford, School of Computing, Science and Engineering, Manchester, UNITED KINGDOM.
[2]University of Portsmouth, School of Earth and Environmental Sciences, Portsmouth, UNITED KINGDOM.
[3]University of Sheffield, Department of Civil and Structural Engineering, Sheffield, UNITED KINGDOM.

e-mails: l.augusthusnelson@salford.ac.uk, gareth.swift@port.ac.uk, c.c.smith@sheffield.ac.uk, m.gilbert@sheffield.ac.uk, c.melbourne@salford.ac.uk

SUMMARY

Experimental research has demonstrated that the interaction between soil fill surrounding the arch barrel of a masonry arch bridge structure has a significant influence on ultimate limit state behaviour. However, little work has been carried out to date to examine the behaviour of soil filled bridges at the permissible limit state, PLS (defined as the state which, if not exceeded, will ensure the lifespan of the bridge is not measurably reduced by repeated live loading). Tests on a masonry arch bridge backfilled with crushed limestone fill material and subjected to 10^6 cycles of 50kN peak cyclic load amplitude (approximately one third of the collapse load) indicate that the initial stiffness and ultimate load carrying capacity of the soil-arch system is not adversely affected by this level of cyclic loading. However in follow up tests on a similar bridge, the peak cyclic load amplitude was incrementally increased from 60kN to 100kN (with 10^5 cycles at each load level), which was found to cause increasing levels of damage to the arch barrel, in turn reducing the stiffness of the bridge to applied loads.

Keywords: *Masonry arch bridges, cyclic loading, permissible limit state.*

1. INTRODUCTION

Masonry arch bridges form a vital part of the road and rail transport infrastructure of the UK, where 40-50 percent of the total bridge spans are masonry [1]. These masonry arch structures have been subjected to much higher dynamic loadings than originally designed for [2]. Furthermore, other actions can exacerbate the situation, causing permanent damage to the arch barrel and piers, such as freeze-thaw action and foundation settlement due to scour effects [3], [4]. As a consequence, many masonry arch bridges, mostly built more than a century ago, have started to show signs of distress, for example through a progressive reduction in the mechanical properties of the constituent materials or through a loss of bricks in the arch barrel. In order to ensure that our masonry arch bridges are employed effectively, and are available for continued use by future generations, they need to be properly managed. To assist with this, a permissible limit state (PLS) should be established, below which permanent deterioration is not induced.

The reliable assessment of masonry arch bridges has been an important concern of bridge owners over the last few decades. Masonry arch bridge assessment techniques vary in complexity, with the tools employed ranging from the simple semi-empirical MEXE method, to limit analysis based approaches, to advanced non-linear analysis methods [3], [5], [6]. Most assessment methods were developed to estimate the failure load at the ultimate limit state (ULS). In order to verify that the serviceability limit state (SLS) is satisfied the UK highway bridge assessment code BD21 [7] suggests that the load should be limited to half the ultimate failure load. In contrast the 'SMART' assessment method suggests that a range of load reduction factors (factors of safety) should be used to estimate working load capacity [8].

The lack of widely accepted serviceability criteria for masonry arch bridges was the motivation behind a physical modelling study involving the application of cyclic loads to soil-filled masonry arch bridges. Furthermore, providing physical evidence to help verify or otherwise the SLS partial safety factors currently used in codes such as BD21, and to help establish a new PLS were the main aims of the present study.

2. EXPERIMENTAL INVESTIGATION

A series of large scale physical tests on masonry arch bridges have recently been conducted at the University of Salford. The test arrangements and instrumentation setups have been described elsewhere [9]–[11]. This paper reports on tests carried out on two full scale bridges subjected to variable intensity cyclic loads.

