



## PRELIMINARY FINITE ELEMENT ANALYSIS OF A MASONRY ARCH BRIDGE WITH NEAR-SURFACE REINFORCEMENT

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### ABSTRACT

A three-dimensional finite element analysis of a typical brick arch highway bridge with near-surface reinforcement is presented. The effects of the fill over the barrel, spandrel walls, patch loading and the presence of defects such as cracks were included in the analysis. Initially an elastic analysis of the bridge under design service loading was undertaken. The effect of reinforcement was found to be most significant in the presence of existing cracks. In order to assess the effect of near-surface reinforcement on the load-carrying capacity of the bridge, a non-linear finite element analysis with incrementally applied UK standard highway loading was used. Preliminary results showed that the reinforcement delays the onset of cracking and increases the collapse load by up to 45%.

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## **INTRODUCTION**

Near-surface reinforcement (also known as retro-reinforcement) is a minimum-disruption method of strengthening stone or brick masonry arch structures such as bridges, culverts and tunnel linings. The technique consists of grouting stainless steel reinforcing bars into pre-drilled holes or pre-sawn grooves in the near-surface zones of the masonry where tensile stress levels are likely to result in cracking (Garrity 2001).

Most of the research into masonry arches with near-surface reinforcement has been limited to the study of two dimensional arch rings. In order to generate increased confidence in the strengthening technique and to provide further design guidance, it was considered necessary to study the influence of near-surface reinforcement on a complete single span highway bridge including the effects of the fill material, spandrel walls, wingwalls, patch loading and existing defects such as cracks. This was undertaken using the ABAQUS finite element (FE) code (Hibbit et al 1998) enhanced with the PATRAN 3D Computer-Aided Engineering software system (MacNeal-Schwendder Corp. 1995). The need to apply non-symmetrical loading and to model defects in the bridge required the use of a full three-dimensional model. As a result, the use of separate elements to simulate individual bricks and mortar joints in the FE model was discounted as it would prove to be too expensive computationally. Consequently, a continuum mechanics approach was used with the average properties of masonry represented as an homogenised medium.

Various researchers have used a linear elastic constitutive model for masonry under service load conditions, that is, prior to the formation of structural cracks. For post-cracking behaviour up to collapse, a non-linear constitutive model for a brittle material implemented in ABAQUS was used with material parameters appropriate for masonry. Although the model was developed for plain concrete, it describes cracking and crushing behaviour in qualitatively similar ways to those successfully adopted by other researchers for masonry (Ali and Page 1988, Loo and Yang 1991). No account was taken of any strain softening of the masonry in compression. This was not considered to be a significant omission as the arch ring was under-reinforced and the tensile stresses and strains in the masonry were likely to predominate. If, however, a much thinner arch ring was being analysed where crushing of the masonry was likely to occur, it would have been more important to include strain softening in compression in the constitutive model for the masonry. In contrast, the tensile strength of the masonry and the tensile strain softening parameter have a significant influence on the post-cracking and failure behaviour of arch bridges (Loo 1995) and were included.

An initial validation of the FE model using data from a series of tests on 2m span reinforced arches (Garrity 1995), showed that the reinforcement could be adequately simulated with truss elements included in a three-dimensional brick element mesh for the arch. The need to account for spandrel wall separation in the numerical simulation was also found to be important in order to achieve greater accuracy. The model was then used in a two-stage investigation of a typical 10m single span brick arch highway bridge with and without near-surface reinforcement. The investigation is described below. Initially, a linear elastic analysis was used to study the performance of both bridges under simulated in-service live loading. A non-linear analysis with incrementally applied loading was then carried out to assess the effect of near-surface reinforcement on the load carrying capacity.

## **THE FINITE ELEMENT MODEL**

The finite element mesh of the bridge used in the study is shown in Figure 1. A summary of the assumed construction details and material parameters is given in Table 1. The use of 20-node quadratic elements and 8-node linear elements was considered. Although the quadratic elements provide greater accuracy, capture stress concentrations and permit improved modelling of curved surfaces, their use requires significantly greater processing and memory requirements. In a trial, it was found that the use of 8-node linear elements produced maximum principal strain values within about 15% of those produced using the quadratic elements with about 8% of the processing time and 28% of the memory required. Hence, 8-node linear elements were used in the study reported in this paper.

The fill was also modelled with similar three dimensional elements to simulate the self weight and to provide some dispersal of the live load applied at carriageway level through to the arch barrel. No attempt was made to model the effects of lateral earth pressure. The fill mesh size was chosen to facilitate the application of live loading. The foundation was modelled as an additional layer of finite elements beneath the abutments and wingwalls. This was done to permit the application of a prescribed displacement to one of the abutments to simulate torsion and rotation due to differential settlement. For the purposes of the study, both the fill and the foundation materials were assumed to behave in a linear elastic manner.

