THE EFFECTS OF TRAFFIC INDUCED VIBRATIONS ON HERITAGE BUILDINGS—FURTHER CASE STUDIES

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## CONTENTS

| Abstract |
|----------|---|---|
| 1. Introduction |
| 2. Generation and propagation of traffic vibration |
| 3. Selection of buildings |
| 4. Measurement method |
| 4.1 Vibration measurements |
| 4.1.1 Instrumentation |
| 4.1.2 Structural vibrations |
| 4.1.3 Window vibration |
| 4.1.4 Crack movements |
| 4.2 Noise measurements |
| 4.3 Measurement of road surface irregularity |
| 5. Surveys |
| 5.1 Building surveys |
| 5.2 Traffic surveys |
| 5.3 Soil surveys |
| 6. Analysis of vibration data |
| 6.1 Time domain |
| 6.2 Frequency domain |
| 7. Results |
| 7.1 Vibration levels |
| 7.1.1 Vibrations at foundations |
| 7.1.2 Amplification of vibrations |
| 7.1.3 Source of vibrations |
| 7.1.4 Noise levels |
| 7.1.5 Window vibrations |
| 7.2 Dynamic crack movements |
| 7.3 Road surface movements |
| 7.4 Building surveys |
| 7.4.1 Building A—town house |
| 7.4.2 Building B—church |
| 7.4.3 Building C—timber-framed cottage |
| 7.4.4 Building D—farmhouse |
| 7.5 Traffic surveys |
| 7.6 Soil surveys |
| 8. Discussion |
| 8.1 Exposure to vibration |
| 8.1.1 Radial component of vibration |
| 8.1.2 Vertical component of vibration |
| 8.2 Window vibration |
| 8.3 Causes of damage |
| 9. Conclusions |
| 10. Acknowledgements |
| 11. References |
| 12. Appendix—Reports of the structural surveys by the Historic Buildings and Monuments Commission for England |
| 12.1 1 and 3 Widcombe Parade, Bath |
| 12.2 The parish church of St James, Louth |
| 12.2.1 Comparative study of windows |
| 12.3 The Old Bell Cottage, Norton |
| 12.4 Rising Sun farmhouse near Honiton, Devon |

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THE EFFECTS OF TRAFFIC INDUCED VIBRATIONS ON HERITAGE BUILDINGS—FURTHER CASE STUDIES

ABSTRACT

The effects of traffic induced vibrations on heritage buildings have been studied in four widely different buildings. Three stone-built and one timber-framed building, including a Georgian town house and a large 15th century church, were examined. As part of the study a large farmhouse was surveyed before and after a significant increase in lorry traffic to determine if the increased exposure to traffic vibration had triggered damage. This study extends and complements a previous examination of four relatively small brick built properties reported in RR156 (Watts 1988). All buildings were within a few metres of roads carrying HGVs and they all showed some signs of distress. Vibration and noise measurements were made at the sites in order to characterize the exposure to vibration. All building surveys were carried out by the Historic Buildings and Monuments Commission for England whose main objectives were to report on the state of dilapidation of each building and to determine whether any damage was the result of traffic vibration. It was concluded that although measurable levels of ground-borne vibrations were present in all buildings, reaching perceptible levels in some, the observed damage was likely to have been caused by other site factors rather than the exposure to traffic vibration.

1 INTRODUCTION

Residents living alongside heavily trafficked roads are often concerned about the damaging effects of traffic vibration on their homes (Watts, 1984). There is also a widespread belief that old and structurally weak buildings are particularly at risk from these vibrations (Civic Trust, 1970; Crockett, 1973). A previous report (Watts, 1988) described the first phase of an investigation into the possible link between traffic vibration and building damage. Four grade II listed buildings were studied and all were brick built properties used wholly or partly as private dwellings. In two cases control properties of similar construction and age were also studied in order to aid the assessment of the causes of damage. It was concluded that the observed damage could more plausibly be attributed to factors other than traffic vibration. In this second phase a much wider range of building types in terms of age, size and construction was selected for study and one of the buildings was monitored before and after a substantial increase in lorry traffic. In these ways it was expected that further insights would be gained into the role of traffic vibration in causing damage to buildings of widely different types.

As in the first phase buildings were sought close to roads carrying substantial lorry traffic and where there were obvious signs of building distress. In addition there was a requirement for a site where lorry traffic would increase over the study period. The objectives were firstly to quantify the vibration exposure of various building elements and determine the dominant modes of vibration, secondly to determine the condition of the buildings and lastly to establish as far as was possible the main causes of any observed damage.

The Historic Buildings and Monument Commission for England (English Heritage) who have responsibilities for historic buildings and monuments of national importance collaborated in this research by carrying out structural surveys of the buildings, monitoring crack movements and reporting on their observations and conclusions concerning the effects of traffic vibration.

2 GENERATION AND PROPAGATION OF TRAFFIC VIBRATION

Passing vehicles can induce building vibration in two major ways. Low frequency noise produced by vehicle engines has dominant frequencies in the 50–100 Hz range and enters buildings readily through doors and windows. It can produce vibrations in building elements particularly if they are light and flexible. High levels of vibration can be measured on window panes fronting heavily trafficked roads and this can produce annoying rattles (Watts, 1984). At the most exposed locations, acoustically induced floor vibrations can become perceptible but vibration levels in the hard structure of the building are generally at a much lower level (Watts, 1987b).

Ground-borne vibration has dominant frequencies in a lower frequency band, typically 8–20 Hz. These vibrations are produced by the varying forces between tyre and road and become perceptible in buildings near the road if the axle loads are high and the surface is uneven. Both compression and shear waves are produced and their amplitudes and attenuation with distance depend on a number of factors including the soil composition and the nature of geological layers beneath the surface. Since this vibration enters buildings through the foundations, the hard structure of the building is normally affected to a greater degree than is the case for airborne vibrations.
There are two further mechanisms for the generation of traffic induced vibration that need to be considered where buildings are situated very close to the carriageway. Firstly, the air pressure pulses caused by the bow and stern waves of large vehicles passing at speed and secondly the quasi-static depression of the road surface under the weight of heavy vehicle axles may induce small, short duration vibrations or movements at the front wall. The frequency of any vibration would be expected to be related to the length of the vehicle and its speed and probably would not exceed 3 or 4 Hz in most cases. At one site where site conditions were considered to be conducive to observing these movements, further measurements and analyses were carried out to indicate the scale of the effects.

3 SELECTION OF BUILDINGS

One aim of this second phase of study was to determine the effect of traffic induced vibrations on a greater range of building types and this was achieved by including a timber-framed dwelling and three stone built structures of widely different ages and proportions. Although it was not possible to find similar buildings not exposed to traffic vibration at each site to aid the interpretation of observed damage, it was possible to study the effects of vibration on a two storey building before and after a large increase in lorry traffic. In this way any short-term damage triggered by passing traffic would be readily detected. In addition, because one of the buildings was very large (a church over 60 m long) it was feasible to study damage to parts of the building which were exposed to different levels of vibration. This was particularly relevant to the study of vibration damage to the stained glass windows which at the east end were exposed to significantly high levels of traffic vibration.

At three sites the buildings selected were all within a few metres of roads carrying high lorry flows and traffic vibrations were perceptible at the kerbside or inside the buildings so that exposure was known to be relatively high before measurements commenced. At the fourth site the building was situated close to a quiet country lane carrying only a few two axled heavy vehicles per hour and occasionally an articulated goods vehicle. Although vibrations at this site were not perceptible it was expected that following the opening of a gas pipeline store within 100 m of the house there would be a large growth in the numbers of very heavy vehicles passing the site resulting in increased exposure.

Full descriptions of the buildings and their state of dilapidation are given in the building survey reports in the Appendix and Figure 1 shows simplified plans for each building. Building A is a four storey Georgian terraced town house situated in Widcombe Parade, Bath (see Plate 1). It was built in stone c1760 probably as a private dwelling house but later converted to shop premises on the groundfloor. It is situated 3 m from the A36 which is a two lane one way system at this point. Building B is the parish church of St James in Louth in Lincolnshire. It is a very large building dating from 1430; the spire was added in the 16th century. The east wall containing large stained glass windows adjoins the A16 and the buttresses are less than 1 m from the carriageway (see Plate 2). The road is so narrow at this point that traffic lights control vehicle flow in one direction at a time. Building C forms the south end of a T-shaped group of timber framed buildings at Norton, near Evesham, and is basically a rectangular two storey dwelling. It was probably built in the 15th century with the porch and bell cote added in the 19th century. There is a mid 20th century two storey extension at the rear. The east gable wall is timber framed with lath and plaster infill panels and is situated only 2 m from the heavily trafficked A436 (see Plate 3). Building D stands at a crossroads on an unclassified road about one kilometre south of the A30 leading to Honiton. It was built as the ‘Rising Sun’ inn at the beginning of the century and has since been converted to a farmhouse. It is a two storey detached rectangular building with a front entrance porch only 3.4 m from the edge of the carriageway. The external walls are constructed mainly from flint and stone. Plate 4 shows a pipe delivery lorry passing close to the building, at a gross weight estimated to be over thirty tonnes.

4 MEASUREMENT METHOD

To determine the exposure of the buildings to vibration, peak amplitudes and rms values of particle velocity and acceleration were measured at various parts of the building. Where possible dynamic crack movements were recorded to supplement this information. The frequency content of the vibrations was also computed to indicate the dominant source of vibration. Noise measurements were made at the facades of the buildings to quantify the level of low frequency noise, and measurements were made of the profiles of significant road surface irregularities which were considered to be chiefly responsible for ground-borne vibrations.

4.1 VIBRATION MEASUREMENTS

4.1.1 Instrumentation

Peak particle velocities (PPVs) of major building elements were measured in three orthogonal directions. Long travel geophones were employed with a sensitivity of 28mV per mm/s and with a level frequency response down to 5 Hz. The geophones were bolted to the sides of an aluminium cube and in most cases attached to the measurement surfaces with Plaster of Paris. The signals from the geophones were suitably conditioned by operational amplifiers
Plate 1 Georgian town houses in Bath. Measurements taken in second house from far end of terrace (Site A)

Plate 2 East wall of church at Louth (Site B)

Plate 3 Timber-framed cottage at Norton, near Evesham (Site C)

Plate 4 Farmhouse with pipe delivery lorry passing close to front porch (Site D)
which provided the input to an intelligent interface unit (CED 1401) driven by a microcomputer which was programmed to calculate PPVs on up to 16 channels and could provide printed time histories if required. It was also capable of sampling continuously on one channel and computing the number of vibration events which exceeded given levels. Because of space constraints at site C it was not possible to operate the equipment indoors and so an FM instrumentation tape recorder, set up in an estate car, was used to acquire these data. This recorder was also used at other sites to record vibrations continuously during 15 min periods so that rms levels could be later calculated. At site D, where vehicle flow was low, the rms value was very small and so only peak values were recorded as individual heavy vehicles passed by.

