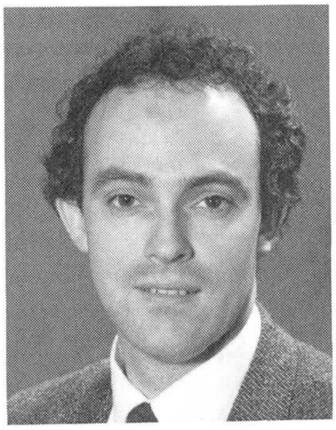
Behaviour of a Prestressed Brickwork Diaphragm Wall Bridge Abutment

Comportement d'un voile de culée de pont en maçonnerie précontrainte

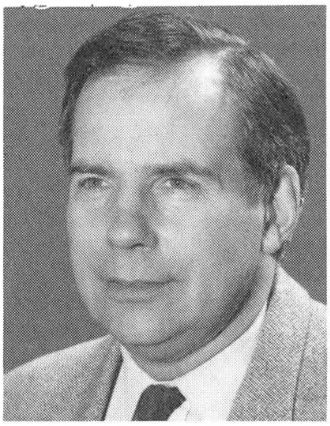
Verhalten einer Brückenflügelmauer aus vorgespannten Mauerwerk

Stephen GARRITY
Senior Lecturer
Bolton Inst. of Higher Educ.
Bolton, Lancs, UK

Thomas GARWOOD
Senior Lecturer
Bolton Inst. of Higher Educ.
Bolton, Lancs. UK



Stephen Garrity, born in 1956, graduated from Nottingham University in 1977. For eight years he worked mainly as a bridge engineer with Sir Alexander Gibb and Partners and Greater Manchester Council. He has a particular interest in the properties and performance of concrete and masonry.



Thomas Garwood, born in 1940, graduated from Cambridge University in 1963. He then spent five years with John Laing Construction Limited and four years as a research student at Salford University before being appointed to his present position. He has written several papers on reinforced and prestressed brickwork.

SUMMARY
A prestressed brickwork diaphragm wall, which represented part of a full size bridge abutment, was constructed and tested in the laboratory. The test loading simulated both the earth pressure forces and the longitudinal load from the bridge deck. Under the service load condition, there was no cracking in the brickwork. At the final stage of the test, when the shear force and bending moment resisted by the abutment exceeded the service load values by more than 75%, there were no signs of impending failure.

RÉSUMÉ
Un voile précontraint en maçonnerie de brique, représentant une partie de culée de pont grandeur nature, a été construit et testé en laboratoire. La charge d'essai simulait à la fois les poussées de la terre et la charge longitudinale du tablier de pont. Aucune fissure n'est apparue dans le voile sous les surcharges de service. Dans la dernière phase de l'essai, alors que la culée était soumise à une force de cisaillement et à un moment fléchissant qui dépassaient les valeurs des surcharges de service de plus de 75%, aucun signe de rupture imminente n'a été constaté.

ZUSAMMENFASSUNG
Eine vorgespannte Flügelmauer, Teil eines Brückenwiderlagers, wurde in voller Grösse im Labor getestet. Es wurden sowohl die Bodendruckkräfte als auch die Vertikallast vom Brückendeck simuliert. Unter Betriebslastbedingung konnte in der Mauer keine Rißbildung festgestellt werden. In der Endstufe des Tests, als die Scherkraft und der Biegemoment, die Betriebslastwerte um mehr als 75% überschritten, gab es keine Anzeichen eines unmittelbar bevorstehenden Fehlverhaltens.

1. INTRODUCTION

Curtin et al [1,2,3] have shown that the prestressed masonry "diaphragm" or cellular wall is an efficient structural form which can be used economically to resist high shear forces and bending moments resulting from lateral loading. In view of the problems which have occurred with reinforced concrete bridges and other highway structures as a result of reinforcement corrosion, it is possible that prestressed brickwork diaphragm wall construction could be a cost-effective alternative to reinforced concrete for bridge abutments. Furthermore, prestressed brickwork is likely to have greater aesthetic appeal.