The longitudinal and transverse reinforcement was modelled as a 3-D truss element mesh acting on the arch intrados. In practice, the reinforcement would be installed in pre-sawn grooves or in pre-drilled holes in the outer zone of the masonry; this was accounted for by reducing the arch thickness by an amount corresponding to the depth of the grooves or holes. It was assumed that the reinforcement remained fully bonded to the masonry substrate at all times; no attempt was made to account for any premature failure due to de-bonding.

## **LOADING**

The unit weight of the masonry was assumed to be  $20 \text{ kN/m}^3$ ; the same unit weight was assumed for the fill. When assessing the capacity of existing masonry arch highway bridges in the UK, the regulatory authorities usually require consideration of several alternative types of live loading. For simplicity the authors only considered one form of live loading in this study, namely 30 units of highway type B (or “HB”) live loading (Highways Agency et al. 1988). This consists of a total characteristic imposed load of 300 kN which is carried equally on four axles of a notional vehicle known as the HB vehicle. Each axle consists of 4 wheels spaced laterally across the carriageway at 1.0m centres. The spacing of the axles in the direction of the traffic flow is 1.8m; 6.0m and 1.8m, respectively. This represents standard loading in the UK for highway structures carrying a moderate to heavy volume of traffic which is typical of many masonry arch bridges.

## **RESULTS OF LINEAR ELASTIC ANALYSIS**

The principal aim of this analysis was to compare the in-service performance of the bridge with and without near-surface reinforcement. The position of the HB vehicle was varied over the upper surface of the fill to obtain the maximum principal strains in the masonry. The worst case condition was obtained when two of the axles spaced 1.8m apart were located between the midspan and quarter span of the arch and the other two axles were located outside the arch.

As expected, the effect of the reinforcement on the strain distribution in the masonry was very small up to first cracking with the most noticeable effect being a reduction in the maximum principal tensile strain in the masonry of in the order of between 4% and 8%. Parametric studies indicated that the magnitude of the reduction in strain was primarily a function of the value of the elastic modulus of the fill used in the analysis. The presence of spandrel walls was also found to be important; tensile strain reductions of up to 15% were found in the absence of the edge stiffening provided by spandrel walls. Hence, near-surface reinforcement is likely to be of benefit in reducing tensile strains in the masonry in the event of spandrel wall separation.

The main influence of the near-surface reinforcement was found to be where it was used as a strengthening measure to bridge across an existing crack. Initially, a vertical crack (or discontinuity) was created in the parapet and spandrel mesh of the bridge at quarter span. Abutment movement was applied to the model to simulate a vertical differential movement. The analysis showed a large stress concentration at the tip of the crack and large horizontal displacements between the faces of the discontinuity simulating the crack opening up as a result of the differential movement of the abutments. Horizontal reinforcement was then installed in the parapet and spandrel masonry of the finite element model to span the crack and it was found that the tensile stress distribution adjacent to the discontinuity was similar to that in the adjacent uncracked spandrel. In

addition, the two adjacent faces of the discontinuity moved much closer together. Hence, horizontal reinforcement appears to be very effective at preventing further crack growth and restoring structural integrity.

## **RESULTS OF NON-LINEAR ANALYSIS**

This analysis was carried out to examine the influence of near-surface reinforcement on the load to first cracking and the load-carrying capacity of the bridge. The HB vehicle was fixed in the position obtained from the elastic analysis described above and the magnitude of the applied load was increased incrementally until a collapse condition was reached. The load versus deflection results obtained from the FE analysis for a point on the intrados beneath one of the wheels of the HB vehicle, for the unreinforced and reinforced bridges, are presented in Figure 2. The cracking and collapse loads are summarised in Table 2.

Judging from the results, for the bridge under consideration, it appears that near-surface reinforcement produces an increase in the load to first cracking of about 28% and a 45% increase in the load carrying capacity. In addition, the reserve of strength beyond first cracking was increased by about 11%.

## **CONCLUSIONS**

The following conclusions are based on a preliminary three-dimensional FE analysis of a typical 10m span brick arch highway bridge with and without near-surface reinforcement subjected to 30 units of HB live loading.

