VIBRATION TECHNIQUES TO ASSESS THE STRUCTURAL CONDITION OF MASONRY ARCH BRIDGES

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1. INTRODUCTION

A simple empirical method for assessing the capacity of masonry arches to carry traffic was developed by the Military Engineering Experimental Establishment (MEXE) in the 1940's. It was based on Pippard's theory [1], and various experimental studies. Details of the method, as currently used, are set out in [2]. In the method, a permissible axle load is calculated from the span, ring thickness and depth of fill, using a nomogram. This value is then modified by a series of factors which takes into account the actual shape of the arch, the type and condition of the materials, joints and workmanship and the presence and position of cracks. This leads to a value for weight restriction, if necessary. There is some concern that this may not be an accurate indicator. Consequently, some bridges may be unnecessarily strengthened or have a weight limit imposed, while others may be put at risk because no action has been taken. Generally, the method is reckoned to be conservative, but it only assesses the load carrying capacity of the arch barrel which, in some cases, will be affected by the strength of the spandrel walls, wing walls and foundations.

At Reading, vibration techniques have been investigated, for possible use in conjunction with the MEXE assessment. The object is to detect internal faults not revealed by visual inspection and to differentiate between serious and superficial faults. Such faults may be expected to affect the stiffness, damping and linearity of the structure. Initially, random traffic excitation was used, requiring a minimum of equipment on the bridge during tests. Spectral techniques were used in the analysis of the data collected and a small condition monitoring programme was set up, in which five bridges were tested over a period of eighteen months. These bridges were all thought to be deteriorating and so it was hoped that any significant damage would result in a loss of stiffness with a corresponding detectable change in natural frequency. However, this proved impossible as the frequency spectra were so coloured by the nature of the traffic. This led to a study of the responses arising from "standard vehicles" (a 2-ton van and a 16-ton lorry). A total of seven bridges were traversed, in both directions and at various speeds. Correlation methods were developed to compare the response signals, but the measured resonance frequencies were found to be too dependent on such factors as the vehicle, speed, acceleration, line of action and direction of travel and no useful information about the bridge could be gained.

A fresh approach was required. A test was needed that would give a repeatable response that was characteristic of that bridge and no other. Also, from the point of view of a local authority, a contribution from this work will be most useful if it is based on a simple one-off test rather than on tests from a series of visits. For this purpose, an impulse tester was developed which dropped a mass through a fixed height. This paper gives an account of these experiments. The programme of work has concentrated on one-off impulse tests rather than on condition monitoring programmes, with the following objectives in mind:

a) To measure the response of a bridge to a standard impulse and to analyse the result in different ways with the aim of verifying and, perhaps, supplementing the MEXE assessment. This is to include multiple tests for linearity assessment.

b) To compare velocity measurements with those in the German standard DIN 4150 [3], in order to assess the rate of damage of the bridge.

2 TEST PROGRAMME

The present impulse tester can be seen in Figure 1. The mass, which can be varied from 12.5 kg to 75 kg in six steps, is lifted by pulley and held by a bomb release on a tripod system. It is dropped through 1m on to a thin bed of sand that has been placed on the bridge pavement. The impulse time has been made short (10 ms) compared with the fundamental period for this type of structure (usually about 50 ms).

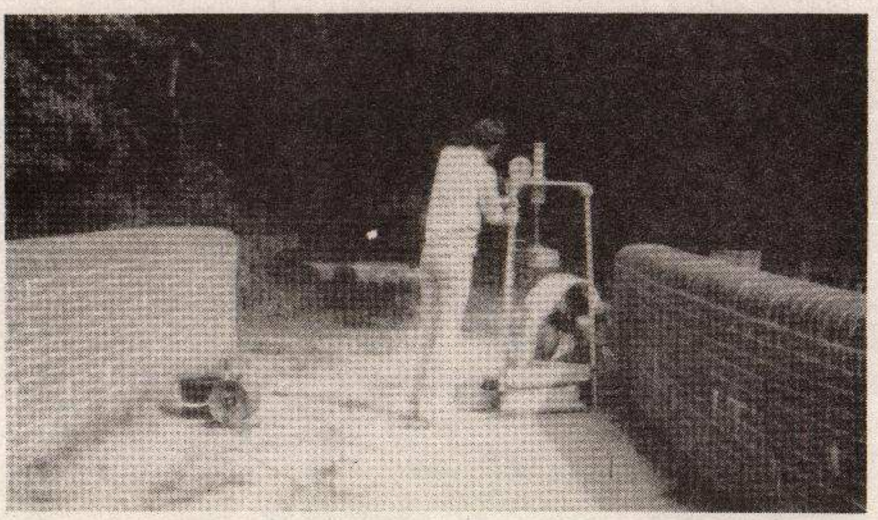

Figure 1.

