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17–19 May 2018, Zadar, Croatia

Road and Rail Infrastructure V

Stjepan Lakušić – EDITOR

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APPLICATION OF FINITE ELEMENT METHODS TO MASONRY BRIDGES

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Abstract

Across Europe, masonry bridges remain vital to the infrastructure networks and most are now more than 100 years old. Good management of these bridges requires a thorough understanding of the structural behaviour of each bridge. This paper explores the application of finite element methods to masonry bridges. Options for idealisation are outlined and recommended; considering issues of soil-structure interaction, material parameters and nonlinearity.

Keywords: bridges, brickwork & masonry, stress analysis

1 Introduction

Most masonry bridges are backfilled with local material between the external spandrel walls, to provide a near-level running surface. The fill plays a stabilising role, with soil pressures opposing movement in the arch as per [1] section 2.6.3 and [2]. This implies the need for a soil-structure interaction (SSI) analysis. Such an FE model could be constructed using 3D continuum elements. However, such an approach can be rather inefficient since:

1) There is no fixed extent for the model, with soil extending vertically downwards and in both horizontal directions. This can lead to large numbers of elements and nodes in 3D continuum models.
2) Both soil and masonry are inherently nonlinear materials. Solution requires iteration, which is inherently more time-consuming than a simple linear static solution.

The use of 2D continuum models or 3D shell element models is suggested.

2 Continuum modelling in 2D

2.1 Prestwood bridge example

Figure 1 shows results from a 2D plane strain model, based on the loading to collapse of the Prestwood Bridge, as described in [3]. The soil has been assigned a Mohr Coulomb material and the masonry has been assigned a material which can model cracking and crushing, with the parameters shown in Table 1 – as in [4].

Table 1 Material parameters (Prestwood)

Soil		Masonry	
E'	50E3 kN/m²	E	4.14E6 kN/m²
ν'	0,25	ν	0,15
ρ	2,04 t/m³	ρ	2,04 t/m³
c'	7 kN/m²	f_c	4,5E3 kN/m²
φ'	37°	f_t	130 kN/m²
ψ	5°	G_f	0,03 kJ/m²

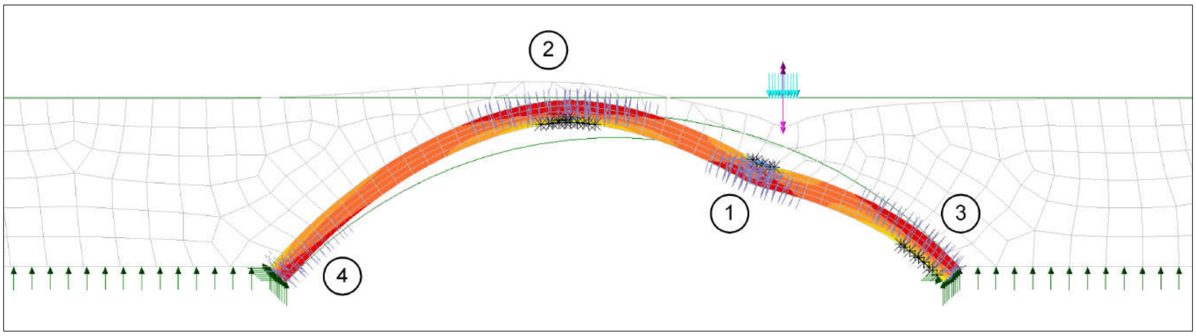

Figure 1 2D model of Prestwood Bridge at numerical failure; P=0.85×F.

The formation of 4 hinges, in the order numbered (in Figure 1), shows good correspondence to that observed in the test, and the predicted failure load (P), at 85 % of the test load at collapse (F), is quite reasonable. Cracking planes shown in grey; crushing indicated with black symbols. Moreover, the model offers the engineer the possibility of varying properties in order to determine the effects of changes in assumptions on the results. The following sections describe the key components of this model.

2.2 Soil material nonlinearity

There are many nonlinear material models designed to represent soil behaviour. The Mohr-Coulomb model is widely used as per [1] section 4.4.3.5. However, determination of input parameters appropriate to an existing bridge is often problematic. Sample values may be obtained from [5] section 8.1.4.5 and a helpful summary of the likely influence of values on collapse load is given by [1] section 4.5.8.2. A sensitivity analysis is advisable.

