INTELLIGENT MONITORING OF JACK ARCH STRUCTURES: THE RESULTS OF A PRELIMINARY STUDY

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ABSTRACT
A cast iron beam, instrumented with deflection gauges, strain gauges and two acoustic monitoring systems, was subjected to a series of load cycles. The aim of the test programme was to determine whether acoustic emission (AE) techniques could be used to form the basis of an intelligent monitoring system. The results support the view that AE was generated by a load-related mechanism - most likely due to micro-cracking of the cast iron - and so AE techniques might indeed form the basis of a monitoring system, but further work is required to develop a practical system.

INTRODUCTION

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Many old jack arch structures are still in service as short-span road bridges over canals and railways. Cast iron beams manufactured in the 19th century are also still in service in major public buildings, textile mills and warehouses. The structural condition of such bridges and beams is difficult to assess. Owners, and those charged with the maintenance of such structures, therefore, have a problem with managing such assets. On the one hand it is essential to identify sub-standard structures that pose a risk to users, but on the other it is desirable to minimise expenditure on the strengthening of perfectly adequate structures.

As part of the Transport Research Foundation’s reinvestment programme, a project was initiated at TRL Limited to examine various techniques for monitoring the in-service performance of cast iron structures. These might form the basis of an intelligent system that, ideally, was capable of continuously monitoring a structure so that it could forewarn of structural deterioration and thereby trigger remedial works. As a necessary precursor to field trials, various techniques were used to monitor the performance of a cast iron beam in a load test carried out in the laboratory. The beam was instrumented with strain gauges, deflection gauges and two acoustic monitoring systems. Details of the test set-up are provided herein. A summary of the results of the test is also provided, along with recommendations for further work.

CAST IRON

Cast iron is any iron alloy that has a carbon content of between 2 and 4 per cent, and a silicon content of between 1 and 3 per cent. It might also contain trace amounts of manganese, phosphorus and sulphur.

As the name implies, cast iron products are made by pouring molten alloy into a mould and allowing it to solidify. There are two basic types of cast iron. Grey cast iron forms as a result of relatively slow cooling, in which case some of the carbon solidifies as graphite flakes sitting in a matrix of what approximates to steel. White cast iron (or chill iron) forms as a result of relatively rapid cooling, in which the carbon is unable to solidify as graphite flakes but instead combines completely with the iron. Because the rate of cooling is dictated by the volume/surface area ratio, the strength of a casting varies with cross-sectional shape and size. A thin flange may cool relatively quickly resulting in a strong but brittle cast iron, conversely the material at the centre of large poor-quality castings can be spongy or porous in appearance.

The properties of cast iron can be compromised by the presence of chemical impurities, and by the frequency and size of flaws. The stress-strain behaviour of grey cast iron is a function of the quantity and coarseness of the graphite flakes, and so it can vary considerably. The flakes act as planes of weakness, resulting in low tensile strengths, but are able to transfer compressive forces so that the material has a near elastic behaviour in compression. In general, white cast iron has a higher tensile strength than grey cast iron.

Swailes (1995) published a small sample of the results of an investigation into the strengths of different British iron alloys undertaken by Fairbairn in the 19th century: as shown in Table 1, the properties can vary considerably.

Rank***	Name of iron	Specific gravity	E* (kN/mm^2)	Breaking load** (kN)	Colour	Quality
1	Ponkey, no. 3, cold-blast	7.122	119	2.584	Whitish grey	Hard
5	Beaufort, no. 3, hot-blast	7.069	116	2.300	Dullish grey	Hard
10	Beaufort, no.2, hot-blast	7.108	112	2.108	Dull grey	Hard
15	Oldberry, no.2, cold-blast	7.059	99	2.024	Dark grey	Rather soft
20	Blania, no.3, cold-blast	7.159	98	1.993	Bright grey	Hard
25	Carron, no.3, cold-blast	7.094	112	1.970	Grey	Soft
30	Wallbrook, no.3	6.979	106	1.957	Light grey	Rather hard
35	Level, no.2, hot-blast	7.031	105	1.908	Dull grey	Soft
40	Coltham, no. 1, hot-blast	7.128	107	1.886	Whitish grey	Rather soft
45	Coed-Talon, no. 2, cold-blast	6.955	99	1.837	Grey	Rather soft

* E obtained from the deflection at about one sixth of the breaking load
** Breaking load at centre of a 4ft 6in (1.37 m) span
*** Ranking in terms of strength

Table 1 Selected results from Fairburn’s tests on British iron alloys, after Swailes (1995)

JACK ARCH STRUCTURES

According to Hilton (1997), jack arch construction originated in the UK in the early part of the 19th century: the first recorded structure was built in 1801. This form of construction allowed short to medium span structures to be built with a much shallower depth than was possible with masonry.

Jack arch bridges typically comprise cast iron beams, at say 1 to 1.5 metre centres, with brick arches, usually at least two courses thick, spanning transversely to the beams. The brick arches spring from a mortar bed laid on the bottom flanges of the beams, and have a rise of about half the depth of the beams. The space below the running surface is filled with soil or concrete. Transverse steel ties might be provided between the beams at mid-span. Apart from the bottom flange, the beams are usually buried within brickwork and fill, and so it is difficult to determine the material condition and, hence, the structural stability of the arch.

Hythe End Bridge

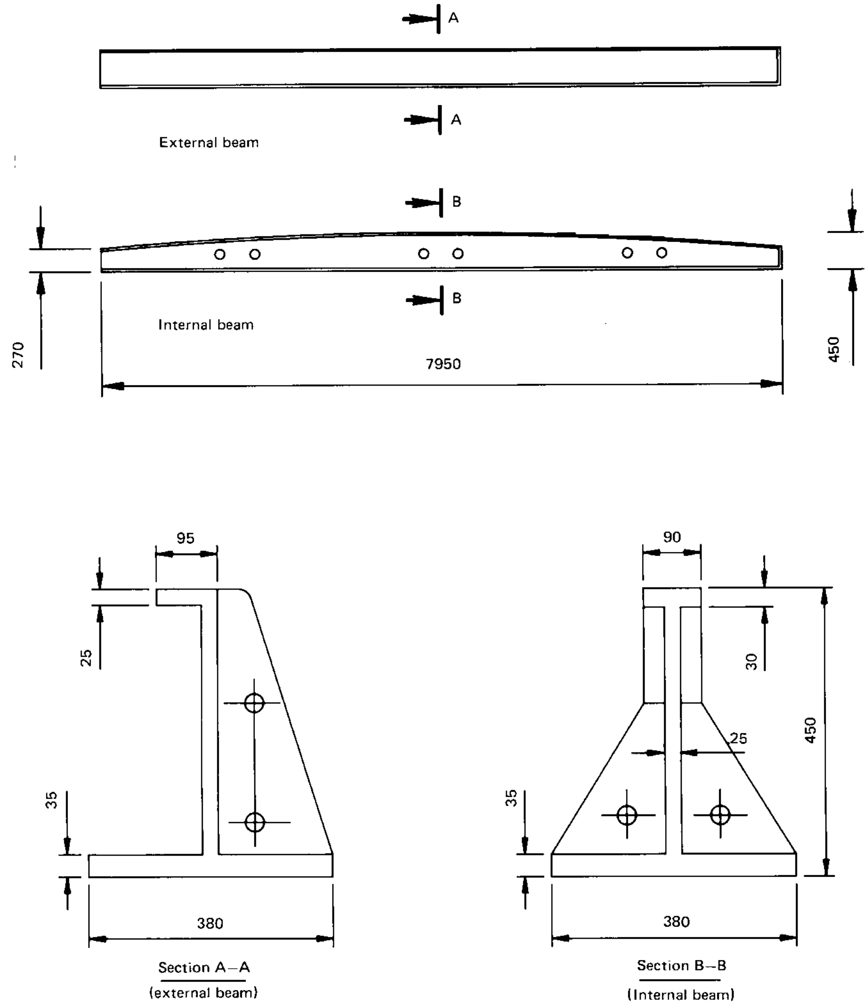
The Hythe End bridge is a good example of a jack arch structure: the test on an edge beam from this structure is the main focus of this investigation. The bridge was constructed in 1852 to cross the River Coln at Hythe End near Wraysbury, Berkshire. It was constructed as a two-span bridge with four beams per span. The beams were supported on brick abutments and on a brick pier at midstream to give two clear spans of 6.9 m; the width of the bridge between kerbs was 4.06 m.