The resulting movement is detected using a seismometer, positioned by the opposite parapet. At the moment, the output is recorded on tape, and is taken back to the laboratory for digitization and computer analysis, but the system could be streamlined by directly digitizing the seismometer signal. The list of bridges tested, along with their condition and span, is given in Table 1.

A series of tests on a number of arches showed that each one produced a distinctly characteristic response. A range of different types of analysis was used, including spectral analysis. These are described in section 3. The responses from different arches were compared in an attempt to relate condition and bridge dimensions to any of the response characteristics. Three of the bridges (8, 9 and 10) were along the same canal and were of particular interest, being very similar in both construction and materials but in different states of repair.

2.1 Repeatability : To be meaningful, results need to be repeatable. Each test is repeated at least three times and the resulting traces compared using correlation techniques. In general, good correlation was obtained, (correlation coefficients of around 0.99), showing that the traces resembled each other very closely. The average was then taken as being characteristic of this test. There was more discrepancy found at the lowest weights, mostly due to the weights not falling squarely. Repeatability was further checked by repeating the whole test on three bridges over a period of three months. The final test was made when the bridges were partially frozen and this gave an indication of the degree of change that might arise from serious damage, see Figure 2. This change was much greater than the changes due to "natural" variability. Correlation between visits was not as good as within visits, (correlation coefficients above 0.98).

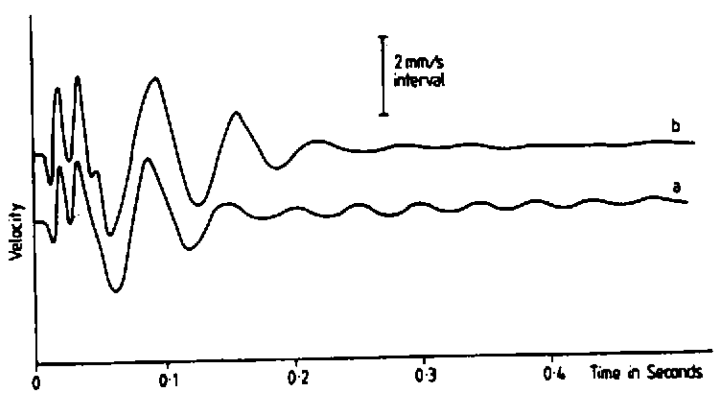

Figure 2: Responses From Bridge No. 7 a) When Partially Frozen, b) As Normal.

2.2 Linearity : To investigate the linearity of bridge response, the full range of drop masses was used giving a set of response signals for each bridge. Tests were carried out on a number of bridges and, again, repeatability needed to be established. An obvious cause of non-linearity would be the closing of cracks as the response increases, making the structure stiffer. Figure 3 shows the range of linearity obtained. In order to quantify comparisons, an arbitrary index of linearity was required. The simple one used here was calculated as the ratio of final and initial gradients. This linearity index, L, has the advantage of being 1.0 for a linear response, decreasing to zero for responses that are increasingly non-linear. It has the disadvantage of not accounting for the detailed shape of the linearity curve.

3. SIGNAL ANALYSIS

A typical average response waveform is shown in Figure 2a, and this illustrates how some of the following features have been derived.

3.1 Peak-to-peak velocity : The maximum value of peak-to-peak velocity is not only easy to find, but also may give an indication of the rate of damage of the bridge. In DIN 4150, it is suggested that this value should not exceed 4 mm/s for ancient structures. All structures in this study were built before 1940 and some are more than 200 years old.

3.2 Root mean square velocity : This is calculated by taking a root mean square over 1 second, which is roughly equal to the duration of the transient response. This period has to be kept the same if valid comparisons are to be made. The root mean square velocity is less variable than the peak-to-peak and gives a good indication of the general level of the response.

3.3 Natural frequency : Using FFT analysis, a frequency spectrum for each drop-mass at each bridge can be found. The position and shape of the peaks, particularly the fundamental, can be compared. Damping coefficients can also be found for well-defined peaks, although results of this analysis are not yet available. The linear frequency range of the instrumentation used to measure vibration is 1 to 50Hz. This includes most of the relevant natural frequencies for the arches in this study.