4.1.2 Structural vibrations
At each building, measurements were taken at foundation level on the front wall close to the major source of ground-borne vibration (position 1 in Figure 1) which was either a poorly backfilled trench, sunken drain cover or patch of repaired road surface. Simultaneous measurements were taken at various levels on the front and rear wall so the overall response of the building could be determined and the amplifications with respect to the levels at the front foundations could be computed (see points 1–5 in Figure 1 for the principal measuring positions). Vibrations at the front foundations were monitored for 15 min period each hour throughout the day so that the daily vibration dose could be characterised.

4.1.3 Window vibrations
Short term measurements of window vibration were made at sites A, B and C with a small piezoelectric accelerometer (sensitivity 3.75 mV per m/s²). The accelerometer was attached with beeswax to the middle of a window pane on the exposed facade of the building. A measuring amplifier was used to condition the signal and recordings were made on the instrumentation recorder. Additional recordings were made at the church (site B) since a number of the stained glass windows rattled and buzzed as vehicles passed by and there was concern that these windows might have suffered damage. Measurements were made on two stained glass windows near the exposed south east corner and on two very similar windows near the south west corner which were not so exposed to traffic vibration in order that any excess damage due to high levels of traffic vibration might be detected (Figure 1 shows the measurement positions).

4.1.4 Crack movements
Dynamic movements of cracks can occur as vibration propagates through building elements. These movements were recorded at sites A, C and D but not in the church (site B) since the principal cracks were inaccessible, being high up in the chancel arches. The measurements were made with a strain gauge which could resolve movements of the order of one micron. The measurement points are indicated by the letter ‘S’ in Figure 1. A bridge and operational amplifier were used to condition the signals and simultaneous measurements were made with the vertical vibration of the front foundations. At the town house (site A) measurements were made across a vertical crack in a party wall on the first floor of the building near the front facade. At the timber-framed cottage (site C) the measurements were taken across a crack in an outside wall on the groundfloor again near the front facade. Measurements were taken above the porch door fronting the road at the farmhouse (building D).

4.2 NOISE MEASUREMENTS
At buildings A and C, external low frequency noise measurements were made 1 m from the front facade at first floor level (see letter ‘M’ in figure 1). At the church (site B), measurements were made at a height of approximately 3 m above ground level in front of the east and west wall windows. Traffic levels were very low past the farmhouse (site D) and although measurements were attempted they were found to be contaminated with the noise from the construction plant operating in the pipe store less than 100 m away and for this reason recordings were not made. All measurements were made with a ½ inch condenser microphone connected to a measuring amplifier which provided the polarization voltage. A 22 Hz high pass filter was incorporated so that low frequency surges produced by air turbulence would be reduced.

4.3 MEASUREMENT OF ROAD SURFACE IRREGULARITY
A laser plane was used to measure variations in height near wheel tracks at each site where it was evident that an irregularity was contributing significantly to the measured vibration levels. At the farmhouse (site D) measurements were made over an identical profile in the before and after period to check whether the large increase in the number of very heavy vehicles had caused any further deterioration of the road surface which might in turn lead to increased vibration amplitudes at the building. The laser plane consists of a portable battery operated laser source and measuring rod. The source was placed at the side of the road and emitted a rotating beam of laser light in a horizontal plane. The measuring rod was usually placed at 100 mm centres along the profile which was defined by a strip of white tape. Height readings were taken at each point by holding the rod vertically and activating a moving detector which came to rest in the plane of the laser light.
Fig. 1 Location plan for geophone, accelerometer, microphone and accelerometer measurement points
5 SURVEYS
In addition to the measurement of vibration and noise, surveys of the buildings, traffic and soil were completed.

5.1 BUILDING SURVEYS
Building surveys were carried out by structural engineers from English Heritage at the same time or shortly after measurements were made. Their objectives were to describe the state of dilapidation of the buildings, for example the degree of cracking, the extent of any distortions in building elements and any settlement problems and then to infer the most likely causes of this damage based on available site evidence and also on a knowledge of problems in buildings of similar type and age. The positions of significant cracks were marked on drawings and at the farmhouse (site D) small studs were placed across these cracks and width readings taken with a Demec gauge. Several measurements were taken over a period of months to determine if the increased heavy traffic had caused deterioration. The complete reports of the surveys are reproduced in the Appendix.

5.2 TRAFFIC SURVEYS
The volume and composition of the traffic at a site is one of the important factors determining the rms and peak levels of vibration recorded in a building. At sites A, B and C estimates of the daily variation were obtained by recording vehicle flows during 15 rain periods each hour between 9:00 or 10:00 and 19:00 hours. Because of low traffic flow at site D, measurements were taken over complete hours during the before and after periods. Vehicles were categorised into 'light' (cars and small vans with a gross weight less than 1.5 tonnes), 2, 3, 4 and 5 or more axle goods vehicles greater than 1.5 tonnes, and buses and coaches. Additional information on traffic levels in past years was provided by the local authority.

5.3 SOIL SURVEYS
The aim of these surveys was to provide a simple description of the soil type and its moisture content in order to confirm the description obtained from large scale drift deposit maps. Where possible the findings were supplemented by detailed information on geological strata in the vicinity, obtained from borehole data held by the British Geological Survey. Soil samples were taken at various depths in the range 0.5 to 0.7 m close to the building foundations and sufficient samples were collected to enable the particle size distribution and moisture content of the soil to be established. Standard test procedures were followed (British Standards Institution, 1975).

6 ANALYSIS OF VIBRATION DATA
Time and frequency domain analyses were used to analyse the data. Most of the time domain analyses were carried out on site.

6.1 TIME DOMAIN
The CED 1401 intelligent interface unit, which was connected to the geophones and strain gauges as detailed above, was programmed to carry out time domain analyses as follows:

(i) The vibration dose at the foundations recorded over each 15 min was divided into 8s intervals and the number of intervals during which the PPV in the vertical direction exceeded levels of 0.14 mm/s and 0.3 mm/s was logged. The figure of 0.14 mm/s corresponds to the base curve specified in BS 6472 (BSI, 1984) below which comments or complaints about vibration in buildings are rare. The value of 0.3 mm/s corresponds to the threshold of perception for continuous sinusoidal vibrations established by Reiher and Meister (Steffens, 1974). The highest vibration amplitudes in the 15 min periods were also recorded. Because of the small number of lorries at site D the equipment was triggered manually for each passing HGV and the peak values were established in hourly intervals and not during 15 min periods.

(ii) The amplification of ground-borne vibrations in various parts of the structure was determined by calculating the ratios of the PPVs of major building elements to the corresponding PPVs of vertical vibration at the front foundation. Peak amplitudes were captured for several of the largest vibration events at each measurement position and the average amplifications calculated.

(iii) Up to 12 channels of data from four separated geophone arrays at the front and rear of the building were plotted to indicate the significant modes of vibration.

(iv) Recordings of the free vibration of suspended floors produced by a small vertical impulse were plotted to determine the natural frequency of oscillation.

(v) The maximum amplitudes of window vibration and dynamic crack movements were computed.

6.2 FREQUENCY DOMAIN
A two channel digital signal analyser connected to a plotter was used to compute and display the frequency content of the recorded signals. In most cases the analysis was restricted to the range DC-100 Hz and the corresponding resolution was 0.39 Hz. The spectral densities of vibrations at the
TABLE 1
Vertical vibration levels at foundations by site and time of day

<table>
<thead>
<tr>
<th>Site</th>
<th>Number of events in 15 min period when peak velocity &gt;0.14 mm/s</th>
<th>Number of events in 15 min period when peak velocity &gt;0.3 mm/s</th>
<th>Peak level during period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A† B C</td>
<td>A B C</td>
<td>A  B  C</td>
</tr>
<tr>
<td>10:00</td>
<td>27 0 18</td>
<td>8 0 6</td>
<td>0.92 0.13 0.42 0.02 0.12</td>
</tr>
<tr>
<td>11:00</td>
<td>31 1 25</td>
<td>12 0 8</td>
<td>0.46 0.26 0.44 0.05 0.14</td>
</tr>
<tr>
<td>12:00</td>
<td>22 1 17</td>
<td>6 1 4</td>
<td>0.92 0.30 0.45 0.07 0.12</td>
</tr>
<tr>
<td>13:00</td>
<td>22 0 26</td>
<td>8 0 9</td>
<td>0.61 0.10 0.46 0.09 0.16</td>
</tr>
<tr>
<td>14:00</td>
<td>26 0 22</td>
<td>10 0 2</td>
<td>0.54 0.08 0.37 0.11 0.13</td>
</tr>
<tr>
<td>15:00</td>
<td>29 2 16</td>
<td>5 1 5</td>
<td>0.44 0.33 0.37 0.09 0.16</td>
</tr>
<tr>
<td>16:00</td>
<td>21 0 19</td>
<td>6 0 2</td>
<td>0.67 0.12 0.41 0.07 0.13</td>
</tr>
<tr>
<td>17:00</td>
<td>20 0 9</td>
<td>2 0 3</td>
<td>0.36 0.11 0.39 0.06 0.16</td>
</tr>
<tr>
<td>18:00</td>
<td>16 2 17</td>
<td>7 0 1</td>
<td>0.49 0.19 0.37 0.09 --</td>
</tr>
</tbody>
</table>

†Site A: town house in Bath.
Site B: church in Louth.
Site C: timber-framed cottage near Evesham.
Site D: farmhouse near Honiton.
* Because of small numbers of HGVs, peak values in hourly periods were recorded.
D₁ are results for the period before the increase in lorry traffic.
D₂ are results for the first period when lorry traffic was relatively high.

foundations, front facade, and middle of suspended floors and window panes during 15 min periods were determined using signal averaging*. The sound pressure at facades was also analysed in a similar way. For the vibration records this enabled the dominant frequencies to be identified and rms values in the low (4-20 Hz) and high (20-100 Hz) frequency bands to be determined by integration of this spectral density function. In the case of the noise recording, the decibel scale was used and integration yielded the dB level in the 20-100 Hz band.