In 1975, the bridge was repaired and strengthened: some brick arches were replaced with in situ concrete, permanent traffic lights were installed to enforce single lane traffic, and an 11-ton weight limit was placed on the bridge. In 1985, the bridge was assessed to BD 21 (DMRB 3.4.3) by the owners, Berkshire County Council. They determined that the structure had no capacity for highway loading and immediately closed the bridge. Prior to demolition, it was offered for testing to the (then) TRRL: the results of the test are given by Daly and Raggett (1991). Following the test, the bridge was demolished: Figure 1 shows the demolition works in progress, and reveals the inner structure of the bridge. All eight cast iron beams were subsequently recovered and stored at the TRL. Figure 2 shows elevations and sections of the internal and external beams. Calculations indicate that:

(i) the design load of the beam was about 72 kN, and

(ii) the collapse load of the beam was between about 150 kN and 220 kN.

Figure 1 Demolition of the Hythe End bridge, from Daly and Raggett (1991)



(All dimensions in mm)

Figure 2 Details of bridge beams from Hythe End bridge, after Daly and Raggett (1991)

DETAILS OF MONITORING SYSTEMS AND TEST ARRANGEMENT

Acoustic Emission

Many materials subjected to an applied load generate acoustic stress waves - these are mechanical vibrations that radiate out from an energy source. Crack propagation in metals and concrete, the debonding of composite materials, and sliding between particles of soil have all been shown to generate such waves. In a cast iron element, micro-cracking might be initiated by the separation of the graphite flakes and surrounding material. The rate and extent of cracking could be expected to be related to the magnitude of the load and, therefore, also to the acoustic emission (AE) generated.

The AE sensor, a piezoelectric transducer, is attached to the structure under examination. As the structure deforms or deteriorates, an emission is generated by some mechanism (for example, micro-cracking) and propagates through the structure as a series of stress waves. The transducer converts the stress waves (or vibrations) into a voltage. The voltage is processed usually by removing unwanted frequency components (filter) and amplifying the remaining signal. An analogue-to-digital conversion board samples the signal and converts it to a digital format: the data are then stored, usually on disk. An example of the resulting output is given in Figure 3 - this depicts a typical decaying sinusoidal transducer-generated response, termed an ‘event’. A number of ways of defining the output are shown on the figure.

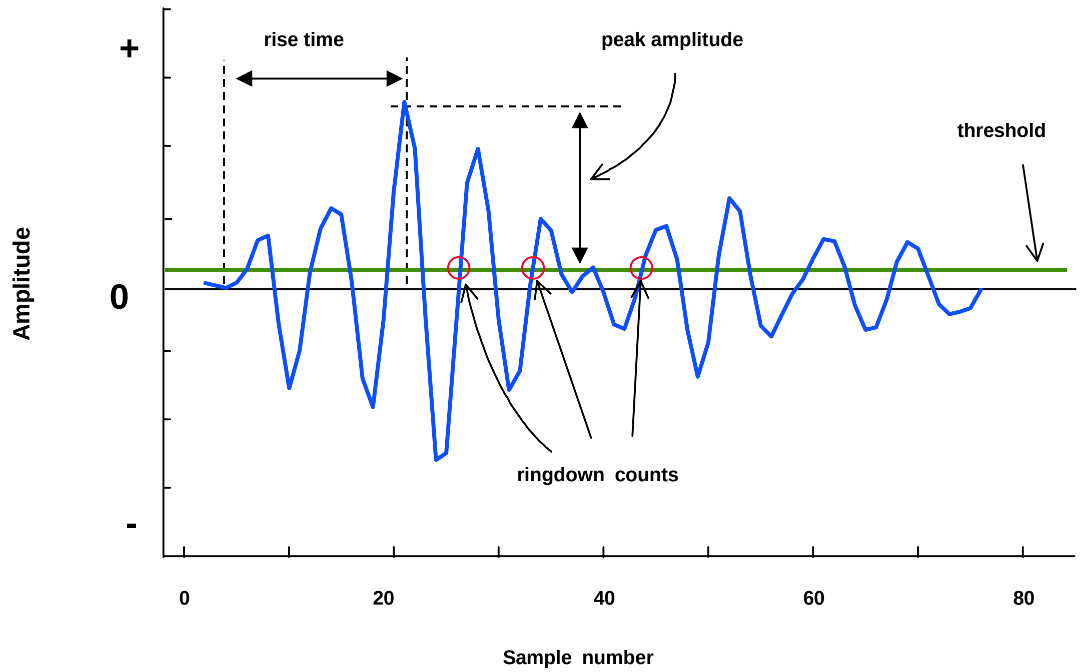


Figure 3 Transducer response to an event and its definition

<u>SoundPrint Acoustic Monitoring System</u>
The SoundPrint system was developed by Pure Technologies Inc to detect wire breaks in steel tendons and has a good track record in that area. It employs a slightly different philosophy to that used in conventional practice, in that the mechanism responsible for the generation of the acoustic stress wave is the catastrophic break of a wire rather than one reflecting a gradual deterioration in condition. The system can be 'tuned' to detect the release of energy from lower energy events, such as micro-cracking. In this system, each piezoelectric transducer is connected to the data acquisition system and the time taken for the signal to reach each sensor is used to identify the location of the signal. The system has been successfully used to detect wire breaks in a number of bridges. Details are given by, amongst others, MacNeil and Cullington (1998).