4. RESULTS

Bridge Number	d mm	c mm	b m	Span S m	CF	MAL tonnes	L	P-P6 mm/s	Comments
1	740	830	10*	25.8	N/A	N/A	0.66	2.30	reinforced concrete
2	355	1064	5.15	5.26	0.9	38	0.21	3.33	sandstone
3	457	737	5.5*	9.40	0.9	14	0.63	4.92	sandstone
4	470	800	8.5*	8.28	1.0	12	0.09	3.60	brick railway arch
5	405	1230	7.08	6.70	0.65	12	1.0	2.27	sandstone, 6 ties
6	335	830	6.5*	6.03	0.7	12	0.28	1.96	brick
7	565	785	5.3	8.62	0.7	7.5	0.62	5.51	brick, shallow arch
8	340	760	2.93	6.57	0.8	12	0.15	3.41	brick canal bridge
9	353	648	6.17	6.55	0.8	11	0.42	4.69	brick canal bridge
10	340	740	9.27	6.55	0.6	8.5	0.45	1.34	brick canal bridges

where d = arch ring thickness c = depth at the crown
b = breadth CF = condition factor
MAL = modified axle load, as calculated in the MEXE assessment
L = linearity index, as defined in section 2.2
P-P6 = peak-to-peak velocity with a drop-mass of 75kg (6 mass units).
* These figures are estimated.

Table 1: Summary of the dimensions and results obtained from the standard impulse test for the ten bridges.

5. DISCUSSION OF RESULTS

5.1 Linearity : Figure 4 shows a graph of linearity index (L), plotted against the condition factor from the MEXE assessment. The condition factor depends on visible defects in the arch barrel which affect its load carrying capacity. These include longitudinal cracks due to differential settlement of the abutments, lateral cracks or permanent deformation of the arch which may be caused by partial failure of the arch or movement at the abutments and diagonal cracks which may be due to subsidence at the sides of the abutment. Condition factor might be expected to correlate with linearity. However, on the basis of these tests, this assumption appears to be invalid. There is the intriguing possibility that the hypothesis is actually correct and that the condition factor may be meaningless because it only takes into account superficial condition. At the moment, these results are inconclusive. Further work will be valuable if it can pinpoint the cause of non-linearities in vibration response.

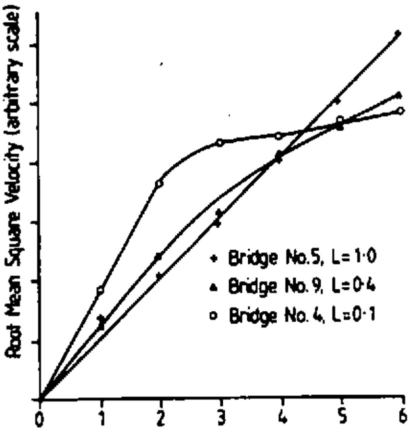

Figure 3.

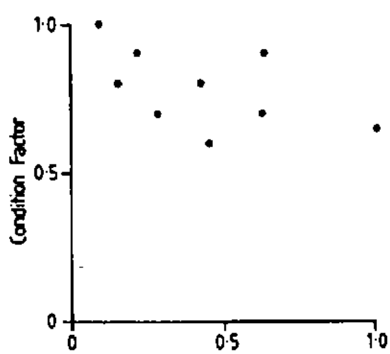

Figure 4.

5.2 Peak-to-peak velocity responses : The 75kg drop test (P-P6 on Table 1), has been shown to give a response roughly equivalent to that obtained from the passage of a 10-ton lorry. The peak-to-peak values can be directly compared with the DIN 4150 limit of 4mm/s, and Bridges 3, 7 and 9 exceed this, but only by a small margin. The modified axle load (MAL on Table 1), can lead to a weight restriction for the bridge. The minimum MAL is for Bridge 7, for which the recommended restriction from MEXE is 17.5 tonnes. Several other bridges have low MAL values, including Bridge 9, but not Bridge 3. There is thus some correlation between peak-to-peak response and MAL, but it is not strong.

5.3 RMS Velocities: The bridges are assumed to be effectively linear up to a drop-mass of 25kg, (2 mass units). The RMS value for this drop-mass has therefore been included in Table 2 (column RMS2), as it may indicate something of the linear behaviour of the structure. In particular, it may indicate stiffness values considerably different from a norm. The nature of the test is such that the arch responds significantly in both arch ring bending and torsion and these two elements are present in the response traces. Because of the multi-modal behaviour, the general level of response will depend on mass, stiffness and damping and, usually, the greater these factors, the less the response. The raw RMS2 data needs to be modified to take account of the bridge dimensions and materials. Damping will be omitted from this discussion as this has not yet been studied.

Bridge	RMS2	MF	SF $\times 10^{-4}$	RMS2 x MF $\times 10^{2}$	RMS2 x SF $\times 10^{-3}$
1	3.96	214	2.4	8.5	0.9
2	6.23	29.2	16.3	1.8	10.2
3	7.58	38.1	6.5	2.9	4.9
4	7.35	56.4	15.4	4.2	11.3
5	3.68	58.5	16.3	2.2	6.0
6	4.12	32.4	11.8	1.3	4.9
7	14	36.0	15.4	5.0	21.6
8	7.03	14.5	4.0	1.0	2.8
9	4.8	26.6	9.3	1.3	4.4
10	2.76	45.4	12.7	1.3	3.5

Table 2: Values for root mean square velocity, mass and stiffness factors (for definitions see text).

