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THE EFFECT OF SPANDREL WALL STRENGTHENING ON THE LOAD CAPACITY OF MASONRY ARCH BRIDGES

M.Miri[1] and T.G. Hughes[2]
[1]Research Student, Cardiff School of Engineering, Cardiff University, Cardiff, CF24 3TB, MiriM@cf.ac.uk
[2] Professor, Cardiff School of Engineering, Cardiff University, HughesTG@cf.ac.uk

ABSTRACT

This paper reports the results of a series of small-scale centrifuge model tests on undamaged, un-strengthened (benchmark) arches and the corresponding strengthened (repaired) structures. This work is part of a series of investigations into the effectiveness of arch repair strategies both on the effect at serviceability and at ultimate load.

The model arches tested were 1/12th scale three dimensional models of a notional 6 metre single span brickwork arch bridge. Two arch geometries were tested with span/rise ratios of 2 and 4; the arches were identical in all other respects. The models include the arch ring, the spandrel wall and the backfill material; they do not include parapets or wing walls. The undamaged and un-strengthened arches were tested under service and just short of ultimate live load. The arches were then repaired by applying reinforced concrete on the inner face of parts of the spandrel walls. The repaired arches were subsequently retested initially with a rolling load and then with a line load which was progressively increased up to full failure.

Test results indicate a significant improvement in the load capacity of the repaired arch. The loads at the failure of the strengthened models were approximately twice the un-strengthened failure load for both the shallow and deep arch geometries. Under service loading the strengthened arches also demonstrated improved (stiffer) performance. The advantage of this strengthening method is that it is efficient whilst being relatively low cost with minimum disruption to traffic using the bridge and has no external visibility.

KEYWORDS: arch bridges, repair, spandrel walls

INTRODUCTION

Significant experiments on single arches have been carried out and reported by Hendry [1], Page [2], Hughes [3] and others. Most of the early tests on arch bridges and their results were reported by Page [4]. The results of these experiments were initially directed at load assessment and have provided significant understanding on the response of these types of structures to loads. Most of the existing structures in use are more than a hundred years old and therefore there is significant interest in repair and strengthening techniques. In recent years experimental work has moved more from assessing existing capacity towards investigations of methods to increase capacity. The effect of various strengthening methods on full scale arches was reported by Sumon [5] and

Melbourne et al. [6]. Baralos [7] applied different types of strengthening method on 1/12th scale two-dimensional single span arch models. The effects on arch load capacity of applying a concrete saddle assessed by small scale centrifuge models are presented by the authors elsewhere [8].

In most of the above referenced works two-dimensional arch models were studied. However, it is known that a significant number of arch bridge defects are due to transverse effects. Lateral earth pressure on spandrel and wing wall tends to cause the overturning of the walls and often causes longitudinal cracks between the spandrel and the arch ring. Longitudinal cracks can also be caused by the lane directionality in the loading, but these tend to be on the centreline rather than at the spandrel ring interface. The edge longitudinal cracks may result in the simple separation of the arch barrel and spandrel wall. Additionally, the backfill downward pressure due to self weight and traffic loads on the barrel which is restrained in each side between two relatively stiff spandrel walls may cause transverse bending in arch barrel. This suggests that three dimensional and spandrel wall effect on these structures should also be considered. Three dimensional numerical and experimental works by Fanning and Boothby [9] and Boothby and Robert [10] have shown bending effect and transverse behaviour of arches under service loads tests.

A survey of ninety-eight arch bridges by Page et al. [11] showed that twenty-three bridges had longitudinal cracks and sixty-three bridges had spandrel wall defects. Despite the magnitude of this problem, little research has been carried out on understanding the behaviour of spandrel walls and to the authour's knowledge nothing specifically on the effect of strengthened walls on the arch behaviour under service and ultimate loads. The strengthening methods related to spandrel walls have two possible objectives: to increase the resistance of the spandrel walls to overturning (as any earth retaining wall) and to increase the connection between the stiff spandrel wall and the arch ring to provide additional overall arch bridge load capacity.

The results of two small scale model tests on undamaged and on spandrel wall strengthened arch are presented in the current paper. The emphasis in the paper is the load capacity of the bridge and although there is nothing specifically preventing spandrel walls overturning, it was not the primary goal of the studies.