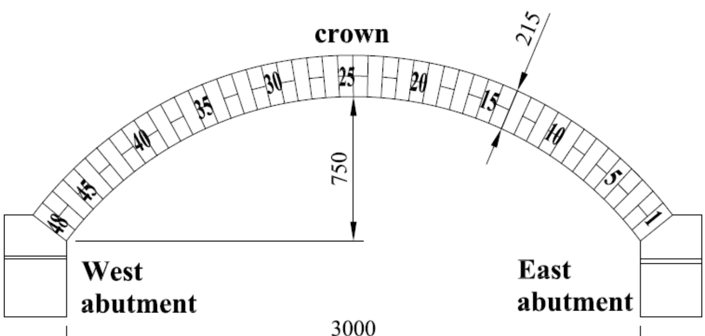


Fig. 1. *Elevation of arch barrel.*

A segmental arch barrel with span:rise ratio of 4:1 was constructed with a header bonding pattern so as to prevent ring separation. The arch barrel shown in Figure 1 was backfilled to a depth of 300 mm above the crown using compacted graded crushed limestone (unit weight = 20.0 kN/m^3; effective angle of friction = 56°). Figure 2 shows a schematic view of the test chamber used for this experimental investigation. The cyclic loading test was carried out using five actuators, with actuators positioned directly above the crown, above both abutments and above both quarter-span points [10], [11]. A servo-controlled system was used to control the actuators (which have a loading capacity of 0-

200 kN). Figure 3 shows the position of actuators relative to the arch barrel and backfill. The cyclic load was applied in the manner of a wave moving at constant velocity over the arch from the East abutment towards the West abutment. Each actuator applied the load in the form of a sine wave 180 degrees ahead of the following actuator, so that when it reaches its peak load level the sine wave of the following actuator is at zero. As a result the total load applied to the bridge remains constant throughout, and is equal to the peak load applied by a single actuator. Displacement transducers, pressure cells and electronic resistance strain gauges were placed in various positions to measure respectively the deformation of the arch barrel and test chamber, the soil pressures on the extrados of the arch barrel and strains across the mortar joints [11]. Displacement measurements taken during the cyclic loading tests are discussed in this paper.

Fig. 2. *Masonry arch bridge test chamber located at the University of Salford.*

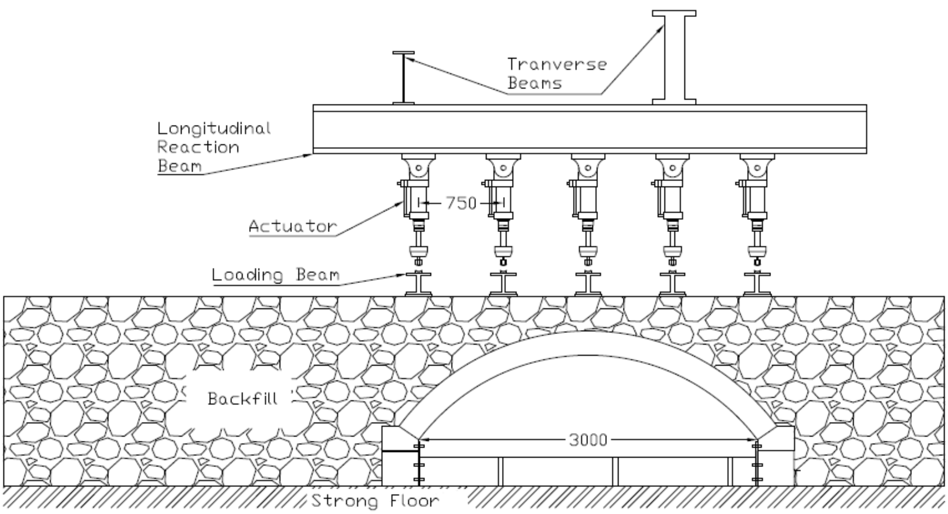


Fig. 3. *Cyclic loading test setup.*

The first bridge to be subject to cyclic loading, bridge EP1, was subjected to 10^6 cycles of 50 kN peak load intensity with a frequency of 2Hz. This load was applied to simulate highway traffic loading, with the in-service cyclic load being considerably less than the ultimate failure load (found to be approx. 140 kN). A series of stiffness tests (cyclic loads applied with a frequency of 0.01 Hz and a peak cyclic load intensity of 50 kN) was conducted periodically within the main 10^6 cycles campaign in order to check the evolving stiffness of the bridge.