7 RESULTS
A large quantity of data was collected and analysed and the most important results are presented below.

7.1 VIBRATION LEVELS
7.1.1 Vibrations at foundations
The vibration dose at each building was characterised by the frequency with which perception levels were exceeded at the foundations in the vertical direction and by the maximum recorded amplitudes. Table 1 lists these values for 15 min sampling periods throughout the day at sites A, B and C. At the farmhouse (site D), peak values in a full hour of monitoring are given both for the period before the increase in traffic and during the period when lorries were delivering the pipes to the store. The highest vibration amplitude of 0.92 mm/s was recorded at the town house (site A) and the frequencies with which the threshold was exceeded at this site also tended to be greater than at other buildings. An examination of the table shows that there is no obvious systematic pattern in exposure throughout the day. At building D the significant increase in peak levels in the after period can be clearly seen in each hour. Figure 2 shows this change in vibration dose in greater detail. Only one event produced a PPV greater than 0.1 mm/s in the before period yet in the after period 32 exceedences occurred in a similar period. These increases were almost entirely due to the pipe delivery lorries.

The frequency contents of the vertical vibrations at the front foundations are shown in Figure 3 where the spectral density is given for sites A, B and C during a 15 min sampling period. It is clear that at all sites the dominant frequencies are below 20 Hz and the peaks of the distribution are in a narrow range from 9-12 Hz. By integrating the spectral density function and taking the square root, the rms value in mm/s was calculated for low and high frequency bands (see Table 2) and this again indicates the dominance of low frequency vibrations. The rms levels are very low at the church (site B) since it was only occasionally that heavy vehicles crossed a small sunken drain cover producing perceptible vibrations.

* The spectral density referred to is the autospectrum and is the magnitude squared of the Fourier transform of the time signal. It indicates the distribution of power in the vibrations across the selected frequency band. A linear average of several hundred spectra was computed for each analysis period.
Fig. 2 Frequency of vibration events by peak velocity produced by HGVs at Site D in the before and after periods from 9:00—18:00 hours.

Fig. 3 Spectral densities for the vertical components of particle velocity at the front foundations.
TABLE 2
RMS levels for vertical vibration at foundation level

<table>
<thead>
<tr>
<th>Site</th>
<th>Description</th>
<th>4–20 Hz (mm/s)</th>
<th>20–100 Hz (mm/s)</th>
<th>Frequency at peak (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Town house in Bath</td>
<td>0.027</td>
<td>0.022</td>
<td>9.4</td>
</tr>
<tr>
<td>B</td>
<td>Church in Louth</td>
<td>0.005</td>
<td>0.002*</td>
<td>11.3</td>
</tr>
<tr>
<td>C</td>
<td>Timber-framed cottage near Evesham</td>
<td>0.015</td>
<td>0.010</td>
<td>10.9</td>
</tr>
</tbody>
</table>

* Value estimated due to extraneous measurement noise.

at the foundations. Because of this, extraneous noise was a problem and resulted in the spurious peaks between 60 and 80 Hz. It is estimated that the rms value for the higher frequency band was inflated by about 40 per cent and the corrected value is given in the table.

7.1.2 Amplification of vibrations

The amplifications of the vibrations that occurred in the buildings are shown in Table 3 where the PPVs at various points in the building are related to that measured in the vertical direction at the front foundations. The numbers in brackets refer to the measurement points shown in Figure 1. Radial and transverse measurement directions are also defined in this Figure. In all cases at the front foundations the vertical component was largest. Vibrations of the front facade tended to be greater at higher levels and this was especially true of vibrations in the radial direction where amplifications of over four occurred. On the rear facade vibrations were lower but the increase with height was still apparent. Vertical vibrations of suspended floors were in all cases much higher than at the front foundations, the amplifications in this case ranging from 2.5 to 5.1. The increase with height is well illustrated at the town house (site A) where amplification increases from 0.9 on the ground floor which was solid, to 5.1 on the suspended wooden floor at third floor level. Table 4 lists the natural frequencies and dimensions of these suspended floors together with the dominant frequency of vertical vibration at the foundations.

A print out of particle velocity at several positions on the front facade at site A showed the principal mode of vibration was a rocking motion on the foundations at about 11 Hz since the vibrations at various heights were appreciably in phase and the radial motion at top floor level was nearly four times the corresponding level at the foundations. The church (site B) was over 60 m long which meant that most components at the west end were much reduced due to the attenuation with distance. The time delay for vibrations to reach the far wall was 0.34s (wave speed 180 m/s) and opposite walls were vibrating out of phase as a consequence. There was relatively little movement of the east and west walls in the radial direction even at a height of 30 m in the tower at the west end of the church. In contrast there was relatively large amplification in the radial direction at first floor level at sites C and D. There was little attenuation of this motion at the rear of the farmhouse (site D) at first floor level, and both walls appeared to rock in phase on their foundations. There was however a significant reduction in the radial motion at this level at the timber-framed cottage (site C), since the amplification reduced from 3.4 to 1.2

7.1.3 Source of vibrations

Building C was only 2 m from a de-restricted stretch of road where HGVs frequently passed at speed and it was thought that there was a possibility that detectable low frequency vibrations due to air pressure pulses or quasi-static depression of the road surface might be evident. As explained in section 2, the frequency of these vibrations would probably not exceed 3 or 4 Hz in most cases. To examine this possibility, a spectral density plot of the radial component of velocity at the front wall at first floor level was computed from the average of eight of the most significant vibration events caused by HGVs passing close to the building. This is shown in Figure 4, and even allowing for the reduction in sensitivity of the geophone below 5 Hz (eg sensitivity at 2 Hz is approximately ½ that at 10 Hz) there is no evidence of any substantial vibrations at these lower frequencies which might result from these mechanisms.

To demonstrate that the source of the observed vibrations at this site was due to ground-borne vibrations, simultaneous measurements in the radial direction were made at the kerbside and on the front wall at first floor level. Time histories of the vibrations for the period when a large three-axled lorry passed close to the building at speed are given in Figure 5. By close inspection it can be seen that the first six major oscillations in the vibration traces for the kerb and wall geophone are similar. The dominant vibrations in both records occur at a frequency of 11 Hz and cross-correlation analysis
### TABLE 3

Peak particle velocity (PPV) at locations in the buildings as ratio of the vertical PPV at foundations

<table>
<thead>
<tr>
<th>Site</th>
<th>Front facade — foundations</th>
<th>Location*</th>
<th>Front facade — upper floors</th>
<th>Location</th>
<th>Front rooms</th>
<th>Location</th>
<th>Rear facade — foundations</th>
<th>Location</th>
<th>Rear facade — upper floors</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>v</td>
<td>r</td>
<td>t</td>
<td>v</td>
<td>r</td>
<td>t</td>
<td>v</td>
<td>r</td>
<td>t</td>
<td>v</td>
</tr>
<tr>
<td>A (Town house)</td>
<td>1.00</td>
<td>0.90</td>
<td>0.41 (1)</td>
<td>1st</td>
<td>0.79</td>
<td>2.09</td>
<td>0.42 (2)</td>
<td>2nd</td>
<td>0.70</td>
<td>0.94</td>
</tr>
<tr>
<td>B (Church)</td>
<td>SE</td>
<td>1.00</td>
<td>0.58</td>
<td>0.21 (1)</td>
<td>0.99</td>
<td>0.75</td>
<td>0.37 (2)</td>
<td>Stone sill of stained glass window 3.3 m above ground level</td>
<td>1st floor</td>
<td>2.47</td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>1.00</td>
<td>0.32</td>
<td>0.46 (1)</td>
<td>1.32</td>
<td>3.43</td>
<td>0.82 (2)</td>
<td>1st floor</td>
<td>3.73</td>
<td>1.98</td>
<td>1.98 (3)</td>
</tr>
<tr>
<td>D (Farmhouse)</td>
<td>Porch</td>
<td>1.00</td>
<td>0.68</td>
<td>0.76 (1)</td>
<td>NE</td>
<td>1.92</td>
<td>4.18</td>
<td>0.99 (2)</td>
<td>SE</td>
<td>2.07</td>
</tr>
</tbody>
</table>

*For location of geophones see Figure 1. 

v = vertical component. 
r = radial component measured normal to road alignment. 
t = transverse component measured parallel to road alignment.

### TABLE 4

Amplification of vertical vibrations in suspended floors related to natural frequency

<table>
<thead>
<tr>
<th>Site</th>
<th>Location</th>
<th>Floor dimensions (m)</th>
<th>Natural frequency (Hz)</th>
<th>Predominant frequency at foundations (Hz)</th>
<th>Amplification with reference to foundation levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (Town house)</td>
<td>1st floor</td>
<td>4.75 x 3.6 m</td>
<td>13.5</td>
<td>9.4</td>
<td>3.65</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>4.40 x 3.6 m</td>
<td>—</td>
<td>9.4</td>
<td>4.16</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>4.21 x 3.6 m</td>
<td>13.11</td>
<td>9.4</td>
<td>5.10</td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>1st</td>
<td>3.70 x 3.5 m</td>
<td>17.8</td>
<td>10.9</td>
<td>2.47</td>
</tr>
<tr>
<td>D (Farmhouse)</td>
<td>1st</td>
<td>3.9 x 3.8 m</td>
<td>15.3</td>
<td>10.3</td>
<td>3.73</td>
</tr>
</tbody>
</table>

* No measurement of natural frequency carried out on second floor. 

NB No suspended wooden floors at site B.
Based on the average of eight spectra

![Graph showing spectral density for the radial component of particle velocity of the front wall at Site C at first floor level](image)

**Fig.4** Spectral density for the radial component of particle velocity of the front wall at Site C at first floor level

Radial velocity at kerb

![Graph showing radial velocity at kerb](image)

**Fig.5** Time histories of vibration during the passage of an HGV at Site C

Radial velocity at first floor level on front facade

![Graph showing radial velocity at first floor level on front facade](image)

shows a delay of 80 ms. It would appear that the wall is lightly damped and continues to vibrate at a frequency close to the forcing frequency after the major vibrations at the kerb have decayed. It is also apparent that the wall vibration is later re-excited by minor peaks of ground-borne vibration.

### 7.1.4 Noise levels

Table 5 lists the linear noise levels recorded at the building facades in the 20–100 Hz band and the frequencies at the peaks of the spectra. Although low frequency noise produces the greatest response on flexible structures such as suspended floors (Watts, 1987b) the rms values for floor vibration given in the table indicate the importance of ground-borne rather than airborne vibration as levels in the low frequency band 4–20 Hz are greater than in the high frequency band where acoustically coupled vibrations normally tend to dominate.

