Load test to collapse on a full scale model six metre span brick arch bridge

by C Melbourne and P J Walker
(Bolton Institute of Higher Education)

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Bridges Division
Structures Group
Transport and Road Research Laboratory
Old Wokingham Road
Crowthorne, Berkshire RG11 6AU

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LOAD TEST TO COLLAPSE ON A FULL SCALE MODEL SIX METRE SPAN BRICK ARCH BRIDGE

1 INTRODUCTION

The test described is one of a series which are currently being undertaken in the UK to study the behaviour of masonry arch bridges. The main aim of the research is to improve the method of assessment as present methods (1, 2) are generally considered overly conservative or inappropriate.

The TRRL programme of research was to comprise model tests and theoretical analyses in addition to a series of full-scale tests. This report relates to the sixth of a planned series of about ten full-scale tests (3). The brickwork bridge had a span of 6 metres and a rise of 1 metre.

Earlier tests had shown the relative importance of the soil-structure interaction and stiffening effects of the spandrel walls. A series of elastic 'point' loadings were undertaken to simulate wheel loadings. Vibrating wire strain gauges were attached to the ring in an attempt to determine the position of the thrust line. Additionally embedment strain gauges were installed to monitor ring separation.

The construction and elastic tests were undertaken during the summer of 1988 culminating in the testing to failure on 16 September 1988. The work was carried out at the Boiton Institute of Higher Education in their large scale testing facility.

2 DESCRIPTION OF TEST BRIDGE

2.1 Constructional details

Details of the completed bridge are given in figure 2.1

The segmental arch barrel (radius of circle 5m) consisted of two rings of brickwork across a 6m width. The barrel had an internal span of 6m with a rise of 1m.

The bridge was constructed and loaded on a purpose built reinforced concrete testing slab, measuring 14m x 8m. Reinforced concrete abutments provided a span of 6m, the clear height from the base of the slab to the springing was 500mm. Loading to failure was achieved through 'CCL' prestressing tendon anchorages cast into the slab along the quarter span. Additionally, 'macalloy' bar and coupler type anchorages were provided along the span of the bridge in order to apply point loads across the span and width of the bridge.

The centring for the bridge construction was provided using a 'RMD' arch centring system. Timber was used to form the profile required for the arch. A release agent was applied to the timber prior to bricklaying to avoid bonding of the bricks to the centring. Bricklaying was carried out over a five week period in June and July 1988.
The brickwork was built in a 'stretcher' bond with no bonding between rings, other than through the mortar bed-joint. The total thickness of the completed arch barrel was 220mm. To accommodate 25 pressure cells into the arch barrel, cavities were formed in the brickwork bonding and the pressure cells were later embedded in mortar. Additionally three ducts were built into the barrel to allow the loading tendons to pass through the bridge at the quarter span. The three ducts were placed at the mid-width and at 2m either side.

The spandrel, parapet and wing walls were built in an 'English' bond, cross-sections shown in figure 2.1. Brickwork retaining walls were provided at each end of the bridge to retain the backfill. The vertical cross-section of the walls was the same as the wing walls. The width of the retaining walls, between the wing walls, was 4.46m. The vertical faces between the retaining and wing walls were de-bonded by including a sheet of PVC between the mortar joint and brickwork. Cavities were built into the west spandrel wall to accommodate pressure cells.

The properties of the backfill are given in the Appendix. Filling was achieved using a 10 tonne overhead crane facility and a concrete discharging skip. Compaction of the backfill was carried out in 100mm layers using a vibrating compacting 'wacker' plate. The bridge was filled to 200mm above the crown. After filling was complete a 100mm thick road surface was provided by subcontractors. Once the construction process was complete the centring was removed. Throughout the filling, road surfacing and centring removal operations the backfill pressures, brickwork strains and deflections were monitored.

3 TEST ARRANGEMENTS AND INSTRUMENTATION

The details of the loading required and the instrumentation used to monitor the structural response are described below.

3.1 Loading arrangements

The bridge was subjected to two separate load tests. A series of 'point' load tests was conducted, followed by the application of a 'knife edge load' (KEL) at the quarter span point through to collapse of the bridge.

3.1.1 Point load tests

A series of 100 kN point load tests were to be applied across the span and width of the bridge to simulate wheel loading. However, during application of the first load test at position D7 (figure 3.1) the bridge cracked along the ring/spandrel wall interface and ring separation was also recorded. As a result the maximum loading was revised to 50 kN. The loads were individually applied using hydraulic jacks through a steel reaction beam and applied to the road surface through a circular 340mm diameter mild steel plate. Loading was applied and released incrementally and the response of the bridge was recorded. The results are discussed in Section 4.

3.1.2 Load test to failure

Loading (KEL) was applied, at the quarter span across the full width of the bridge, incrementally to failure through a 750mm wide reinforced concrete beam. The load was provided using three 1000 kN, 600mm stroke, hollow cylinder hydraulic jacks and applied using the prestressing tendons anchored in the test slab and
3.2 Instrumentation

3.2.1 Deflection

A total of fifty-six linear displacement transducers were positioned around the bridge to monitor the movement of the arch barrel, spandrel walls, retaining walls, wing walls and road surface. The layout is shown in figures 3.2 - 3.4. The directions of the arrows indicate a positive deflection on the graphs.

3.2.2 Backfill pressure

The backfill pressures around the extrados and spandrel wall interfaces were measured using thirty "Gage Technique" BSP type vibrating wire earth pressure cells incorporated into the bridge. Plate 3.1. The position of each gauge is shown in figures 3.5 -3.7.