Although prestressed brickwork diaphragm walls have been used for bridge abutment construction [4], no full-scale structures of this type have been tested. This paper describes a test carried out on a full-scale prestressed clay brickwork bridge abutment built in the laboratory. The headroom available limited the height of brickwork to 4.275m. This would be the height of brickwork, from the top of foundation to the capping beam soffit, in an abutment of 6.6m overall height as shown in Figure 1a.

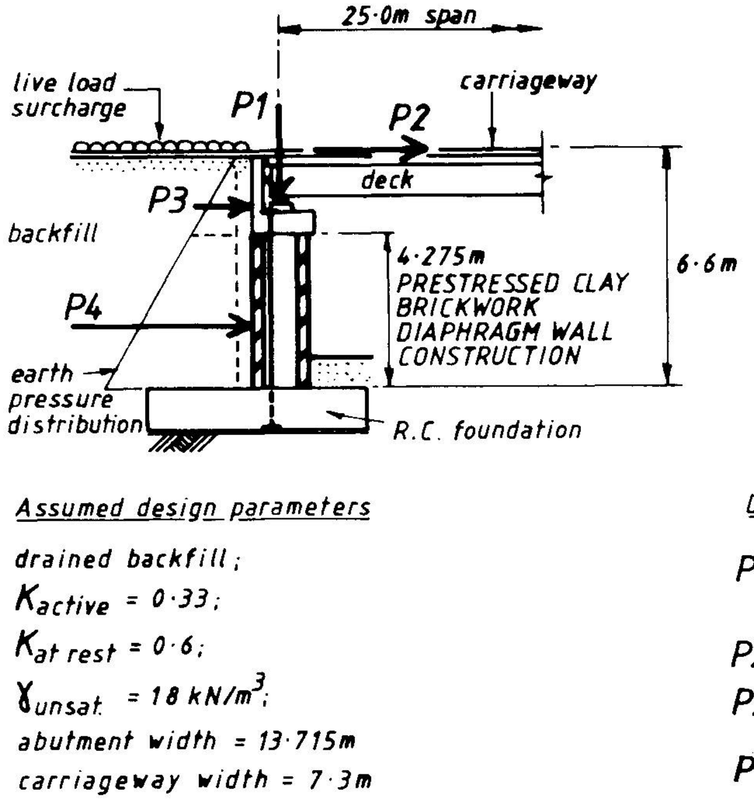


Fig. 1a Abutment detail showing design loads

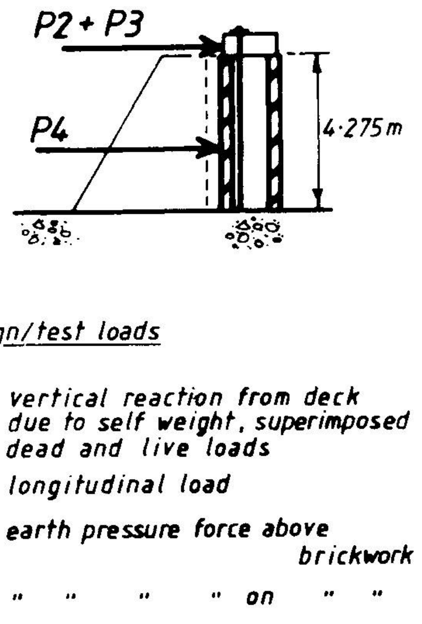


Fig. 1b Loads simulated in test

2. TESTING

2.1 Construction details

Details of the test arrangement are shown in Figure 2. Engineering bricks having an average crushing strength of 103 N/mm² and 5.8% water absorption were used with a 1:¼:3 (cement:lime:sand) volume batched mortar. The prestressing force was provided by 6 No. 40mm diameter Macalloy bars at an eccentricity of 250mm. The prestressing force in each bar was 910kN; the prestress in the brickwork was 1.02 N/mm^2 and 3.74 N/mm^2 in the front and back flanges respectively. The total loss of prestress, in the 5 month period between prestressing and testing, was estimated to be approximately 5%. The test abutment construction and losses of prestress are described in greater detail elsewhere [5].