### 7.1.5 Window vibrations

In the case of window vibration the dominant frequencies are much higher and peak acceleration amplitudes exceed 10 m/s² (Table 6). At site B,
### TABLE 5
Noise levels at facades and upper floor vibrations

<table>
<thead>
<tr>
<th>Site</th>
<th>Position</th>
<th>Linear levels at facade (dB)</th>
<th>Frequency at peak (Hz)</th>
<th>RMS vertical floor velocity (mm/s)</th>
<th>Frequency at peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20–100 Hz</td>
<td>4–20 Hz</td>
<td>20–100 Hz</td>
<td></td>
</tr>
<tr>
<td>A (Town house)</td>
<td>First floor</td>
<td>89.2</td>
<td>59.2</td>
<td>0.106</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>East side</td>
<td>85.4</td>
<td>40.6</td>
<td>0.027</td>
<td></td>
</tr>
<tr>
<td></td>
<td>West side</td>
<td>74.0</td>
<td>35.0</td>
<td>0.012</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Both measurement points at a height of 3 m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>First floor</td>
<td>86.5</td>
<td>76.8</td>
<td>0.044</td>
<td>15.6</td>
</tr>
</tbody>
</table>

At side D, due to low traffic flows, rms vibration levels and noise levels were both very low and affected by extraneous factors.

### TABLE 6
Maximum recorded window vibrations

<table>
<thead>
<tr>
<th>Site</th>
<th>Position of window</th>
<th>Dimensions of window pane</th>
<th>Peak acceleration(m/s²)</th>
<th>Dominant frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (Town house)</td>
<td>First floor fronting road</td>
<td>0.34 × 0.27 m</td>
<td>10.4</td>
<td>200</td>
</tr>
<tr>
<td>B (Church)</td>
<td>Stained glass window near SE corner exposed to traffic</td>
<td>1.95 × 0.55 m</td>
<td>5.7 (traffic)</td>
<td>39.9</td>
</tr>
<tr>
<td></td>
<td>Very similar windows near SW corner—not exposed</td>
<td>1.95 × 0.55 m</td>
<td>3.5 (organ playing)</td>
<td>42.1</td>
</tr>
<tr>
<td></td>
<td>Ground floor fronting road</td>
<td>0.78 × 0.24 m</td>
<td>1.4 (traffic)</td>
<td>43.8</td>
</tr>
</tbody>
</table>

Recordings not taken at site D because of adverse weather conditions.

### TABLE 7
RMS window acceleration at church (site B)

<table>
<thead>
<tr>
<th>Position of windows</th>
<th>Window pane dimensions (m)</th>
<th>4–20 Hz m/s²</th>
<th>20–100 Hz m/s²</th>
<th>Linear noise level (dB) near window</th>
</tr>
</thead>
<tbody>
<tr>
<td>East wall—exposed</td>
<td>2.30 × 0.47</td>
<td>0.012</td>
<td>0.231</td>
<td>85.4</td>
</tr>
<tr>
<td>West wall—not exposed</td>
<td>2.30 × 0.47</td>
<td>0.011</td>
<td>0.055</td>
<td>74.0</td>
</tr>
<tr>
<td>South wall—exposed</td>
<td>1.95 × 0.55</td>
<td>0.021</td>
<td>0.393</td>
<td></td>
</tr>
<tr>
<td>(near east end)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>South wall—not exposed (near west end)</td>
<td>1.95 × 0.55</td>
<td>0.004</td>
<td>0.078</td>
<td></td>
</tr>
</tbody>
</table>
measurements were made when the organist played and traffic levels were low in order to provide a context for judging the importance of traffic vibration exposure compared with that resulting from the normal use of the building. The organist was instructed to play the full range of notes in order to excite resonances in the stained glass window panes. In this way it was hoped to create the worst case conditions likely to be produced by the organ. The Table shows that the maximum vibration amplitude produced by the organ was lower than that produced by traffic although of the same order. Since a detailed study of the conditions of similar windows exposed to and removed from traffic was made at site B a more detailed examination of the vibration dose at two pairs of windows was made. Table 7 lists the rms levels of these four window panes and also records the linear noise levels at 1 m from the exterior of the windows at opposite ends of the church. The window on the south wall at the east end ‘buzzed’ as vehicles passed by and it was clearly driven by low frequency noise. The rms levels are substantially greater in the higher frequency band as can be seen in the table. These levels for the exposed windows are 4–5 times greater than the similar windows away from traffic reflecting the large differences in sound pressure levels.

7.2 DYNAMIC CRACK MOVEMENTS
At all sites the maximum recorded crack movements produced by passing heavy vehicles were very small and of the order of one micron, and were in most cases hardly distinguishable from the background noise level of the instrumentation. At site D, where the strain gauge was attached across a crack in the brickwork directly above the porch, slamming the front door produced a peak amplitude of 22 microns.

7.3 ROAD SURFACE PROFILE
The maximum depths and heights of significant road surface irregularities at the sites are given in Table 8. The largest depression at any of the sites was 50 mm deep and was due to the presence of a misaligned service cover and a poorly backfilled trench. However, this did not result in the largest peak levels being recorded at this site (D) since the propagation distance to the building was over 30 m. It was thought that this profile might deepen as a result of the large increase in lorry traffic and this might contribute to higher vibration levels in the after period. Figure 6 shows the profile in the before and after periods and it can be seen that differences are very small being of the order of a few millimetres at maximum and may in part be due to measurement error. It was considered that the measured irregularities at all the sites produced the largest contributions to the vibration levels observed but it is likely that other surface irregularities in the vicinity would have made some contribution. This was probably the case at sites A and D where the road surfaces were generally uneven.

<table>
<thead>
<tr>
<th>Site</th>
<th>Maximum depth (mm)</th>
<th>Maximum height (mm)</th>
<th>Distance from foundation measurement position (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (Town house)</td>
<td>5</td>
<td>9</td>
<td>4.9</td>
</tr>
<tr>
<td>B (Church)</td>
<td>32</td>
<td>4</td>
<td>6.5</td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>9</td>
<td>3</td>
<td>2.3</td>
</tr>
<tr>
<td>D (Farmhouse)</td>
<td>50</td>
<td>1</td>
<td>33.0</td>
</tr>
</tbody>
</table>
party wall next to the stairs between first and second floors. On the external facade a crack had developed above the beam over the shop front. This followed the joints in the ashlar blocks and was probably caused by the sagging of the beam spanning the shop front. The front wall was reasonably plumb, the variation from the vertical up to a height of 5.8 m was only 22 mm.

### 7.4.2 Building B—church

The church was in very good condition structurally. There were only two lines of fracture that were noted as being of any importance. These were at the crown of the arches of the chancel both on the north and south side and indicate a slight outward lean of the east gable wall. It was concluded that this cracking was probably due to the greater weight of the east gable wall compared with the smaller weight of the side walls. Very many churches of all ages suffer fractures adjacent to the east gable. The condition of the stained glass windows exposed to high levels of vibration and those not so exposed was very similar.

### 7.4.3 Building C—timber-framed cottage

This building has a timber-framed gable wall which leans towards the road and has a bulge at first floor level. However there are no indications that this movement is continuing and in fact some distortion is apparent on an early photograph reproduced in a book published in the early 1940s (Russel, 1941). Unfortunately the building appears quite small in this picture since the photograph was taken from the other end of the village so it is not possible to infer the degree of tilt accurately.

### 7.4.4 Building D—farmhouse

The building near Honiton was fully inspected on four occasions. On the 28th January 1988 the first survey was carried out before the increase in lorry traffic. It was next surveyed on the 20th April which occurred in the middle of a six week period when the pipes were being delivered to the store and then again on the 17th June in the middle of a similar period when the pipes were being hauled back past the house to be delivered to various parts of the pipeline. Inspections were also carried out on the 12th August 1988 and 9th February 1989 after deliveries had finished. With the exception of the porch, the main building was found to be in good structural condition. The north gable wall had some minor cracking and flaking to the render. The front and side walls of the porch had an outward lean of approximately 40 mm over a height of 1.5 m and there were some cracks on the north elevation above the porch door. The settlement of the porch and associated cracking had possibly started some considerable time ago and had been caused by inadequate foundations or poor subsoil. Little or no further movements had taken place in the porch over the monitoring period: for example, the recently repointed corners between the porch wall and the main wall had not cracked and measured crack movements were relatively small (the largest being 0.09 mm) and were probably due to seasonal changes.

### 7.5 TRAFFIC SURVEYS

The hourly flows are listed in Table 9. For sites A, B and C these were estimated from 15 min counts during each hour. The A435 adjacent to the timber-framed cottage (site C) carried the highest lorry flow

<table>
<thead>
<tr>
<th>Site</th>
<th>Light*</th>
<th>Heavy**</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (Town house)</td>
<td>908 904 976 972 932 968 1008 1368 1344 1016</td>
<td>108 144 184 108 128 128 140 96 60 48</td>
</tr>
<tr>
<td>B (Church)</td>
<td>456 568 504 492 588 580 820 564 460</td>
<td>108 64 112 108 72 88 64 36 44</td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>672 608 616 512 648 752 956 900 780</td>
<td>184 148 140 192 184 136 140 88 136</td>
</tr>
<tr>
<td>D (Farmhouse)</td>
<td>48 40 39 45 42 38 42 42 56 70</td>
<td>6 6 10 4 10 7 5 5 1</td>
</tr>
</tbody>
</table>

* Light: Cars and goods vehicles ≤1.5 tonnes
** Heavy: Goods vehicles >1.5 tonnes, buses and coaches.

<table>
<thead>
<tr>
<th>Site</th>
<th>Total flow</th>
<th>Percentage of heavy vehicles (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (Town house)</td>
<td>8472</td>
<td>10.4</td>
</tr>
<tr>
<td>B (Church)</td>
<td>4572</td>
<td>12.5</td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>5664</td>
<td>17.6</td>
</tr>
<tr>
<td>D (Farmhouse)</td>
<td>3722</td>
<td>11.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Site</th>
<th>Before</th>
<th>1st After</th>
<th>2nd After</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light</td>
<td>48 40 39 45 42 38 42 42 56 70</td>
<td>43 49 44 40 41 50 41 52 71</td>
<td>65 51 60 64 60 73 79 101 105</td>
</tr>
<tr>
<td>Heavy</td>
<td>6 6 10 4 10 7 5 5 1</td>
<td>11 9 16 10 11 11 17 8 6</td>
<td>19 17 18 15 13 11 14 18 12</td>
</tr>
</tbody>
</table>
both in terms of the total number and also when expressed as a percentage of all vehicles. The farmhouse (site D) was exposed to the lowest flow of HGVs, there being only 48 in the measurement period before the pipe store was operational. However this rose to 118 in the second ‘after’ period when pipes were being delivered to store and was largely due to the opening of a subcontractors depot nearby at the beginning of the second ‘after’ period. Information obtained from the site office suggested that these increased lorry flows were fairly constant over the two six week periods when pipes were being moved. It is likely that the normal lorry flows on this road have not changed very much in recent years since flow data for October 1986 obtained from the Highway Authority shows a similar level (61 in a 12 hour period 7:00 to 19:00).

