Model tests on masonry arches

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The Author's conclusions concur with my own work on 1 m, 1·5 m, 3 m, 5 m, and 6 m span bridge test[17–21] which have resulted in the idealization shown in Fig. 17. It is from this theoretical model that the assessing engineer may start and thus determine the relative significance of each component. This allows the different modes of failure to be studied. If the contribution by the spandrel walls to barrel stability is high (which in most cases it will be), transverse distribution becomes significant. This has been observed during the monitoring of a two-span stone bridge during the passage of an abnormal load.[22] The bridge showed no signs of distress longitudinally, but transversely, local to the wheel loading, tensile strains were recorded which were commensurate with crack formation. As it was likely that transversely there was no precompression (unlike longitudinally where compression is guaranteed), it followed that the recorded tensile strains did cause cracking. Fig. 18 shows the transverse strain and deflexion profiles. This raises the question as to whether or not the adoption of a full width KEL as the standard loading case is restrictive, as it increases the likelihood of a classical four-hinge mechanism or a three-hinge snap-through failure depending on the spandrel wall stiffening and barrel backfill. It was reassuring, therefore, to see confirmation of this concern by the Authors' use of a half-width loading case. Other types of loading may cause local distress or even a mode of failure not hitherto considered. Certain types of arch bridge are more likely to be susceptible to non-standard modes of failure, e.g. skew, elliptical or multi-ring brickwork arch bridges. It has been shown[21, 22] that point loads can cause local distress to an arch.

47. The real value of model testing lies in the prediction of the mode of behaviour and the load-carrying capacity of full-scale structures. The Authors' analysis considered only one of the variables, i.e. vertical loading. A more extensive analysis using the parameters identified in Fig. 17 results in a more complex situation. Assuming the densities of the prototype and model are the same for each of the constituents, it follows that, using the Authors' notation, P_p/P_m for the various parameters is related to L_r^2 (for the vertical dead load), L_r^3 (for the horizontal longitudinal soil pressure), L_r^3 (for dead load of the spandrel wall), L_r^2 (for cohesion of backfill with the spandrel wall), L_r^3 (for frictional resistance of the backfill with the spandrel wall due to self-weight) and L_r^4 (for frictional resistance of the backfill with the spandrel wall due to the applied loading). Additionally, as the Authors correctly explored, there is the effect of relative strength.

48. It follows that any extrapolation of the results for application at full scale should be treated with care as the scaling factors are not the same for each parameter.

49. In spite of the doubts raised, it is interesting that the correlation between model and full-scale behaviour is good. This is further demonstrated in Figs 19 and 20 which shows tests on model and large-scale multi-span bridges respectively. Details of these tests are presented elsewhere.[23] Important observations relevant to the present discussion are that the spandrel walls played a significant role in the determination of the hinge positions and hence of the load-carrying capacity, that the mode of behaviour of the model and large-scale tests was similar, and also that a modified mechanism analysis was appropriate.

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We agree that the factors portrayed by Fig. 17 are important in the assessment of masonry arch bridges. However, these factors are by no means exclusive—geometric proportions of