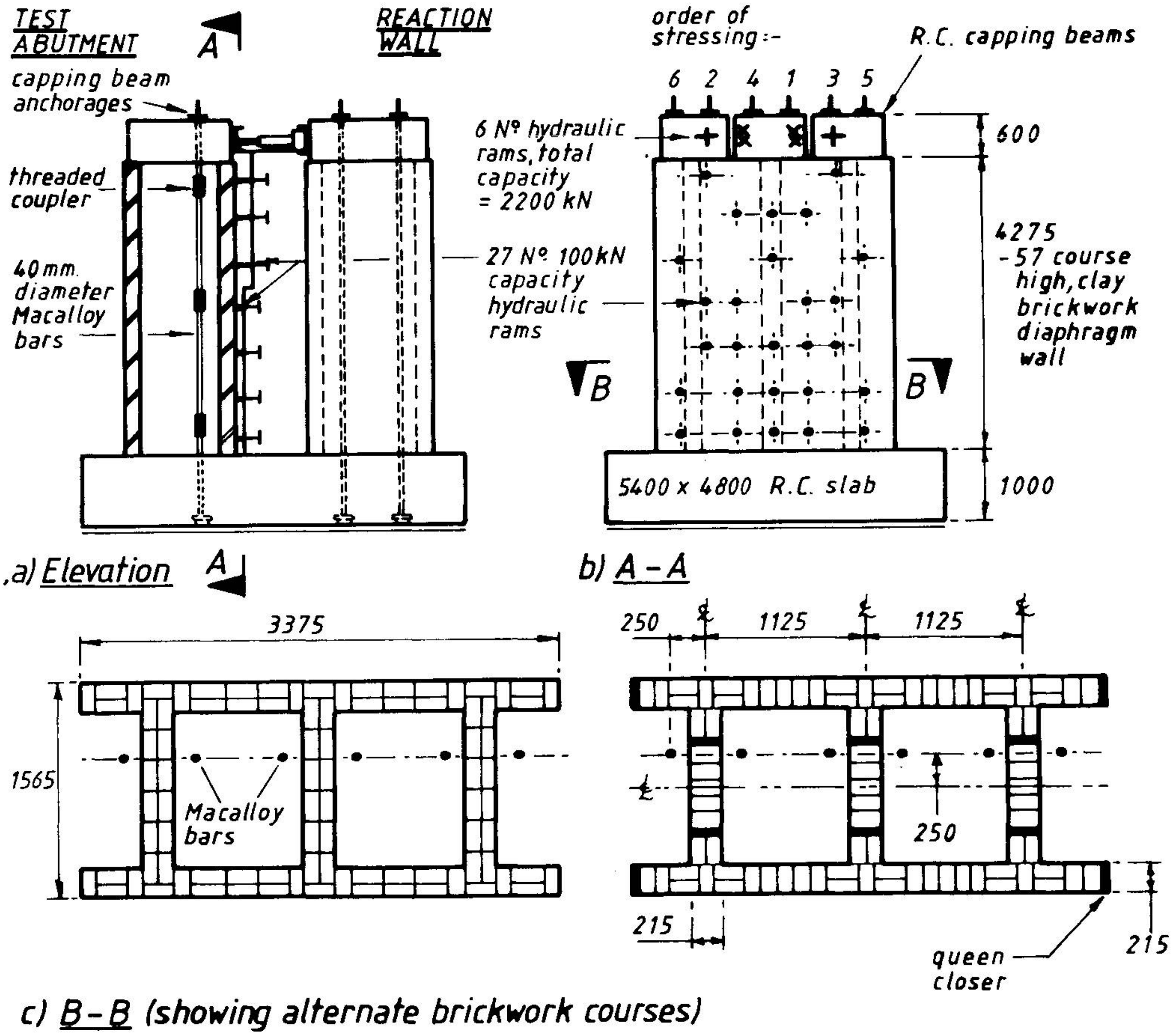


Fig. 2 Details of test arrangement (All dimensions are in millimetres)

2.2 Test loading

Most bridge abutments are designed to withstand the combination of forces shown in Figure 1a. However, force P1 was not simulated in the test as it would have a small beneficial effect. Forces P2 and P3 were combined and applied using six large capacity hydraulic rams. Twenty seven smaller rams, each with a load capacity of 100kN and connected to a single electrically controlled pump, were used to provide force P4. These rams were arranged in seven levels, as shown in Figure 2, to produce the trapezoidal distribution of earth pressure loading that would be applied to the full 6.6m height of the abutment.