Additional flow information was obtained for the other sites. At the town house (site A) the total flows of light and heavy vehicles between 9:00 and 19:00 hours in February 1982 were 7666 and 981 which are similar to those recorded in this study. At the church (site B) in 1982 the total flow of heavy vehicles between 8:00 and 18:00 was 685 indicating that this flow had not changed substantially in recent years. At the timber-framed cottage (site C) the flow of light vehicles recorded in 1981 between 10:00 and 18:00 was 5238 and the total for heavy vehicles was 1331, again similar to levels recorded in this study.

7.6 SOIL SURVEYS

The particle size distributions and the moisture contents of the soil samples taken at each site are given in Table 10. It is evident that there was a large range of soil types: for example, the soil at the farmhouse (site D) contains a high percentage of very fine particles consistent with a clay while at the timber-framed cottage (site C) there is a much larger percentage of larger particles which could be described as sandy gravel. Borehole data showing the types and depths of various geological strata are given in Table 11. Only at the town house (site A) are soft layers recorded and for such ground conditions other work shows that for a given set of conditions relatively large vibrations can be induced in nearby buildings (Watts, 1989).

8 DISCUSSION

8.1 EXPOSURE TO VIBRATION

All the buildings were situated within a few metres of roads carrying HGVs and they were exposed to a range of traffic vibration levels. The principal vibration frequencies were below 20 Hz and peaked in the 10 Hz region indicating that the source was ground-borne vibration produced by the dynamic loading of the road surface by passing heavy vehicles. This was indicated at the timber-framed cottage (site C) and has been confirmed at other locations near road surface irregularities (Watts, 1987a). At sites A and C these levels were relatively high and vibration amplitudes exceeded the level of perception (0.3 mm/s) many times a day. The presence of soft ground at the town house (site A) may partly explain the relatively high vibration levels. At the church (site B) there were few events which exceeded this threshold and this was probably due to the fact that the small sunken drain cover causing the high dynamic loading was only struck occasionally. At the farmhouse (site D) the exposure was very low before the opening of a pipe store close to the building but this increased substantially when the store became operational although it was still below the exposures at other sites.

8.1.1 Radial component of vibration

Generally vibration levels in the radial direction (horizontally, away from the point of loading on the road) were higher on upper levels of the buildings and this has been found in previous studies (Watts, 1987b and 1989) and probably results from the lack of restraint offered by the ground. The predominant mode of vibration in the dwelling houses was a rocking of the front wall about the foundations. At

<table>
<thead>
<tr>
<th>Site</th>
<th>Clay &lt;0.002 mm (%)</th>
<th>Silt 0.002–0.06 mm (%)</th>
<th>Sand 0.06–2.0 mm (%)</th>
<th>Gravel 2–60 mm (%)</th>
<th>Cobbles 60–200 mm (%)</th>
<th>Moisture content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (Town house)</td>
<td>3</td>
<td>26</td>
<td>47</td>
<td>24</td>
<td>0</td>
<td>38</td>
</tr>
<tr>
<td>B (Church)</td>
<td>7</td>
<td>23</td>
<td>43</td>
<td>27</td>
<td>0</td>
<td>24</td>
</tr>
<tr>
<td>C (Timber-framed cottage)</td>
<td>12</td>
<td>7</td>
<td>35</td>
<td>46</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>D (Farmhouse)</td>
<td>43</td>
<td>27</td>
<td>22</td>
<td>8</td>
<td>0</td>
<td>29</td>
</tr>
</tbody>
</table>

TABLE 10

Soil characteristics near building foundations
the town house (site A) this effect was probably exacerbated by the type of construction since the front wall of ashlar blockwork was only 175 mm thick yet over 8 m high. The side and rear walls were nearly three times as thick and being stiffer responded to the ground-borne vibration to a much smaller extent. The church (site B) had thick stone walls with massive buttresses (see Plate 2) and as expected the radial vibrations of the exposed wall were relatively small. At the timber-framed cottage (site C) there was a relatively large amplification of the radial component probably due to the flexible nature of the timber frame which was poorly damped and also because the natural frequency may have been close to the principal frequency of ground-borne vibration. This was not the case on the rear wall as this was brick built and therefore more substantial. At the farmhouse (site D) both front and rear walls vibrated in phase at a relatively high amplitude. The absence of thick partition walls on the first floor and continuous joists running from front to back probably contributed to this effect.

8.1.2 Vertical component of vibration

There was generally an increase of the vertical component of vibration with height but this was not so large as the amplification of the radial component. This was also noted in a previous study and is thought to result from the reduction of dead load with height (Marshall and Hood, 1987). Amplification of the vertical component was very noticeable in the middle of floors at the fronts of the buildings. At the town house (site A) the peak amplitude increased monotonically with height reaching over five times foundation level on the third floor. The amplification of the suspended floors relative to the foundation level depends also on the mass and stiffness of the floor. Where the mass is relatively high and stiffness low, the natural frequency will be closer to typical frequencies of ground-borne vibration and the response will tend to be high. Thus at site C where the natural frequency was highest the amplification was lowest (2.5) and the greatest response occurred on the floor with the lowest natural frequency where the amplification was 5.1 (site C). This trend was also noted in the first series of case studies (Watts, 1988).

Peak vertical levels on the rear walls of the buildings were less than the corresponding levels at the front since there was some attenuation with distance especially at site B where the building was over 60 m long. In the absence of dissipative losses in the underlying ground, the minimum attenuation expected can be calculated from the assumed circular spreading of surface waves from the road surface irregularity. This leads to an attenuation rate
8.2 WINDOW VIBRATIONS

Low frequency noise levels at all sites where measurements were taken were relatively high ranging from 85 to 89 dB at the exposed facades. This was expected as these buildings were close to carriageways carrying high levels of lorry traffic. Consequently window vibration levels were high at these sites and the maximum acceleration of over 10 m/s² was recorded at site A where noise levels were highest. The dominant frequencies in the vibration spectra were in the range 40–200 Hz and clearly result from acoustic coupling rather than ground-borne vibrations. These frequencies tended to increase with decreasing window pane size. This is probably due to a decrease in natural frequency with window area (Steffens, 1975).

8.3 CAUSES OF DAMAGE

The maximum amplitude of vibration of a structural element of any of the buildings was well below levels known to cause even minor damage directly. Some minor damage from other vibration sources eg blasting and pile driving has been found to occur just above a PPV of approximately 10 mm/s (Nelson and Watts, 1987). However damage may possibly be triggered in a building element weakened through other causes and there are the further possibilities that low level vibrations over a period of many years may cause damage through material fatigue or induce differential settlement through soil densification or migration.

The most significant damage noted at the town house (site A) was probably a result of the alteration from a private dwelling house to a shop. The deflection of the beam over the shop front due to the weight of the masonry above probably caused the crack in the stonework. This is a common occurrence under such circumstances and it is unlikely that the damage was caused by the action of traffic vibration.

At the church (site B) the major cracks were in the chancel arches close to the major road. However as pointed out in the Appendix, cracking at these points in similar churches not exposed to such vibration is common and is thought to result from the movements caused by the greater mass of the east gable wall compared with the less massive side walls. Measurements were made of these crack widths by the church architects between 1977 and 1981 (Benny 1987) and show some movements. The largest increase was 2 mm and the largest decrease was 1.4 mm. No control measurements were made of similar cracks in parts of the building not so exposed to traffic vibration at the time so it cannot be concluded on this evidence alone that heavy traffic had contributed to the movement. It should also be noted that cyclic thermal movements of building materials and components are to be expected (BRS 1970). For example a 6 m length of unrestrained dense limestone would expand 1 mm if the temperature increased by 30 Celsius. These movements can be progressive particularly if there is an ingress of debris into these cracks. The stained glass windows chosen for study were of a very similar age and construction and although the exposed windows at the east end were subjected to average vibration levels 4–5 times that measured near the west end there were no differences in their conditions which could be attributed to the action of traffic vibration. This is despite the fact that the window near the traffic on the south wall was found to have been exposed to a peak acceleration amplitude of nearly 6 m/s² and ‘buzzed’ noticeably as vehicles passed. It is often the case that normal use of a building can produce vibration levels in parts of the structure which are similar to, or higher than, those produced by traffic (Watts, 1987b; Watts, 1988). For example, the peak acceleration of a stained glass window produced by the church organ approached the level caused by passing traffic and bell ringing can produce very high levels in a church steeple. Steffens (1974) records a peak velocity due to bell ringing of 17 mm/s at 1.4 Hz at a height of 26 m in a tower 50 m high. This level is far greater than that which can reasonably be expected from passing traffic.

At site C the timber-framed gable wall next to the road was highly distorted and responded readily to ground-borne vibration. However an old photograph indicates some distortion was present many years ago and before the advent of modern heavy traffic. In addition a timber-framed building within a few miles of this site but situated away from heavy traffic showed a similar but more severe distortion (see Appendix 12.3).

At the farmhouse (site D) the large increase in exposure to traffic vibration has not been associated with any significant changes in the building. Changes in the crack widths on the porch and north wall have been very small, and these may well be due to seasonal changes in temperature and humidity. Some minor cracking on the south gable wall that was noticed during the second after period may have been caused by substantial maintenance work that was carried out over the period of the surveys.

At all sites where dynamic crack movements were measured there was little or no movement due to passing vehicles and it is therefore likely that these cracks are not a direct result of exposure to traffic vibration. Domestic activity such as slamming doors has been shown to produce measurable movements.
in cracks adjacent to the doors. These movements can be greater than those produced by traffic vibration (Watts, 1987b; Watts, 1988).