<u>Other Measurement Systems</u>
The cast iron beam was also instrumented with deflection gauges and strain gauges. In contrast to AE techniques, such gauges only measure the change at a point and this may not be representative of the structure as a whole.

<u>Limitations of Test Programme</u>
As described later, the test involved the application of a sequence of loads each applied for a fixed time before being removed. The maximum load generally increased through the sequence, but this does not replicate the loading history of an in-service structure. Furthermore, the stability of jack arch structures is considerably enhanced by the support provided by the adjacent brick jack arches and fill. This support was not available in the test conducted on the single beam.

<u>Test Set-Up</u>
The test set-up is illustrated in Figure 4. The beam was loaded using a computer-controlled 500 kN test rig at the TRL. The computer recorded the movement of the cross head of the rig and the applied load - calculated from the load cells housed in each of the rig supports. As shown in Figure 5, an additional and more accurate load cell, of 450 kN capacity, was placed between the loading platen of the rig and spreader beam.

Figure 4 Cast iron beam under test rig

Figure 5 View of load cell and spreader beam

The test beam was fitted with six deflection gauges, eighteen strain gauges (in six triplets), five SoundPrint acoustic sensors and four Physical Acoustics (PA) AE transducers. The locations of the instruments are shown schematically in Figures 6 and 7, and the orientation of the strain gauges is shown in Figure 8. The middle set of strain gauges (i.e. 3, 9, 15, 23, 29 and 35) were parallel with the longitudinal axis of the test beam.

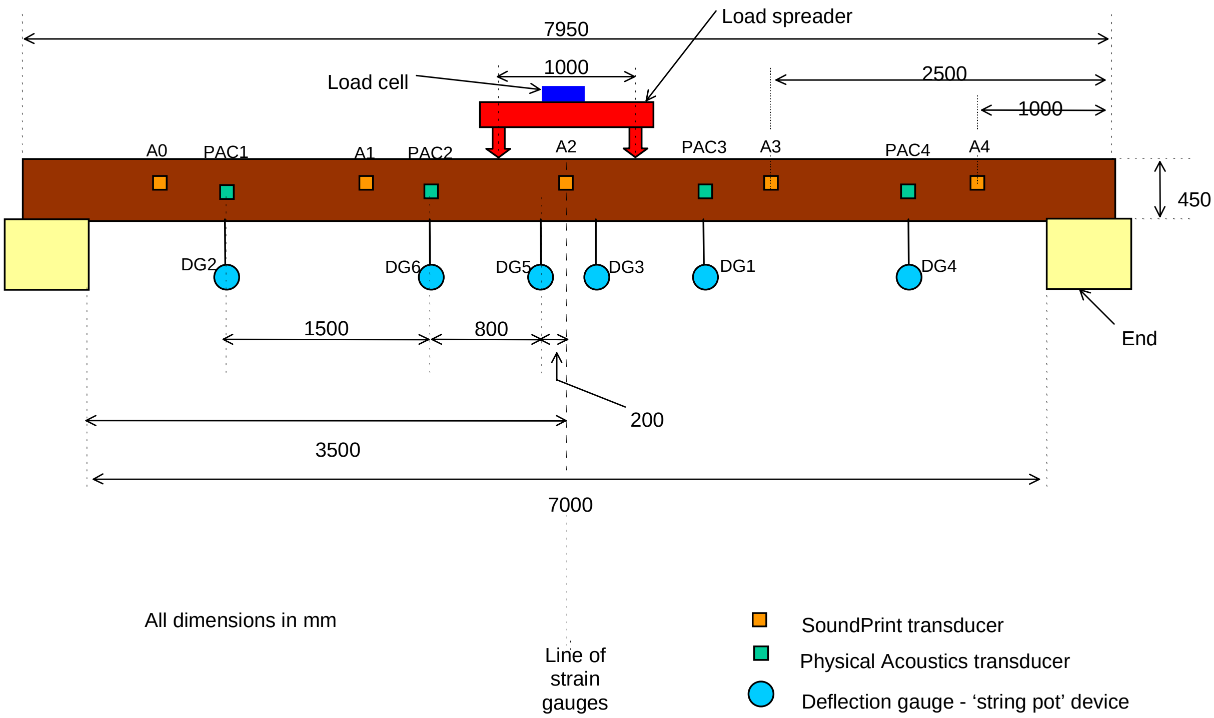


Figure 6 Elevation of beam showing location of instruments

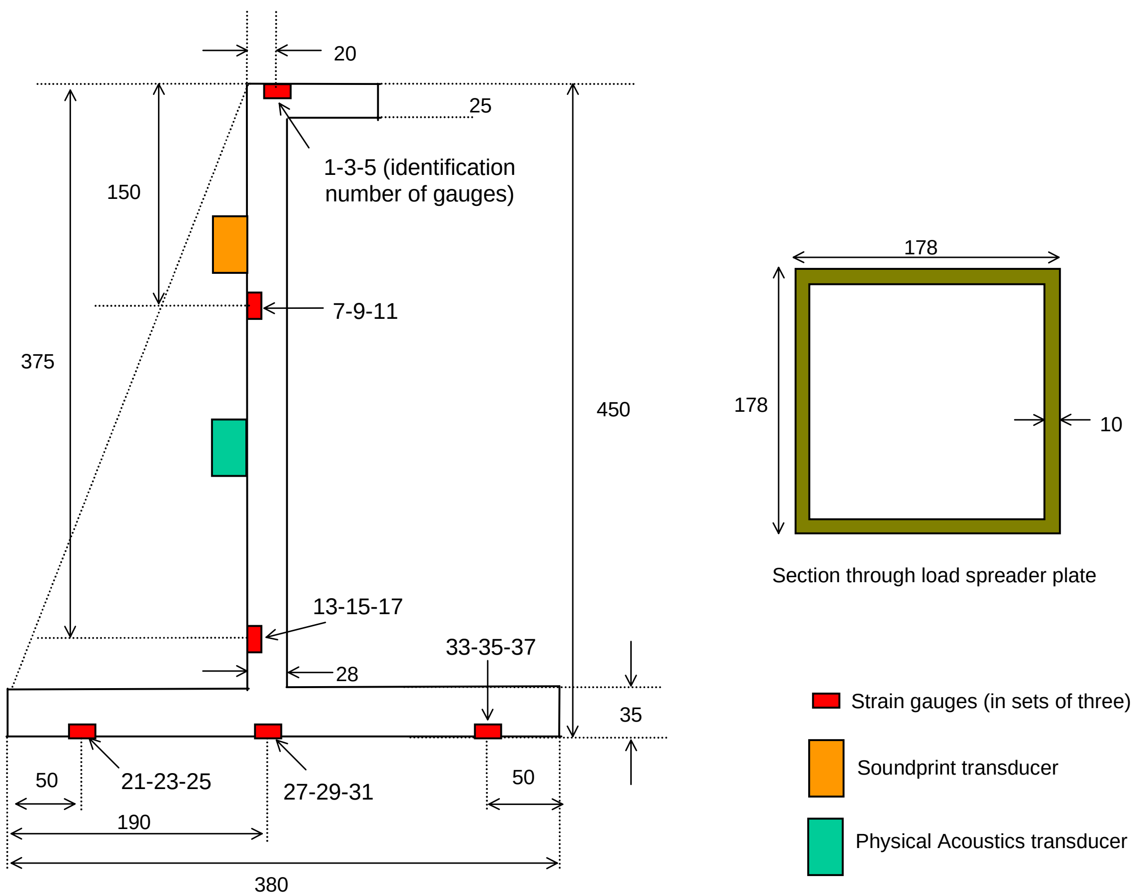


Figure 7 Cross section through beam at mid-span (showing location of strain gauges and lines of acoustic transducers) and through load spreader plate