3.2.3 Brickwork strains

Internal strains of the brickwork were measured using electrical resistance type strain gauges. The surface brickwork strains were measured using vibrating wire type gauges. The position of each type of gauge is shown in figures 3.8 - 3.12.

3.2.3.1 Embedment strain gauges

To monitor the development of ring separation and ring/spandrel wall separation twenty electrical resistance type embedment strain gauges were placed in the mortar joints between the two rings of brickwork.

3.2.3.2 Vibrating wire strain gauges

'Gage Technique' surface mounted vibrating wire strain gauges (with a gauge length of 140mm) were used to measure the brickwork surface strains around the arch barrel and across the ring/spandrel interface.

3.2.4 Temperature

A total of twenty 'integrated circuits' were used to measure the temperature both inside the backfill and on the external surface of the brickwork. They were used to provide temperature compensation of the other instrumentation.

3.2.5 Load

The load applied to the bridge was measured using two 'RDP' 1000 kN electrical resistance type load cells.

3.2.6 Data loggers

Three data loggers were used to monitor the instrumentation: two Solartron Orions and a Soil Instruments Geoscan.

3.2.7 Visual observation

Cracking of the brickwork was observed and reported throughout each of the tests.
To aid crack detection the arch soffit and the east spandrel wall were painted white making it possible to observe cracks 0.1mm wide with the naked eye. The west spandrel wall was left unpainted.

4 POINT LOAD TEST RESULTS

4.1 Introduction

A series of point load tests was carried out, the positions of load application are shown in figure 3.1. Initially, the tests were planned for incremental application of load up to 100 kN, as stipulated in the contract. However, during the first load test at position D7, structural damage occurred at 80 kN and so the loading was reduced to 50 kN for the subsequent positions.

Referring to figure 3.1, the order of loading was:
D7, D3, D4, D5, DSA, C5A, C5, C7, C1, B1, B7, B5A, B5, H5A, H5, H7, H1, A1, A7, A5, A5A, F5A, F5, F7, F1, E1, E7, E5 and E5A.

The time for one load cycle was approximately 1 hour.

4.2 Results

4.2.1 Brickwork surface strain

In figures 4.1 the typical results for surface brickwork strains recorded under point loading are presented. The layout of the strain gauges on the structure is given in figure 3.11.

During load application at the first position, D7, cracking around the west spandrel wall/arch barrel joint was observed at 80 kN at the quarter point adjacent to the load. The crack extended equi-distant either side of the quarter point for a distance of approximately 500mm. The maximum crack width was estimated to be 0.2mm. In figure 4.1 the strains measured on the arch barrel during the application of the load, position D7, are presented. The results are presented for gauges G46 and G48, no other strain gauges demonstrated any significant change in the level of strain. Both graphs illustrate an initial linear 'elastic' increase in the brickwork strain. Compressive strains were measured at the intrados at the north abutment, gauge G46, and tensile strains were recorded on the intrados at the quarter point (adjacent to the load), gauge G48. The orientation of the compressive and tensile strains measured was as expected, i.e. further loading would suggest the formation of hinges at the north abutment intrados and at the extrados at the quarter span. Figure 4.1 indicates that from the outset of loading tensile strains were induced at the quarter point intrados. As loading increased a change in slope of the graphs occurred at approximately 70 kN. This seems likely to have been due to the spandrel wall/arch barrel separation although cracking was not observed until loading reached 60 kN. Beyond 70 kN a rapid increase in the brickwork strains occurred, however the responses remained 'linear' and so the barrel may be considered to have remained 'elastic'. The greater increases in strain after cracking was due to the reduction in structural stiffness resulting from the wall/barrel separation. Increases in the tensile strain up to 100 kN at gauge G48, figure 4.1, indicates a shift of the thrust line further outside the middle third of the barrel. On reaching 100 kN the load was released and residual strains of -27 microstrain (compression) and 41 microstrain (tension) were recorded by gauges G46 and G48 respectively. At a load of 100 kN
the crack opening was estimated to be 0.3 - 0.4mm, after removal of the load the crack closed and was no longer visible to the naked eye.

After load position D7 the maximum point load to be applied was reduced to 50 kN. To monitor changes in strain across the wall/barrel separation a further vibrating wire strain gauge was placed perpendicularly across the crack at the quarter point, gauge G71 (figure 3.11). All subsequent brickwork surface strain measurements corresponded to across the wall/barrel crack, some of the results are given in figures 4.2 and 4.3. None of the other vibrating wire strain gauges demonstrated any significant change in the level of strain throughout the remaining point load tests.

The experimental load/strain relationships indicate a non-linear increase in strain; the rate of change in strain increasing with load. For any particular load the strains were greatest for a loading position nearest the west edge of the bridge, this was due to the greater moments induced when loading was nearer to the edge of the arch barrel.

In figures 4.2 and 4.3 the strains measured at gauge G71 are reported for loading positions C, B, A, F and E (no deformation occurred for loading at position H). With the exception of position A1 the strain changes were only recorded when the load was at positions 5 and 7. Other than when loading was at the crown the experimental relationships showed an increase in strain with load; the rate of increase in strain increasing with load. On load removal residual strains were recorded during decreasing load. The level of strain was greatest when the loading was applied nearest to the west spandrel wall. The significance of the load position across the width in respect to the spandrel wall was also dependent upon load position in relation to the span. For example, at load positions C, the strains for C7 at 50 kN were 1100% greater than when loading was at C5, a distance of only 530mm nearer to the spandrel wall. However, a similar movement of the load nearer the west wall caused an increase in strain of only 150%, movement from position F5 to F7.