DESCRIPTION OF ARCHES UNDER TESTS

The two geometries of the two experimental models, with both shallow and deep arches, are shown together with their respective instrumentation points in Figure 1. The arch models were single spans of total width 405 mm in both cases; the bridges were built in a rigid box with a total width of 450 mm leaving enough free space between the box rigid side walls and the spandrel walls. The fill and spandrel wall were extended to be restrained to the rigid end of the box at both longitudinal ends, although some flexible packing was included to allow in-plane spandrel wall movement. Both the arch vaults and spandrel were made of small scale brick masonry where the brick units themselves were manufactured by being diamond cut from whole bricks. The arch vault was constructed of three rings in both models which were laid as separate stretcher bonds without any cross bonding headers. In the UK, with the exception of waterway arches, most arch bridges are constructed without headers. The spandrel walls were laid on top of the arch barrel with a thickness of 30 mm and were constructed from full width single small bricks which simulate block masonry in the prototype. The major geometrical parameters of the arches are summarized in Table 1.

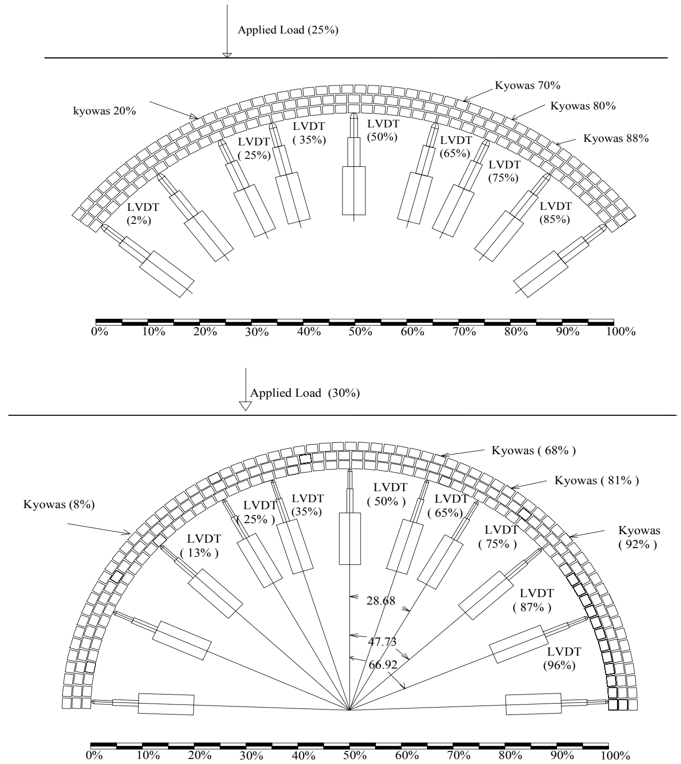

Figure 1 – Arch geometry and model instrumentation location

A UK mortar type V with a mix content of 1:3:12 (cement:lime:sand) was used as the joint material. Three rows of Linear Variable Direct Transducers (LVDT) were installed under the longitudinal middle line and at both edge of the arches to record the arch barrel deflections during the tests. Due to the limited numbers of available channels, different numbers of LVDTs were installed in these three rows with the maximum number used (as shown in Figure 1) in the centre row. Three LVDTs at 25%, 50% and 75% of arch span recorded the deflection of the top of spandrel walls from above. Small diaphragm pressure gauges (Kyowas), also shown in Figure 1, were used to measure the normal soil pressures between the arch barrel and the backfill material.

Table 1 – Summary of geometry and properties of arch models

Properties	Shallow	Deep
Span (mm)	500	500
Span/rise	4	2
Ring thickness (mm)	30	30
Depth of fill on crown (mm)	13	13
Load Point(% Span)	25	30
Concrete thickness (mm)	17	17
Concrete compression (N/mm2)	56	56

These pressure cells are located within special manufactured brick units, laid within the extrados brick course, with their active face flush with the extrados; two pairs of these small gauges were used across the arch at each location shown in Figure 1.