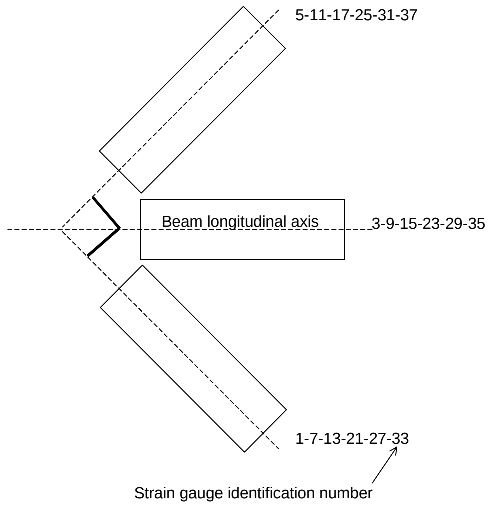


Figure 8 Configuration of strain gauges

<u>Details of Test</u>
The test was undertaken on an edge beam taken from the Hythe End bridge: the beam was forged in 1852. The beam was 7.95 m long and weighed approximately 15 kN: its dimensions are given in Figures 6 and 7. The beam was placed longitudinally and centrally under the test rig. A load spreading plate was fixed to two bearings attached to the top flange of the beam. The bearings were 1 m apart and arranged equidistant from the centre of the beam. The beam was supported at both ends by concrete blocks. To provide some acoustic insulation a thin sheet of rubber was placed between each end of the beam and the supporting concrete block.

The test consisted of a series of nine load cycles in which the maximum applied load generally increased in sequence. Prior to the application of load, the cross head of the loading rig was moved downwards until it was close to the load cell. The rig was then placed under computer control and the cross head moved downwards at a constant loading rate of 3 kN per minute, the slowest rate the rig would accommodate, until the target load was reached. The peak load was left on for some time while monitoring continued. The load was then removed, usually rapidly, and monitoring continued for a short period following unloading.

Whilst under computer control, but just prior to the cross head coming into contact with the load cell, the data-logger and PA AE system were primed. The SoundPrint system ran continuously over the entire test period but its clock was synchronised with the other AE system. In this way it was possible to match the data from the rig computer with the other data sets. A manual record of times, events, points of interest and so on was also made. Details of the load cycles are given in Table 2.

SUMMARY OF RESULTS
The deflections recorded at the peak loads are plotted in Figure 9: it is evident that the deflection of the beam was slightly asymmetric. As shown in Figure 10, there was a more or less linear relationship between the recorded deflection and the peak load. The deflections plotted in Figures 9 and 10 are those recorded from the start of each cycle of load, but the beam did not return to its original position at the end of each cycle. Although there was some asymmetry in the response, the residual displacement was a little less than 1 mm.

Load cycle	Peak load (kN)	Approximate time peak load left on hold (minutes)	Approximate time monitoring continued after load removed (minutes)
1	51	25	0
2	68	10	0
3	73	10	10
4	52	10	10
5	87	15	10
6	104	20	10
7	116	1020	90
8	130	20	15
9	151	15	15