When the point load was applied at the crown, the crack at the greater span was recorded as closing (reduction in strain). This was due to the typical action of a live load applied at the crown of a symmetrical arch. However, the level of strain induced in the ring at other parts of the barrel was insufficient to illustrate a strain reversal throughout the arch.

In figures 4.2 and 4.3 the strain responses are compared as a function of the load moving across the span of the bridge. Both graphs clearly illustrate the significance of the load position on strain development. The graph for the loading position nearest to the spandrel wall, figure 4.2, shows the greatest deformation to have occurred when the load was nearest the cracked section, position C7. This is to be expected since the largest moment will be produced at the cracked section when the load is at that section. The effect of a 50 kN load moving across the span is demonstrated as an 'influence line diagram' in figures 4.2 and 4.3.

The results for the deformation at the quarter span point have also been presented in terms of an increase of the crack width. Predominantly, throughout the loading the crack widths induced by the point load were significantly less than 0.1mm. However, cracking of this magnitude may be considered unacceptable for a limit state in masonry arches and subject to repeated loading may lead to fatigue problems for the brickwork.
4.2.2 Embedment strains

During application of the point load at position D7 spandrel wall/arch barrel separation occurred adjacent to the load point at a loading of approximately 70 kN. The cracking of the brickwork should have been detected by the embedment strain gauge OR2/125 (figure 3.8) but due to an apparent gauge malfunction the cracking was not detected until a load of 80 kN.

Gauge OR2/105, placed at the quarter span of the barrel to monitor development of ring separation (figure 3.8), illustrated a large increase in tensile strain during load application at position D7. The compressive and flexural brickwork tests suggested that a strain of approximately 75 microstrain corresponded to tensile cracking (ring separation). In figure 4.4 the tensile strain response at OR2/105 for load stage D7 is illustrated. From zero to a load of 40 kN there was little change in the level of strain, however, by 70 kN a tensile strain sufficient to cause ring separation had been measured. At 100 kN a tensile strain of 2250 microstrain was recorded, showing that a significant deformation had been caused to the arch barrel. After removal of the load a residual strain of 850 microstrain was recorded, the arch barrel was, therefore, subject to a significant permanent defect. Ring separation was not reported at any other area of the arch barrel during loading at D7.

Subsequent 50 kN loads positioned elsewhere on the bridge road surface caused the ring separation cracking to develop further. The results for ring separation at gauge OR2/105 are presented in figure 4.4. Loading placed at positions B, C, D, E and F caused the tensile ring separation strain to increase with load, the experimental load/strain responses were similar in format for all loading positions. The rate of increase of strain increased with loading up to a maximum value at 50 kN. On load removal significant residual strains were present, leading to a 'hysteresis' type envelope. Generally the ring separation strains were increased only when the loading was at positions 7 and 5. The behaviour was similar to the surface cracking of the bridge, section 4.2.1, in that the magnitude of the strains induced were subject to the loading position. The largest strains were recorded when the load was nearest to the deformation, at the edge of the ring and adjacent to the quarter span.

The influence of the load position across the width is clearly illustrated in figure 4.2. A movement of only 630mm (from D3 to D5) caused the strain to increase by over 200%, from 140 to 300 microstrain. The increase in ring separation strain at the quarter span was zero when the loading was at the abutment and at the crown, the deformation reached a maximum of over 300 microstrain when loaded at C7.

Further loading, nearer to the crown, caused ring separation to develop at the crown, gauge OR2/109, during load stage B5. The greatest strains were induced by loading at the crown, for load position A7 a strain in excess of 1100 microstrain was measured.

4.2.3 Backfill pressures

The backfill pressure measurements due to the point loads display a linear increase with increasing load. In all cases the pressures measured relate to cells in the immediate vicinity of the applied load. The pressure response was therefore local as no pressure measurements were recorded in the 'un-loaded' side of the bridge. As might be expected the backfill response was not truly elastic.
due to compaction by the point load. Although the "Wacker" plate applied a vibration compaction, this was not equivalent to a point load of 50 to 80kN. Hence further local compaction took place resulting in a residual increase in backfill pressure. Current soil mechanics theory suggests that pressures in excess of "at rest" pressures, \( K_0 \) are possible when dealing with shallow depths of fill. A figure closer to pressures on unloading may be more applicable

\[ K_1 = \frac{1}{K_0} \]

For pressure cells at the crown, depth of material 300mm, assuming no distribution of the load shows very good agreement with the measured pressures. However, for the pressures measured away from the crown a better correlation was shown if a 45° distribution of the load was assumed.

4.2.4 Deflection

Throughout the point load tests a comprehensive series of deflection measurements was taken. However, no deflection response was measured during any of the loading cycles. The resolution of the transducers along the spandrel walls and arch barrel edge was 0.1mm, whereas, along the intrados centre line the accuracy was 0.02mm. The crack widths recorded along the edge were, in general, significantly less than 0.1mm and hence no deflection was recorded adjacent to the cracking. As the deflections of the arch were very small, it seems unlikely that deflection will provide an adequate serviceability limit state criteria for masonry arches. The resolution of the transducers indicated that the deflection throughout the point load tests was less than \((1/60000) \times \text{span}\).

4.3 Summary

(i) A point load of only 70 kN was sufficient to cause separation of the spandrel wall/arch barrel leading to ring separation in the arch barrel. Subsequent load positions caused the cracking to open with increasing load and close with decreasing load.

(ii) During each load cycle increases in the backfill pressure were measured only in a position immediately underneath the load. The load/backfill pressure response was linear and the pressure was shown to be directly related to the load.

(iii) No deflection response of the bridge was recorded throughout the point load tests.