A further bridge, bridge EP5, was subject to cyclic loads with a peak load intensity ranging from 50kN to 100 kN, applied at a frequency of 2 Hz. As with bridge EP1, a series of tests were conducted to check the stiffness of the bridge. Table 1 summarises the loading sequences for bridges EP1 and EP5. A series of quasi-static load tests was subsequently performed to determine the residual capacity of both arch bridges, though consideration of these tests is beyond the scope of the present paper.

Table 1. *Summary of applied cyclic loadings before quasi-static loading test until failure for arch bridges EP1 and EP5.*

label	Description	
	EP1	EP5
CYC1	0.01 Hz – 6 cycles – 50 kN peak load	0.01 Hz – 6 cycles – 50 kN peak load
CYC2	2 Hz – 10^4 cycles -50 kN peak load	2 Hz – 10^5 cycles -50 kN peak load
CYC3	0.01 Hz – 6 cycles – 50 kN peak load	0.01 Hz – 6 cycles – 50 kN peak load
CYC4	2 Hz – $9x10^4$ cycles -50 kN peak load	2 Hz – 10^5 cycles -60 kN peak load
CYC5	0.01 Hz – 6 cycles – 50 kN peak load	0.01 Hz – 6 cycles – 50 kN peak load
CYC6	2 Hz – $9x10^5$ cycles -50 kN peak load	2 Hz – 10^5 cycles -70 kN peak load
CYC7	0.01 Hz – 6 cycles – 50 kN peak load	0.01 Hz – 6 cycles – 50 kN peak load
CYC8	N/A	2 Hz – 10^5 cycles -80 kN peak load
CYC9	N/A	0.01 Hz – 6 cycles – 50 kN peak load
CYC10	N/A	2 Hz – 10^5 cycles -90 kN peak load
CYC11	N/A	0.01 Hz – 6 cycles – 50 kN peak load
CYC12	N/A	2 Hz – 10^5 cycles -100 kN peak load
CYC13	N/A	0.01 Hz – 6 cycles – 50 kN peak load

3. RESULTS AND DISCUSSION

This section reports new results obtained from a series of cyclic loading tests conducted on bridge EP5. Results from tests carried out on bridge EP1 were reported in Swift et al. [10], and show that the cyclic load of intensity 50 kN applied at a frequency of 2 Hz for 10^6 cycles (which can be considered to be below the expected 'serviceability limit state' load, given that the ultimate failure load was 141 kN) does not adversely affect the peak load. Also, the initial stiffness of the masonry arch bridge system was found to be increased due to densification of soil.

3.1. Effect of cyclic loading

Figure 4 shows arch barrel deformation at the quarter span during the entire cyclic loading campaign on bridge EP5. Permanent deformation of the arch barrel and increases in the amplitude of deformation with higher cyclic load levels were observed as expected.

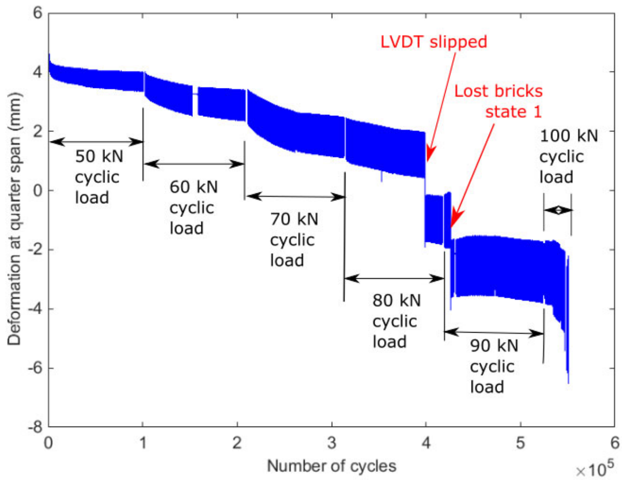


Fig. 4. *Increase of displacement amplitude at the quarter span against number of cycles –EP5.*