Finally, in case studies of this type it is not possible to exclude the possibility that traffic has in some way contributed to the observed damage and this was a conclusion of the previous study (Watts, 1988). Plausible reasons have been advanced to account for the state of dilapidation based largely on the judgement of structural engineers who have specialized in heritage type buildings. Wide experience of common types of damage in other buildings of similar types to the ones examined in this study provided the framework for assessing whether vibration effects are likely to be important. However it remains a possibility that traffic vibration may exacerbate structural problems that are clearly due to other causes (e.g. cracking above the shop window due to alteration works at site A) or may be assisting soil settlement (e.g. the ground under the east wall of building B causing it to lean towards the road). In other studies where tighter control of extraneous factors was possible there was no evidence that traffic vibration has any significant effects. In one study where a pair of semi-detached houses was exposed to high levels of simulated traffic vibration at levels which would be intolerable to most people (Marshall and Hood 1987) the only observed damage was minor cracking of plasterwork. It was concluded that this damage would probably go unnoticed in a normally decorated house. There was no movement of the building despite the fact that it was built on a fairly loose sand and some densification of the sand was expected. In another study the condition of houses on alluvial soils exposed to traffic vibration was compared with that of very similar properties in the neighbourhood that were situated well away from heavy traffic to determine if any excess damage was detectable. It was concluded that there was no evidence of significant differences in the condition of the properties that could reasonably be attributed to the effects of traffic vibration. Thus while it is not possible to exclude the possibility that traffic vibration has some effect on the buildings in the present study it is likely that the observed damage was mainly due to other site factors.

9 CONCLUSIONS

The effect of traffic induced vibrations on four heritage buildings has been examined. This study forms an extension to a previous examination of four relatively small brick built dwellings. The range of building considered in this further phase was far greater in terms of age, size and type of construction.

The main findings from an analysis of the large amount of physical measurements and structural investigations are similar to the first phase and are as follows:

(i) At all sites ground-borne traffic induced vibration was the most significant source of building vibration. When the surface is even, airborne vibration dominates, but in the cases studied surface defects up to 50 mm deep gave rise to relatively high levels of ground-borne vibration. At two sites levels were relatively high, regularly exceeding the level of perception at ground level.

(ii) Maximum vibration amplitudes were greater on upper floors and walls at the fronts of the buildings than at foundation level.

(iii) Window pane vibration levels were relatively high and at one site where stained glass windows exposed to high levels of airborne traffic vibration were compared with similar windows not so exposed, no differences in their condition were found which were attributed to the effects of traffic vibration.

(iv) Damage surveys carried out by structural engineers from the Historic Buildings and Monuments Commission for England identified a range of defects in the buildings ranging from cracks in plaster finishes to more substantial structural damage such as cracked stonework, distortion of walls and foundation settlement. In all cases, however, it was concluded that the main causes were likely to have been other site factors rather than exposure to traffic vibration.

10 ACKNOWLEDGEMENTS

The work described in this report was carried out in the Vehicles and Environment Division of the Vehicles Group of the TRRL. The assistance of the Historic Buildings and Monuments Commission for England is gratefully acknowledged.

11 REFERENCES


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12 APPENDIX

REPORTS OF THE STRUCTURAL SURVEYS BY THE HISTORIC BUILDINGS AND MONUMENTS COMMISSION FOR ENGLAND

Although the immediate physical effects of passing traffic in terms of vibration, crack movements and noise can be measured objectively using a variety of instruments, assessment of damage to the structure of buildings is a matter of judgement based on experience. The conclusions drawn in this report rest largely on the expert judgements of structural engineers who have a wide experience of assessing damage in heritage buildings. Since the results of their surveys cannot readily be summarised without neglecting some aspects which might be helpful to the reader, their reports are reproduced in full.
12.1 1 & 3 Widcombe Parade, Bath

Report on condition of building for traffic vibration study

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November 1987

Introduction

This report concerns the condition of the above building in connection with a study into the effects of traffic on historic buildings. The study was instituted by the Transport and Road Research Laboratory with English Heritage providing reports on the buildings under study. The buildings were inspected on the 8th October 1987. Plans and photographs are included in the report.

HISTORY

Widcombe Parade was constructed as domestic dwellings during the 1760’s. At a later date the ground floor of the terrace was converted to shop premises with the other floors above remaining in use as living quarters. No 1 was then converted into a garage and No 3 into a public house. No 3 then reverted back to a shop before both premises came under single ownership. At this point access doors were formed through the party wall and it is in this form that the buildings survive today. Above ground floor the exterior and interior of the buildings have survived in what looks to be near original in layout. Unfortunately the decoration and fittings have suffered through neglect and vandalisation by squatters who occupied the premises after the death of the previous owner.

DESCRIPTION OF 1 & 3 WIDCOMBE PARADE

a) Ground Floor.

3 Widcombe Parade.

The ground floor is of solid construction with a rear wall and party walls of 450–500 mm thick solid stone. The front wall has been removed to ceiling height over its full width and a timber and glass shop front sitting on a 150 mm thick wall has been inserted. (see photographs 1 & 2). The ground floor is one room with a chimney breast on the party wall with No 5 and downstand beams on the lines of the internal walls above. In addition to the internal walls the quarter turn stair has also been removed between ground and first floor level. A small light weight partition has been erected to form a passage to a door which has been formed in the party wall between No 1 and No 3. In the rear wall there are two large openings and these both lead into a large ground floor extension which in turn leads into a lean to. The extension roof is a flat roof constructed of boards on joists on timber beams and occurs at second floor level. The rear wall of the extension is of a timber and glass construction for most of its height and leads to the lean to which is constructed of timber and glass.

1 Widcombe Parade.

The ground floor is of solid construction with the rear and party walls of a similar thickness as those at No 3. A passage has been formed running from front to back with a door at each end. In addition to the door into No 3 the passage also contains a straight flight of stairs to a landing which connects to the original quarter turn stairs. The remainder of the ground floor is open plan with large openings in the front and rear walls which give access to the extensive garage area to the rear of both No 1 and No 3. These openings are approx 3.7 m wide and 3.2 m high with the walls above supported on steel beams. The first floor has been removed to accommodate the large openings and additional steel beams have been inserted to carry the internal walls and chimney breast above.

b) First Floor.

3 Widcombe Parade.

The first floor is of timber construction with rear and party walls of the same thickness as those at ground floor. The front wall is solid stone 175 mm thick with three 940 mm wide x 1440 mm high sash windows. The main room and stairwell are formed by internal walls which follow the lines of the downstand beams visible below and a passage and bathroom are formed by an additional timber board partition. In the main room the chimney breast has a newer brick flue connected into its side. This flue runs across the alcove from the rear wall and serves the fire in the ground floor rear extension. The rear wall in this room also contains an existing opening which is now a cupboard with a plaster and lathe skin on the outer face of the wall. This room opens onto the passage and the quarter turn stairs which lead up to the second floor. At the other end of the passage a doorway has been formed in the rear wall and this leads onto an enclosed landing which is approx 1.125 m x 2.7 m and projects into the rear extension. The landing contains a WC cubicle and gave access to the ground floor down a stairway which has been removed.

1 Widcombe Parade.

The only surviving part of the first floor is the landing at the top of the straight stairway which leads to the start of the quarter turn stairs which are the mirror image of those in No 3.

c) Second Floor.

3 Widcombe Parade.

The second floor is of timber construction, the external walls are the same thickness at first floor level and the three sash windows on the front elevation are repeated again on this floor. The main internal walls carry on up through from the first floor and form a second floor consisting of a large room as below, a smaller bedroom and a stairwell containing a landing and quarter turn stair. The large room remains in its
original form with a chimney breast on the party wall with No 5, two sash windows on the front wall and a completely solid rear wall. An original window opening exists in the rear wall of the bedroom, this opening rises from floor level to a height of 2.0 m and is approx 600 m wide. At the junction of the party wall and the internal wall of the stairwell an opening has been cut through to No 1 and later closed using a thin screen wall, this has left a recess in the bedroom wall 820 mm wide x 1.81 m high x 460 mm deep.

1 Widcombe Parade.
Apart from very minor differences, No 1 is a mirror image of No 3 at this floor level and should be regarded as such for the purposes of this report.

d) Third Floor (Attic).
The attic floor is of timber construction, the size of the joists could be measured through a hole in the ceiling below and are 160 mm deep x 40-45 mm wide spanning front to back at 300 mm crs. The joists bear into 75 mm deep pockets cut out of the stonework of the front wall and most likely sit on top of the rear wall. The party walls in the attic are the same thickness as below and the front and rear walls are timber construction with plaster and lathe finish inside and slate and tile outside. The internal walls are continued up from below to duplicate the room layout from the second floor. Natural light is provided for in the attic through two dormer windows at the front, one in the large room, one on the landing and a window in the rear walls serves the small bedroom.

1 Widcombe Parade.
This also mirrors No 3 and should be regarded as such for the purposes of this report.

e) Roof.
1 & 3 Widcombe Parade.
The front wall and pitched roof of the attic rooms are covered in slates with drainage to the front of both 1 and 3 via a gutter behind the parapet wall, this drains through an outlet and downpipe between 3 and 5. Clay tiles cover the rear wall of the attic rooms with exception of the bottom two oversailing courses which are stone tiles. This side of the roof drains into a normal gutter and downpipe.

b) Internal.
On first appearances the buildings look in a very poor state but this is due mainly to neglect of the decoration and more recently vandalism by squatters. There seems to have been major alterations to the front and rear walls at various stages of the buildings history. In particular the ground and first floors seem to have taken the brunt of the alterations, surprisingly the only visible sign of distress is a crack on the party wall in the stairwell of No 3 between first and second floor. (see photo 3). There is no sign of this crack on either side of the party wall below first floor or above second floor level although something may be hidden behind the house and deteriorating dry wall lining on the ground floor of No 3.

There are two areas of wet floor, one on the second floor and one directly above in the attic. These are caused by missing tiles on the attic wall letting water which then seeps through to the floor below.

Water leaking from the parapet gutter has caused rot in at least one of the ends of the attic floor joists.

c) External: Front.
The building on the other side of No 1 to No 3 has been demolished leaving only a 800 mm wide strip of wall which runs the full height of the building. The line of this part of the building was 85 mm in front of No 1 and No 3. There is a vertical crack running up this step in the wall which may have been caused during the demolition or by other factors related to the demolished building which at this stage can no longer be determined.

Another crack is visible on the front of No 1, this starts at the top corner of the garage door farthest from No 3 and follows the joints of the stonework up and across to the sill of the second floor window. The most likely causes of this crack are a slight settlement at a support and deflection of the beam over the garage door. There is also a fracture in the lintel over the entrance door, caused by differential settlement due to the higher point load exerted on one side of the door. (see paragraph 4). On No 3 a crack has formed above the beam over the shop front at the end nearest No 5, this crack follows the joints of the stonework up and across to the first floor window lintel and can be attributed to deflection of the beam spanning the shop front. (see photograph 5).