5 LOAD TEST TO FAILURE

5.1 Introduction

The load was applied through a steel loading beam via a system of three hydraulic jacks (Figure 2.1). Due to the defect on the west side of the bridge caused during the point load tests, it was necessary to ensure a balancing of the load across the full width of the bridge. The level of load was maintained constant throughout each load increment.

The load was initially applied in increments of 10kN, increased to 20kN on reaching 280kN, and increased again to 60kN on reaching 530kN. Including the dead
weight due to the loading beam and hydraulic jacks a total of 26 increments were
applied, taking approximately 6.5 hours; an average loading rate of 180 kN/hour.

The progressive development of cracking on both sides of the bridge is shown in
figures 5.1 - 5.4. Prior to loading, cracking along the spandrel/barrel interface
was present on the west side. On loading the cracking defect on the west side was
observed to open and increase further around the arc of the barrel. Separation
of the spandrel/barrel interface commenced on the east side at 360 kN, figure 5.2.
Further loading caused the cracking to spread around the arch barrel. At 400 kN
the first hinge in the barrel was observed underneath the load line. At 640 kN
vertical and shear cracking of the spandrel walls was observed.

Ring separation on both sides, between the north abutment and crown, was also
noted. At a loading of 820 kN vertical cracking of the parapet/spandrel wall
was observed, this was due to the lifting of the arch barrel crown under
def ormation.

Failure of the bridge was due to the formation of a four hinge mechanism at a
total applied loading of 1173 kN. Prior to failure, the only hinge which was
visibly developed was underneath the load point, failure was very sudden. On
formation of the four hinge mechanism, figures 5.3 and 5.4, the spandrel/
wing/parapet walls were lifted and rotated as shown. Horizontal splitting of
the wing walls occurred along the bed-joint level with the abutments. Hinges
formed at each abutment, underneath the load and at the crown. Extensive ring
separation was coincidental with failure and the two brickwork rings were almost
completely separated around the full arc of the barrel. It was clear from the
formation of the hinges that two separate thrust lines existed, with two separate
compression zones, one in each ring. Immediately upon formation of a mechanism
the load reduced to 700 kN. After a full set of readings were taken the
instrumentation was removed and the bridge was slowly loaded through to collapse.
It is worth noting that the failure mode of the full-scale bridge was very similar
to that reported for 1.5m span model bridges tested at Bolton Institute (4).

5.2 Deflection

Horizontal and vertical deflections of the arch barrel adjacent to the abutments,
quarter span points and crown were measured at the west wall, east wall and along
the centre-line of the intrados (figure 3.2).

Throughout loading to failure no movement, either horizontal, or vertical, was
recorded at either of the abutments.

The deflection responses of the arch barrel at the quarter point underneath the
load are presented in figures 5.5 and 5.6. Both the horizontal and vertical
deflection responses show a similar format. An initial linear 'elastic' stage
until cracking and hinge formation occurred may be clearly observed. This was
followed by rapid increases of deflection with load leading to a levelling off
of the load/deflection relationships up to ultimate. The deformations recorded
after achieving ultimate load are also shown in the graphs. Horizontal
deflections at the barrel edges were in close agreement, however, horizontal
movement at the centre-line was noticeably less. Downward deflections at the
quarter point were in very close agreement across the full width of the bridge.
Formation of the first hinge was observed underneath the load-point at 400 kN, the
vertical load/deflection response indicated an increase in the deflection
coinciding with the hinge formation. Throughout loading the vertical (downward)
movement exceeded the horizontal movement by a ratio of approximately 2:1. At ultimate load the total downward movement of the quarter point was estimated at 8mm, or \((1/750) \times \text{span}\).

The horizontal and vertical (hogging) deflections at the crown are given in figures 5.7 and 5.8. The slope was similar to those at the quarter span and typical of load/deflection relationships for a masonry arch. In the horizontal direction the deflection response was linear to 700 kN whereupon a rapid increase in deformation was observed, this coincided with separation of the spandrel wall/arch barrel at the crown and vertical cracking of the spandrel. Deflection of the crown upwards shows three different curves across the width of the arch, figure 5.8. This may have been caused by ring separation, present on the west side (OR2/61), permitting greater movement at that side. The ratio of horizontal to vertical movement was approximately one beyond hinge formation. The total upward movement of the crown at ultimate load was 2.5mm, \((1/2400) \times \text{span}\).

The horizontal and vertical (hogging) deflections at the quarter span opposite the load point are given in figures 5.9 and 5.10. Prior to 800 kN little movement was recorded, however subsequently there was a rapid increase in deflection corresponding to the extensive cracking of the arch and hinge formation. The ratio of vertical to horizontal movement was approximately 2:1, total upward movement was approximately 4.5mm at ultimate, \((1/1333 \times \text{span})\). A good indication of the overall deformation of the centre line of the arch at the load increment prior to ultimate is given in figure 5.11.

The outward movements of the spandrel walls were measured around the arc of the bridge, both near to the wall/barrel joint and at the top of the parapet walls. The spandrel walls moved under loading; this effect at the loaded quarter point may be ascribed to deviatoric stresses induced in the backfill. At ultimate the walls had moved out by approximately 1-2mm. Outward movement of the walls at the opposite quarter point may be explained by dilation of the backfill as it was pushed against by the deforming arch barrel. The magnitude of deformation was significantly less than near the loaded area, only 0.25mm at ultimate.

Above 700-800kN the south retaining wall rotated and translated in response to the induced soil pressures. Additionally the spandrel walls rotated and translated in response to the interaction not only with the arch barrel but also with the backfill frictional stresses.