EXPERIMENTAL PROCEDURE

The arch model barrels were initially built (over several days) on pre-bent steel support formwork with connected rigid steel blocks used as the abutments. A week after its completion, the arch barrel was placed in the test box and the spandrel wall building started. Care was taken to construct both side walls at the same time to prevent any unwanted, unsymmetrical loading on the barrel during the building process. The arch was backfilled 28 days after the completion of the spandrel walls and then tested. Testing is undertaken immediately following backfilling in order to try to maintain the moisture level in the backfill material. The centrifuge tests are undertaken at a stable 12 g acceleration with initially a service load being applied to the models. The service load is made up of three roller weights which act as a line load across the full width of the arch between the spandrels. The dead load rollers are moved slowly across the arch bridge, taking about 15 minutes per pass. Details of the applied load are presented elsewhere [12]. After service loading was complete, a fixed location knife load was applied across the whole width of arch. The ultimate load tests were stopped when the arch exhibited signs of failure behaviour, for example when any crack was observed in the spandrel or barrels (by camera) or when the load deflection plot started to level off. The arch was then repaired using the reinforced slab concrete detailed in the next section. The repaired arches were retested after 28 days (curing period) under the same service and ultimate loads, but the knife loading was taken up to the full failure at this stage. Although most of the original backfill was retained during the repair to help support the arch, all the remaining backfilling was removed and then replaced with new material immediately before re-testing.

The rational behind the test/retest procedure, as designed, is that the repair (strengthening) is to be applied to a damaged rather than a "new" arch, and that if possible any construction defects in the initially un-strengthened arches are retained in the strengthened tests. Since it is difficult to judge when the initial "ultimate" load test is to be stopped, due to concerns over the inability to strengthen a completely fragmented arch, separate un-strengthened tests have been taken to complete collapse. The results of the strengthening can therefore be compared to both the actual same un-strengthened case as well as the average (benchmark) of a series of other un-strengthened tests.

SPANDREL WALL STRENGTHENING METHOD

Various stages of the arch repair are shown in Figure 2. Retro-reinforcement is suggested as a method of reinforcing existing masonry to increase its ability to resist bending and shear forces. Test results on a series of un-reinforced and reinforced 2 m span clay brick model arch bridges [13], reinforced with 6 mm diameter stainless steel bars into grooves up to 75 mm deep, have previously indicated improvements in the in-service performance as well as the load carrying capacity of reinforced arch. A principal feature of retro-reinforcement repairs is that they often result in a change in the appearance of the masonry arch bridge and this is an important issue with many of these aesthetically appealing structures. This method may also not be applicable where the mortar joints are very thin or where un-coursed masonry has been used. Use of tie bars is another method which is frequently used to prevent lateral movement of the spandrel walls relative to the barrel. These repairs mainly involve the installation of steel rods through both the spandrels and the existing fill; the rods extending into some end plate, visible on the spandrel walls, or terminating within some strengthen anchor section of the spandrel wall. This is a low cost method with little effect on traffic flow [14], but tests on full scale models of a 5 m span with and without tie bars have indicated that the tie bars had a negligible impact on the stiffness of the bridges under serviceability and higher loads [6].

Figure 2 – Shallow arch after initial test, during repair and after repair tests

A method used in the UK, sometimes referred to as the "Stratford" method, involves the excavation of a trench parallel, and immediately adjacent, to the spandrel walls that extends downwards to the arch barrel. The trench is backfilled with reinforced concrete placed part on the extrados of the arch barrel to which both the spandrel walls and the arch ring can be stitched using structural steel ties. The construction advantages of this method are that it can be undertaken without closing the bridge (being normally restricted to the footpath), it can be constructed without access beneath the structure and, if extended only up to the road surface, it has no external visibility; the concrete itself can also provide a key for enhanced parapet provision, if required. In the present study the vertical extent of the reinforced concrete was restricted to 100 mm in depth (about 1.2 m in the prototype). This would require only limited support during construction. The concrete was manufactured using 2 mm aggregate as the coarse aggregate, Chelford 95 silica sand as the fine, and OPC. The mix proportion of 1:1.8:2.8:0.6 (cement: fine: coarse: water) by mass was used and the 28 day compressive strength of 25 mm cube samples achieved 56 N/mm^2. The concrete was reinforced using 20 mm square mesh (UK type 304) manufactured of 0.8 mm diameter mild steel. To apply the strengthening method holes with a depth equivalent to one brick course were initially drilled in the barrel and 2 mm steel rods were installed in those holes using an epoxy resin.