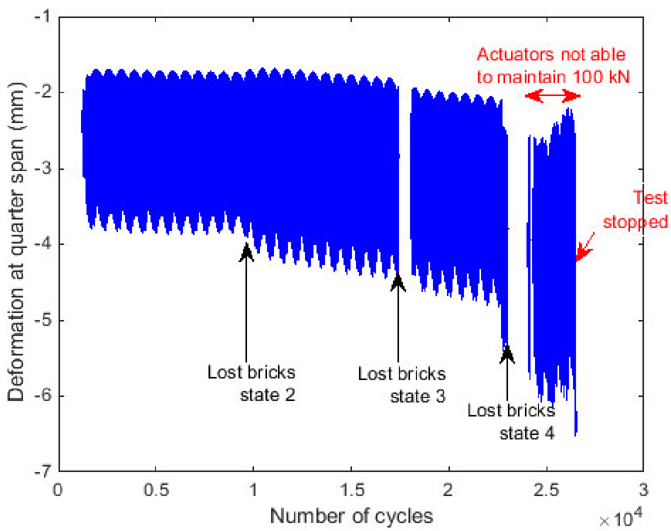


Fig. 5. *Increasing the quarter span displacement amplitude vs. number of cycles under the action of a 100 kN cyclic load –EP5.*

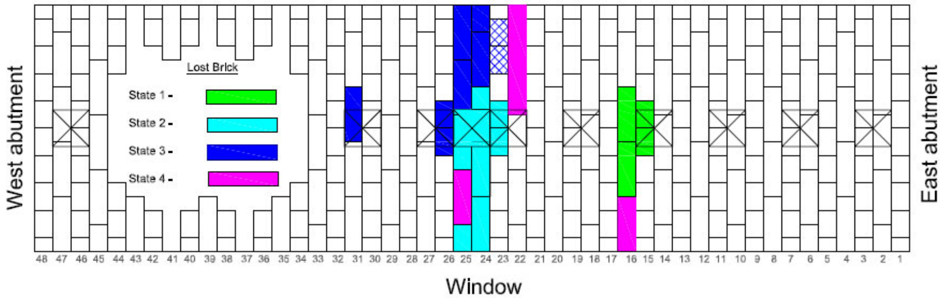


Fig. 6. *Location of lost bricks during 90 kN and 100 kN cyclic load tests.*

a)

b)

c)

Fig. 7. *Lost bricks at: a) state 1, b) state 2, and c) state 3 and 4.*

The barrel did not shown any sign of distress during the 50 kN and 60 kN cyclic loading stages. In BD21 a factor of two on the ULS load is applied to compute the safe working load, giving a load of 70 kN in this case (the ultimate failure load of EP1 was approx. 140 kN). During the 70 kN of cyclic loading, no significant changes to the arch barrel were observed, though a hairline crack at the joint between the abutment and the skewback was noticed. However, during the 80 kN cyclic loading stage bricks started to dislocate. This caused an LDVT deflection gauge to slip. Several more cracks around the dislocated bricks in the intrados of the arch barrel developed and the barrel showed distress during this cyclic loading stage. The first set of bricks near the quarter span at the east end of the bridge became dislocated during the 90 kN cyclic loading stage (denoted state 1 in Figures 4, 6 and 7). This occurred very early in the loading stage. During the rest of the 90 kN loading stage the arch barrel deformed significantly more than during the 80 kN loading stage.