Vertical deflection of the road surface was monitored throughout loading, figure 3.3. The results indicate a total downward movement at the load point of 10mm at ultimate, figure 5.12. However, taking into account the movement of the arch structure beneath, the net deflection is much reduced and indicate the fill to have been compacted by only 4mm and thus had probably not failed in bearing.

5.3 Surface brickwork strain

Surface mounted vibrating wire strain gauges were positioned on the bridge to monitor three aspects of strain behaviour of the structure during loading, namely:

(i) Position of the thrust-line
(ii) Lateral separation of the spandrel wall from the arch barrel
(iii) Development of existing cracks.
The layout of the strain gauges is given in figures 3.10-3.12. However the following malfunctioned during testing: G35, G37, G39, G41, G43, G44, G50 and G59.

5.3.1 Development of thrust line

Ten strain gauges were placed around the arc of the barrel along its west face, figure 3.11, and a further sixteen gauges were positioned around the intrados and extrados of the arch.

Gauges G45 and G46, placed across the springing, show an initial linear increase in compressive strain until approximately 600 kN, coincidental with extensive cracking of the bridge. Beyond 600 kN a rapid increase in strain occurred at the intrados. Throughout testing the thrust-line would seem to have been inside the middle third at this point as tensile strains were not measured. Once the applied load had exceeded 400 kN large tensile strains were measured on the extrados of the barrel, three courses above the north abutment (gauge G36). A rapid increase in strain occurred after development of the first hinge and the graph levelled off approaching failure indicating the likelihood of a hinge at this position. This was confirmed by visual inspection.

Under the load point there was a steady increase in tensile strain with load at the intrados. At a loading of 400 kN the first hinge was observed at this point, the rapid increase in tensile strain (G48) supported that observation. A tensile strain of 6000 microstrain at 1100 kN was over 100 times that necessary for cracking. Gauge G48 which illustrated a development of a very large tensile strain at the quarter point intrados was positioned across a hinge. It seems gauges G56 and G63 were placed across joints adjacent to the hinge on the intrados and hence they show little change in strain, suggesting a stress relief in the brickwork due to the hinge.

At the crown a steady increase in tensile strain at the extrados face of the arch, gauges G49 and G40, show that the thrust-line was outside the middle third. The magnitude of the strain approaching failure, greater than 2000 microstrain, also shows the development of a hinge at the crown. This was confirmed by observation. Likewise the compressive strain development at gauge G61 on the intrados confirms that a hinge was forming at the crown. Compressive strains greater than 2000 microstrain would suggest that the thrust line was very close to the outer face of the intrados.

At the southern quarter point, the compressive strains at both gauges G51 and G52 indicate that the cross-section was fully compressive throughout and therefore the thrust-line remained within the middle third. However, the strain gradient derived from measurements at the intrados and extrados, within the width of the bridge developed a tensile strain at the intrados indicating that the thrust-line was outside the middle third. Although the tensile strains (greater than 100 microstrain) were sufficient to crack the brickwork, the magnitude of strain compared to other sections would suggest that a hinge did not form at this point.

The south springing section remained in compression throughout loading. However, tensile strains were measured on the extrados of the arch barrel at gauge G44, a section three courses above the springing where a hinge was observed to form. The position of hinges and strain development was therefore similar to that at the north abutment.
The experimental strain measurements were used to determine the position of the thrust-line. Throughout loading the thrust-line under the KEL was close to the extrados. The effects of ring separation recorded at 300-400kN can be detected by a further shift of the thrust-line position. The assumption of "plane sections remaining plane" can no longer be considered valid when ring separation has occurred.

Also of interest is the experimental thrust-line positions for the south quarter span, figure 5.13: one represents that at the edge next to the spandrel wall and the other is for a section towards the centre of the arch barrel. A considerable difference in the location of the thrust-line is clearly apparent, the stiffness of the spandrel wall has, for most of the loading, ensured the thrust-line to remain in the middle third. However, away from the spandrel wall in the much less stiff section the thrust has remained near to the intrados throughout. if not for the stiffening effect of the spandrel walls a hinge would have formed across this section.

5.3.2 Lateral separation of spandrel wall

In a number of existing masonry arches the spandrel walls have become detached from the arch barrel. The separation of the spandrel wall has manifested itself in a number of bridges by a longitudinal crack on the intrados between the springings in line with the thickness of the spandrel wall. To monitor any possible development of such a crack ten gauges were placed around the intrados of the arch, underneath the line of the extent of the spandrel walls, figure 3.12.

Tensile strains were observed indicating that the backfill pressure was increasing and trying to push the spandrel wall off the arch. Under the load the barrel was spanning as a fixed ended beam. High transverse horizontal stresses were also being applied to the spandrel wall through the fill by the KEL. The combination of these effects was to cause the intrados locally to become compressive.

5.3.3 Development of ring-wall separation

During the point load tests spandrel wall/arch barrel separation was observed at the loaded quarter point. To monitor further development of this crack gauges G71-G73 (figure 3.11) were placed perpendicularly across the crack around the arch.

The strain gauge placed across the initial crack at the loaded quarter point, gauge G71, demonstrated a steadily increasing tensile strain across the crack as the load was applied, reaching a value of 10 000 microstrain (equivalent crack width 1.3mm) at only 800-850 kN. Beyond this loading the strain capacity of the gauge had been exceeded. Further loading caused the crack to open wider, reaching approximately 5-10mm at ultimate load and between 50-75mm (observed) just before collapse.

At the remaining two locations, crown and unloaded quarter point, compressive strains were measured across the barrel/spandrel wall interface. The measurements confirmed that the arch barrel in this region was pushing against the spandrel wall leading to rotation of the brickwork walls.
5.4 Brickwork strain: embedment

Twenty electrical resistance strain gauges were placed in the arch barrel to monitor:

(i) ring separation
(ii) spandrel wall/barrel separation.