Table 2 Details of load cycles

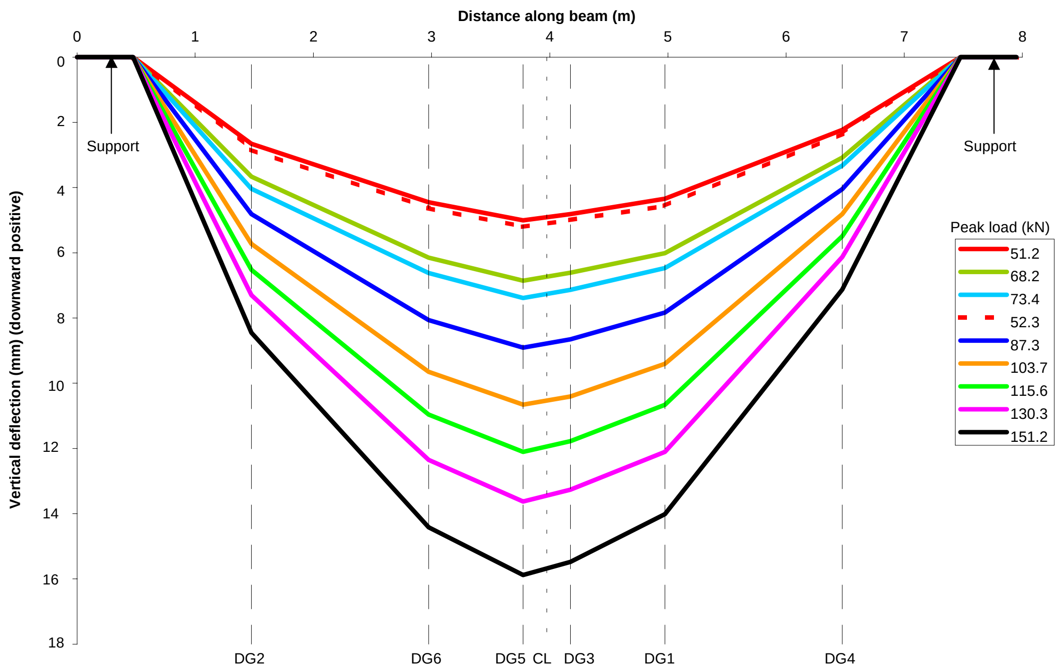


Figure 9 Displacement of beam at peak load for each load cycle

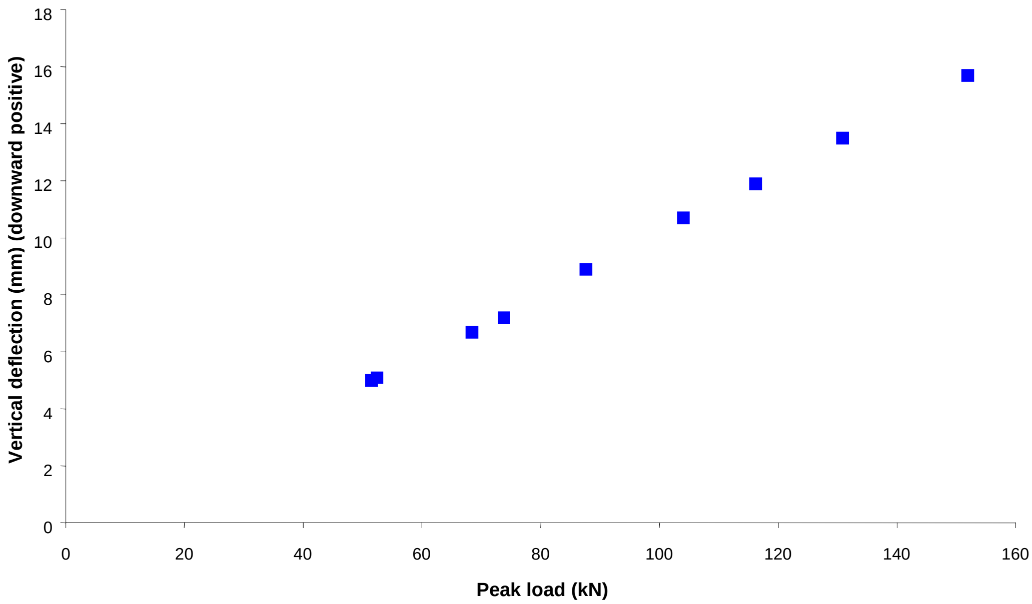


Figure 10 Relation between deflection and peak load

Figure 11 indicates reasonably linear relations between the strains recorded on the top and bottom flanges and the peak load. The maximum strain readings at the peak load of about 151 kN, were 946 uE (compression) recorded by gauge No. 3, and 508 uE (tension) by gauge No. 29.

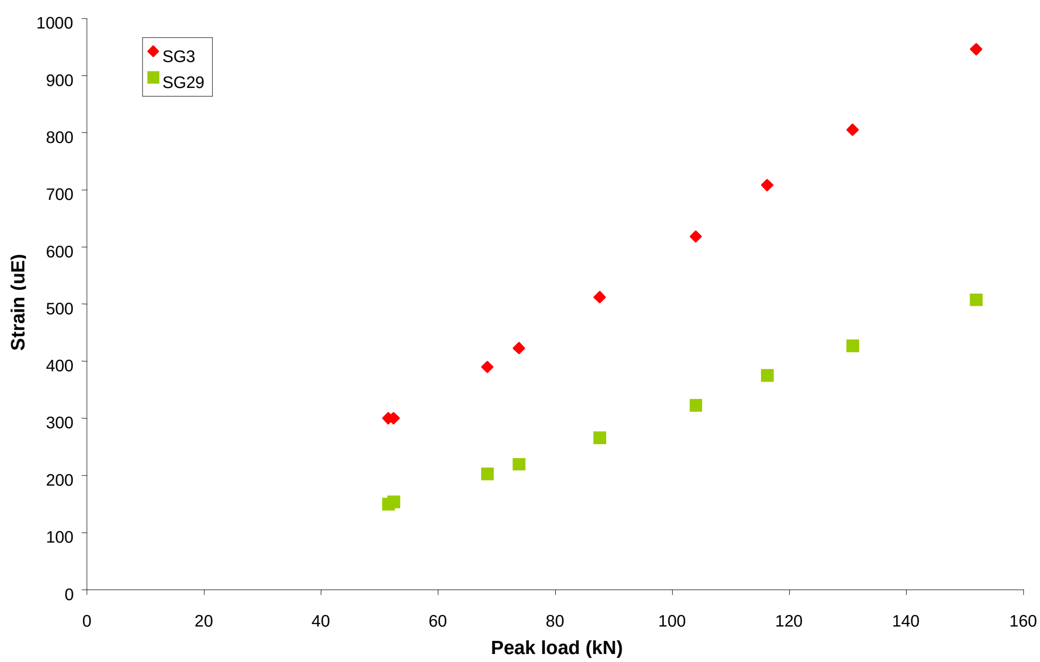


Figure 11 Relation between strain and peak load (position of instruments shown in Figures 7 and 8)

Figure 12 shows a plot of the cumulative AE count on the attainment of the peak load as a function of load. The data point for cycle 4, which was a repeat of the first cycle, has been omitted and a best-fit least squares straight line drawn through the others.

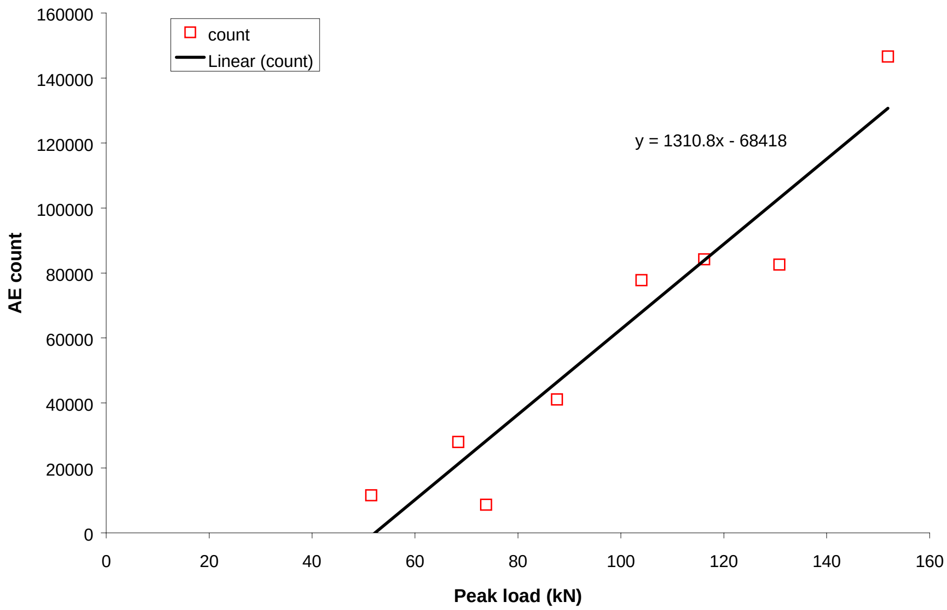


Figure 12 Relation between AE count and peak load

Figure 13 shows the cumulative AE count as a function of time. Were the rate of loading the same for each cycle (as it was supposed to be), the time scale would be proportional to load. The peak loads are defined on the figure, but note that minor variations in the loading rate led to some inconsistencies in the positions of the peak loads. A similar pattern of behaviour was detected by each of the four sensors but the two central sensors (numbers 2 and 3), and in particular number 2, recorded much greater levels of AE than numbers 1 and 4 (see Figure 6). In some cycles, sensor 1 detected noticeably greater levels of AE than sensor 4.