TESTS RESULTS

Crown arch deflections under different types of service loads (when positioned at the crown) for the shallow arch are detailed in Table 2. The results are for both a full a width load of 15 kg and a half width load of 7.5 kg (there are slight differences between the original and repaired loading, but the deflections have been ratioed to suit). The shallow benchmark (un-strengthened) deflections are approximately the same under both the edge and the middle of the arch for the full width load. These reduce, as expected, when the load remains symmetric but of reduced width (and weight); there is also some indication of transverse bending. This effect is repeated with the Service Load 3 with the edge under the load now deflecting more than the arch middle. The repaired arch indicates a significantly stiffer response, especially at the edges (where the concrete was placed); the back reading is an anomaly.

Table 2 – Arch deflection under service load (S-Shallow D-Deep,)

Test ID	LVDT location	Service Load -1	Service Load -2	Service Load -3
S-Benchmark (50%Span)	Front	-	-0.04	-0.07
	Middle	-0.10	-0.06	-0.03
	Back	-0.11	-	-
S-Repair (50%Span)	Front	-0.02	-0.01	-0.03
	Middle	-0.06	-0.03	-0.03
	Back	-	-	-0.11
D-Benchmark (35%Span)	Front	-0.03	-0.01	-0.02
	Middle	-0.03	-0.01	-0.02
	Back	-0.03	-0.01	-0.00
D-Repair (35%Span)	Front	-	-	-
	Middle	-0.02	-0.01	-0.01
	Back	-0.01	-0.00	-

Service load-1: rolling load applied to the whole width of model
Service Load-2: half width rolling load applied along the middle of the model width
Service Load-3: half width rolling load applied to the front edge of the model

The deep arch deflections at the third point of the span are presented in Table 2 for different service loads positioned at the third point. The deep arch appears stiffer than the shallow arch but deflections at the one third points would anyway be less. The transverse bending is also less apparent for both centre and edge loading. However, the overall effect of the strengthening appears to produce a similar increase in stiffness over the un-strengthened arch.

The load-deflection plots for the shallow arch tests for both the benchmark and repaired arches are presented in Figure 3. The results show the deflections at the Front edge (F), Middle (M) and Back edge (B) of the arch at the quarter point, immediately under the applied knife edge load. The results show the two dimensional behaviour (this will be affected by the rigid nature of the loading beam) but more importantly the significant increase in both strength and stiffness whilst maintaining the ductility of the overall behaviour. The result of the Benchmark also shows that the test was indeed taken up to the ultimate load capacity (without arch destruction). This is a very satisfying initial outcome.

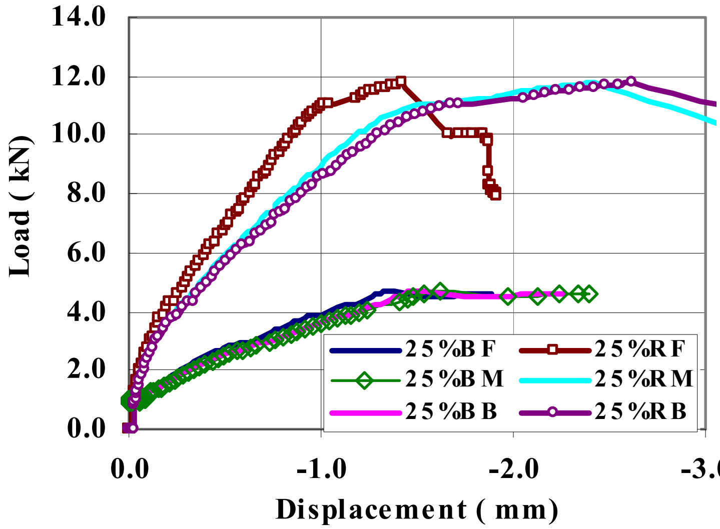

Figure 3 – Load- deflection curve for shallow arch (B-Benchmark, R-Repair)

The effect of arch movement and developed passive pressure for the shallow arch is detailed in Figure 4 as determined from the ultimate load tests. The recorded pressure was zeroed before starting the loading for this figure so as to represent change in pressure solely as a result of the