The layout of the gauges is given in figure 3.8.

5.4.1 Ring separation

In the course of point load tests ring separation was detected at the loaded quarter point (OR2/105) and later also at the crown (OR2/109), both on the west side of the barrel. During the load test to failure it was of interest to consider the development of existing ring separation and also to monitor the progression of further cracking.

Initially the ring separation was present only at gauge OR2/105, however, by a loading of 300kN ring separation was detected at gauge OR2/107. Ring separation therefore occurred across the width of the barrel at the north quarter point. Up to approximately 400 kN the curves followed very similar paths. Beyond 400 kN the load/strain relationships diverged. This was probably caused by the re-distribution of stresses due to hinge formation at this point. The gauges continued to monitor strains up to 2500 microstrain. Visible ring separation was observed at the edges of the barrel, with crack widths of about 10mm at failure. The ring separation at the crown, present before loading to failure, continued to develop with loading to failure. A discontinuity in the experimental relationship at 800 kN may have been caused by hinge formation. Unlike ring separation development at the quarter point the ring separation did not develop across the full width of the arch ring until after ultimate load.

Prior to loading no ring separation was present in the ring adjacent to either abutment. However it occurred at the north abutment at 400kN, corresponding to hinging at the quarter point and the inherent gross deformations. Similarly ring separation was reported across the width of the load at the south abutment from a loading of 600kN. No ring separation was reported at the unloaded quarter point until after reaching the maximum load.

5.4.2 Spandrel wall: barrel separation

Vibrating wire strain gauges were installed across the arch barrel/spandrel wall interface. Separation was recorded beginning at 400kN and eventually extended around the entire span except at the south quarter span where, as might be expected, compressive strains occurred as the arch barrel pushed into the spandrel wall.

5.5 Backfill pressures

A total of thirty-four earth pressure cells were incorporated into the bridge. Unfortunately, seven of the gauges malfunctioned during the test. Figures 5.14 - 5.22 shows the load/pressure response for the main locations on the extrados. In each case the initial response appeared to be linear. This is in keeping with the elastic deformation of brickwork. By the formation of the first hinge this had ceased to be the case.

At the north abutment, pressure increases are directly attributable to the KEL
and the confined situation of the backfill.

The pressure cells beneath the KEL at the quarterspan showed a good agreement with a 45° distribution. Pressures on the crown changed little during the test.

At the south quarter point the pressure increased steadily with load reaching a maximum of 30kN/m² which represents approximately 60% of full passive pressure.

At the south abutment pressures decreased with negative pressures being recorded in the latter part of the test. This was probably caused by arching of the fill over the gauges as the arch barrel hinged just above them and additionally the backfill displayed some adhesiveness.

Pressures on the spandrel walls followed the expected pattern. Adjacent to the KEL, a progressively increasing pressure was recorded peaking just prior to the spandrel wall cracking. Additionally, the pressures adjacent to the south abutment increased steadily until hinges formed, at which stage significant increases were observed. These changes were commensurate with the stresses induced by the longitudinal pressures generated during hinge formation.

5.6 Summary

(i) The bridge failed due to the formation of a four hinge mechanism at an ultimate load of 1173kN. The spandrel walls provided a significant restraint to the arch barrel. The failure mode was identical to that observed in small scale model tests.

(ii) The load/deflection response for the arch barrel was initially linear until hinging of the barrel. This was followed by a rapid increase in deflection with load up to eventual failure.

(iii) Passive pressure was not recorded, however, the backfill did provide a significant lateral restraint to the arch.

(iv) The experimental thrust-line was successfully determined using measured brickwork surface strains and material properties for the brickwork.

(v) The surface strain measurements confirmed that the thrust was confined to the middle third at the three quarter span by the stiffness of the spandrel walls. Away from the influence of the walls the thrust-line was outside the middle third.

6 ANALYSIS OF STRUCTURE

Collapse loads for the brickwork arch were predicted using a MEXE analysis (2) and a modified mechanism analysis. Both methods are outlined below and the corresponding loadings are compared with the experimental collapse load of 1173kN.
6.1 MEXE method

The MEXE analysis carried out in accordance with Advice Note BA 16/84 (2) is set out below.

Span = 6.0 m (L)
Rise at crown = 1.0 m (r_c)
Rise at quarter span = 0.77 m (r_q)
Arch thickness = 0.22 m (d)
Depth of fill at crown = 0.3 m (h)
Provisional axle load (PAL) = (740 (d + h)^2)/613 = 19.48 tonnes.

Modifying factors:

\[ F_{st} = 0.75 \]
\[ F_p = 2.3(r_c - r_q)^{0.6}/r_c = 0.95 \]
\[ F_b = 1.2; F_f = 0.7 \]
\[ F_s = ((F_b \cdot d) + (F_f \cdot h))/(d+h) = (1.2 \times 0.22) + (0.7 \times 0.3))/0.52 = 0.91 \]

\[ F_j = F_w \cdot F_d \cdot F_{so} = 0.9 \times 1 \times 1 = 0.9 \]
\[ F_c = 1.0 \]

Modified axle load (MAL) = 0.75 \times 0.95 \times 0.91 \times 0.9 \times 1.0 \times 19.48

= 11.37 tonnes

(axle load for double axle bogie)

Axle load for a single axle = 14.55 tonnes.