On the basis of Pippard's theory, and other considerations, the effective bending stiffness and mass of an arch bridge may be expected to be approximately proportional to:

stiffness: $b.d^3/S^3$ = SF (stiffness factor)
mass: $b.c.S$ = MF (mass factor)

where S is the span and the other factors are as defined on Table 1.
If the RMS2 data is multiplied by the SF or MF, this will normalise it with respect to the particular bridge. Table 2 shows the result of this calculation, (the actual units are arbitrary). The final two columns need to be looked at for abnormalities. Bridge 1 is obviously abnormal, but this can, to some extent be explained by the fact that it is a long span, shallow, reinforced concrete arch. All of the other bridges are short span and of traditional brick or masonry construction. Bridge 1 is now left out of further consideration. The mass corrected values are then seen to be reasonably consistent at a value of about 2, except for Bridge 7 and possibly Bridge 4. The stiffness-corrected values are less consistent but Bridge 7 is again particularly high. High values in these two columns indicate a higher than normal response. A similar analysis has been conducted for torsional motion of the arch ring, but, generally, this is less conclusive. However, it also points to a possible weakness in Bridge 7. A satisfying aspect of the data in Table 2 is the close consistency of results for the three similar canal bridges (8, 9 and 10).

Bridge	1 Test Bending Frequency Hz	2 BBF	3 Ratio	4 Test Torsion Frequency Hz	5 TTF	6 Ratio
1	6.3	1.05	6.0	19.4	2.71	7.2
2	23.9 ?	7.47	3.2	39.6 ?	7.61	5.2
3	11.8	4.10	2.9	-	7.02	-
4	-	5.23	-	44 ?	5.11	8.6
5	14.6	5.27	2.8	25	4.98	5.0
6	17.4	6.04	2.9	26.4	5.58	4.7
7	15.3	6.55	2.3	-	10.6	-
8	15	5.22	2.9	23.6	11.9	2.0
9	15	5.90	2.5	15	6.28	2.4
10	15	5.29	2.8	11.8	3.75	3.1

Table 3: Test frequencies and calculated factors for bending and torsion.

5.4 Natural frequencies : Some difficulties have been experienced in the analysis of frequency spectra for many of the bridges in this study. Although it is often obvious that two modes predominate, (see Figure 1), it is not clear which is bending and which is torsion. The data in Table 3 is, therefore rather tentative in some respects. Once again, it has been necessary to attempt some normalisation of results, using Pippard's theory and other sources. The natural frequencies in bending and torsion are expected to be approximately proportional to:

$$\text{bending frequency : } d^{\frac{3}{2}}/c^{\frac{1}{2}}S^{2} = \text{BFF (bending frequency factor)}$$

$$\text{torsion frequency : } d^{\frac{3}{2}}/c^{\frac{1}{2}}bS = \text{TFF (torsion frequency factor)}$$

The symbols have the same meaning as before. The data is presented in Table 3 and question marks indicate uncertainty as to modal attribution of the measured natural frequencies. Columns 3 and 6 show the ratios of measured frequencies to the frequency factors given above. The results in column 3 show a remarkable degree of consistency with the exception of Bridge 1 which may be disregarded on the grounds given before. Bridge 7 gives the lowest ratio, which may indicate lack of stiffness, though the indication is not nearly so clear as it has been in discussing response levels. Column 6 in Table 3 shows rather less consistency and is of less value because of the lower reliability of the data and the apparent absence of some of the values. In the case of Bridge 7, some measurements indicate a torsion frequency at about 64Hz, but because this is beyond the normal range of the instrumentation, the value has been discarded.

6. CONCLUSION

The analysis of impulse response curves for arch bridges has shown that there may be some useful indicators of damage which can be defined. The consistency of prediction of the arch ring bending natural frequency is satisfying and response levels are also useful in two distinct ways. The first is to make a comparison with the suggested velocity limit of 4mm/s from DIN 4150; the second is to check whether the level is abnormally high when compared with the response of other similar structures. All of these techniques have consistently shown one of the bridges to be in need of a more careful assessment.

When there is such a wide variety of materials and quality in existing arch structures, it is not suprising that some of the work described here has not led to consistent results capable of being used in assessment procedures. The work on linearity, in particular, has presented a confused picture. Further techniques also remain to be assessed. For example, one is the damping of impulse-excited vibrations; another related technique is the use of the Hilbert Transform to calculate the envelope functions of the response. When it becomes clear that some of these techniques can be usefully applied, there will then be a need to develop a simple test system for field use.

7. ACKNOWLEDGMENTS

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8. REFERENCES

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