2.3 Interface options

Some studies assume full contact between soil and structure [6], but sliding may take place when approaching collapse, as per [1] sections 4.4.3.5 and 4.5.8.2. Where required, the interface may be represented in an FE analysis by way of [4]:
1) Joint elements & materials [7], section 4.12.
2) Contact slidelines [7], section 5.4.
3) Elasto-plastic interface materials [7], section 4.4.2.2.

2.4 Masonry material nonlinearity

Approaches to the modelling of masonry are also discussed in [4]. The 'smeared' approach is recommended along with a cracking and crushing material such as that described by [8] and [7] section 4.7. Figure 2 shows how this 'smeared' cracking and crushing material replicates the expected behaviour, as load is applied at approximately quarter-span on an arch barrel.

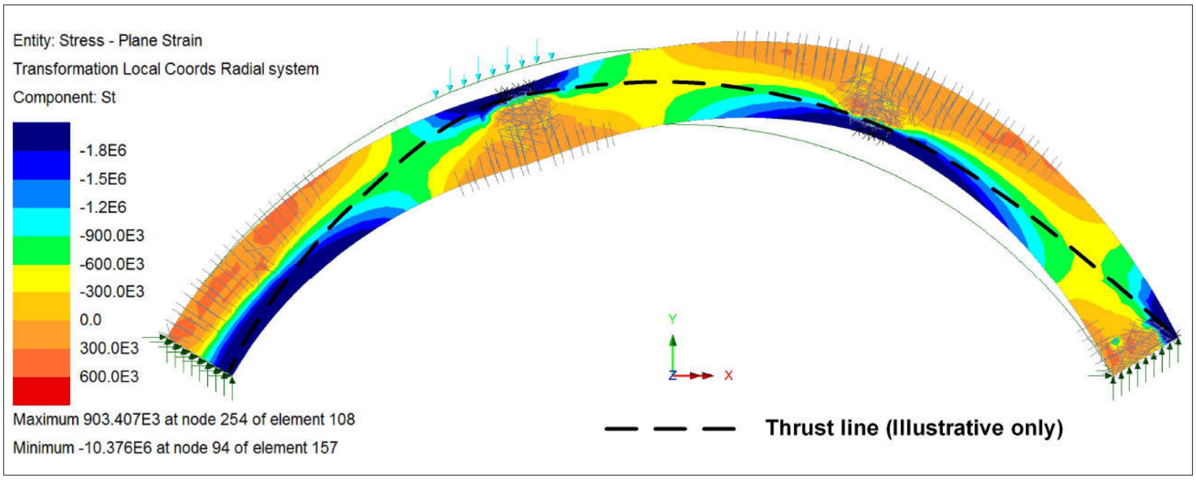

Figure 2 Formation of 4-pinned arch using plane strain model with concrete material

Determination of suitable values for masonry material parameters is, again, often a challenge for an existing structure. Testing is usually of limited use since obtaining a statistically representative sample would cause damage to the structure. Again [1] provides guidance and sample values, and further references for the example considered are given in [4].

2.5 Large displacements

The cracking behaviour and deformation leads to a displaced thrust line, passing through the uncracked material, as shown in Figure 2 (also refer to [3]). This warrants treatment using large displacement theory as failure is approached – handled by invoking a suitable geometric nonlinear option in the FE solution ([7] section 3)

2.6 Ring separation

Under cyclic loading, the 'fatigue capacity' of multi-ring masonry arches has been found to be of the order 50 % of the static strength ([5] section 8.1.4.4). Often these effects are overlooked in bridge assessments. In the fatigue tests of [9] all the multi-ring arches tested failed by ring separation as opposed to the 4-pin mechanism widely anticipated and illustrated in Figure 2 above. With this in mind, it may be appropriate to model the arch with interface planes between such rings. Figure 3 illustrates the changed behaviour as compared to Figure 2, and corresponding to a significantly reduced ultimate load.