Calculations derive a loading for a single axle of 14.55 tonnes. As the bridge was sufficient for two carriageways this provides a total loading in a similar format to the KEL of 29.1 tonnes. Compared with the actual experimental failure load a factor of safety of 4.1 is arrived at:-

FS = 1173/285 = 4.1

Although this factor of safety may seem more than adequate against failure a single axle may indeed be sufficient to cause significant cracking of the bridge. A single axle load of 14.55 tonnes (143 kN) would produce two equal wheel loads of 71.5 kN. A load of 70 kN placed at 970mm from the outer edge was sufficient to cause spandrel wall/barrel separation leading to ring separation, clearly the single axle load would also be sufficient to cause structural damage to the bridge if applied nearer to the edge at the quarter span or, possibly, at the crown.
6.2 Mechanism analysis

A modified mechanism analysis was conducted to determine a collapse loading for the bridge. The method adopted was similar to that proposed originally by Pippard and Baker (5) and later by Heyman in the guise of 'Plastic' theory (6). However, the method has been modified to account for lateral pressures and stiffening due to spandrel walls. The theoretical physical model is given in figure 6.1.

Twelve separate analyses were carried out using a simple mechanism analysis with hinges assumed to be in the same position as those for the Dead Load only case. This is not strictly correct but it allows a relative 'feel' for the significance of each restraining parameter.

It can be seen from Table 6.1 that by allowing for the self weight of the spandrel wall and the effects of friction/cohesion between the backfill and the spandrel wall: a load at which a mechanism forms can be predicted.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Description</th>
<th>Quarter Pt. Load at Failure (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Vertical DL soil &amp; barrel only</td>
<td>280</td>
</tr>
<tr>
<td>2</td>
<td>Case 1 + (K₀h) horizontally</td>
<td>307</td>
</tr>
<tr>
<td>3</td>
<td>Case 2 + DL spandrel wall (sw) + all resistance (cφ)</td>
<td>687</td>
</tr>
<tr>
<td>4</td>
<td>Case 1 + (K₁h) horizontally</td>
<td>554</td>
</tr>
<tr>
<td>5</td>
<td>Case 4 + (sw)</td>
<td>783</td>
</tr>
<tr>
<td>6</td>
<td>Case 4 + (cφ)</td>
<td>734</td>
</tr>
<tr>
<td>7</td>
<td>Case 4 + (sw) + (cφ)</td>
<td>1010</td>
</tr>
<tr>
<td>8</td>
<td>Case 1 + (K₁h) horizontally</td>
<td>750</td>
</tr>
<tr>
<td>9</td>
<td>Case 8 + (sw)</td>
<td>978</td>
</tr>
<tr>
<td>10</td>
<td>Case 8 + (cφ)</td>
<td>931</td>
</tr>
<tr>
<td>11</td>
<td>Case 8 + (sw) + (cφ)</td>
<td>1159</td>
</tr>
<tr>
<td>12</td>
<td>Case 8 + (sw) + (cφ) + (cφ © KEL)</td>
<td>1175</td>
</tr>
</tbody>
</table>

Table 6.1 Mechanism Analysis

Load case 2 represents the case of the arch barrel unrestrained by the effects of spandrel wall and wall friction. More realistically, because of the compaction of the backfill, horizontal stresses will be close to those given by K₁ (coefficient of pressure for unloading and is usually taken as K₁ = 1/K₀ (K₀ = coefficient of pressure at rest). This suggests that a load of 554 kN (case 4)
would result in movement and a reliance on support from other parameters to prevent failure. Two cases are considered, firstly spandrel wall stiffening. The lower bound is that of dead load effects only which is considered in case 5. (If spandrel wall separation had existed then the cohesion/friction between the spandrel wall and the backfill would have resisted motion (case 6)). Either way at around 734 - 783 kN major deformation would be likely. The large rotations associated with the mechanism would thus result in the interaction between the spandrel walls and the backfill to be mobilised (in addition to the spandrel wall dead load). Thus suggesting a maximum load of 1010 kN.

If spandrel wall separation had been present then a maximum load of 734 kN (case 6) would have been predicted.

Cases 8 to 12 (Table 6.2) follow a similar logic but for the full passive soil resistance case. Although the predicted loads are closer to the observed loads, it is important to note that the passive soil resistance was not recorded during the test.

Cases 5 and 6 predicted a change in stiffness of the bridge, which was observed in the graphs eg. figure 5.3. This may form a basis for determining a serviceability limit state, by applying a factor of safety of 3 to this load. For this bridge it would have given a safe working load of about 250 kN (cf. MEXE 285 kN). The arch ring is quite slender relative to 'normal' bridges which would normally have a four ring barrel for a 6 metre span. The thicker barrel would greatly increase the theoretical carrying capacity but would not affect the MEXE assessment. In fact the test probably highlights one of the situations where MEXE should not be considered to be quite so conservative.

Additionally, the friction on the extrados will offer some resistance to movement. No measurements were taken to quantify this effect but a crude assessment would indicate a load carrying capacity enhancement of at least 100 kN.

7 CONCLUSIONS

The following conclusions may be made:-

(i) The bridge failed due to the formation of a four hinge mechanism. The failure mode was identical to that given by small-scale model tests.

(ii) A point loading similar to that given by a MEXE analysis for a wheel load was sufficient to cause structural damage, ring separation, to the bridge. Further loading led to propagation of the cracking.

(iii) The backfill provided a significant lateral restraint to the deformation of the arch ring, however, passive pressure was not recorded.