The provision of near-surface reinforcement was found to:-

- a). Have a minimal effect on the magnitude and distribution of tensile strain in the masonry up to first cracking;
- b). Arrest further growth of an existing crack and to restore structural integrity;
- c). Help reduce tensile strains in the event of a loss of edge stiffening resulting from spandrel wall separation;
- d). Increase the load at which cracking first occurs by about 28%;
- e). Increase the reserve of strength beyond first cracking by about 11%;
- f). Increase the load carrying capacity by about 45%;

The three-dimensional FE analysis included all the features of a typical masonry arch bridge including the fill and the foundations. In addition, the effects of patch loading

and existing defects in the masonry such as cracking were taken into account. Linear elastic and non-linear FE analysis could both be used in the design of near-surface reinforcement for a masonry arch structure if:-

- a). An improved constitutive model for the masonry is implemented in the FE model;
- b). More reliable material parameters are obtained from calibration exercises using the data from large-scale tests on arches with near-surface reinforcement;
- c). The effect of parameter variation on the design is assessed by parametric study.

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Table 1: Construction details and material parameters  
(to be read in conjunction with Figure 1)

<b>Construction Details</b>		<b>Assumed Material Parameters</b>
<b><i>Masonry</i></b>		
Arch ring (4 courses thick)	Segmental profile; 0.44m thick; 2.5m rise; 10.0m square span between faces of abutments. 9.3m wide between inside faces of parapets.	Elastic modulus = 5 kN/mm <sup>2</sup> (Crisfield 1984, Hendry 1990, Hughes & Blackler 1997)
Spandrel walls	0.44m thick; min. height 0.4m (at crown)	
Parapets	0.215m thick, 1.0m high (above level of fill/spandrel walls)	Limiting tensile strain (onset of cracking): $1.5 \times 10^{-4}$
Wingwalls	All 4 wingwalls identical. Each 6m long (measured from face of abutment) x 4.5m high	Ultimate tensile strain: $3.0 \times 10^{-4}$
Abutments	1.5m thick x 1.2m high (measured from springing to ground/foundation level)	Tensile strength: 0.75 N/mm <sup>2</sup> Compressive strength: 7.5 N/mm <sup>2</sup>
<b><i>Reinforcement</i></b>		
Longitudinal steel	Pairs of 6mm dia. bars spaced at 0.215m centres.	Elasto-plastic material model assumed. $E = 200$ kN/mm <sup>2</sup> , Poisson's ratio = 0.3, yield strength = 460 N/mm <sup>2</sup> .
Transverse steel	6mm dia. bars spaced at 0.45m centres.	
<b><i>Fill &amp; Foundations</i></b>		
Fill	Horizontal surface; min. depth 0.4m at crown. Top of fill coincident with top of spandrel walls	Linear elastic behaviour assumed. $E_{\text{fill}} = 0.5$ kN/mm <sup>2</sup> $E_{\text{foundation}} = 1.0$ kN/mm <sup>2</sup>
Foundations	0.5m thick elastic compressible layer supporting abutments, wingwalls and fill behind the abutments	Poisson's ratio = 0.18 (fill & foundation)

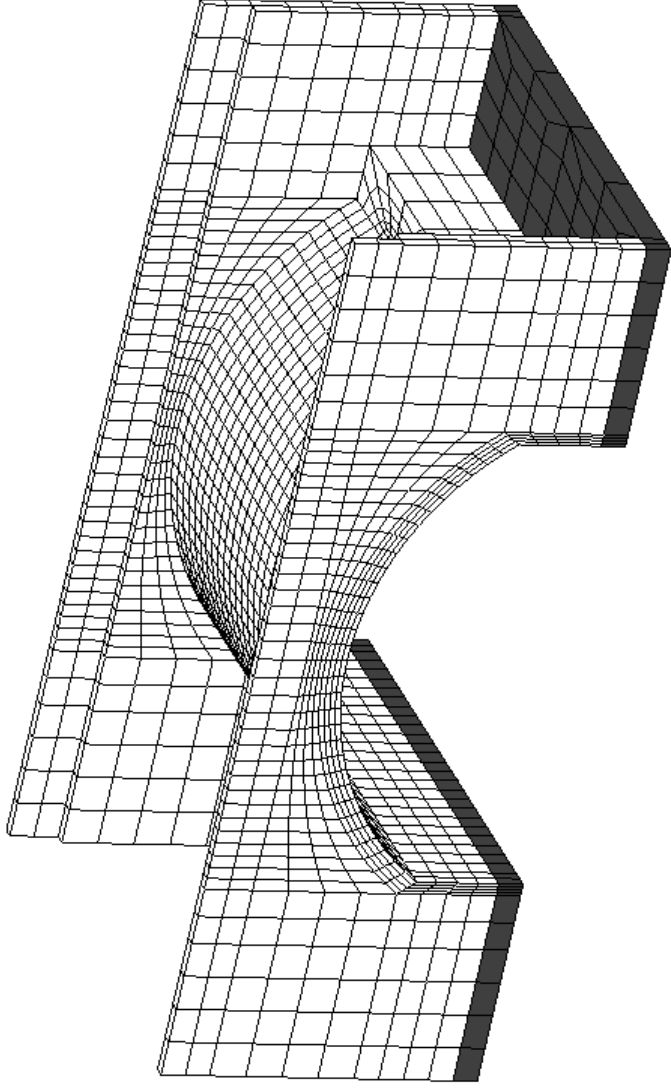


Figure 1. Finite element mesh for the bridge and foundation  
(fill and reinforcement meshes omitted for clarity)