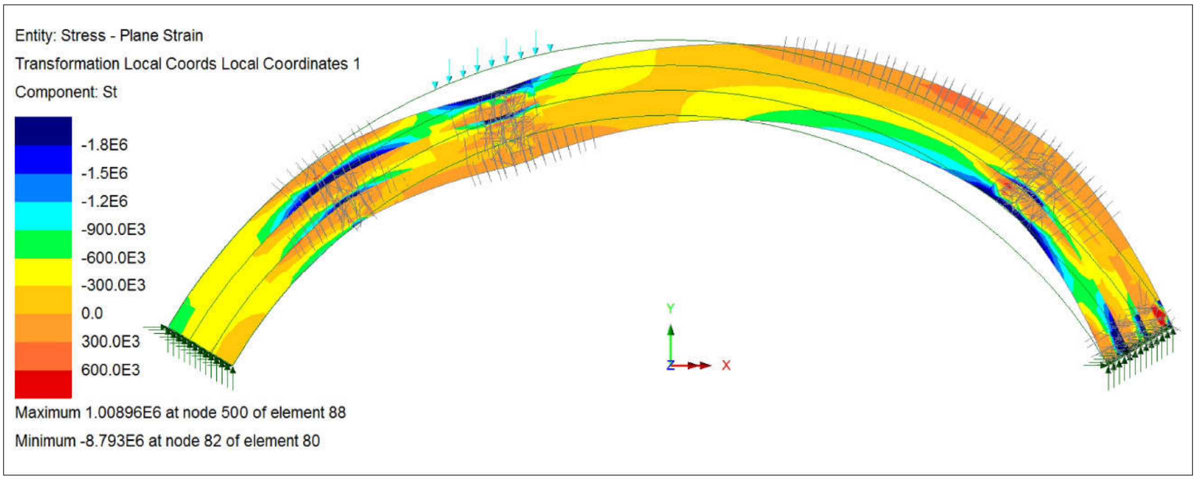

Figure 3 Failure of arch in a model including interfaces to allow ring separation

2.7 Support conditions

In general, support conditions have a significant influence on results from bridge analysis, and masonry bridges are no exception. In the examples above, rigid supports have been used, however, in order to obtain realistic stress distributions, sprung supports or explicit modelling of the subgrade using continuum elements is recommended.
When modelling a structure interacting with a soil mass, the extent of the model is not straightforward to define: vertical and horizontal boundaries must be imposed on the soil mass at some distance from the structure. Where such boundaries cannot be reasonably defined to match physical boundaries (e.g. free soil face, bedrock) they need to be determined by comparing key results from several models which are identical except for the assumed width or depth. Settlement, subsidence and scour are identified as important causes of defects in [10]. Support movements cannot be considered by many available tools – a significant limitation when attempting to make sense of the crack patterns or other damage observed in an old bridge. They can, however, be included in FE models.

2.8 Model development and consideration of remedial options

As described, it is likely that there will be much uncertainty in parameters for soil, masonry, supports, and any interfaces. But if the purpose of an FE analysis of a masonry bridge is to promote understanding of the behaviour of the system, then this does not require accurate values for parameters. Instead, it requires a synergistic comparison of model behaviour and results such as crack patterns against observations from site.
Planning of any intervention intended to strengthen a structure must be with great care. When the example of Figure 3 is modified for the insertion of radial dowels, the collapse load may be apparently doubled [4], but the failure mode is found to be more brittle and therefore likely to be more sudden in practice. With this knowledge, a client may prefer to increase monitoring, rather than carry out the intervention.

3 3D shell element modelling

2D models provide a good starting point for validation of a modelling approach using benchmark problems, for the study of SSI effects and sensitivity of the model to assumed parameters, and may in some cases be adequate for the purpose of the analysis. However, it is identified in [1] section 2.1.4 that even “modest span railway bridges often have internal spandrel walls directly below the rails”. These are likely to act as stiffeners to the barrel and may have great effect on the behaviour of the structure. The external spandrels and parapets may also stiffen the edges of the arch. No 2D analysis method (including the 2D continuum approach above) can properly take account of the influence which these stiffeners may have upon the structure as a whole. If they cannot be neglected, then a 3D approach is required.
Figure 4 shows results from a 3D model based, again, on the loading to collapse of the Prestwood Bridge [3]. In this instance, the size of model is reduced as compared to a corresponding 3D continuum model, by using shell elements to represent the masonry and joint elements to represent the soil. Further information on transverse behaviour and spandrel wall failure is given in [11].