Finally there is a vertical crack which runs the full height of the building and occurs at the junction of the properties. This crack appears to be an opening up of a straight joint but removal of some of the paint covering the crack and a close examination from a ladder or scaffolding is required to confirm the diagnosis (see photograph 6).

d) External: Rear.
The rear of the building is to a large extent obscured by the extensions and garage and what is not obscured is difficult to view. What could be seen of the rear, although of a rougher and more random type of stonework, seems very stable with no signs of distress. There is however an external flue from a heating appliance which runs up the wall between No 1 and No 3 which due to a lack of proper bonding to the main wall has come adrift by up to 25 mm at second floor level.
MONITORING

Monitoring was considered but due to access problems it would not be practical at the present time. When renovation and conversion of the properties begins, ladders and other methods of access will be available and with the developers' consent monitoring of the building could be instigated.

On the day of the survey the variation from vertical of the front wall of No 3 was checked in two places using a plumb line. The maximum deviation from vertical found over a height of 5.75 m was 22 mm at approximately 3.25 m above ground level.

CONCLUSIONS

The cracks caused by the overloading of certain areas of the foundations and the deflection of the support beams are unsightly but they are old and not severe. The building has moved and settled and if movement is continuing it is so small that only close monitoring will show it.

Although traffic vibration can be felt through the timber floors, especially the attic floor, none of the damage to the building can be directly attributed to the passing of heavy traffic. The constant vibration from the traffic may not have caused the damage but it may exacerbate the already damaged areas. This can only be proven or disproven through monitoring which can hopefully be set up at a later date.

Investigation of the foundations was not possible at the time this survey was carried out but should be undertaken in the future to determine any likelihood of further settlement of the support columns for the beams carrying the front wall. It must be said that the cracks and fractures found on this building are not uncharacteristic for a building of this type and age considering the extensive alterations to the facade at ground floor level.
Photograph 1  Front facade of numbers 1 and 3 Widcombe Parade

Photograph 2  Widcombe Parade
Photograph 3  Crack on party wall in stairwell

Photograph 4  Fracture in the lintel over entrance door
Photograph 5  Cracking due to deflection of beam spanning shop front

Photograph 6  Vertical crack at junction of properties
Numbers 1 and 3 Widcombe Parade (ground floor)
Numbers 1 and 3 Widcombe Parade (first floor)

- First floor
- Bathroom
- No first floor exists due to removal for increased bedroom in the garage.
- Staircase from 3rd floor to 1st floor, has been removed.
- Timber frame.
- Extending roof conservation.
- Brick flue in alcove.
- First floor landing.
Number 3 Widcombe Parade (second floor)

- Wet area due to water retention from roof area.
- Opening (12 m high)
- APART FROM MIRROR CHANGES OF INTERNAL PARAPET WALLS IN WIDCOMBE TERRACE, HOUSE NO. 3.

Typical section

Number 3 Widcombe Parade (third floor)

- Wet area due to running roof tiles.
- Attic (3rd floor)
- 160 mm cavity (400 mm cavity at 600 mm).
- APART FROM MIRROR CHANGES OF INTERNAL PARAPET WALLS IN WIDCOMBE TERRACE HOUSE NO. 3.
12.2 THE PARISH CHURCH OF ST JAMES, LOUTH

Structural Engineering report for a traffic vibration study

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November 1987

GENERAL DESCRIPTION AND HISTORICAL NOTE

St. James', Louth has been described as one of the finest perpendicular churches in the country. It is 182 feet (55.5 m) long and 72 feet (22 m) wide. The spire is 295 feet (90 m) high. A plan of the Church is attached at Figure 1.

According to the Guide Book to the Church by Hedley Warr, the first Church on this site was erected in 1170, although no above ground trace of this now remains. A second Church was built about 1247, but it was shorter and narrower than the one which now exists. The present building dates from between 1430 and 1441 and it suggested that much of the material from the earlier building was re-used, including the nave piers.

The splendid spire was added between 1501 and 1515 at a cost of £305 7 shillings and 5 pence. The financial records for this still exist.

The nave and aisles were re-roofed in 1825 and the chancel was similarly treated in 1828.

A major restoration of the Church took place between 1861 and 1869. The C19 saw a very great many restorations at Medieval churches as well as the erection of large numbers of new churches of both the established and non-established churches. The restoration at Louth was carried out under the direction of James Fowler and was more of an internal re-ordering than a major structural alteration. However, the Medieval south porch was replaced with the existing one and the north porch was added.

RECENT MAJOR WORKS

It appears that a ring beam was inserted in the tower in 1937 and that the north arcade previously noted as being re-used from an earlier building was stabilised in 1954.

No other major structural works are thought to have taken place, although works of a rather more maintenance nature such as replacing of decayed stone and re-leading of roofs has been done.

STRUCTURAL CONDITION OF CHURCH

The building was inspected by the author of this report on Monday 19 October 1987. The assistance of Mr Crust, Church Warden, is gratefully acknowledged concerning his knowledge of the Church and its history.

The Church is in very good condition structurally as well as in all other ways generally. Only 2 lines of fractures of any importance were noted. These are at the crown of the easternmost arches of the chancel arcade both north and south sides of the chancel. These cracks are obviously of considerable age. There are brass screw measuring pins fixed at the crown of the arch across the crack and the cracks were reported by the English Heritage commissioned Architect in October 1978. These cracks are noted on the plan at Figure 1 and must indicate a slight eastward (outward) lean of the chancel east gable wall.

There is a boiler flue at the north-east corner of the chancel arch north pier. The rendering of this pier has quite widespread map cracking. This form of cracking is caused by the heat of the flue gases rather than by any structural movement.

The remainder of the interior of the Church was limewashed in the spring of 1987 and has no fractures showing other than the occasional hairline crack to indicate any live movement.

The north aisle parapet when viewed from the tower, is distinctly bowed outwards at the centre.

COMPARISON WITH THE ‘AVERAGE’ CHURCH

Clearly, there is no such animal as the ‘average’ church as each one is distinctive in its size, style, age and condition, but it would be useful to compare the problems or the lack of them as noted at St James', Louth, with what might reasonably be expected, basing such expectations on the author’s experience in considering the structural condition of many churches over a period of some 10 years.

The first point to note is that St James’ has far fewer signs of structural distress than might reasonably be expected on a building of this size.

Referring first to the fractures in the east end of the chancel arcade, very many east churches of all ages suffer fractures adjacent to the east gable. Very often, the east gable is fractured in the area over the window and again, many fractures occur where the east gable wall abuts the north and south chancel walls. This form of structural distress is related to the great weight of the east gable as compared with the lesser weight of the side walls. Therefore, the cracking at the east end of the chancel arcade at St James’, Louth, is by no means unusual.

Moving further along the Church, we come to the chancel arch and its piers. Again, this is a location where very often, cracks are found in churches, due again, to the great weight of this part of the structure. At St James’, Louth, no cracking of any magnitude was noticed in the chancel arch. The jambs of the chancel arch are often found to be distorted or leaning outwards due to the thrust from the chancel arch
itself. The jambs of the chancel arch at St James', are quite plumb and true to line. Looking now at the nave of the Church, it is very often found that the nave walls and the walls of the north and south aisles where they exist lean outwards due partly to the thrust from the roof and partly due to inadequate foundations. As already noted, there has been outward movement of the columns to the nave arcades, but this has now been stabilised. The north wall of the north aisle is quite plumb as is very often found, and the south wall of the south arcade which in many cases leans outwards due to moisture changes in the ground, in an area which is subject to more wetting and drying than the north side of the Church, at St. James', Louth, is relatively plumb.

Another common place to find fractures in churches is the tower. These very often have vertical fractures running up one or more faces of the tower and it is commonly supposed and is probably quite possible that these fractures are caused by the ringing of church bells as the tremendous weight of these causes stresses on the tower which the tower is not really capable of taking. A ring beam has been installed as has already been noted, at St James', Louth, so there may have been a past history of fractures. However, inspection of the tower showed no such fractures at present existing.

CONCLUSIONS

In spite of the fact that St James', Louth, is a very large Church, with very high walls and extremely high tower; apparently the highest parish church tower in the country, there are few signs of structural distress and the Church as has already been mentioned, is in very good condition. It seems unreasonable to apportion any of the blame of the cracking at the east end of the Church to the traffic, in spite of the fact that this is a very busy road and very very close to the Church.

12.2.1 COMPARATIVE STUDY OF WINDOWS

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5th October 1988

1. Introduction

On September 17th 1987 and November 25th 1987 the interior and exterior of the east and west windows of the south aisle were examined from ground level and cill level by hand and eye and with the assistance of a telescope. The Department of Transport studied two additional south aisle windows i.e. first eastwards of the south entrance and the second window from the east end. These were only examined from ground level and by telescope, interior and exterior in November. In September the conditions were dry, windy and warm; in November, wet, calm and cold with the heating in operation.

2. Description

2.1 East window, south aisle.
Five lights, 5 tiebars. Subject: Life of St Stephen. Commemoration: Stephen Fryche d. 1882. The window is not datable from documentary sources, and the workshop/artist is not recorded. Date: post 1882, certainly late C19. (See Plate 1).

2.2 West window, south aisle.
Identical in form to above but with six interior ties. Subject: Joseph, Gideon, St Michael, David and Jonathan. Commemoration: dated 1869. The window is not datable from documentary sources and the workshop/artist is not recorded. Date: 1870s. (See Plate 2).

2.3 South window, second from east end.
Three lights, six tie bars. Subject: Christ ministered to by angels in the desert after the Temptation, the Baptism and Agnus Dei. Commemoration: 1879. The window is not datable from documentary sources and the workshop/artist is not recorded. Date: 1880s. (See Plate 3).

2.4 South window, first east of entrance.
Three lights, six tie bars. Subject: Benjamin weeping, Joseph presents Jacob to Pharaoh, Israel blesses Ephraim and Manasseh. Date: 1902 by Heaton, Butler and Baynt (source Louth parish church guide by R D Goulding 1916, reprinted 1930). (See Plate 4).
3. DAMAGE OBSERVED

3-1 The east window of the south aisle.
This was constructed of abnormally large narrow panels with H-profile leads.

The deformation of the base resulted from the vertical pressure of the over-large main light divisions and lack of support rather than traffic vibration. There were no cracks, fissures, crazing or other intrinsic observable weaknesses, the leads were weakened but still flexible and watertight.