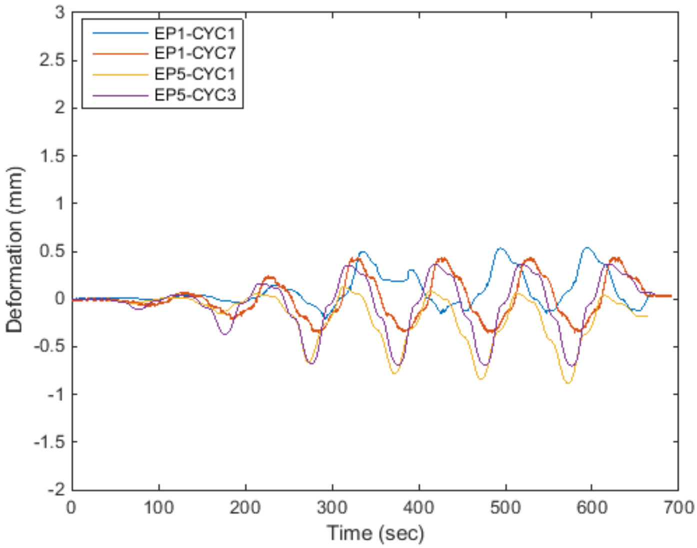
The arch barrel deformed significantly at the start of 100 kN cyclic loading stage. At around 10^4 cycles, bricks fell out at the crown (denoted state 2 in Figures 5, 6 and 7). Immediately afterwards the amplitude of the arch deformation increased by approximately 0.5 mm at the quarter span. Further bricks were lost at the crown at around $1.7\text{x}10^4$ cycles (state 3) and at $2.3\text{x}10^4$ cycles (state 4). Immediately after state 4, the loading actuators were not be able to reach the peak load of 100 kN due to significant reduction in the stiffness of the bridge. (This was because the hydraulic pump was unable to supply oil at a sufficient rate to maintain the desired cyclic loading regime.)

3.2. Changes in stiffness of arch barrel

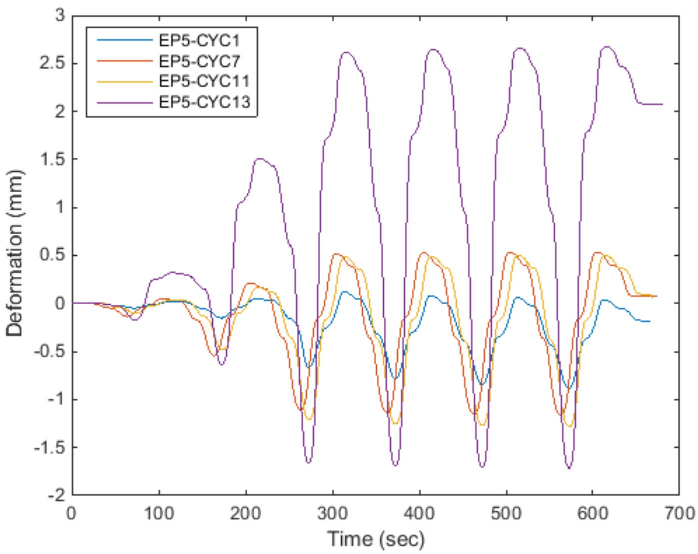
The stiffness tests, involving a load of peak intensity of 50 kN, applied at a frequency 0.01 Hz, were used to allow the stiffness of the bridge to be monitored periodically throughout the various cyclic loading regimes. These tests were undertaken for both bridges EP1 and EP5. Figure 8a) shows the relative quarter span displacements of the masonry arch barrels before and after the 50 kN cyclic loading stage was applied in the case of bridges EP1 and EP5.

Changes in the amplitude of the displacement in the case of EP1 after 10^6 cycles and EP5 after 10^5 cycles were approximately the same. i.e. the stiffnesses of bridges EP1 and EP5, although somewhat different, were not significantly altered by the application of the 50 kN cyclic loading regime.