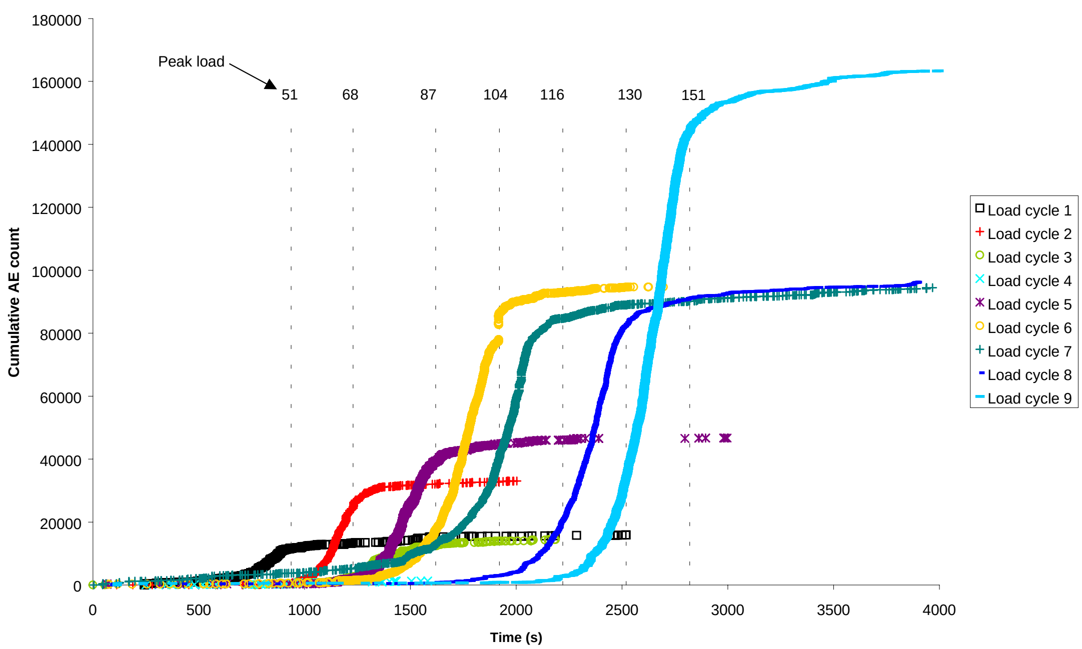


Figure 13 Cumulative AE counts for each load cycle

The data from the SoundPrint system are summarised in Table 3, and the calculated locations of the source of the events are shown in Figure 14. The error in the location of most of the events should be within 300 mm, but for a few might it might be up to 600 mm. Accuracy could be improved by changing the positions of the sensors and through modification of the software.

Cycle	Max test load (kN)	No. of possible cracking events	No. of observed cracking events	Total
1	51.2	0	0	0
2	68.2	1	0	1
3	73.4	0	0	0
4	52.3	0	0	0
5	87.3	4	0	4
6	103.7	0	1	1
7	115.6	8	0	8
8	130.3	2	0	2
9	151.2	4	0	4

Table 3 Number of cracking events detected during each load cycle

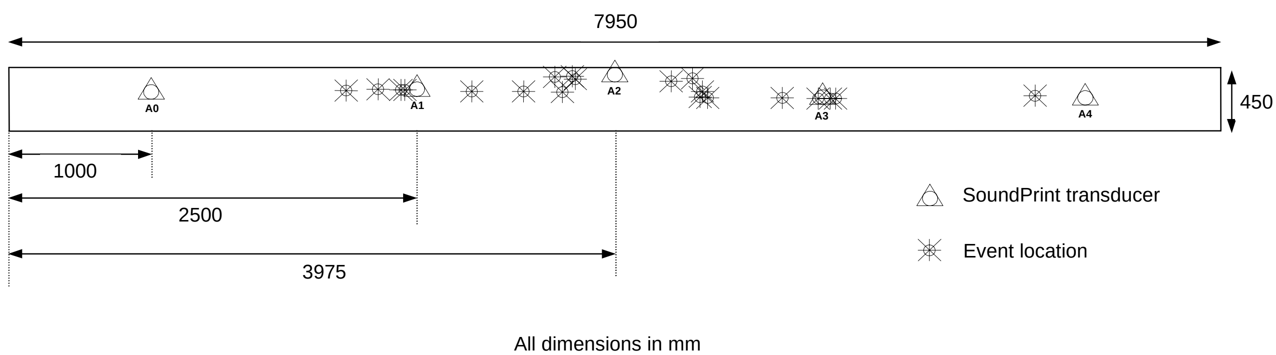


Figure 14 Location of acoustic emission

The fact that the AE sensors located at the centre of the beam recorded far more activity than those towards the ends, indicates that the supports were not a major source of emission. Similarly, the events recorded by the SoundPrint system were located towards the mid-span of the beam. It is of interest to note that the data from both the AE sensors and the deflection gauges showed a slightly asymmetric response of the beam.

Further details of the test and results are given in a TRL report by Kavanagh and Brady, to be published shortly.

DISCUSSION

Interpretation of AE Data

There is little doubt that the AE activity resulted from some load-related mechanism. Given the relation shown in Figure 12, it is probable that AE activity was associated with micro-fracturing of the cast iron beam, perhaps between the graphite flakes and the surrounding matrix. In which case, and as shown in Figure 13, AE activity should reduce as the beam reaches equilibrium with the applied load. Removal of the load would not be expected to generate much AE but some might be generated by the closure of fractures. During reloading, existing fractures would reopen, but this would not generate much activity until the previous maximum load had been exceeded. This model fits the data reasonably well but there are two apparent anomalies that require explanation.

Firstly, as shown in Figure 12, the cumulative AE count recorded at the peak load of about 73 kN in the third load cycle was (at 8,700) substantially lower than recorded (28,000) during the previous cycle where the peak load was 68 kN. It is relevant to note that the rate of AE generated following the attainment of the peak load in load cycle 3 was relatively low compared to the rate generated in other load cycles. Furthermore, the AE activity in the second cycle might have been boosted by the acoustic event recorded by the SoundPrint system (see Table 3). If the number of ‘substantial’ fractures is not a function of the applied load, the relation between AE and peak load will not be smooth. As shown in Table 3, the number of events recorded by the SoundPrint system did not increase uniformly with increasing peak load.

Secondly, the data presented in Figure 13 show that there is little difference in the cumulative AE counts recorded in load cycles 6, 7 and 8 for peak loads of 104, 116 and 131 kN respectively. A significant audible event was noted in load cycle 6, and eight events (individually of a lower magnitude) were recorded during load cycle 7. Also, in most of the load cycles the peak load was only maintained for between about 10 and 25 minutes but it was applied for 17 hours in load cycle 7. Data for this cycle show that the rate of AE was still reducing at the end of this time indicating that the beam had still not reached ‘equilibrium’ – at least in terms of micro-cracking and AE activity. By implication, the other load cycles were terminated well before an ‘equilibrium’ condition was approached (if indeed such a state would be reached). Thus, the relation between AE (however quantified) and load might be strongly dependent on the rate of loading and on the length of time a particular load is applied for. It is, therefore, feasible that the low AE count for load cycle 8 was due to damage inflicted in the previous cycle. This has important implications for the application of AE.