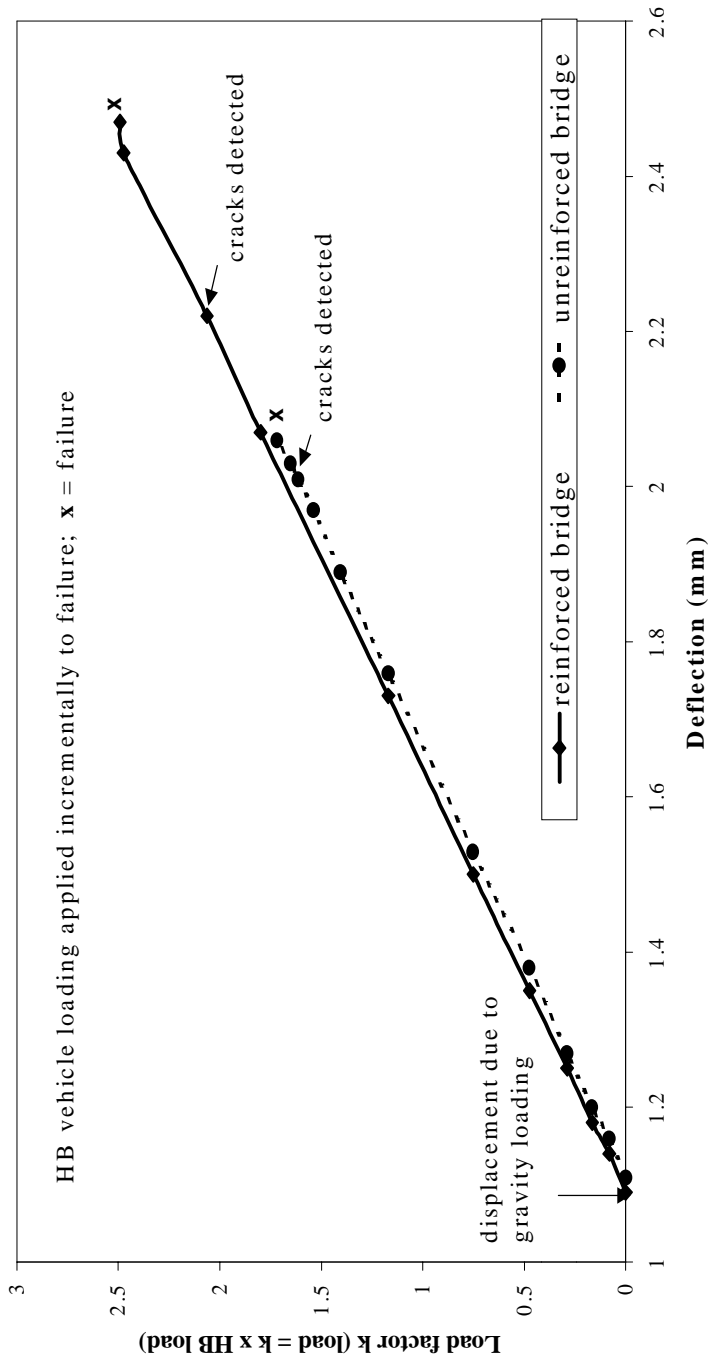


Figure 2. Comparison of results from non-linear FE analysis of reinforced and unreinforced bridges subjected to 30 units of Highway Type B live loading (Highways Agency et al. 1988)

Table 2: Summary of loads causing cracking and collapse from FE analysis for unreinforced and reinforced bridges subjected to 30 units of HB live loading

Arch bridge	Load at first cracking [kN]	Collapse Load [kN]	Cracking Load ÷ Characteristic Load	Collapse Load ÷ Characteristic Load	Collapse Load ÷ Design Ultimate Load
Unreinforced	484	515	1.61	1.72	0.78
Reinforced	619	748	2.06	2.49	1.13

*Notes*

- a). Total Characteristic Live Load (30 units of HB) = Design Service Load = 300 kN
- b). Total Design Ultimate Load =  $300 \times 2.0 \times 1.1 = 660$  kN where 2.0 = partial safety factor for imposed load ( $\gamma_{li}$ ) and 1.1 = partial safety factor for load effect ( $\gamma_{fs}$ ) in accordance with BD 21/97 (Highways Agency et al. 1997)