3-2 West window, south aisle
This has sustained extensive paint loss due to underfiring but is otherwise sound structurally.

3-3 South window, second from east end.
This was sound, no observable deterioration.

3-4 South window, first east of entrance.
Sound, no observable deterioration.

4. CONCLUSION
There is no perceptible physical evidence of any damage attributable to traffic vibration in any of the windows exposed directly to high levels of traffic and low frequency vibrations. There appeared to be no difference in the condition of the windows in direct relation to their location in the church and distance from the traffic. Any deterioration of glass, lead and ferramenta could all be attributable to the method of construction, design, infrastructure and effectiveness of the support system.
Fig. 1 Plan of St. James Church
Plate 1 East window in south aisle

Plate 2 West window in south aisle

Neg. no. R737/87/3

Neg. no. 737/87/6
Plate 3 South window, second from east end

Plate 4 South window, first east of entrance
THE OLD BELL COTTAGE, NORTON

Report of structural survey

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December 1987

12.3 THE OLD BELL COTTAGE, NORTON

(54 Evesham Road)

GENERAL DESCRIPTION

This house is the east-west cross wing of a T-shaped group of buildings numbered 51, 53 and 54 Evesham Road. Evesham Road, Norton is the very heavy trafficked A435 leading from Evesham to Birmingham and Coventry.

It is basically a rectangular building of probably 15th century, with a 19th century porch and bell cote added and an additional mid 20th century rear 2 storey kitchen/bathroom extension.

The original part of the house has 2 rooms downstairs, that to the rear having an open plan quite modern staircase leading to the upper floor, which also has 2 bedrooms, the rear of which has had an area partitioned off to provide a passageway to the newer bathroom.

The east gable wall which fronts the A435 and the north flank wall are timber framed, the front wall having lath and plaster infill panels, whilst the north flank wall has brick noggins.

The south flank wall has been rebuilt at some stage in brickwork and reuses an earlier window to the front bedroom. The rear wall of the original cottage is now hidden behind the finishes of the new extension. This extension is brick built and has a roof of a lower pitch than the main house, the pitch of which is quite steep.

A sketch plan (not to scale) is at Fig 1. The construction of the half timbered walls is typical with timber corner posts, sills plates and bressumbers at first floor and roof levels, together with various infill posts and bracings. Both timber framed elevations are built on a brick plinth, also a typical form of construction. The timber posts never went into the ground as this would, of course, cause severe rot problems, the builders realising this.

STRUCTURAL CONDITION OF THE COTTAGE

The Old Bell Cottage is in good order structurally, even if it is somewhat distorted from its original shape.

The front gable wall leans towards the road generally and the roof gable itself more so than the rest of the wall. This wall also has a mid-bulge at first floor level. There are, however, no indications that this movement is continuing. Indeed there are signs that movement occurred many years ago. The joint between the front wall and the ceiling of the front room downstairs was specifically inspected regarding this movement. No significant movement has taken place recently and there is only the slightest indication of a fracture at one point of this junction. The south east corner post is quite vertical and abuts the vertical modern south wall, whilst the north east corner post, however, is quite out of plumb by several inches. Close examination has suggested that the bottom of this north east corner post has in the past shifted outwards at the bottom rather than the top moving in. This may well have been associated with decay of its face. It now appears to be stable.

It is this outward movement of the base of the north east corner post which in my opinion has caused the distortion of the plasterwork on the north wall of the front room downstairs. There is quite a severe bulge in the plaster and the owner was quite rightly concerned about this. However, I feel that there is no current problem and the solution seems to be to take out the damaged plaster and to replaster to a more level surface. The only other cracking noted internally was very minor cracking on the original rear wall, now the partition wall between the rear room downstairs and the kitchen. This minor cracking may well be shrinkage of plaster and is of no structural significance.

EFFECTS OF TRAFFIC VIBRATION ON THIS BUILDING

My inspection of the structure of this building suggested to me that it was indeed in extremely good condition with very little signs of movement, no more movement than would be expected and indeed a lot less than might well be expected. The severe distortion of the front gable wall has clearly been in existence for very many years and may well, and almost certainly did, exist before heavy traffic of todays standards began to use the road outside. I attach to this report a number of photographs of No 54 Evesham Road. I also attach a photograph of a building in Evesham which is nowhere near any busy road at all, and is perhaps 50 yards from a fairly lightly used road. It will be noted that the front wall of this building is of somewhat similar proportions to the Old Bell Cottage and also has a very very marked lean outwards. The point of this photograph is to suggest that the lean of the front elevation of the Old Bell Cottage is by no means unreasonable and the distortion cannot therefore be blamed on traffic vibrations.
Fig. 1 Plan of Old Bell Cottage

No. 53 Evesham

Timber frame

Lounge (Bedroom over)

Brick wall

Porch

Brick wall

Dining room (bedroom & passage over)

Timber frame

Evesham Road

Kitchen (bathroom over)
The Old Bell Cottage

Distortion of front elevation
Internal damage in lounge

View of house in Evesham
12.4 RISING SUN FARMHOUSE NEAR HONITON, DEVON

Structural survey report
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February 1989

Introduction
This report concerns the structural condition of the above building which was chosen by the Transport and Road Research Laboratory as part of their study into traffic vibration and its effect on historic buildings.

The initial survey was carried out on the 28th January 1988.

GENERAL DESCRIPTION
The building stands isolated, at a crossroads, on an unclassified road about half a mile due south of the main A30 leading to Honiton. The building was selected by the TRRL as a good example of a 'before' and 'after' case for traffic density. At the present time (February 1988) there is very light traffic passing the building, this being the occasional car or lorry. However a new gas pipe depot is currently being built opposite and this will give a good opportunity to study the effects of vibration from the heavy lorries leaving the depot with their deliveries. The majority of these lorries should have to pass close by and within a few feet of the front entrance porch.

The building was once an old inn but has now been converted into a dwelling house. It is basically a two storey rectangular building with a small front entrance porch. There are four rooms downstairs plus entrance hall with a timber staircase leading to the upper floor of traditional timber construction and which comprises five bedrooms and a bathroom. The external walls, which support a bitumen coated slate roof are of substantial thickness and are a mixture of stone and flint construction.

The majority of the windows have brick arches and brick facings to the jambs. The gable wall at the north end has been rendered and the corners of the main building are brick faced.

Appended to this report are plans and elevations showing general layout, monitoring positions and location of photographs.

STRUCTURAL CONDITION
With the exception of the porch the main building, generally, is in good structural condition. The roof line is slightly uneven but has no missing slates. All the main walls are reasonably plumb and there are no signs of differential settlement or cracking. The south gable wall and rear wall have recently been re-pointed. The rendered face to the north gable wall has some minor cracking and flaking of the render.

Although not serious, the main structural problem area concerns the entrance porch which has some external cracking to the north elevation and has generally suffered from settlement in the past.

The front and side walls of the porch have an outward lean of approximately 40 mm over a height of about 1.5 metres. A tie rod has been inserted in the past just above door height to help restrain the north and south walls from outward movement. This tie appears to have been put in place some considerable time ago. The outward lean of the walls has caused the brick arch to settle and crack at the springings with the settlement cracks on each side of the arch continuing through the external stone face up to ridge level. The corner cracks which obviously had once occurred between the porch and main wall have been re-pointed and had not re-opened. This shows that any recent movements between the porch and main building is now possibly less of a problem and that the porch has become stable. All the internal walls to the porch have been re-pointed and no cracks were evident.

Some areas of the internal wall surfaces are prone to dampness caused either by direct penetration of rainwater through porous brickwork and mortar joints or by faulty roof flashings.

A combination of re-pointing and a suitable water repellant silicone treatment to the exterior wall surfaces has largely helped to overcome this problem, particularly to the south gable wall. Other areas of wall still have damp problems and this treatment should be extended wherever possible.

MONITORING
Demec strain gauge points were fixed across the major external cracks to measure any possible future changes in crack width.

The positions were recorded and are shown on the relevant elevations. The initial readings were taken.

When the Pipe Depot is completed and the traffic density has substantially increased then a second visit will be made and a set of readings taken at the weak areas of the structure to determine possible movements. From these results conclusions can be reached about any possible movements related to vibration or other causes.
A second visit was made on the 20th April during the period when delivery of the pipes was being made to the Depot and a second set of demec readings was then taken.

A third visit was made on the 17th June when the pipe lorries were leaving the Depot with their full loads for delivery and these were passing very close to the front of the house at approximately 30 minute intervals.

A third set of demec readings was again taken at this time.

During the visit of the 17th June some vertical cracking to the recently pointed south gable wall was noted between the windows and also to the render over the ground floor window. This cracking was not apparent on the initial visit and may have been caused either by the internal work which involved stripping the plaster or the subsequent drying out of the wall following repairs to the chimney flashing.

Additional Demec points (8) and (9) were added to monitor these cracks. Also an additional Demec point (10) was added to the south wall of the porch to monitor a small crack above the brick plinth.

These additional points are shown on Elevations D-D and E-E.

Visits we made on the 12th August and 9th February of the following year, after the lorry flow had decreased substantially. Further sets of demec readings were taken.

The recently re-pointed corners between the porch wall and the main wall had not shown signs of movement or cracking during the previous year.

CONCLUSIONS

In addition to the normal demec readings the strain gauge was placed across the points on the porch wall at positions (1), (2) and (3) to coincide with the passing of the pipe lorries which were either full or empty.

No movement of the dial gauges or vibration of the wall could be detected at the time the lorries were passing the front porch. The settlement of the porch and associated cracking has possibly been caused by inadequate foundations or poor subsoil and it would appear that the problems associated with the porch have been in evidence for some considerable time.

The relatively minor cracking to the gable walls are caused by thermal movements and also the past problem of damp penetration to the internal core of the wall which is now drying out following the remedial work.

The major damp problems to the exterior walls has, according to the owners, now been overcome following the damp treatment to the wall surfaces and the remedial work to the chimney.

The general movement of the lorries passing close to the building during the past year does not appear to have contributed to the deterioration of the fabric.

The crack movement sheets which are appended to this report show the general trend of the cracks with regards to opening or closing. These results are relatively small and are more related to changes in seasonal temperature.

Readings will continue at six monthly intervals.
ELEVATION D-D

ELEVATION E-E

PORCH

RISING SUN
FARMHOUSE
NEAR HONITON
DEVON
Plate 1 Rising Sun farmhouse

Plate 2 North wall of porch
Plate 3 Showing lean of front porch wall

Plate 4 South wall of porch