Figure 15 shows the relation between the rate of AE generated as the peak load is approached and the peak load. Note that the data points for load cycle 3, 4 and 8 fall well below the best-fit line: as discussed above, the relation is a function of load history and so could not be expected to be particularly smooth. Figure 16 shows the relation between the rate of AE generated immediately following the attainment of the peak load and the peak load. In contrast to Figure 15, the data points for cycles 1 and 4 fall close together in Figure 16, but the rate for cycle 8 is quite depressed. Note that two data points are shown for load cycle 7: the higher rate corresponds to the attainment of the peak load whilst the lower corresponds to the end of the loading cycle. The rates differ by an order of magnitude, thus again showing the time-dependency of the relation. Further tests are required to characterise the shape of the AE-load relations over much longer loading times.

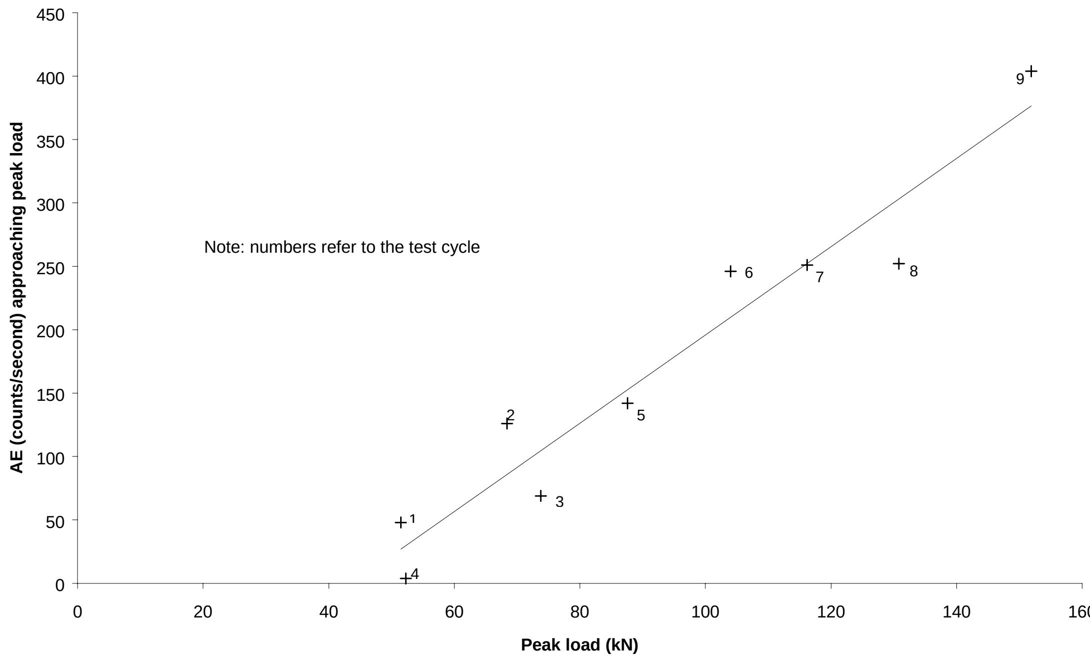


Figure 15 Rate of AE generation approaching peak load for each test cycle

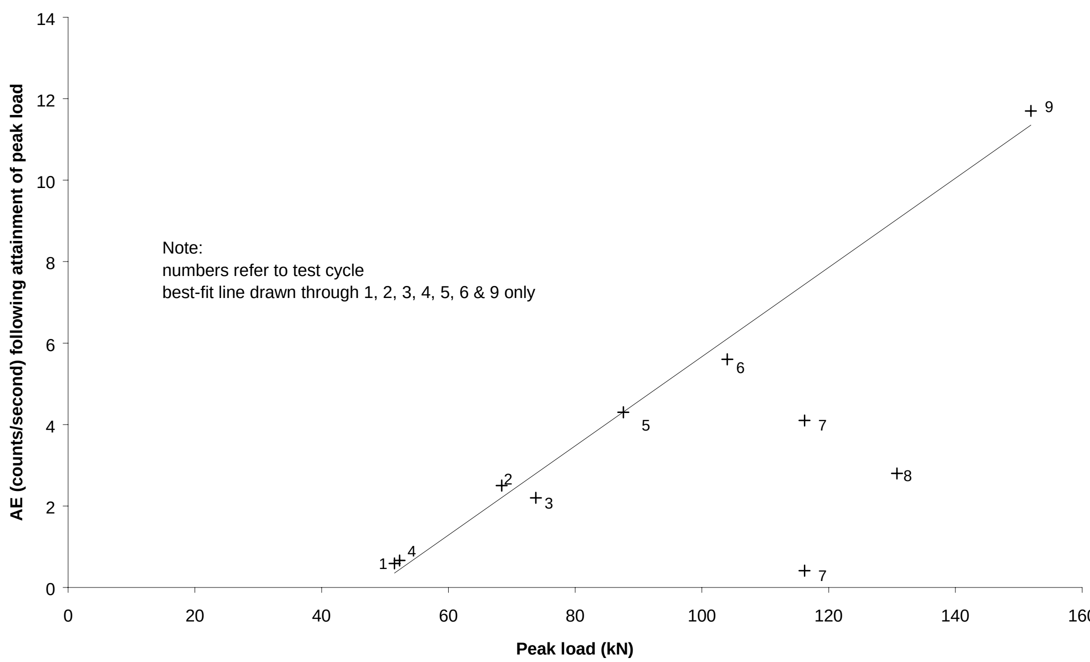


Figure 16 Rate of AE generation following attainment of peak load for each load cycle

Estimate of Maximum In-service Load

An estimate of the maximum load sustained in-service from measurements of deflection and strain might be based on the premise that there is some noticeable change in stiffness or some permanent deflection as that load is exceeded. But it is quite likely that, in-service, the test beam was within its 'elastic recoverable' range. According to the data from the test, some permanent deflection was noted at an applied load of about 70 kN. Furthermore, there was some small permanent strain induced by the applied load (more noticeably from the third load cycle onwards). A bi-linear relation fitted to the data points in Figure 11, such that the lower line passed through the origin, indicates a (slight) change of slope at a load of about 90 kN. However, the load-strain plots for the various cycles show no noticeable change in slope where a previous maximum load was exceeded.

At this stage, the damage commensurate with the AE activity measured during the test cannot be translated into other units (such as the extent of micro-cracking), but it would seem that such damage did not much affect the load-deflection nor the load-strain relation for the beam. This suggests that neither deflection gauges nor strain gauges would be of much use in determining the condition of a cast iron jack arch bridge.

The maximum in-service load might be fixed by the first substantial acoustic event, but there remains the problem of setting the trigger level for such an event. In this test the first acoustic event picked up by the SoundPrint system was at a load of about 68 kN.