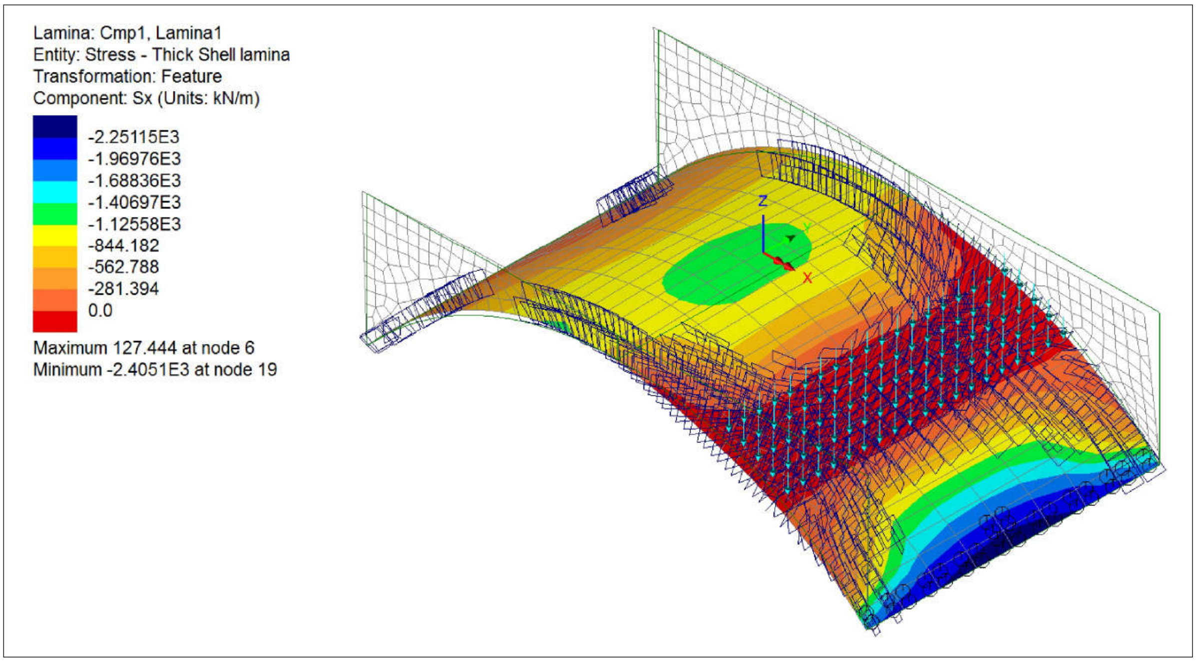

Figure 4 3D shell model of Prestwood Bridge at numerical failure; P = 1.047×F. Cracking planes and crushing locations in underside of arch illustrated.

3.1 Representing the masonry

Shell elements carry in-plane forces and in-plane shears, and transverse loads by flexure, twisting and out-of-plane shears. Crucially shell elements can be formulated in such a way as to allow gradual through-section plastification [7] – or in this case, cracking – enabling them to replicate the softening of masonry due to such damage using a macro-modelling approach. The considerations for modelling the masonry material in the 3D model are then the same as those described for the 2D alternative in section 2.4 above. However, it is not possible for ring separation or the remedial of section 2.8 above to be incorporated in a 3D shell element model. A full 3D continuum model of the bridge would be required

3.2 Representing the soil

Nonlinear joints have been used in this example, as an alternative to the use of 3D continuum elements, to represent the soil. These joints, acting as springs placed between a notional rigid boundary and the masonry structure (modelled with shell elements) reflect a pressure/deflection relationship such as that illustrated in Figure 5 below.

Critically Figure 5 incorporates not only the horizontal stiffness of the soil, using a horizontal modulus of subgrade reaction (k_h), but also at-rest earth pressures (σ'_o, based on K_o). Neither quantities are considered when designing structures using limiting earth pressure methods, however they are essential components of SSI analyses. Typically all the quantities represented in Figure 5 – active and passive 'yield points', the spring stiffness, k_h, and the at-rest pressure – vary with depth.