Table 1 gives the loads that were applied by the hydraulic rams at certain stages of the test. From load stages 1 to 8, the earth pressure forces were increased by equal amounts with no longitudinal load applied. At load stage 8, the total earth pressure load was 1015kN; this was approximately twice the lateral earth force on the abutment shown in Figure 1, calculated on the basis of active earth pressure and assuming a soil density of $18kN/m^3$, an earth pressure coefficient of 0.33 and a surcharge of $10kN/m^2$. From load stages 9 to 18, the simulated earth force was kept approximately constant and the longitudinal load was increased in increments of 75kN.

Load stage	Total earth pressure load [P3+P4] (kN)	Longitudinal load [P2] (kN)	Shear force (kN)	Bending moment (kNm)
1	139	0	139	309
4	501	0	501	1139
8	1015	0	1015	2314
9	1027	75	1102	2648
10	966	150	1146	2886
11	985	225	1210	3200
12	997	300	1297	3647
15	1056	525	1581	4791
18	1075	750	1825	5924

Note: Decompression bending moment = 3665kNm

Table 1 Details of significant load stages

3. OBSERVATIONS DURING TESTING

At load stage 11, small vertical cracks appeared in the webs at the foot of the abutment. This cracking was probably caused by the hogging curvature of the base which produced large horizontal tensile strains in the top surface of the concrete; these strains were transmitted through the bottom bed joint to the brickwork. The cracks did not develop significantly with increased loading. At the next load stage, cracks became visible in the concrete base, the position of some of the cracks coinciding with the aforementioned vertical cracks in the brickwork webs. At load stage 15, horizontal cracking developed in the bottom bed joint of the rear flange of the abutment. Decompression had occurred, i.e. the prestress had been annulled, and furthermore, the flexural strength of the brickwork had been reached. At the final load stage, the horizontal crack at the foot of the rear flange had opened to a width of approximately 5mm. Additionally, the crack had propagated along the web bed joint to within approximately 550mm of the front face of the abutment. The abutment was, in effect, rotating bodily about the foot of the front flange. However, at this stage there was no indication that either shear or flexural failure was imminent.

It is interesting to note that there was some diagonal cracking in the base resulting from the vertical shearing action caused by the downward line load from the front flange and the upward forces from the prestressing bars.

On removal of the load, the abutment returned to its original position and all the cracks closed up. The development of cracks in the brickwork abutment and reinforced concrete base is summarised in Figure 3.