Because of the effect of loading history it might be difficult to determine the peak in-service load from an analysis of the data from the PA AE equipment. Figures 12, 15 and 16 show relations between AE activity and peak load for the various load cycles. A number of the data points could be overlooked in interpreting such relations: nonetheless, extrapolations for various combinations of data points suggest that the maximum previous load sustained by the beam was between about 40 and 50 kN. However, it should be appreciated that,

(i) the load regime applied in the test and in-service were quite different, and
(ii) the AE relations are time-dependent and whilst the beam was in-service for more than a century the first load cycle was completed in about an hour or so.

An examination of the data from the first and second load cycles shows that the plots for the former have a much more pronounced S-shape than of the latter. Much more AE activity took place at low loads during the first load cycle than in any of the other cycles. It is, however, difficult to decide what rate or change in rate of AE could be taken to represent a change in performance that might result from the application of a new peak load. The assumption that no (significant) AE is generated until the previous peak load is approached leads to the conclusion that the maximum in-service load was less than about 20 kN.

<u>Estimates of Collapse Load</u>

It is improbable that a load-deflection relation, as shown in Figure 10 for example, could be used to predict the collapse load of a jack arch structure. A brittle failure might show as a dramatic, and perhaps unpredictable, discontinuity. A plastic failure would be preceded by an upturn in the relation, but such a change in slope might show that the structure has been weakened (and a load test should not venture into that territory). Such a relation might be extrapolated to determine the (serviceability) load for some limiting and acceptable deflection. For example, according to Figure 10 a limiting deflection of 1/1000 of the clear span of the beam (7 metres) is generated at a load of about 70 kN. It should be appreciated that rarely would such an extended load test be feasible for an in-service structure. Furthermore, the relation for an in-service jack arch structure would be substantially affected by the interaction of the beams, brickwork, and backfill.

An estimate of the collapse load might be based on measured strains, but there are a number of interacting factors that make it well nigh impossible to do this, including,

(i) the origin of the relation cannot be defined for a structure,
(ii) the distribution of load might not be known well, or be controllable,
(iii) the point of maximum strain might not be identifiable - and if it could it might not be possible to install strain gauges at that point,
(iv) the shape of the load-strain relation for any particular point on a structure will vary according to the pattern of loading, and might not be known a priori,
(v) due to the presence of flaws, the load-strain response of a beam might be quite different to that of a small-scale laboratory specimen.

According to Swailes (1995), the mean ultimate tensile stress of cast iron is 108 N/mm^2 and is developed at a tensile strain of about 0.0017. At the maximum test load of 150 kN the maximum tensile strain recorded by the gauges was about 0.0005. This suggests that there was a large margin of safety at this load, but the gauge might not have recorded the maximum strain generated on the beam.

As shown in Figure 12, AE activity increased, more or less, with increasing load. If activity were related to the size or number of micro-cracks, which could be taken as a measure of damage, it might be possible to extrapolate the load-AE relation to assess the collapse load. Problems inherent in the data obtained from this test include the apparent time-dependency of such relations, and that the number of significant acoustic events did not increase in any definable pattern with the applied load. Furthermore, (a) it is difficult to base predictions of failure on a linear relation and (b) setting the level of AE to represent an 'allowable level of damage' is problematic. Even if a level could be set for this beam, or a range of beams, it would not necessarily be relevant to an in-service structure whose response is affected (perhaps overwhelmingly so) by its interaction with the surrounding backfill and brick arches.

At this stage, it is not known whether the AE activity generated by reloading was due to the opening of existing micro-cracks, the extension of existing cracks or the creation of new cracks - or what combination of these occurred. In some load cycles, AE activity was not negligible during reloading. This might indicate that cast iron beams are susceptible to cyclic loading: indeed Swailes (1995) reported a fatigue failure of a cast iron structure. Long-term experiments are required to assess whether or not the rate of AE during reloading could be related to the residual fatigue life of a structure.

In-service Condition

The aims of a structural assessment are to check that the structure in question is performing adequately and that it has an acceptable margin of safety against collapse. But assessment via calculation alone is complicated by various structural interactions, which might not be understood particularly well. Furthermore, there might not be relevant information on structural form and material properties; such problems could be expected to increase with the age of a structure. Instrumentation can be installed to quantify in some way (such as through measurements of deflection) in-service performance, but there remains the problem of fixing a weight restriction at an appropriate level. Although strain and/or load-measuring devices and deflection gauges could have some role to play they might not warn that a structure is deteriorating. On the other hand, AE techniques can provide information on the damage sustained in service, but the data from AE devices has to be 'calibrated' to be of direct use.

In many cases, the only way of estimating the level of safety of a structure is through analysis of a load test as described, for example, by Daly and Raggett (1991). Analysis of the data from such tests is complicated by structural interactions and there are also limits on the load that can be applied. Owners might be concerned that load tests would damage, and hence weaken, the structure. Again, measurements of strain and deflection might have some role to play here, but they only provide information local to the position of the instrument. As shown herein, AE has a good potential for site monitoring works, and might, therefore, reassure an owner of the safety of undertaking a load test.

RECOMMENDATIONS FOR FURTHER WORK

It is necessary to conduct further investigations to confirm the findings of this test as well as to investigate the practical application of the equipment to an intelligent monitoring system.

The next step would be the investigation of the time-dependency of the relation between AE activity and load. A particular load might have to be applied for several days, if not weeks, to determine whether or not an 'equilibrium' state was achieved. Depending on the results obtained, it might be possible to load a beam to failure, or close to it, to determine how AE activity changes as failure is initiated.

As a second step, a load test could be undertaken to investigate the AE response at load levels up to and slightly in excess of the maximum in-service load. Again, the load cycles need to be over a much longer period than used in the test described herein. This might provide an insight to the change in AE response at and around a load equivalent to the maximum supported in-service.

In either of the above, if not both, the AE response to cyclic loading (to simulate live loading) should be examined.

Following the above, the AE activity of in-service structures could be monitored - these need not be structures deemed to be at risk of failure. Opportunity should also be taken to gather AE data from load tests.

CONCLUSION

A series of nine load cycles with a generally increasing peak load, were conducted on a cast iron beam recovered from an in-service structure. The beam, instrumented with deflection gauges, strain gauges and acoustic sensors, was loaded to about 150 kN, and in excess of the 'equivalent' maximum in-service load. The results of the load test show that the response of the beam was slightly asymmetric, and the load generated some small permanent deformation of the beam.

The data captured by the PA AE system generally recorded increasing activity with increasing peak load and, almost always, substantial activity was only recorded once the previous peak load was approached. The activity seemed to be related to the time of application of the peak load. The rate of AE reduced following the attainment of the peak load, it reduced further during unloading and all but stopped on removal of the load. The SoundPrint system recorded a number of acoustic events: one of these was audible and also recorded by the PA AE system.

The results of the test strongly support the view that AE was generated via some load-related mechanism, perhaps related to micro-cracking of the cast iron. Further research is required to confirm and extend the findings of the test, but the results show that AE techniques have the potential for forming the basis of an intelligent monitoring system for cast iron structures.

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