In the case of bridge EP5 significantly increased quarter span displacement was observed in the final stiffness test which results from the considerable accumulated damage to the masonry arch barrel; see Figure 8b). For example, the change in displacement between CYC11 and CYC 13 is considerably higher than the change in displacement between CYC7 and CYC11. The change can be attributed to the loss of bricks during the 100 kN cyclic loading stage.



a)



b)

Fig. 8. *a) Stiffness comparison of EP1 and EP5 after 106 cycles and 105 cycles respectively, and b) stiffness at different stages for EP5.*

4. CONCLUSIONS

Various cyclic loading regimes have been conducted on full-scale backfilled masonry arch bridges tested in the laboratory in order to study the cyclic loading behaviour of soil filled masonry arch bridges. In the case of a bridge subjected to cyclic loads of progressively increasing peak intensity, the test results show that the bridge stiffness gradually reduces. However, when the peak cyclic loading intensity reached approx. 65 percent of the expected quasi-static load capacity falling bricks from the arch barrel led to a sudden loss in stiffness.

ACKNOWLEDGEMENT

The authors wish to acknowledge the support of the UK Engineering and Physical Sciences Research Council (EPSRC), under grant references EP/I014357/1 and EP/I014489/1.

REFERENCES

[1] PAGE J., *Masonry Arch bridges*. London, UK: TRL State of the Art Review: HMSO, 1993.

[2] HARVEY B., Some problems with arch bridge assessment and potential solutions, *The Structural Engineer*, Vol. 84, No. 3, 2006, pp. 45–50.

[3] WANG J., and MELBOURNE C., Mechanics of MEXE method for masonry arch bridge assessment, *Proceedings of the Institution of Civil Engineers-Engineering and Computational Mechanics*, Vol. 163, No. 3, 2010, pp. 187–202.

[4] OLOFSSON I., ELFGREN L., BELL B., PAULSSON B., NIEDERLEITHINGER E., SANDAGER JENSEN J., FELTRIN G., TÄLJSTEN B., CREMONA C., KIVILUOMA R., and BIEN J., Assessment of European railway bridges for future traffic demands and longer lives – EC project 'Sustainable Bridges,' *Structure and Infrastructure Engineering*, Vol. 1, No.2, 2005, pp. 93–100.

[5] HEYMAN J., The estimation of strength of masonry arches, *Proceedings of Institution of Civil Engineers*, Vol. 69, No. PT2, 1980, pp. 921–937.

[6] LOURENCO P. B., Computations on historic masonry structures, *Progress in Structural Engineering and Materials*, Vol. 4, No. 3, 2002, pp. 301–319.

[7] DEPARTMENT OF TRANSPORT, The Assessment of Highway Bridges and Structures: Design Manual for Roads and Bridges, London, UK, 2001.

[8] MELBOURNE C., COLE G., GILBERT M., SWIFT G., and SMITH C., A Road-Map for the Assessment of Masonry, *Proceedings of 7th International Conference on Arch Bridges*, 2013, pp. 605–612.

[9] GILBERT M., SMITH C., WANG J., CALLAWAY P., and MELBOURNE C.,

Small and large-scale experimental studies of soil-arch interaction in masonry bridges, *Proceedings of 5th International Conference on Arch Bridges*, 2007, pp. 381–388.

[10] SWIFT G., AUGUSTHUS-NELSON L., MELBOURNE C., and GILBERT M., Physical modelling of masonry arch bridges, *Proceedings of 7th International Conference on Arch Bridges*, 2013, pp. 621–628.

[11] AUGUSTHUS-NELSON L., SWIFT G., MELBOURNE C., SMITH C., and GILBERT M., Large scale physical modelling of soil-filled masonry arch bridges under cyclic loading, *International Journal of Physical Modelling in Geotechnics - Under Review.*