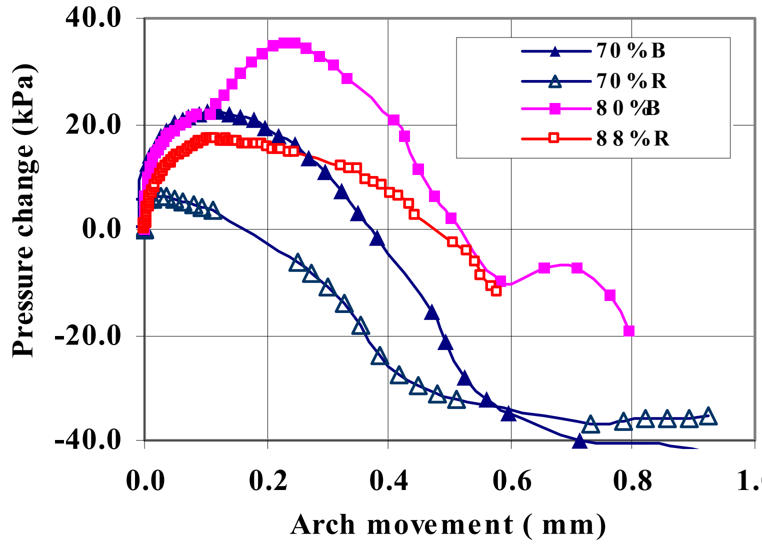

Figure 4 – Pressure / Displacement at different position of the shallow arch

effect of the live load. Under increasing applied load on the far side of arch barrel (25%) the arch initially moves toward the backfill and due to this movement, the passive pressure was mobilized. The benchmark arches indicate slightly higher pressures as the strengthened spandrel wall restricts the arch barrel movement; therefore, movement and change in pressure on the repaired arch is less than the benchmark test. This effect can be seen up to a deflection of about 0.1 mm. Following this, the hinges are starting to form and the overall kinematics of blocks of arch (and soil) cause the soil to move away faster than the arch, resulting in less pressure at these locations. The maximum change in pressure is recorded at 70% of the arch span under movement of about 0.15 mm. The same change in pressure was reported by Burroughs [15] under movement of about 0.25 mm at the same section for a two-dimensional arch.

For the deep arch benchmark, the knife load was increased to 6.9 kN. At this load level some cracks in the spandrel walls were observed and the test was stopped to avoid a collapse and an un-repairable arch. Arch deflections at 75% of the span for the benchmark and the strengthened arch recorded by the different rows of LVDTs are presented in Figure 5. Also included in Figure 5 is the deflection of the front face repaired wall (RWF). The repaired arch attained an ultimate load of 14.0 kN giving just over 100% increase over the un-strengthened (un-failed) initial test. The figure shows a largely two-dimensional response and a very significant increase in the stiffness. Deflection of the repaired arch at an applied load 14 kN is about 55% of the un-strengthened arch at the same position under half of that load.