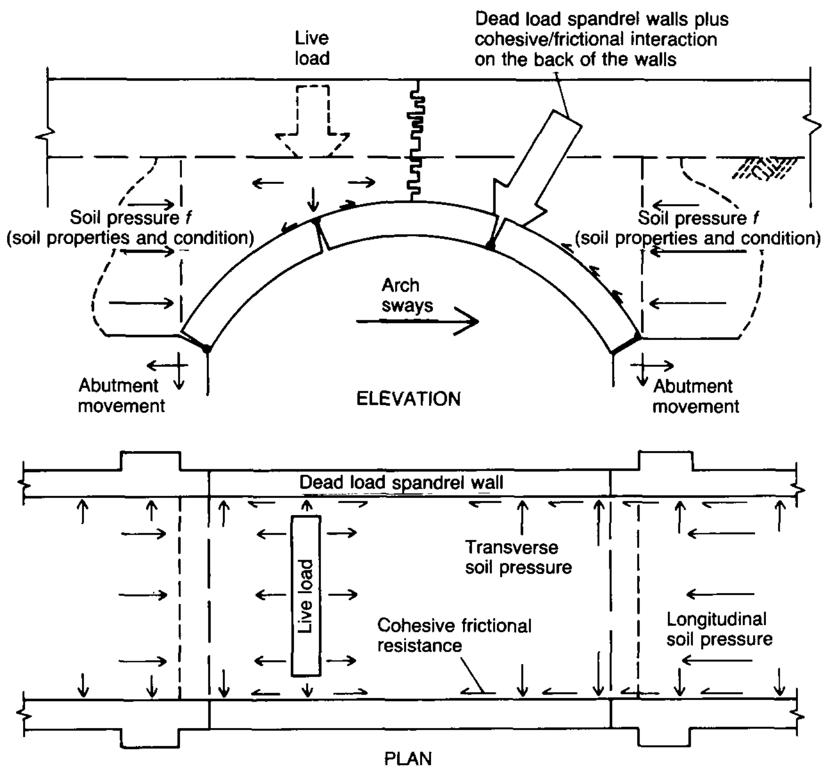


Fig. 17. Theoretical physical model

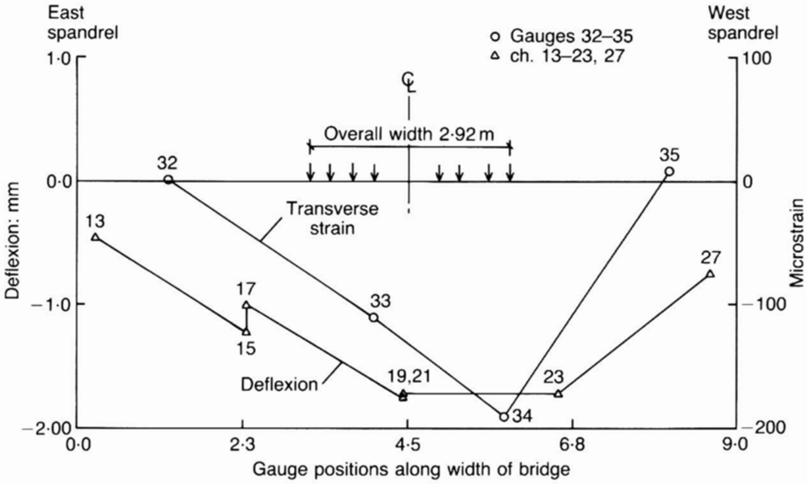
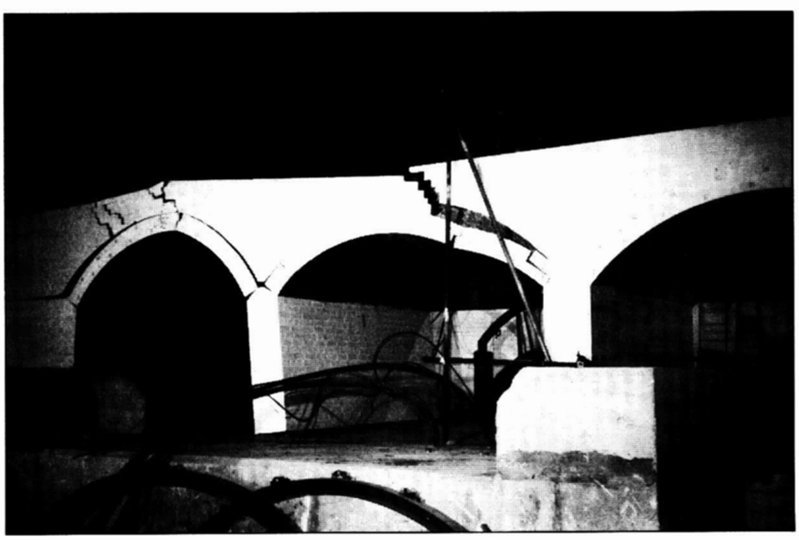


Fig. 18. Egremont Bridge test night-time loading: deflexions and strains at the crown for load stage 8

Fig. 19. Model multi-span bridge test



components such as spandrels, vault and wing walls together with material and support conditions must not be ignored, as was highlighted by the Bargower Bridge and others.[1] In two of the reported bridge model test series—BG and BM—an attempt was made to replicate the fill that existed in the full-scale counterparts. The factors shown in Fig. 17 were present in the complete bridge tests of the BG and BM series and the ensuing scale-up of failure loads was in reasonable agreement with the full-scale values. The question of scaling factors requires careful attention; more model testing would help to clear up uncertainties in this area.

51. The test data from the Egremont Bridge (Fig. 18) emphasize the three-dimensional nature of the masonry arch bridge system evident in the present work. The amount of tensile strain indicated in the transverse direction was less than 200 microstrain (Fig. 18), but it is not clear whether or not any residual strain/stresses were present before the passage of the abnormal load, nor are the material characteristics (e.g. stress–strain relationship) available for a more careful judgement to be made about the onset of cracking.

52. It is interesting to learn, perhaps not surprisingly, that influences on the behaviour of single span masonry arch bridges illustrated by the present work are similarly manifest in multi-span arrangements.

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Fig. 20 (left). Large-scale multi-span bridge test