(iv) Using a modified mechanism analysis, incorporating the lateral backfill pressures, spandrel wall stiffening and backfill cohesion/friction structural interaction, the onset of mechanism behaviour and the collapse load can be predicted. The prediction of the onset of mechanism behaviour could be used to set a serviceability limit state.
8 ACKNOWLEDGEMENTS

The authors wish to acknowledge the support and encouragement given by John Page (TRRL Project Officer) in the execution of the contract. Additionally, the financial support given by NAB and the Bolton Institute to provide the large scale testing facility is recognised. The assistance of the support staff is also acknowledged, particularly Jim Briggs (Principal Technician), Phil Owen, Neil McMillan, Jim Burns and Len Nuttall for constructing and testing the bridge, and Mildred Jones for typing the report.

9 REFERENCES

1 'The Assessment of Highway Bridges and Structures', Department of Transport Roads and Local Transport Directorate, Departmental Standard BD 21/84, Department of Transport, March 1984.

2 'The Assessment of Highway Bridges and Structures', Department of Transport Roads and Local Transport Directorate, Advice Note BA 16/84, Department of Transport, March 1984.


APPENDIX

A1 material properties

23,500 solid concrete engineering bricks, nominal dimensions 215 x 102 x 66mm, were used in the construction of the bridge. The average properties of the bricks, tested in accordance with BS6073, are outlined in Table A1.1.

A1.2 Mortar

A 1:2:9 (cement:lime:sand) mortar mix by volume was used throughout for both the arch barrel and walls. The average properties of the mortar at 28 days are summarised in Table A1.1.

A1.3 Brickwork

A series of small-scale prism tests were carried out to determine the properties of the brickwork. The average compressive, flexural and shear strengths are given in Table A1.1.
The initial tangent elastic modulus for the brickwork was determined by linear regression of the experimental results, average $E_1 = 6.8 \text{kN/mm}^2$.

<table>
<thead>
<tr>
<th>Material</th>
<th>Compressive</th>
<th>Tensile/</th>
<th>Shear</th>
<th>Density</th>
<th>24 hr.</th>
<th>Absorption</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>strength N/mm$^2$</td>
<td>flexural strength N/mm$^2$</td>
<td>strength N/mm$^2$</td>
<td>% wt.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brick</td>
<td>32.0$^1$</td>
<td>-</td>
<td>-</td>
<td>22.2</td>
<td>6.1</td>
<td></td>
</tr>
<tr>
<td>Mortar</td>
<td>2.3</td>
<td>0.11$^2$</td>
<td>-</td>
<td>20.9</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Brickwork</td>
<td>11.2</td>
<td>0.33$^3$</td>
<td>0.27</td>
<td>22.0</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

1 Bed-face direction
2 Splitting cylinder test
3 Flexural strength

Table A1.1 Material Properties

A1.4 Backfill

A 50mm graded limestone 'crusher run' was used for the backfilling of the arch. The properties of the material are given in Tables A1.2 and A1.3 and in figures A.1 and A.2. Shear testing of the backfill was carried out on a sample passing a 20mm sieve in a 300 x 300mm shear box, the loading rate was 0.2mm/min. Results show the backfill to have been a c-$
\phi$ material: \phi = 54$^\circ$ and c = 6.5 kN/m$^2$ (figure A.1).

A1.5 Bituminous road surface

A high density bituminous road surface using 10mm limestone aggregate was provided. The average compacted density was 19.4 kN/m$^3$.

<table>
<thead>
<tr>
<th>% PASSING</th>
</tr>
</thead>
<tbody>
<tr>
<td>SIEVE SIZE</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>75 mm</td>
</tr>
<tr>
<td>37.5 mm</td>
</tr>
<tr>
<td>10 mm</td>
</tr>
<tr>
<td>5 mm</td>
</tr>
<tr>
<td>600 m</td>
</tr>
<tr>
<td>75 m</td>
</tr>
</tbody>
</table>

Table A1.2 Sieve analysis of backfill
<table>
<thead>
<tr>
<th>TEST PROPERTY</th>
<th>TEST RESULT</th>
</tr>
</thead>
<tbody>
<tr>
<td>AIV</td>
<td>12.3</td>
</tr>
<tr>
<td>BULK DENSITY</td>
<td>$21.4^1$ kN/m$^3$</td>
</tr>
<tr>
<td>MAX. DRY DENSITY</td>
<td>19.3 kN/m$^3$</td>
</tr>
<tr>
<td>OPTIMUM MOISTURE CONTENT</td>
<td>8.4 %</td>
</tr>
<tr>
<td>$\phi$</td>
<td>54$^0$</td>
</tr>
<tr>
<td>$c$</td>
<td>6.5 kN/m$^2$</td>
</tr>
</tbody>
</table>

1  in-situ (sand replacement method)  
moisture content = 1.9%

Table A1.3 Properties of Backfill
Plate 2.1 West elevation of bridge

Plate 3.1 Pressure cells
Figure 2.1 Bridge Details
Figure 3.1 PLAN OF 'POINT' LOAD TESTS

Figure 3.2 ARRANGEMENT OF DISPLACEMENT TRANSDUCERS
Figure 3.6 ARRANGEMENT OF PRESSURE CELLS

Figure 3.7 ARRANGEMENT OF PRESSURE CELLS

Figure 3.8 PLAN OF ELEMENT STRAIN GAUGES
Figure 4.1
Comparison of load/microstrain strain responses

Figure 4.2a

Comparison of load/microstrain strain responses

Figure 4.2b

Microstrain

DISC FROM NORTH ABUT. (in)

Figure 4.2b

Microstrain

DISC FROM NORTH ABUT. (in)

Figure 4.2b
Figure 5.3 FAILURE MODE, WEST FACE

Figure 5.4 FAILURE MODE, EAST FACE
Figure 6.1 Theoretical Physical Model
Figure A.1 SRLM BUX TEST BALKFILL

Figure A.2 PROCTON TEST