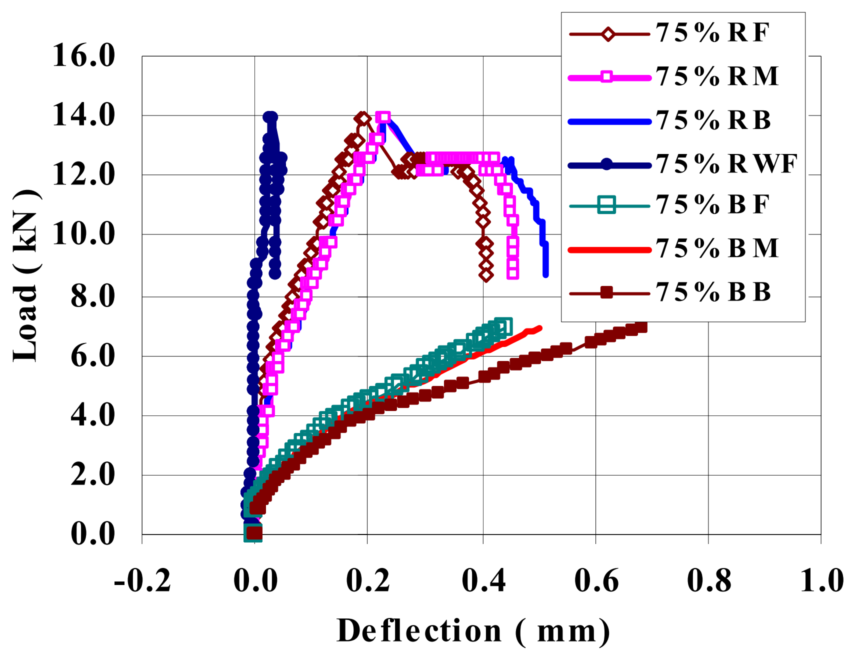

Figure 5 – Load- deflection curve obtained from deep arch test

Soil interaction at different positions in the extrados of the deep arch barrel due to increasing load are detailed in Figure 6 for the benchmark and strengthened arch. The post load peak soil pressures have been removed for clarity. For the benchmark arch the result at 8% (close and to the left of the applied load) show an initial increase in pressure likely under the direct influence of the applied load followed by a reduction as the live load pushes the barrel away from the soil leaving it behind. For the same test the pressure at 81% shows a progressive increase associated with the normal development of passive pressures. For the repaired arches there is a similar response at 8%, but there is a more gradual reduction in pressure (active pressure) as the arch moves away. At 68% there is little effect as there is little soil to develop any normal pressures. At 92% the effect is significant but is limited by the proximity of the rigid abutment. The maximum pressures develop at 81% of the span where the movement is significant and there is sufficient depth of soil to develop significant vertical pressures.

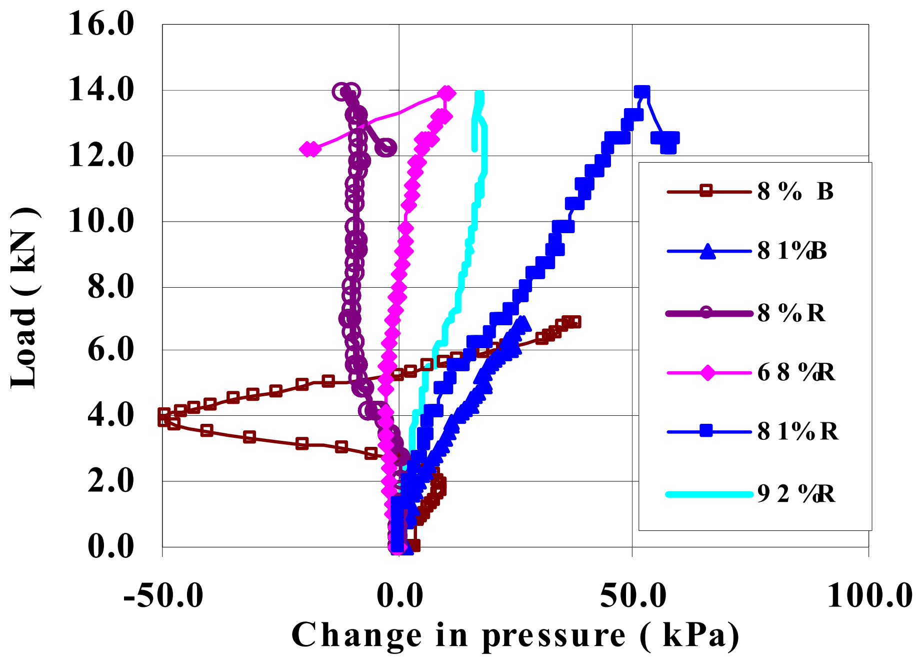

Figure 6 – Arch/Soil interaction obtained from deep arch test

The arches failed because of the formation of ring separation under the load position. The formation of the fourth hinge (most remote form the applied load) was not observed in either of the tests. It appears that the strengthened spandrel wall restrained the barrel very well. No crack occurred between the walls and barrel under the ultimate load in the repaired arches. The observations after the tests have shown no separation between the spandrel and barrel or disconnection of the steel rods connector during either test.

CONCLUSIONS

Laboratory experiments have successfully been carried out on two different geometries of 1/12th scale single span centrifuge arch models. The arches were repaired by applying a 17 mm reinforced slab of micro-concrete on part of the inside of the spandrel walls. The extent was limited to that reasonably attainable in the field. The experiments provided useful information on the effectiveness of this repair method on the failure mechanism and particularly on the service and ultimate load capacity of the arches. Separation between the barrel and spandrel wall observed under low load level and outward movements of the spandrel walls occurred in the un-strengthened tests, especially in deep arch geometry. Test results indicate a significant improvement in the load capacity of the repaired arch. The loads at the failure of the strengthened models were approximately twice the un-strengthened failure load for both the shallow and deep arch geometries. At the low level of applied load and under service load the strengthened arch demonstrated much stiffer behaviour compare with the un-strengthened arch. The strengthened spandrel walls were seen to restrain the arch barrel and the recorded change in pressure on extrados of arch barrel due to barrel movement is smaller than the un-strengthened arch.

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