Such an approach is in keeping with EN1997-1 [12] clause 9.5.4 and [3] section 3.4.3(a) and significantly reduces the size of the model, but comes at the cost of particular assumptions:

- The determination of spring stiffness is problematic, since it is not a fundamental property of the soil
- The weight of the soil must be added to the model as a vertical load
- Dispersal of wheel loads must be handled by assumptions such as those of elastic half-space.
- Increase in lateral pressures local to wheel loads is assumed to be negligible.

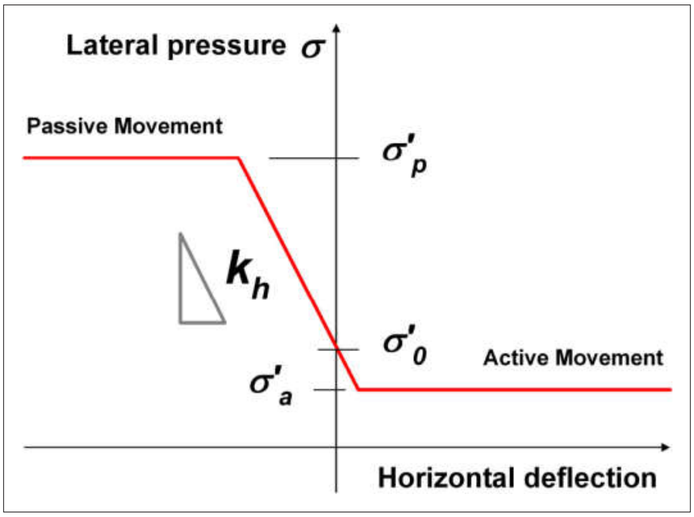

Figure 5 Nonlinear 'soil joint' pressure/ deflection graph (after [5])

4 Comparison of results

Both the 2D (plane strain) model and the 3D shell model give a predicted failure load, P, which corresponds reasonably to that observed in the physical test: 0.85×F and 1.04×F respectively. There is then a temptation to assume that this indicates either model to be adequate and even that the shell model is superior. However, it could be that one or both models are exhibiting a false correlation – that is, model behaviour does not reflect the structural behaviour very well, and the numerical agreement is somewhat a matter of chance, perhaps due to erroneous but compensating assumptions. The possible outcomes when comparing two models of the same structure – applicable to all manner of structural analyses – are set out in [13] along with a note that false correlations are surprisingly common.

In this case, the formation of cracks of specific direction and hinges in a specific order was identified as being agreed between the physical test and the 2D plane strain model. On the contrary, the 3D shell model exhibits longitudinal cracking caused by edge stiffening which was not observed in the test. The behaviour of the 3D model is not very well aligned with site observations, indicating that the close agreement of predicted failure load with the collapse load from the test does appear to be a false correlation.

Rather than simply discard the 3D model, more can be learned. It appears that the spandrel walls do not contribute significantly to the behaviour of this bridge. In other structures they may be stronger, stiffer, and deeper, and the contrary would be true. Sensitivity analysis indicates that the soil stiffness does not have a large effect on the behaviour in this case: owing to low depth of fill and low rise in the arch, lateral pressures are not as significant as they may be for other bridges. The inclusion of soil dead weight, with its precompression effects is, however, found to be significant in the calculations. Observations of this sort can assist the engineer in further study of the structure.

5 Conclusions

FE models of masonry bridges can include:

- Explicit modelling of the behaviour of fill including dispersal of load and stabilising effects using nonlinear materials such as Mohr-Coulomb.
- Crushing/ cracking material for masonry, allowing comparison of crack patterns against those observed on site
- Appropriate modelling of soil-structure interface
- Modelling of ring separation to assist in the understanding of possible fatigue failure modes

- Complete flexibility of geometry, materials and support conditions. 3D models may include haunches, internal and external spandrels and incorporate skew as necessary.
- Ability to model defects & repairs

However, any such modelling should be mindful of the limitations imposed by the uncertainty inherent in modelling older structures. FE analysis is a tool, to be used alongside other analysis approaches, understanding their particular strengths and limitations. But most importantly, the results should be brought together with site observations and monitoring, not only to help quantitatively assess, but moreover to promote the necessary understanding of structural behaviour.

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