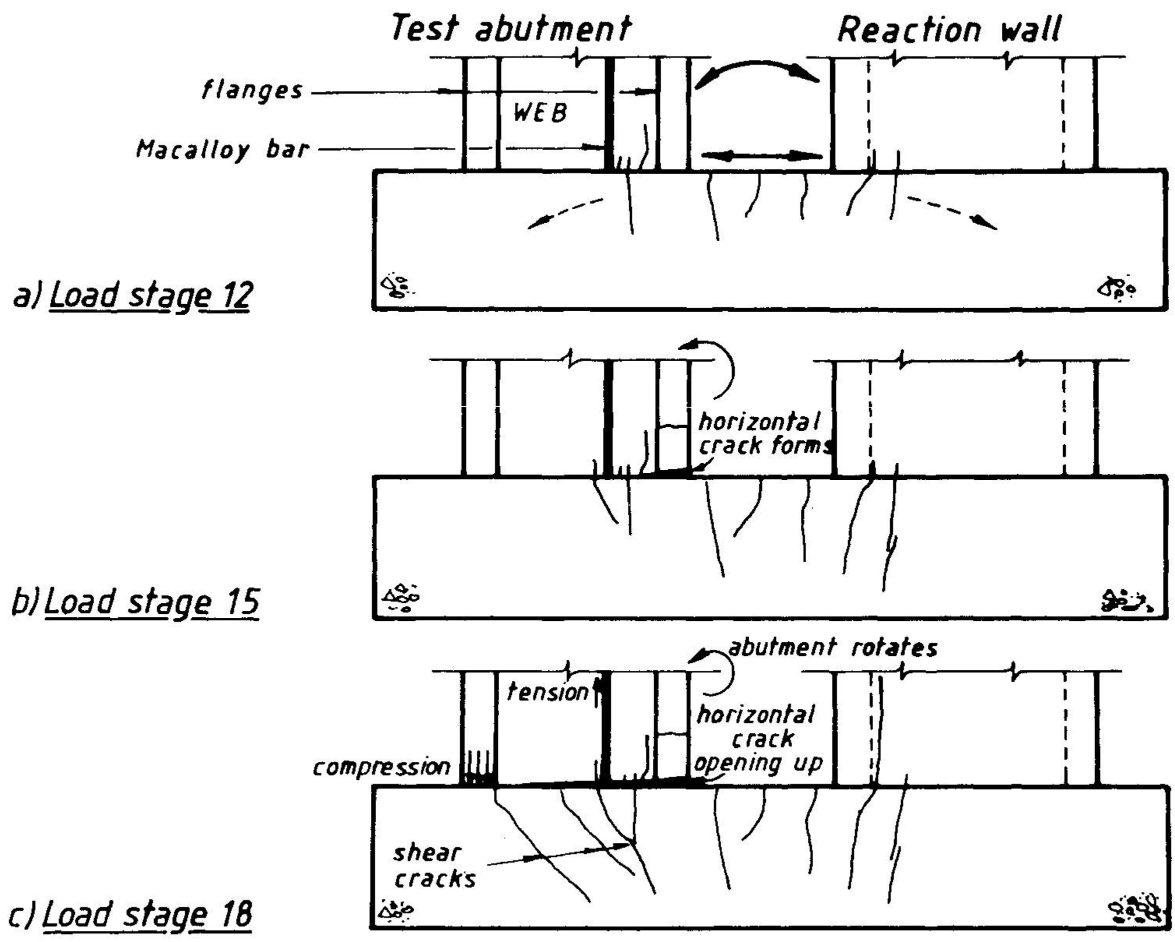


Fig. 3 Development of cracks during testing

4. ASSESSMENT OF STRUCTURAL PERFORMANCE

4.1 Design service condition

Using the recommendations of BS 5400 [6] for the bridge shown in Figure 1a, the total longitudinal load applied over a single notional carriageway width is 408kN. Assuming that this force is uniformly distributed over the full width of the abutment, the longitudinal load appropriate for the 3.375m wide test section would be 100kN. Related to an abutment of 6.6m total height, this would produce a bending moment of 660kNm at foundation level.

Taking the at-rest pressure coefficient to be 0.6, the total earth pressure force acting on the test abutment, caused by drained backfill of weight $18kN/m^3$ and a surcharge of $10kN/m^2$, would be 928kN. The corresponding bending moment is 2188kNm.

Combining the effects of the longitudinal load and the earth pressure forces means that, under service conditions, the test abutment would be subjected to a maximum shear force of 1028kN and a maximum bending moment of 2848kNm. As can be seen from Table 1, these values are very close to the shear force and bending moment resisted by the test abutment at load stage 10, when no cracking had occurred. However, although minor cracking was noted at load stage 11, it was not until after load stage 14, when the shear force and bending moment were 1504kN and 4437kNm respectively, that horizontal flexural cracks were observed.

4.2 Design ultimate condition

Using an effective partial safety factor of 1.375 for the longitudinal load and 1.65 for the earth pressure forces, the design ultimate shear force for the abutment would be 1669kN and the design ultimate bending moment would be 4518kNm. However, in the test, the abutment resisted a shear force of 1825kN and a bending moment of 5924kNm without failure occurring. Hence, although the actual strength of the abutment was not determined experimentally, it has been demonstrated that the abutment was strong enough to resist the design ultimate shear force and bending moment.

5. CONCLUSIONS

- Over the five month period between prestressing and testing, the loss of prestress in the abutment was approximately 5%.
- Under simulated BS5400 design service loading, there was no cracking in the brickwork.
- The abutment was able to resist bending moments and shear forces greater than those produced by the BS5400 design ultimate loads and there was no indication that either shear or flexural failure was imminent.

6. ACKNOWLEDGEMENTS

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